SINCLAIR KNIGHT MERZ

SKM

CHRISTCHURCH CITY COUNCIL
PRK_1474_BLDG_002 EQ2
Scarborough Jet Boat Shed
2 Scarborough Beach

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev B
- 26 February 2013

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on building PRK_1474_BLDG_002 EQ2 located at Scarborough Beach, This building is a two-storey block masonry and timber framed structure that is used by the Scarborough Surf Lifesaving Club. The building was originally constructed as single storey concrete block building in 1976. Later, it was extended in lateral and longitudinal directions (concrete blocks), the construction date of this building modification is unknown. Finally, the timber framed upper storey was added in 2004. An aerial photograph illustrating the building's location is shown below in Figure 1. Detailed descriptions outlining the buildings age and construction type are given in Section 5 of this report.

- **Figure 1 Aerial Photograph of Building PRK_1474_BLDG_002 EQ2 Located at Scarborough Beach**

This Quantitative Assessment report for the building structure is based on the recommendations of "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" issued by an NZSEE Study Group on June 2006, our Geotechnical Desk Study dated 3 April 2012, visual inspections carried out on 26 April 2012 and our intrusive inspection carried out on 27 November 2012.

1.2. Key Damage Observed

Key damage observed includes:

- Cracking to masonry block wall.
- Cracking to plasterboard linings.

Further details describing the level of damage and repair recommendations are given in section 5.5.

1.3. Critical Structural Weaknesses

There are no critical structural weaknesses that have not been incorporated into our analysis.

1.4. Indicative Building Strength (from Quantitative Assessment)

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have assessed the percentage of new building standard seismic resistance using the quantitative method. Our assessment included consideration of geotechnical conditions, existing earthquake damage to the building and structural engineering calculations to assess both strength and ductility/resilience.

The assessments were based on the following:

- On-site investigation to assess the extent of existing earthquake damage including limited intrusive investigation.
- Qualitative assessment of critical structural weaknesses (CSWs) based on review of available structural drawings and inspection where drawings were not available.
- No geotechnical investigation has been undertaken. We have based this report on our knowledge of the site and the absence of liquefaction ejecta on the site.
- Assessment of the strength of the existing structures taking account of the current condition.

Any building that is found to have a seismic capacity less than 33% of the new building standard is required to be strengthened up to a capacity of at least 67%NBS.

Based on the limited structural drawings and using the NZSEE Detailed Assessment Procedure, the buildings capacity has been assessed to be in the order of 10%NBS. This is limited by the capacity of the first floor timber diaphragm.

- **Table 1: Quantitative assessment summary**

Grade	Risk	%NBS	Structural Performance
E	High	10	Unacceptable. Improvement required.

Please note that structural strengthening is only required by law for buildings that are confirmed to have a seismic capacity of less than 34% NBS.

1.5. Recommendations

It is recommended that:

a) A geotechnical investigation is undertaken to confirm the geotechnical parameters used in this report.

b) A full detailed design be carried out to confirm the required strengthening works to bring the seismic capacity of the building up to a minimum of 67% NBS.

c) Barriers around the building are not necessary.

2. Introduction

Sinclair Knight Merz was engaged by the Christchurch City Council to prepare a Quantitative assessment report for building PRK_1474_BLDG_002 EQ2 located at Scarborough Beach following the magnitude 6.3 earthquake which occurred in the afternoon of the 22nd of February 2011 and the subsequent aftershocks.

The scope of this quantitative analysis includes the following:

- Analysis of the seismic load carrying capacity of the building compared with current seismic loading requirements or New Buildings Standard (%NBS). It should be noted that this analysis considers the building in its damaged state where appropriate.
- Identification of any critical structural weaknesses which may exist in the building and include these in the assessed %NBS of the structure.

The Quantitative assessment uses the methodology recommended in the NZSEE Study Group document "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes". This report assesses the likely performance of the structures in a seismic event relative to the New Building Standard (NBS). 100% NBS is equivalent to the strength of a building that fully complies with current codes. This includes a recent increase of the Christchurch seismic hazard factor from 0.22 to 0.3[1].

This report describes the structural damage observed during our inspection and indicates suggested remediation measures. The inspection was undertaken from floor levels and was a visual inspection only. Our report reflects the situation at the time of the inspection and does not take account of changes caused by any events following our inspection.

At the time of this report, limited intrusive site investigation of the building structure have been carried out. Structural drawings were available for this structure and as a result the buildings description outlined in Section 3 is also based on our visual inspection carried out on the 27 November 2012.

[1] http://www.dbh.govt.nz/seismicity-info

3. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

3.1. Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses

- The extent of any earthquake damage

3.2. Building Act

Several sections of the Building Act are relevant when considering structural requirements:

3.2.1. Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

3.2.2. Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67%NBS however where practical achieving 100%NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67%NBS.

3.2.3. Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- in the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- in the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- there is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- there is a risk that that other property could collapse or otherwise cause injury or death; or
- a territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

3.2.4. Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

3.2.5. Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

3.2.6. Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

3.3. Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone. Council recognises that it may not be practicable for some repairs to meet that target. The council will work closely with building owners to achieve sensible, safe outcomes;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33%NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67%NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

3.4. 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.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

a) Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
b) Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

4. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

- **Figure 2: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Figure 2 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

- **Table 2: %NBS compared to relative risk of failure**

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

5. Building Details

5.1. Building Description

Our evaluation was based on our site investigation conducted on the 26 April 2012 and the limited structural drawings provided by the Christchurch City Council.

Building PRK_1474_BLDG_002 EQ2 is a two-storey concrete block and timber framed structure that is used by the Scarborough Surf Lifesaving Club. The building was originally constructed as single storey concrete block building in 1976. Later, it was extended in lateral and longitudinal directions (concrete blocks), the construction date of this building modification is unknown. Finally, the timber framed upper storey was added in 2004. The timber upper storey walls are supported on concrete block walls that form the lower storey. The Level 1 suspended floor is also constructed from timber framing and is supported on the concrete block walls. The cladding to the roof consists of light weight corrugated steel whereas the cladding to the upper storey walls consists of weatherboards. Limited structural drawings show that the lower storey concrete block walls are vertically reinforced and are founded on concrete strip foundations. The ground slab is a concrete slab on grade. The building is situated at the base of a hill however the slope is supported by shotcrete and is not deemed to be in an unstable condition, refer to our Geotechnical desktop study detailed in Appendix 4 – Geotechnical Desk Study.

5.2. Gravity Load Resisting System

As noted above the roof structure is constructed from timber framing and is supported on the upper storey timber framed walls along the east and the west side of the building. The upper storey is supported on ground floor concrete block walls. The Level 1 suspended floor is constructed from timber framing and is also supported on these concrete block walls. The building is founded on concrete strip footings and a concrete floor slab on grade.

5.3. Seismic Load Resisting System

For the lateral analysis of this building the ‘across direction’ has been taken as east-west whereas the ‘along direction’ has been taken as north-south.

Lateral loads acting across and along the building will be resisted at Level 1 by the plasterboard linings present on the ceilings and walls. The lateral loads from the upper storey will then be transferred into the lower storey concrete block walls. The suspended timber floor may act as a nominal diaphragm which will also transfer its load into the lower storey block walls. There is a large opening in the southern block wall which will cause the building to have torsional response when it is loaded in an east-west direction. This torsional response will be resisted by the east and west walls, but only if the first floor has sufficient shear capacity to act as a diaphragm.

5.4. Geotechnical Conditions

A geotechnical desktop study was carried out for this site. The main conclusions from this report are:

- Due to the lack of geotechnical investigation data, an estimation of the ground properties for this site has not been made in this desk study.
- No liquefaction appears to have occurred due to the 22 February earthquakes. However, further site specific investigations would be needed to make a full liquefaction assessment of the site.
- The slope directly behind the jet boat shed has been supported by Shotcrete whereas the slope north and south of the jet boat shed is supported by dry stacked boulders. These appear to have performed satisfactorily with minimal movement adjacent to the boat shed, however, behind the historic boat shed to the south, the boulder wall collapsed during the 13 June event. The road above at hairpin bend looks to have moved approximately 15mm northwest as shown by the crack in the footpath. This is likely to be related to movement of fill or damage of underground services and not an indication of large scale rock movement. From site observations, there does not appear to be any stability issues related to the slope behind the jet boat shed.
- Due to a lack of existing information for the purpose of carrying out a Quantitative Assessment a conservative seismic class should be used. We recommend that seismic class D should be used. Further Geotechnical investigation will be required if design works need to be carried out or a building consent obtained.

The full geotechnical desktop study can be found in Appendix 4 – Geotechnical Desk Study.

5.5. Building Damage

SKM undertook inspections on the 26 April 2012. The following was observed during the time of our inspection:

5.5.1. External Damage

Note that access to the south and east walls was fenced off and therefore external inspections of these areas could not be conducted.

South Wall

1) The following damage was observed on the south wall: -
   - Top course block located on the east side of the roller door has cracked right through the face (PHOTO 4 & 5).

- Minor cracking to block wall mortar joints near entarnce door in the south-east corner (PHOTO 6 & 7). This cracking may be the result of drying shrinkage movement rather than earthquake damage.

West Wall

2) The following damage was observed on the west wall: -

- Hairline cracking through mortar joints and face of the block work in the south-west corner (PHOTO 8 & 9).
- Spalling to the block faces to the top course of the blocks in the north-west corner (PHOTO 10).

5.5.2. Internal Damage

East Stair

The following damage was observed in the east stair: -

- Step cracking present along the block wall mortar joints on the east and south walls (PHOTO 12, 13 & 14).
- Hairline crack present between the block wall and timber wall joint in the south-west corner (PHOTO 15 & 16).
- Hairline cracking present in wall lining joints on the west wall (PHOTO 16 & 17).
- Cracking present between the timber stair stringer and the wall lining joint (PHOTO 19). Timber stair skirting damaged at the base of the stair as well as mid way up the stair (PHOTO 18 & 20). Stringer plate also appears to have separated from the eastern wall.
- Horizontal crack present in wall lining at the top of the stair (PHOTO 21).
- Cracking present along the block wall and timber infill wall joint (PHOTO 22 & 23).
- Horizontal hairline crack present at the wall lining joint on the north wall at the top of the stair (PHOTO 24 & 25).

Garage

The following damage was observed in the garage: -

- Damage present to the wall lining on the east wall. This damage appears to be caused by impact forces and therefore is not earthquake related (PHOTO 27).
- Vertical hairline crack present in the wall lining above the personnel door on the east wall (PHOTO 28).
- Horizontal crack present in wall lining in the south-east corner (PHOTO 29).

Level 1

The following damage was observed on Level 1: -

- Main Room: - Cracking present between the wall lining and ceiling lining joint in the south-east corner (PHOTO 31).
- Main Room: - Hairline cracking present between the full height window architrave and the wall lining joint (window located on the north wall) (PHOTO 32 & 33).
- Southern Office: - Hairline cracking in wall lining above south corner of doorway.

6. Available Information and Assumptions

6.1. Available Information

Following our intrusive inspection carried out on the 27 November 2012, SKM carried out a seismic review on building PRK_1474_BLDG_002 EQ2 located at Scarborough Beach. This review was undertaken using the available information which was as follows:

- Structural drawings by Christchurch City Council dated 1976 (Appendix 2 – Structural Drawings).
- Architectural drawings by David Hopper Architecture dated 2003 (Appendix 3 – Architectural Drawings).
- SKM Geotechnical Desk Study (Appendix 4 – Geotechnical Desk Study)

6.2. Design Criteria and Assumptions

The following design criteria and assumptions made in undertaking the assessment include:

- The soil on site is class D as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. This is a conservative assumption based on our experience of soils around Christchurch. Liquefaction does not need to be accounted for in the foundation design. The latter two assumptions assume that the ground conditions classify as "good ground".
- Standard design criteria for typical office buildings as described in AS/NZS1170.0:2002:
  - 50 year design life, which is the default NZ Building Code design life.
  - Structure Importance Level 2. This level of importance is described as 'normal' with medium or considerable consequence for loss of human life, or considerable economic, social or environmental consequence of failure.
- The building has a short period less than 0.4 seconds.
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- A ductility of 1.25 was taken for reinforced concrete block masonry. For the timber frame structures a ductility of 2.0 was taken.
- Due to the limited damage observed during our visual inspection we have assumed that the reinforcement in the concrete masonry has been undamaged as a result of the earthquake.

The following material properties were used in the analysis:

- **Table 3: Material Properties**

Material	Nominal Strength	Probable Strength Factor	Strength Reduction Factor
Timber frame diaphagm	f_v = 4.2 MPa	f_p = 1.0	Ø = 0.7
Timber walls (10mm GIB one face)	f_y = 2.5 kN/m	f_p = 1.0	Ø = 1.00
Timber walls (Gib bracelines, 12kN connectors)	f_y = 5.5 kN/m	f_p = 1.0	Ø = 1.00
Concrete Masonry (compression)	f'_m = 4 MPa	f_p = 1.5	Ø = 1.00
Concrete Masonry (shear)	v_{bm} = 0.3 MPa	f_p = 1.0	Ø = 0.85
Reinforcing (bending)	f_y = 300MPa	f_p = 1.08	Ø = 1.00

The detailed engineering analysis is a post construction evaluation therefore it has the following limitations:

- It is not likely to pick up on any concealed construction errors (if they exist).
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the structure will not be identified unless they are visible and have been specifically mentioned in this report.
- The detailed engineering evaluation deals only with the structural aspects of the structure. Other aspects such as building services are not covered.

6.3. The Detailed Engineering Evaluation (DEE) process

The DEE is a procedure written by the Department of Building and Housing's Engineering Advisory Group and grades buildings according to their likely performance in a seismic event. The procedure is not yet recognised by the NZ Building Code but is widely used and recognised by the Christchurch City Council as the preferred method for preliminary seismic investigations of buildings[2].

The procedure of the DEE is as follows:

a) Qualitative assessment procedure

   i. Determine the building's status following any rapid assessment that has been done.

   ii. Review any existing documentation that is available. This will give the engineer an understanding of how the building is expected to behave. If no documentation is available, site measurements may be required.

[2] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

iii. Review the foundations and any geotechnical information available. This will include determining the zoning of the land and the likely soil behaviour, a site investigation may be required.

iv. Investigate possible Critical Structural Weaknesses (CSW) or collapse hazards.

v. Assess the original and post earthquake strength of the building (this assessment is subsequently superseded by the quantitative assessment).

b) Quantitative procedure

i. Carry out a geotechnical investigation if required by the qualitative assessment.

ii. Analyse the building according to current building codes and standards. Analysis accounts for damage to the building.

The DEE assessment ranks buildings according to how well they are likely to perform relative to a new building designed to current earthquake standards, as shown in Table 2. The building rank is indicated by the percent of the required New Building Standard (%NBS) strength that the building is considered to have. Earthquake prone buildings are defined as having less than 34% NBS strength which correlates to an increased risk of approximately 20 times that of 100% NBS[3]. Buildings that are identified to be earthquake prone are required by law to be strengthened within 30 years of the owner being notified that the building is potentially earthquake prone[4].

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[3] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*
[4] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

- **Table 4: DEE Risk classifications**

Description	Grade	Risk	%NBS	Structural performance
Low risk building	A+	Low	> 100	Acceptable. Improvement may be desirable.
	A		100 to 80	
	B		80 to 67	
Moderate risk building	C	Moderate	67 to 33	Acceptable legally. Improvement recommended.
High risk building	D	High	33 to 20	Unacceptable. Improvement required.
	E		< 20	

The DEE method rates buildings based on the plans (if available) and other information known about the building and some more subjective parameters associated with how the building is detailed and so it is possible that %NBS derived from different engineers may differ.

This assessment describes only the likely seismic Ultimate Limit State (ULS) performance of the building. The ULS is the level of earthquake that can be resisted by the building without catastrophic failure. The DEE does also consider Serviceability Limit State (SLS) performance of the building and or the level of earthquake that would start to cause damage to the building but this result is secondary to the ULS performance.

The NZ Building Code describes that the relevant codes for determining %NBS are primarily:

- AS/NZS 1170 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard
- NZS4230:2004 Design of Reinforced Concrete Masonry Structures
- NZS 3603:1993 Timber Structures Standard
- NZS 3604:2011 Timber Framed Buildings

7. Results and Discussions

7.1. Critical Structural Weaknesses and Hazards

The following critical structural weaknesses and hazards have been identified.

- Plan Irregularity: Due to the lack of shear walls spanning in an east-west direction the building will be subjected to torsional loadings when it is loaded in an east-west direction.

This critical structural weakness has been incorporated into our assessment of the building. The effect of this will be a lower quantitative assessment result when compared to a building containing no critical structural weaknesses.

7.2. Analysis Results

The equivalent static force method was used to analyse the seismic capacity of the building. The results of the analysis are reported in the following table as %NBS. The results below are calculated for the building in its damaged state. The building results have been broken down into their seismic resisting elements. As the building has elements that are less than 34% NBS any item with a capacity less than 67% NBS will need to be strengthened so that the overall building capacity is greater than 67% NBS.

(%NBS = probable strength / new building standards)

- **Table 5: DEE Results**

Seismic Resisting Element	Action	Seismic Rating %NBS
Timber diaphragm	Shear	10%
Concrete block wall (north)	Shear	18%
Concrete block wall (east and west)	Shear	36%
Concrete block wall (north)	In-plane bending	41%
Timber diaphragm connection	Shear	98%
Timber walls	Shear	100%
Concrete block wall (south)	Shear	100%
Concrete block wall (east and west)	In-plane bending	100%
Concrete block wall (south)	In-plane bending	100%
Concrete block wall (all)	Out-of-plane bending	100%

7.3. Recommendations

The quantitative assessment indicates that the building has a seismic capacity less than 34% of NBS and is therefore classed as being a ‘High Risk Building’. Strengthening of the building is required to bring it up to a minimum of 67% of NBS in order to comply with current CCC policy.

If it is determined that the building should be repaired there are a number of issues which will need to be investigated and associated documents prepared in order to submit a building consent application. These issues will need to be considered during the initial phase of strengthening works.

Listed below are some of the likely items the council may require to be explored:

- A geotechnical investigation will be required and associated factual and interpretive geotechnical reports prepared – the geotechnical reports will be required to enable completion of the strengthening design.
- A fire report along with an emergency lighting design will be required and all necessary upgrades to egress routes, emergency lighting and specified systems will need to be undertaken.
- A disabled access summary will be required including provision for disabled facilities.
- The site amenities (toilets and the like) will need to be reviewed to ensure that there are sufficient facilities for the expected number of people on site.
- Landscaping will need to be considered although we do not anticipate that any modifications will be required since you will not be adjusting the footprint area of buildings on site and will likely only be required for a new build option.

8. Further Investigation

Further investigation is recommended to confirm the following items:

a) Confirmation of geotechnical parameters used in this report.

b) A full detailed design be carried out to confirm the required strengthening works to bring the seismic capacity of the building up to a minimum of 67% NBS.

9. Conclusion

SKM carried out a quantitative assessment on building PRK_1474_BLDG_002 EQ2 located at Scarborough Beach. This assessment concluded that the building is classified as Earthquake Prone.

Table 6: Quantitative assessment summary

Grade	Risk	%NBS	Structural Performance
E	High	10	Unacceptable. Improvement required.

The seismic capacity is based on the weakest structural element of the building and incorporates the critical structural weaknesses and the damage observed during our site inspection. Since the seismic capacity of the building is less than 34% NBS strengthening is required to bring the seismic capacity up to a minimum of 67% NBS in order to comply with current CCC policy.

It is recommended that:

a) A geotechnical investigation is undertaken to confirm the geotechnical parameters used in this report.

b) A full detailed design be carried out to confirm the required strengthening works to bring the seismic capacity of the building up to a minimum of 67% NBS.

c) Barriers around the building are not necessary.

10. Limitation Statement

This report has been prepared on behalf of, and for the exclusive use of, SKM's client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to, and the assumptions made by, SKM. The report may not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, in the event of any liability, SKM's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited in as set out in the terms of the engagement with the Client.

It is not within SKM's scope or responsibility to identify the presence of asbestos, nor the responsibility of SKM to identify possible sources of asbestos. Therefore for any property pre-dating 1989, the presence of asbestos materials should be considered when costing remedial measures or possible demolition.

Should there be any further significant earthquake event, of a magnitude 5 or greater, it will be necessary to conduct a follow-up investigation, as the observations, conclusions and recommendations of this report may no longer apply Earthquake of a lower magnitude may also cause damage, and SKM should be advised immediately if further damage is visible or suspected.

11. Appendix 1 – Photos

Photo 1: South Elevation. No bracing wall at ground floor level

Photo 2: West Elevation

Photo 3: North Elevation

Photo 4: Cracking to Block Work Near Roller Door

	
Photo 5: Close up of Photo 6	Photo 6: Minor Cracking to Block Wall Mortar Joints Near SE Corner

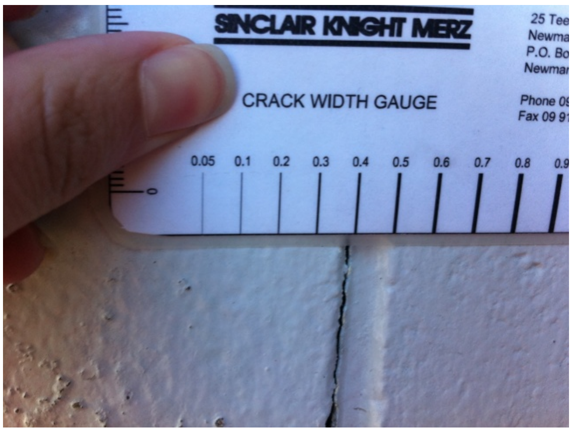



Photo 7: Close up of Photo 6	Photo 8: Minor Cracking to Block Wall Mortar Joints Near SW Corner

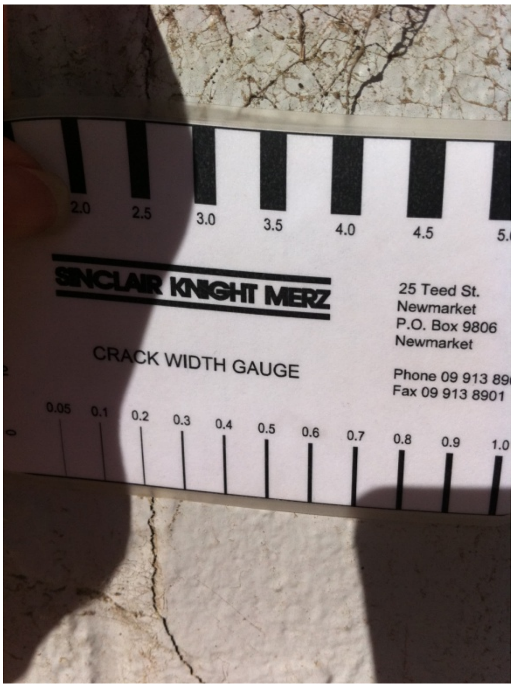


Photo 9: Close up of Photo 8	Photo 10: Spalling to Block Face in NW Corner
	
Photo 11: Internal View of East Stair	Photo 12: Minor Cracking to Block Wall Mortar Joints in SE Corner

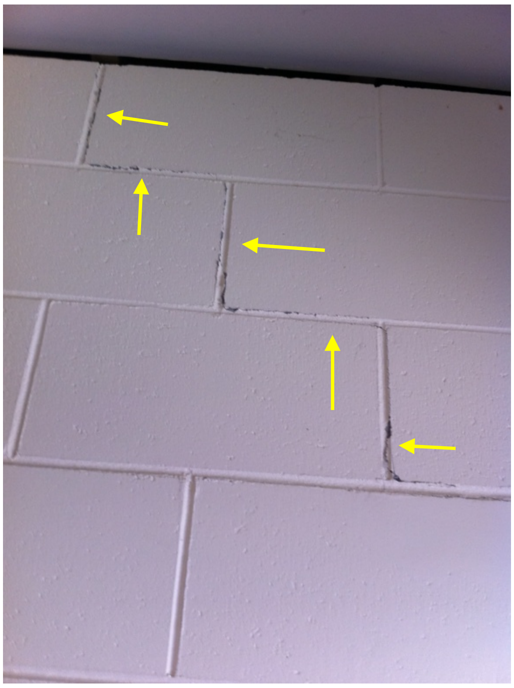


Photo 13: Minor Cracking to Block Wall Mortar Joints on East Wall of East Stair	Photo 14: Minor Cracking to Block Wall Mortar Joints in SE Corner of East Stair

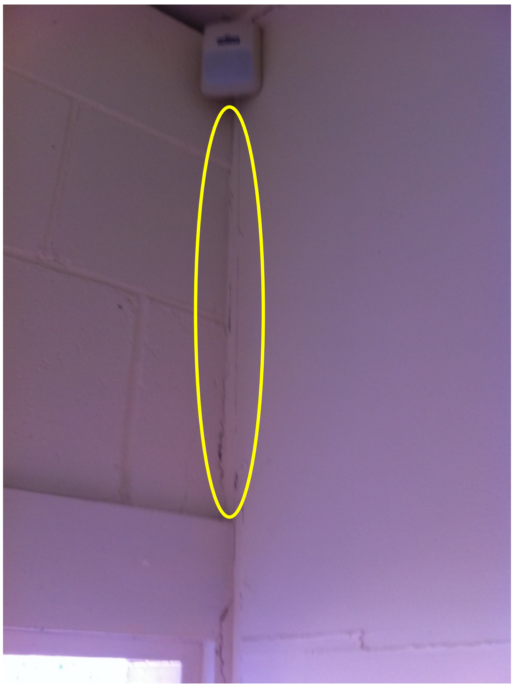
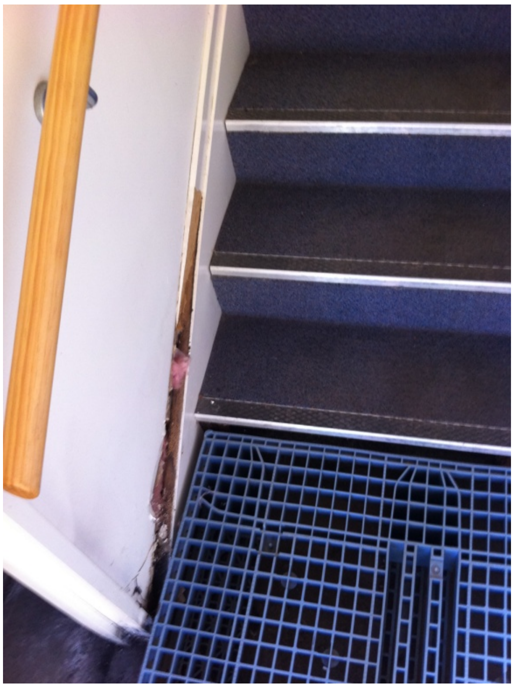
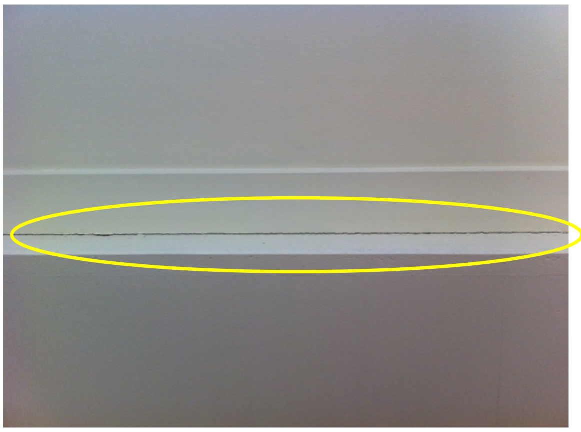



Photo 15: Hairline Cracking between Block and Timber Wall Joint in SW Corner of East Stair	Photo 16: Hairline Cracking at Wall Lining Joints on West Wall of East Stair

	
Photo 17: Close up of Photo	Photo 18: Damage to Timber Stair Skirting
	
Photo 19: Cracking along Timber Stair Skirting Joints	Photo 20: Damage to Timber Stair Skirting – East Wall

	
Photo 21: Cracking to Wall Linings at the Top of the East Stair	Photo 22: Hairline Cracking along the Block Wall and Timber Infill Wall Joint

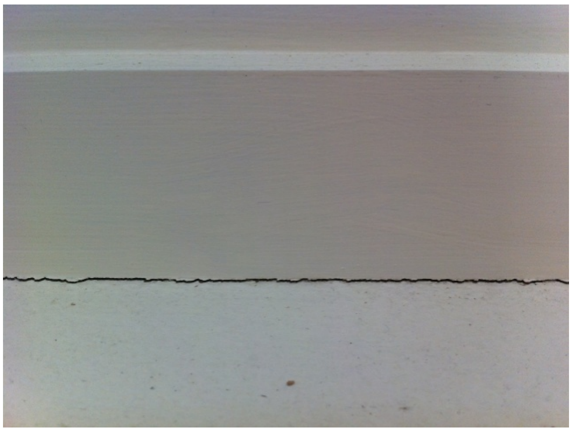



Photo 23: Close up of Photo 26	Photo 24: Horizontal Hairline Crack at Wall Lining Joint at the Top of the East Stair – West Wall

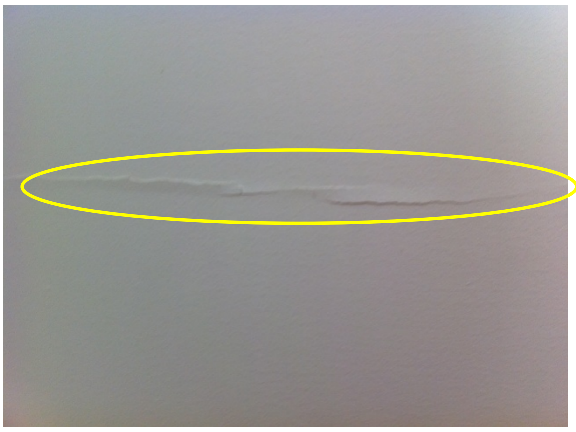


Photo 25: Close up of Photo 27

Photo 26: Internal View of Garage (ground floor)

Photo 27: Damage to East Wall of Garage – Not EQ Damage

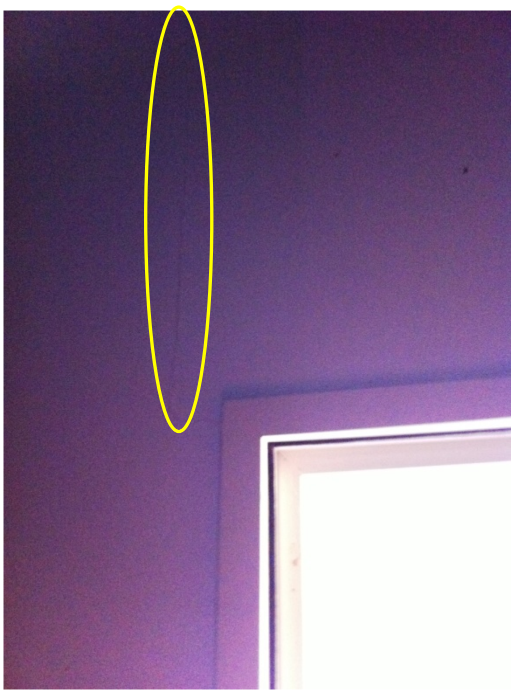
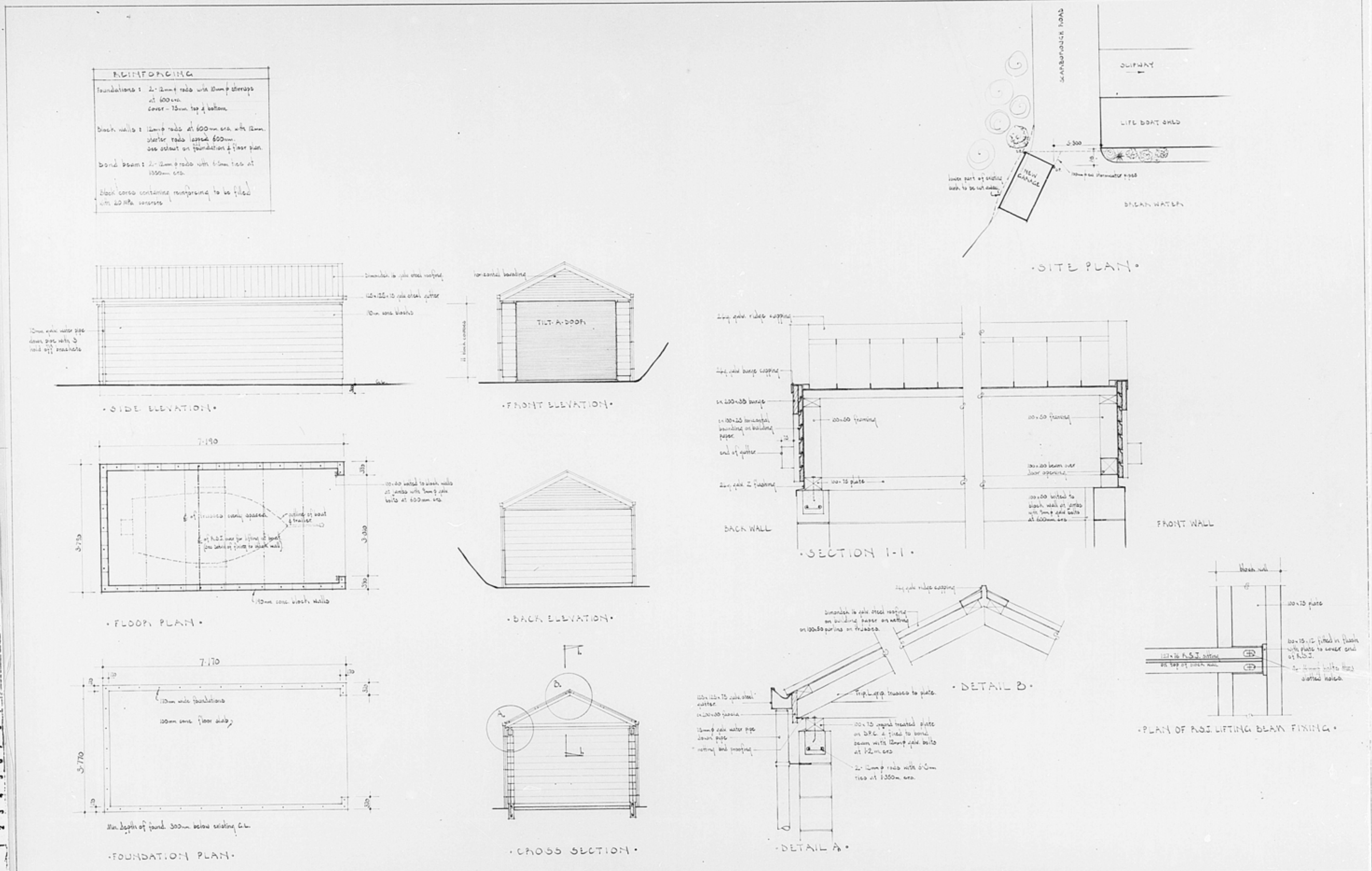


Photo 28: Vertical Hairline Crack above Personnel Door on the East Wall of Garage

	
Photo 29: Cracking to Wall Lining in the SE Corner of the Garage	Photo 30: Internal View of L1
	
Photo 31: Cracking Between Wall Lining and Ceiling Lining Joint	Photo 32: Hairline Cracking Between the Window Architrave and the Wall Lining Joint

12. Appendix 2 – Structural Drawings



CITY ARCHITECT'S DIVISION

CHRISTCHURCH CITY COUNCIL

CITY ENGINEER'S DEPARTMENT

CONSULTANT

NEW GARAGE FOR SUMNER LIFEBOAT INSTITUTION INCORP.

SCALES 1:50 1:100 1:200

DESIGN DSN G.M.L. DRN G.M.L. TCD G.M.L. CKD

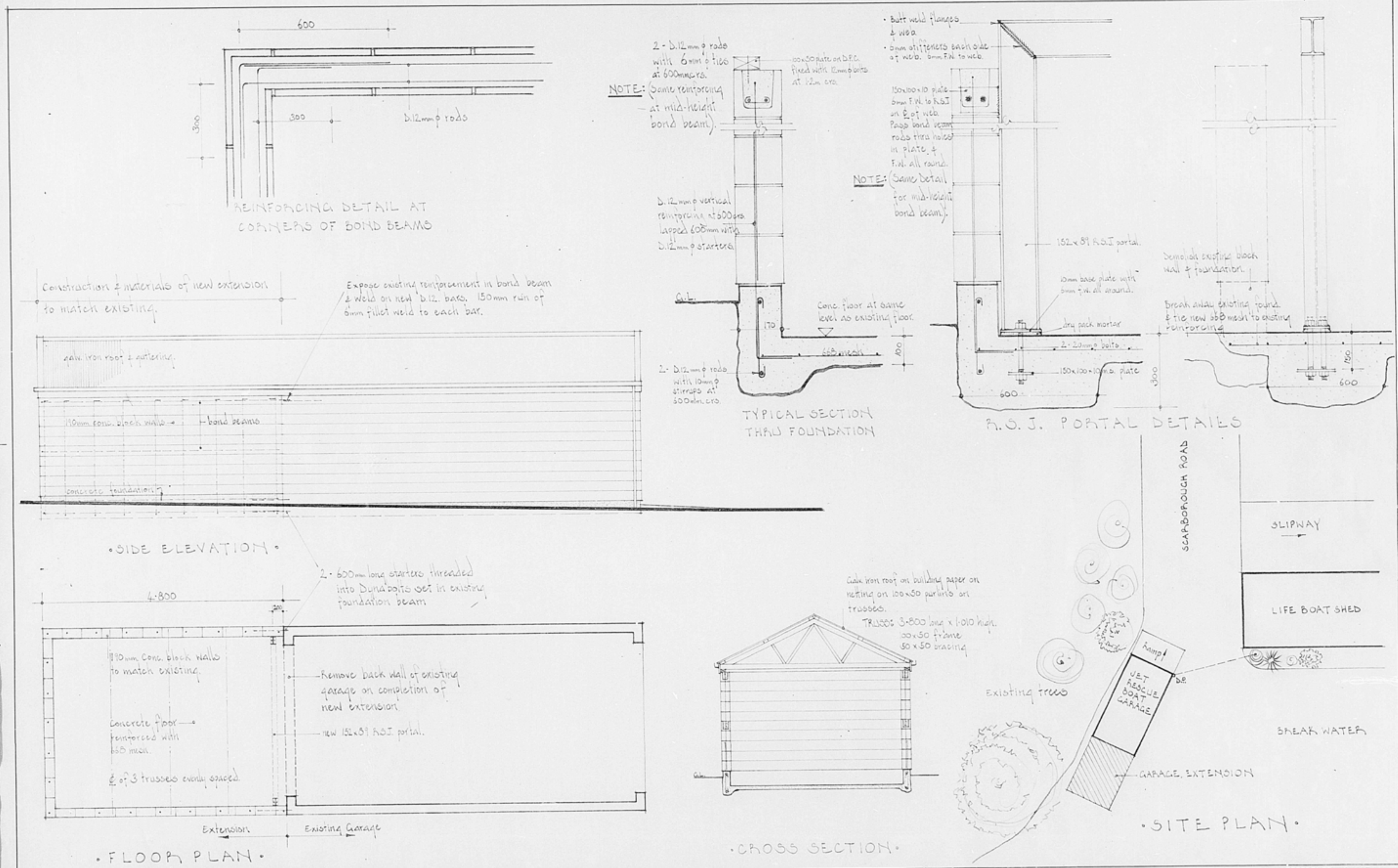
AMENDMENTS

SHEET TITLE PLANS, ELEVATIONS & DETAILS

REFERENCE JOB FILE 56/1/3 CONT

SHEET 1

DATE MARCH '76 CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK OF

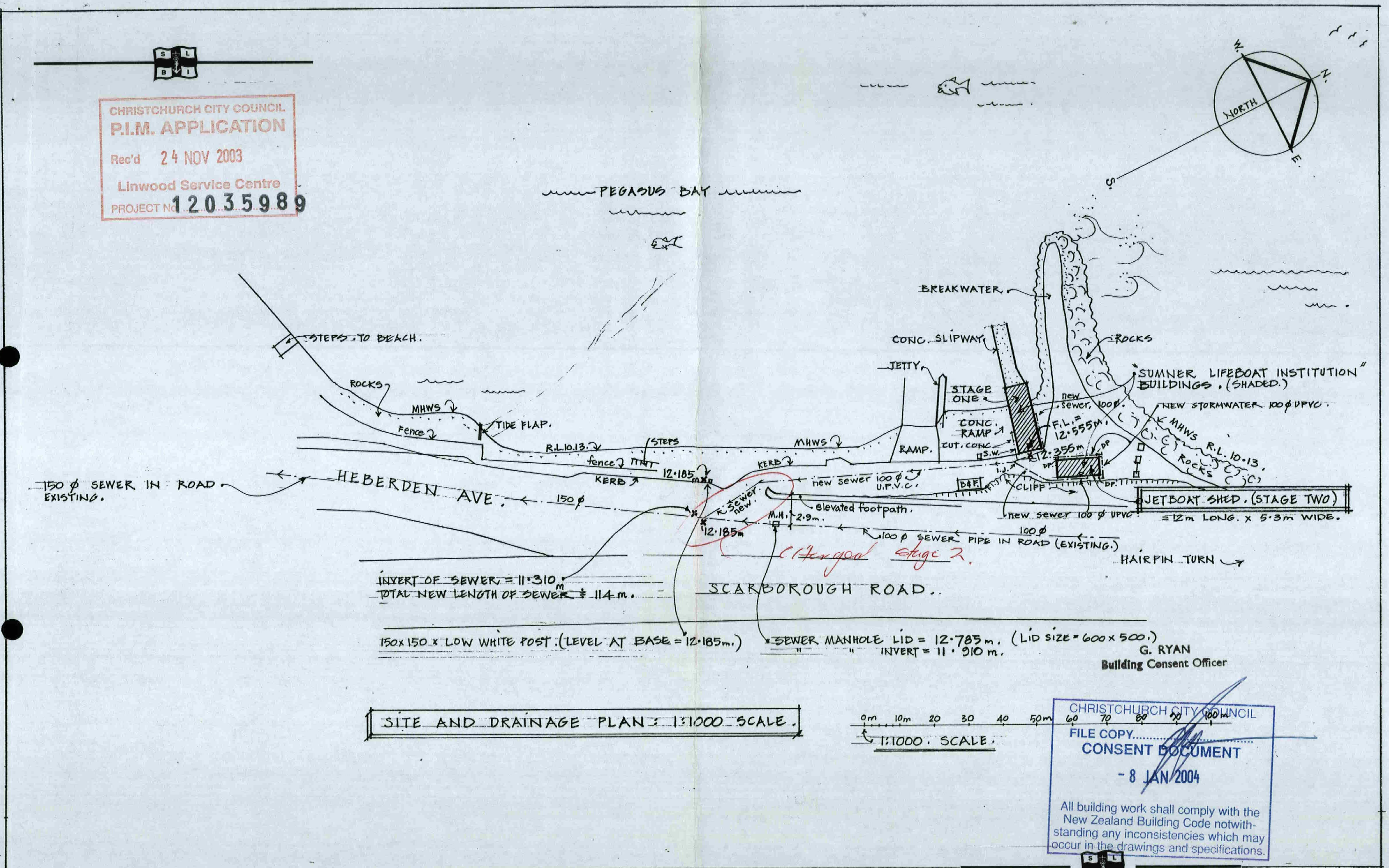


CITY ARCHITECT'S DIVISION CHRISTCHURCH CITY COUNCIL CITY ENGINEER'S DEPARTMENT	GARAGE EXTENSION for SUMNER LIFEBOAT INSTITUTION INC.	SCALES 1:50 1:100 1:200	DESIGN DSN DRN C.M.L. TCD CKD	SHEET TITLE PLANS, ELEVATIONS & DETAILS	REFERENCE JOB 4.14.4 FILE 62/2/3 CONT

SHEET 1 X

DATE 29-3-78 VERIFY ALL DIMENSIONS BEFORE STARTING WORK

13. Appendix 3 – Architectural Drawings



CHRISTCHURCH CITY COUNCIL
P.I.M. APPLICATION
Rec'd 24 NOV 2003
Linwood Service Centre
PROJECT No 12035989

G. RYAN
Building Consent Officer

CHRISTCHURCH CITY COUNCIL
FILE COPY
CONSENT DOCUMENT
- 8 JAN 2004
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

(EST. 1898.)

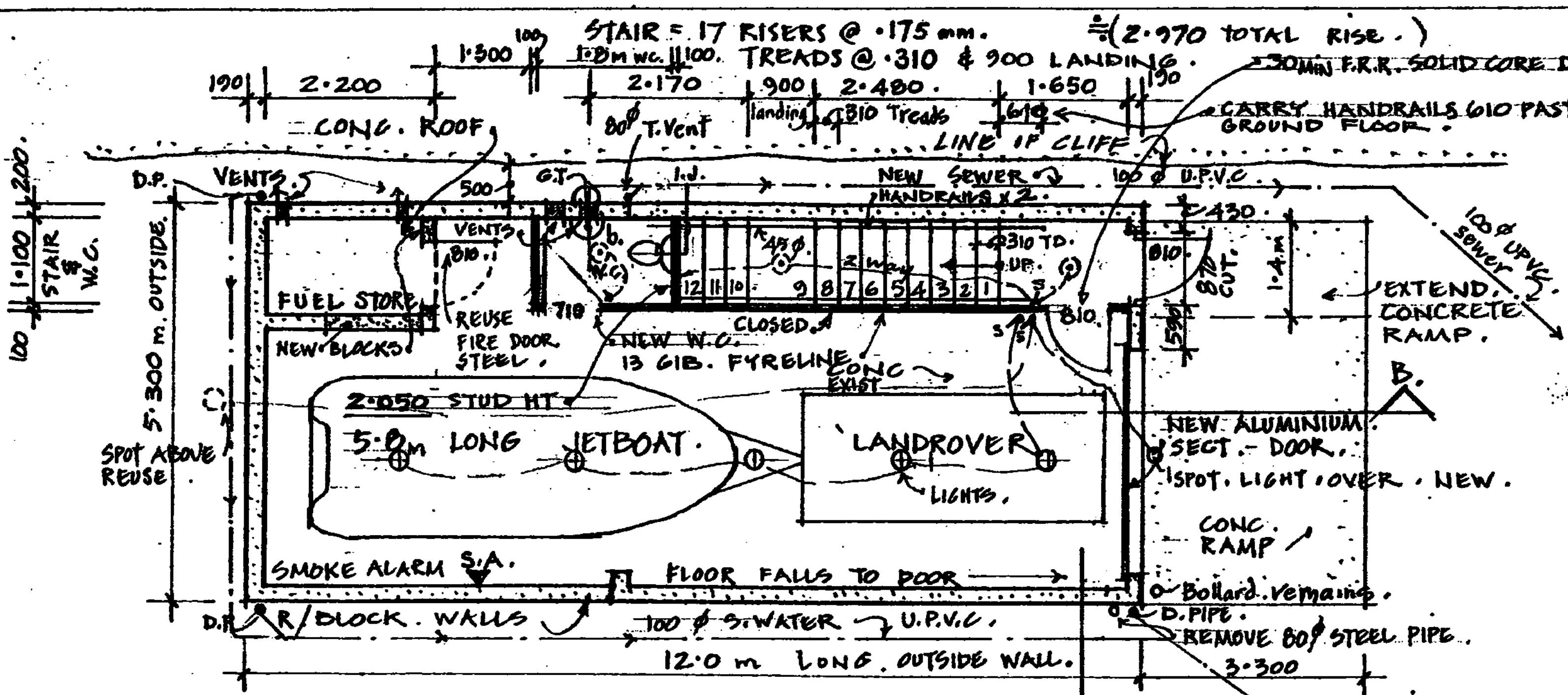
STAGE TWO 2003 : SUMNER LIFEBOAT INSTITUTION.

DAVID HOPPER. B. ARCH. 1977.
ARCHITECTURE PH. 3266032.
70 WIGGINS ST. SUMNER.

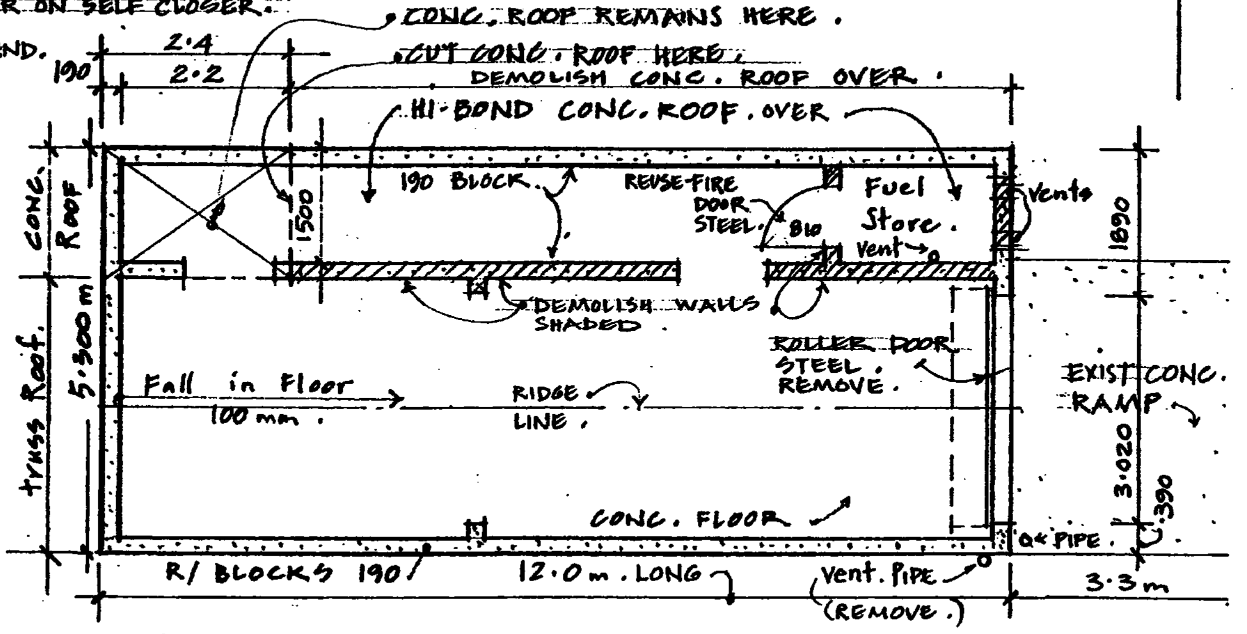
Sumner Lifeboat Institution (Inc.)
PO Box 17-515, Sumner
Christchurch 8008, New Zealand

NOV. 2003.

1



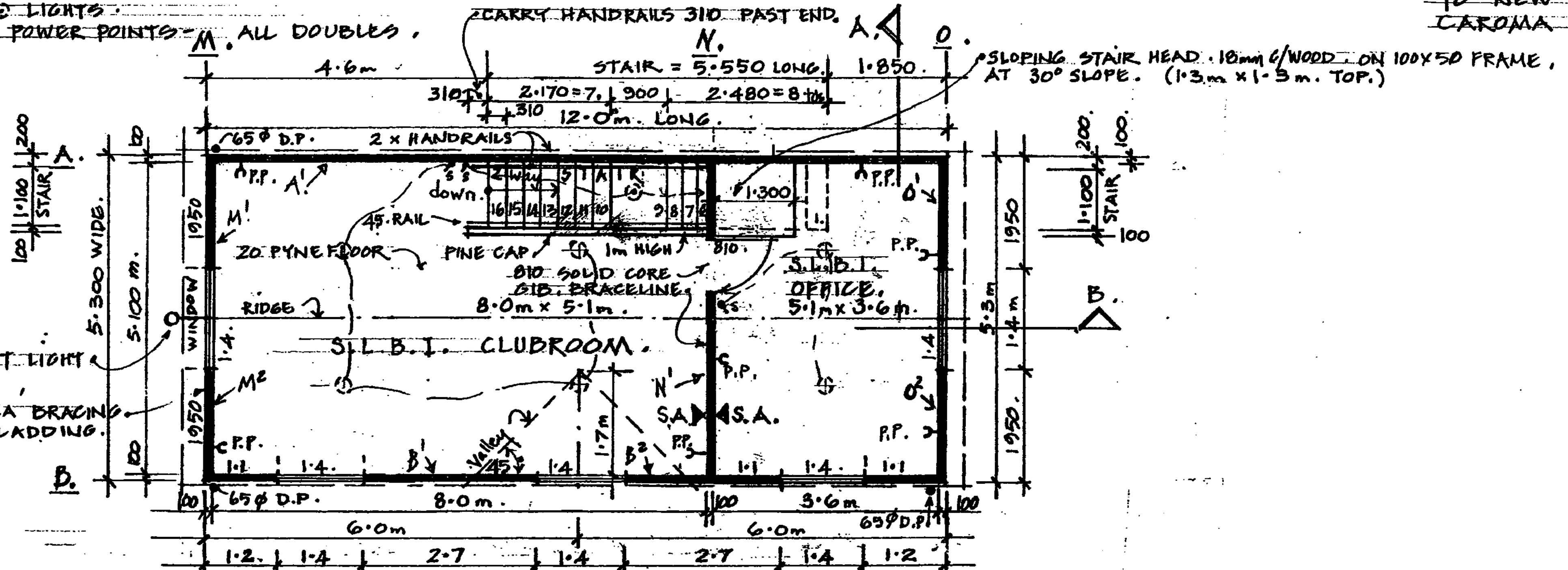
NEW GROUND FLOOR PLAN 1:100 scale.



EXISTING GROUND FLOOR PLAN 1:100.

ELECTRICAL: S.A.: SMOKE ALARM WITH HUSH BUTTON.
FIT ALL LIGHTS, POWER POINTS & SWITCHES AS SHOWN ON PLANS.
S.L.B.I. TO SUPPLY ALL LIGHT FITTINGS. WIRE TO EXIST. BOARD.
LIGHT SWITCHES.
LIGHTS.
POWER POINTS. ALL DOUBLES.

SPECS.
TOILET: NEW 'CAROMA COMPACT 225' WASH HAND BASIN WITH COLD TAP ONLY. (METHVEN BASIN TAP.)
BRING NEW COLD WATER SUPPLY FROM MAIN BUILDING TO NEW TOILET.
CAROMA TRIDENT - SOVEREIGN W.C. SUITE 'P' PAN.



A' = BRACING CODE. (SEE BRACING SHEET.)

P.D.L. POWER POINTS (ALL DOUBLES) NEW FIRST FLOOR PLAN 1:100 SCALE FLOOR AREA = 63·6 m².

AMENDED PLAN

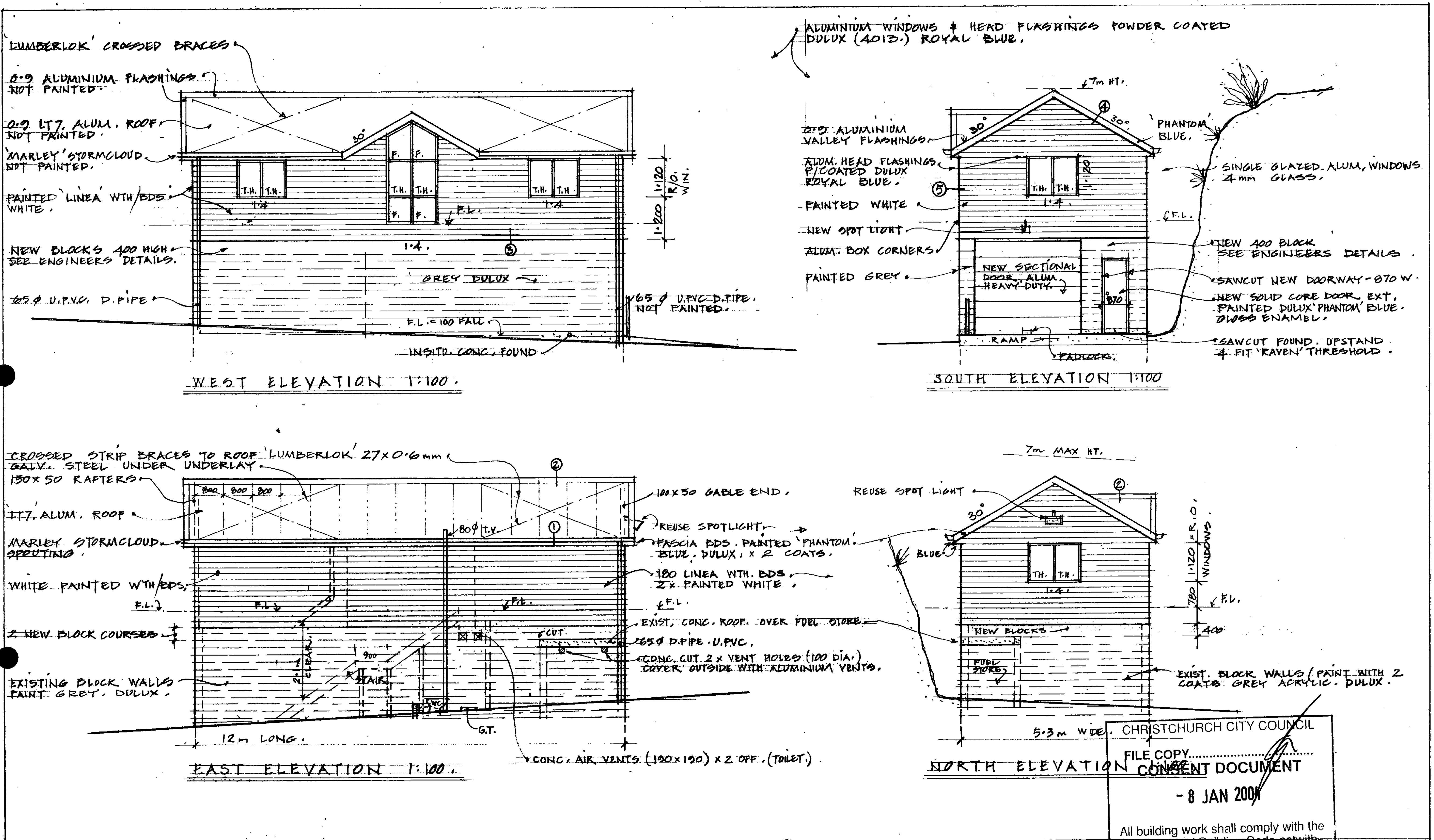
CHRISTCHURCH CITY COUNCIL
FILE COPY..........................
CONSENT DOCUMENT
- 8 JAN 2004
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

"ACCESSIBLE STAIR" - NZBCODE.
900 mm BETWEEN HANDRAILS.
* STAIR AMENDED DEC 17TH 2003
·310 TREADS & 1·100 m WIDE.

DAVID HOPPER ARCHITECTURE
B. ARCH. NZCD.
SUMNER, CHRISTCHURCH 8
PHONE 326-6032

ALTERATIONS & ADDITIONS TO JETBOAT SHED FOR SUMNER LIFEBOAT INSTITUTION.
DEC. - 2003.

2A



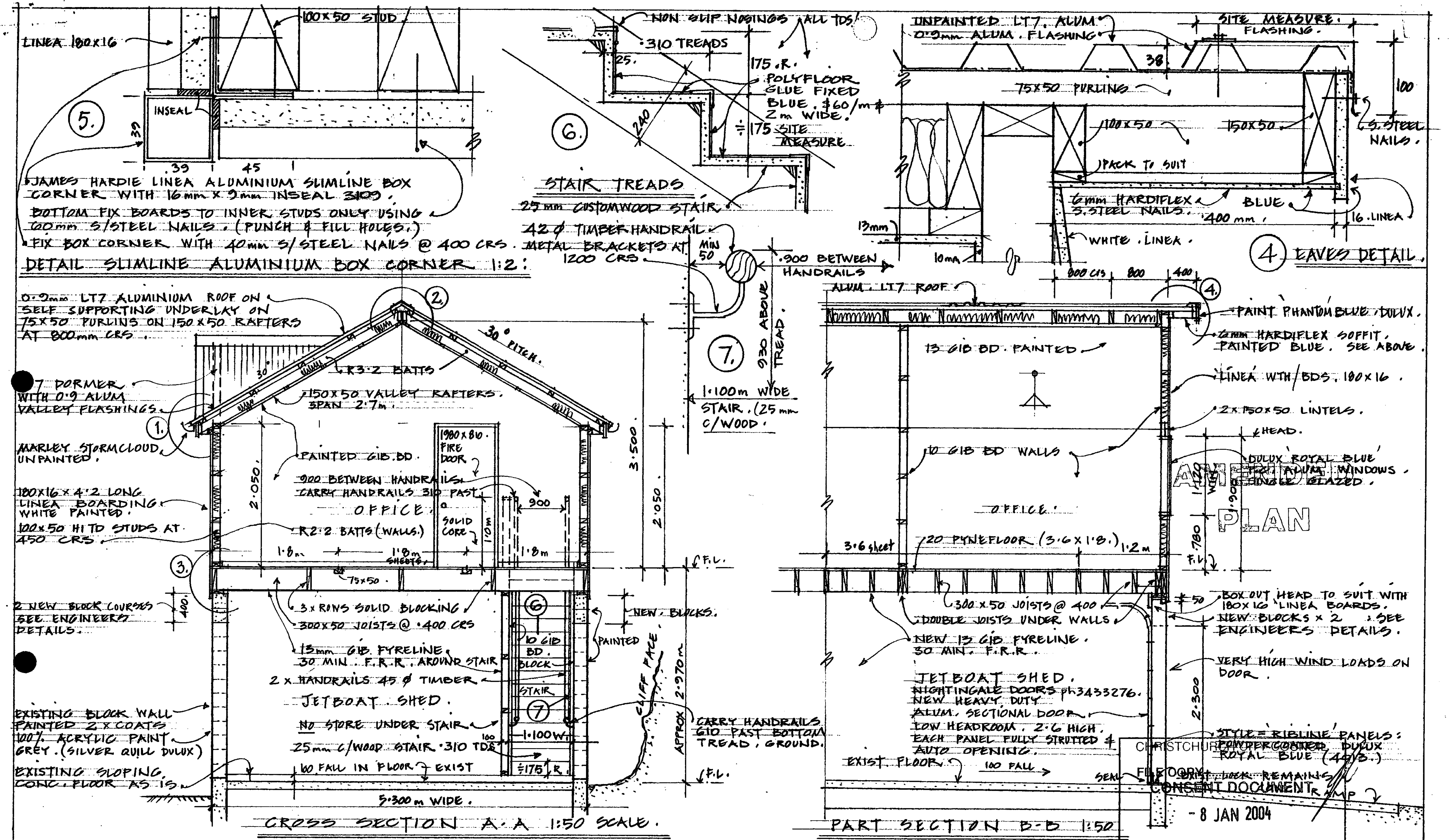
CHRISTCHURCH CITY COUNCIL
FILE COPY
CONSENT DOCUMENT
- 8 JAN 2004
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

DAVID HOPPER ARCHITECTURE
B. ARCH. NZCD.

SUMNER, CHRISTCHURCH 8
PHONE 326-6032

ALTERATIONS & ADDITIONS TO JETBOAT SHED FOR SUMNER LIFEBOAT INSTITUTION.
NOV. 2003.

3



All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the approved documents.

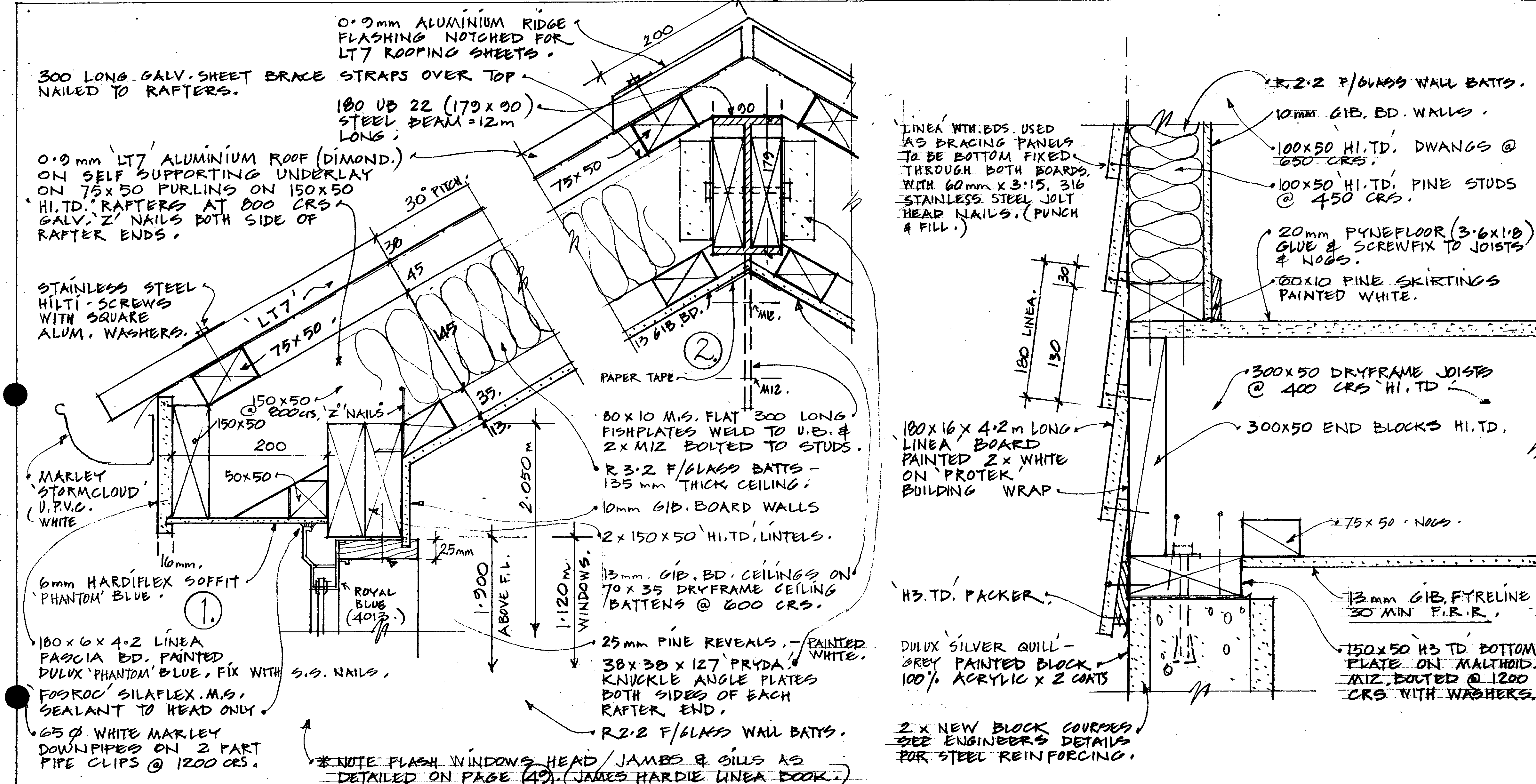
NOTE * STAIR AMENDED TO ACCESSIBLE STAIR (NZBC) 17TH DEC 2003.
1.100m WIDE STAIR. 900mm CLEAR BETWEEN HANDRAILS.
TREADS = 310 MIN & 175mm RISERS (MAX 180)

DAVID HOPPER ARCHITECTURE
B. ARCH. NZCD.

SUMNER, CHRISTCHURCH 8
PHONE 326-6032

ALTERATIONS & ADDITIONS TO JETBOAT SHED FOR SUMNER LIFEBOAT INSTITUTION.
17TH DEC. 2003.

4 A



(3) DETAILS 1:5 SCALE.

NOTE * PLEASE READ. NOTE * "VERY HIGH WIND ZONE." & "SEA SPRAY ZONE." (METAL CORROSION VERY HIGH !!)

BUILDER TO READ JAMES HARDIE BOOK "TECHNICAL INFORMATION ON LINEA WEATHERBOARD SYSTEMS" : ESPECIALLY PAGES 37, 38, 39, 41, 42, 44, 46, 47, 48, 49, 50, 51. ALL WORK TO COMPLY WITH NZS 3604. (1999.) AND "THE NEW ZEALAND BUILDING CODE."

* NOTE ALL EXTERIOR NAILS, BOLTS, SCREWS, HARDWARE, HINGES, FLASHINGS ETC. MUST BE NON FEROUS! WITH LONG 50 YEAR + LIFESPAN!

CHRISTCHURCH CITY COUNCIL
FILE COPY
CONSENT DOCUMENT
- 8 JAN 2004
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

DAVID HOPPER ARCHITECTURE
B. ARCH. NZCD.

ALTERATIONS & ADDITIONS TO JETBOAT SHED FOR SUMNER LIFEBOAT INSTITUTION
NOV. 2003.

5

SUMNER, CHRISTCHURCH 8
PHONE 326-6032

14. Appendix 4 – Geotechnical Desk Study

Sinclair Knight Merz
142 Sherborne Street
Saint Albans
PO Box 21011, Edgeware
Christchurch, New Zealand

Tel: +64 3 940 4900
Fax: +64 3 940 4901
Web: www.globalskm.com

Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	050 to 051
Address	2 Scarborough Beach
Report date	03 April 2012
Author	Ross Roberts / Ananth Balachandra / David Bae
Reviewer	Leah Bateman
Approved for issue	Yes

1. Introduction

This report outlines the geotechnical information that Sinclair Knight Merz (SKM) has been able to source from our database and other sources in relation to the property listed above. We understand that this information will be used as part of an initial qualitative Detailed Engineering Evaluation, and will be supplemented by more detailed information and investigations to allow detailed scoping of the repair or rebuild of the building.

2. Scope

This geotechnical desk top study incorporates information sourced from:

- Published geology
- Publically available borehole records
- Liquefaction records
- Aerial photography
- Council files
- A preliminary site walkover

3. Limitations

This report was prepared to address geotechnical issues relating to the specific site in accordance with the scope of works as defined in the contract between SKM and our Client. This report has been prepared on behalf of, and for the exclusive use of, our Client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and our Client. The findings presented in this report should not be applied to another site or another development within the same site without consulting SKM.

The assessment undertaken by SKM was limited to a desktop review of the data described in this report. SKM has not undertaken any subsurface investigations, measurement or testing of materials from the site. In preparing this report, SKM has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by our Client, and from other sources as described in the report. Except as otherwise stated in this report, SKM has not attempted to verify the accuracy or completeness of any such information.

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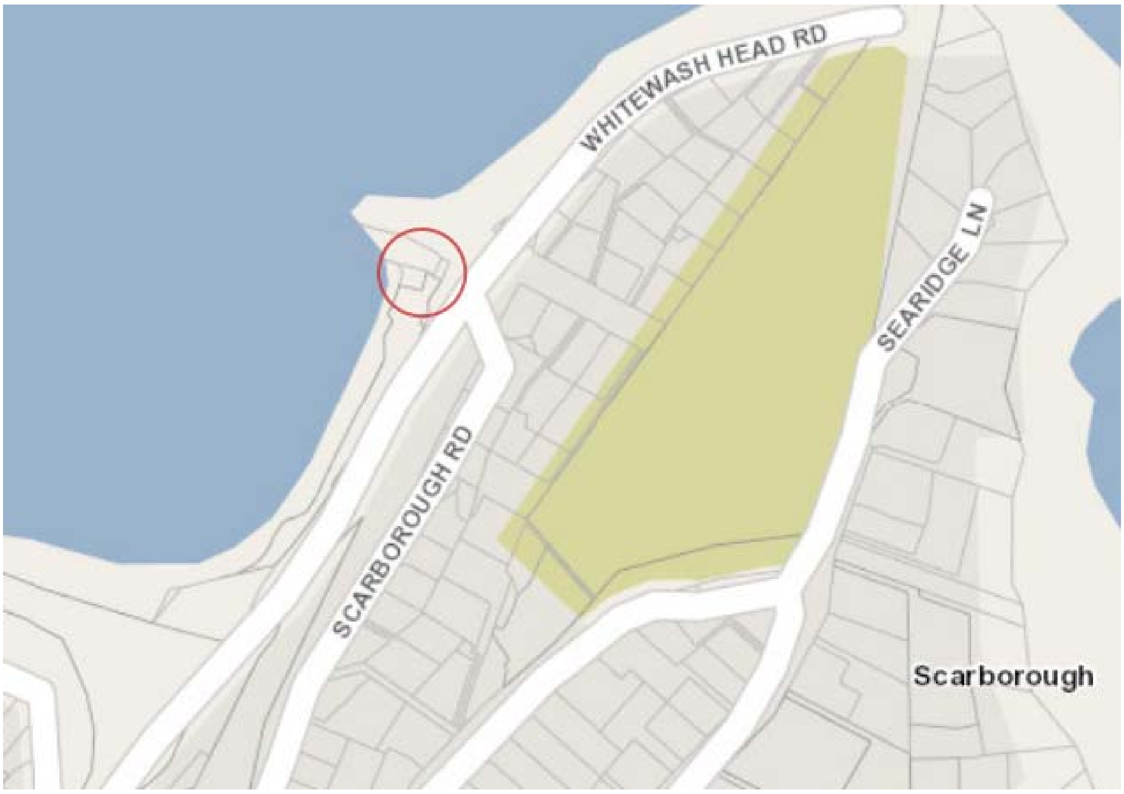
Offices across Australia, New Zealand, UK, South East Asia, Middle East, the Pacific and Americas

This report should be read in full and no excerpts are to be taken as representative of the findings. It must not be copied in parts, have parts removed, redrawn or otherwise altered without the written consent of SKM.

4. Site location

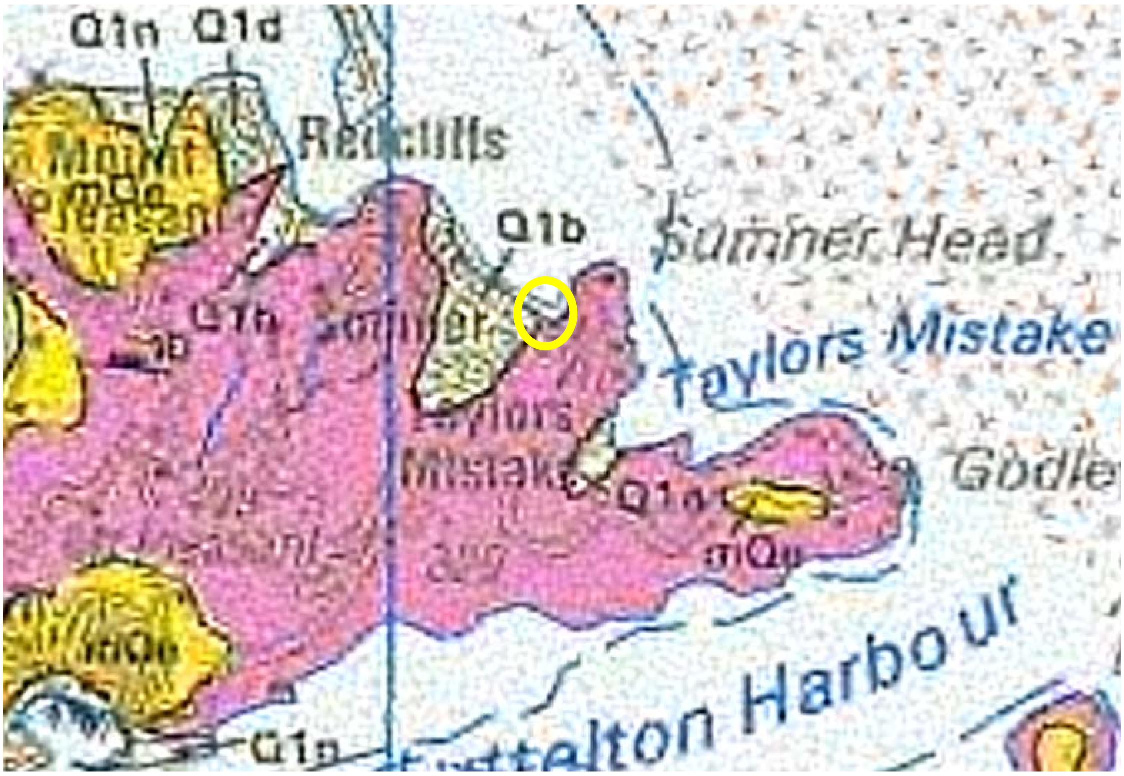


- **Figure 1 – Site location (courtesy of LINZ http://viewers.geospatial.govt.nz)**

These structures are located on 2 Scarborough Beach at grid reference 1581571 E, 5175723 N (NZTM).

5. Review of available information

5.1 Geological maps



- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in yellow.**

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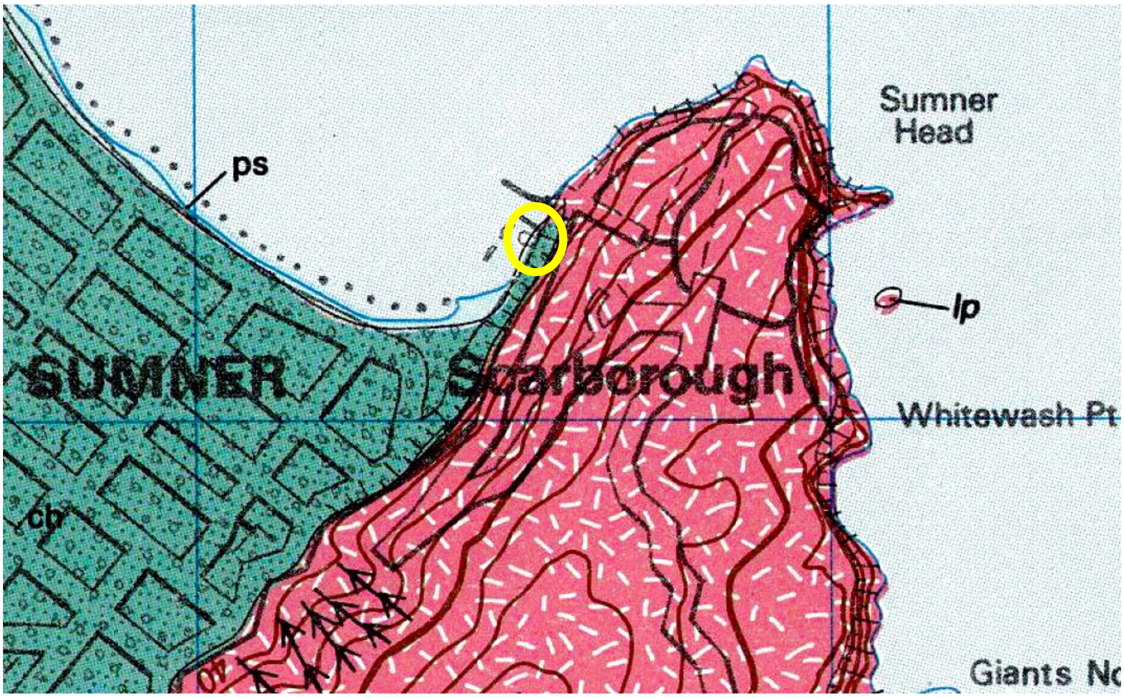

- **Figure 3 – Local geological map (Brown et al, 1992). Site marked in yellow.**

The site is shown to be underlain by Holocene deposits comprising dominantly sands of fixed and semi-fixed dunes and beaches from the Christchurch formation. Immediately to the north, east and south-east of the site, the respective areas are underlain by Miocene deposits comprising dark grey, plagioclase-pyroxene-amphibole, phyric hawaiite through to grey-green trachyte with interbedded red-brown pyroclastic deposits.

5.2 Liquefaction map

The reconnaissance undertaken by M Cubrinovsko and M Taylor of Canterbury University following the 22 February 2011 earthquake did not extend to this area.

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5.3 Aerial photography

- **Figure 4 – Aerial photography from 24 Feb 2011 (http://viewers.geospatial.govt.nz/)**

No damage to the structure, adjacent road and land can be observed from aerial photographs following the 22nd February 2011 earthquake event. Details of land damage are given in Section 5.8. No liquefied material is visible at the surface.

5.4 CERA classification

A review of the LINZ website (http://viewers.geospatial.govt.nz/) shows that the site is:

- Zone: No zone specified. Area surrounding the site is specified as green zone
- DBH Technical Category: On the boundary of N/A (Urban Non-residential) and N/A (Port Hills and Banks Peninsula)

5.5 Historical land use

No record of historical land use for this site was available.

5.6 Existing ground investigation data

No existing ground investigation data near the site was available.

5.7 Council property files

The available council records were limited to building consents applied for the alterations and additions to the existing lifeboat shed and other documents relating to the above construction. The council record identifies the site to be in a very high wind zone. The building is on reclaimed land and classified as being in the sea spray zone. No ground investigation was found in the council property files.

Drawings for the lifeboat shed show a slab-on-grade concrete foundation on the east side of the building. The main foundation consists of reinforced concrete piles with square section of 300 mm by 300 mm spaced along the wall (depth and spacing not recorded). Concrete piles of 600 mm diameter are provided as intermediate support for beams running through the centre of the building. Two shear walls, 1500 mm long by 200 mm wide (depth not recorded), are connected to the edge piles near the slipway. Drawings for the slipway show 600 mm diameter concrete piles were utilised with the existing 150 mm by 100 mm timber posts. The spacing between piles ranged from 1.6 m to 4.0 m.

5.8 Site walkover

A walkover inspection of the site was undertaken by a Geotechnical Engineer from SKM on 11 May 2012.

The site comprises two separate buildings located on the west side of Whitewash Head Road. The buildings are constructed using a combination of timber framed and masonry walls. One of the buildings, the lifeboat shed, is supported on combined slab-on-grade and concrete pile foundations. Damage noted from the external inspection consisted of minor cracking and spalling of the concrete floor slab and masonry wall.

The jet boat shed building has experienced pooling of water on the ground floor since the February 2011 earthquake. Excavation work is currently being undertaken on the east side of the building to prevent further flooding.

During the site walkover, there was no apparent evidence of surface expression of liquefaction. No signs of residual evidence were observed that would indicate liquefaction had occurred, such as undulating ground, ejected materials, settlement of buildings and raised manhole lids in road pavements. Minor cracks in the concrete kerb and pavement were observed during the inspection. Other evidence of land damage such as ground fissures or lateral spreading was not apparent on the site.

The slope behind the jet boat shed has been supported by Shotcrete (see Appendix B, Photos 1 and 2). The annotated aerial photo in Appendix C shows the location of the site walkover observations. The slope north and south of the jet boat shed is supported by dry stacked boulders. These appear to have performed satisfactorily with minimal movement adjacent to the boat shed, however, behind the historic boat shed to the south, the boulder wall collapsed during the 13 June event (see Appendix B, Photos 3 and 4).

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The road above at hairpin looks to have moved approximately 15mm (see Appendix B, Photos 5 and 6) northwest as shown by the crack in the footpath. This is likely to be related to movement of fill or damage of underground services and not an indication of large scale rock movement.

From site observations, there does not appear to be any stability issues related to the slope behind the jet boat shed.

6. Conclusions and recommendations

6.1 Site geology

No inference on the site geology has been made in this desk study due to lack of geotechnical investigation data.

6.2 Seismic site subsoil class

Due to a lack of geotechnical information, seismic class D should be conservatively used.

As described in NZS1170, the preferred site classification method is from site periods based on four times the shear wave travel time through material from the surface to the underlying rock. The next preferred methods are from borelogs including measurement of geotechnical properties or by evaluation of site periods from Nakamura ratios or from recorded earthquake motions. Lacking this information, classification may be based on boreholes with descriptors but no geotechnical measurements. The least preferred method is from surface geology and estimates of the depth to underlying rock.

In this case with a distinct lack of intrusive investigation the least preferred method has been used. The seismic class may be changed following geotechnical investigation.

6.3 Building Performance

Although detailed records of the existing foundations are not available, the performance to date suggests that they are adequate for their current purpose.

6.4 Ground performance and properties

No liquefaction appears to have occurred due to the 22nd February earthquakes. However, further site specific investigations would be needed to make a full liquefaction assessment of the site.

Due to the lack of geotechnical investigation data, an estimation of the ground properties for this site has not been made in this desk study.

6.5 Further investigations

Due to a lack of existing information for the purpose of carrying out a Quantitative DEE then a conservative seismic class should be used. There was no liquefaction observed at the site following the February or June 2011 earthquakes, therefore we can assume that liquefaction risk is insignificant at this site.

If design works need to be carried out or a building consent obtained then a geotechnical investigation will be required as there is considerable uncertainty of where the geological boundary between the sand dunes and volcanic material occurs.. Therefore, further investigations recommended are:

- Two boreholes to a minimum depth of 20m or into 3m of competent rock

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- Two dynamic cone penetration tests to estimate the properties of the surface soil if appropriate.

7. References

Brown LJ, Weeber JH, 1992. Geology of the Christchurch urban area. Scale 1:25,000. Institute of Geological & Nuclear Sciences geological map 1.

Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

Forsyth PJ, Barrell DJA, Jongens R, 2008. Geology of the Christchurch area. Institute of Geological & Nuclear Sciences geological map 16.

Land Information New Zealand (LINZ) geospatial viewer (http://viewers.geospatial.govt.nz/)

EQC Project Orbit geotechnical viewer (https://canterburyrecovery.projectorbit.com/)

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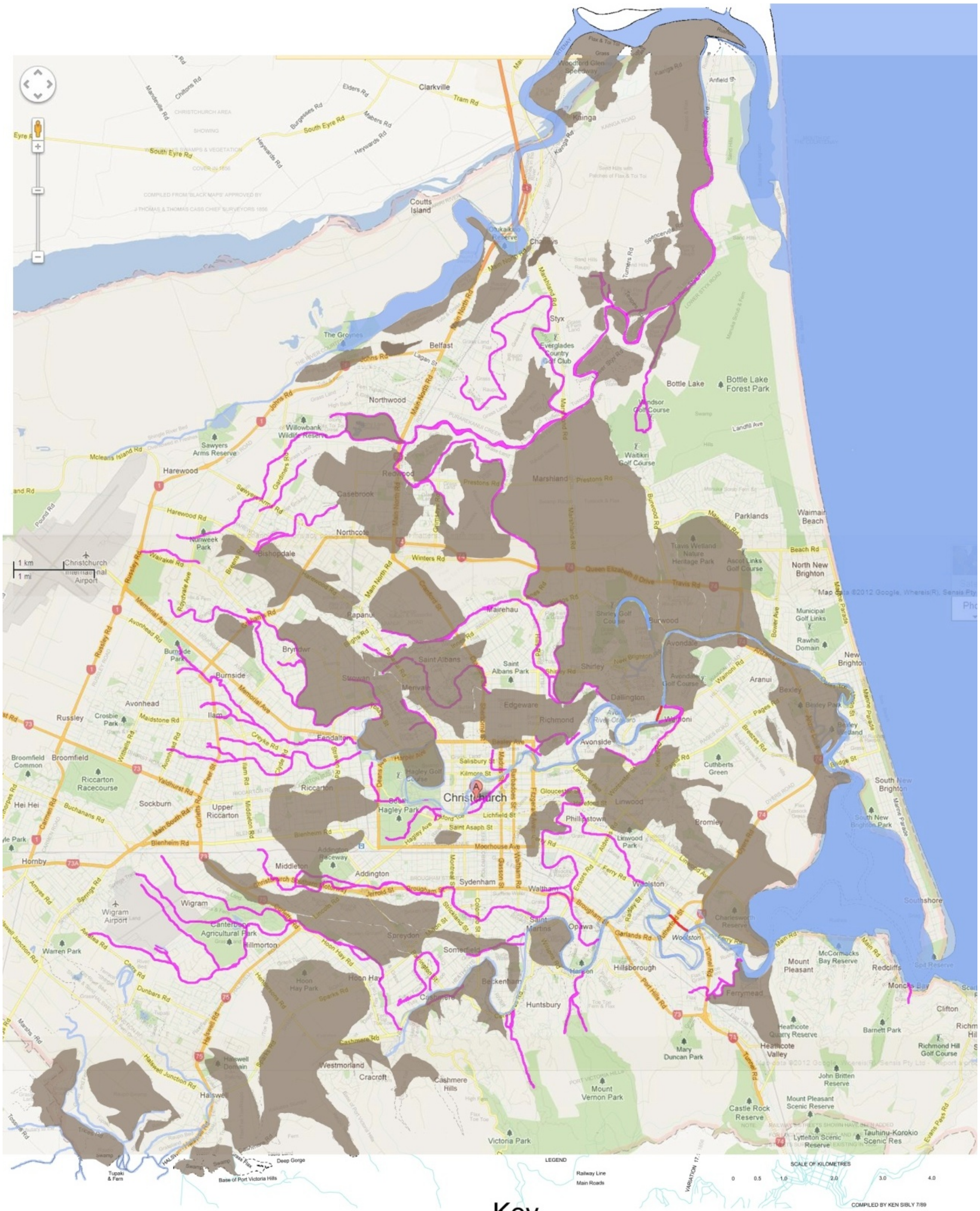
Appendix A – Christchurch 1856 land use



Key
- Previous creeks/rivers
- Existing creeks/rivers
- New creeks/rivers
- Swamp/Marshland

The swamps and previous creeks/rivers from 1856 have been overlayed onto a map of Christchurch in 2012

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Appendix B – Site Photos

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Photo 1 – Shotcrete wall behind jet boat shed.

Photo 2 – Shotcrete wall behind jet boat shed

Photo 3 – Sand bags supporting dry stacked boulder wall that collapsed behind historic boat shed. Wall to right is still standing.

Photo 4 – Rubble removed from behind historic boat shed.

Photo 5 – 15mm offset in footpath due to movement of the cantilevered road section.

Photo 6 – Section of road which has moved approximately 15mm (looking north).

Appendix C – Annotated Aerial Photo

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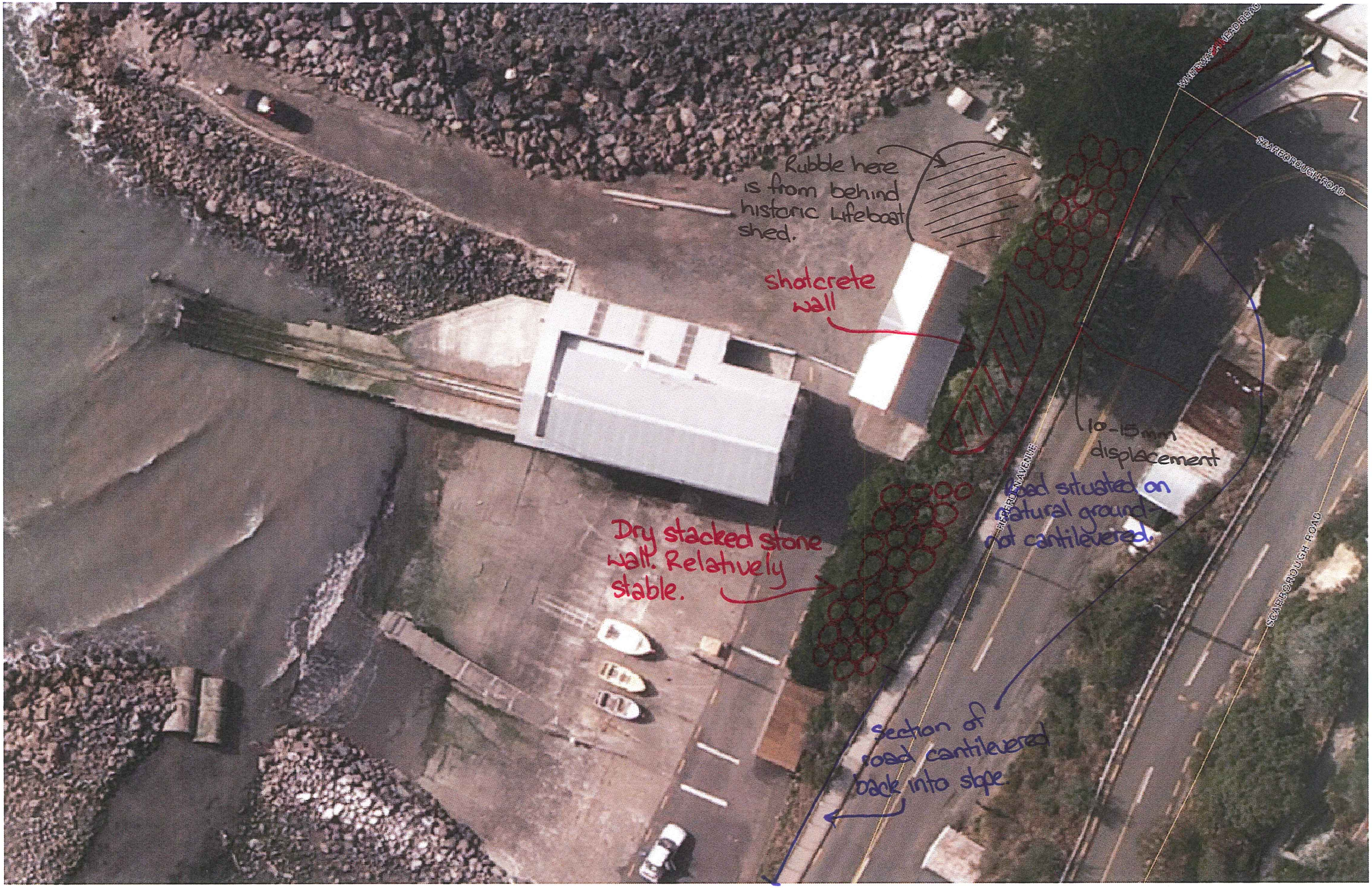
SCARBOROUGH ROAD
SCARBOROUGH ROAD
Rubble here is from behind historic Lifeboat shed.
shotcrete wall
10-15mm displacement
Road situated on natural ground - not cantilevered.
Dry stacked stone wall. Relatively stable.
section of road cantilevered back into slope.