Detailed Engineering Evaluation

Quantitative Assessment Report
THE ROSE CHAPEL – 866 COLOMBO STREET

The Rose Chapel - Detailed Engineering Evaluation

Quantitative Assessment Report

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Contents

Executive Summary

Christchurch City Council (CCC) appointed Opus International Consultants (Opus) to carry out a detailed seismic assessment of the Rose Chapel, 866 Colombo Street, Christchurch. The key outcome of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards. Opus were also asked to provide conceptual strengthening options to improve the building's seismic performance, with a target of meeting at least 67% of the new building standard (%NBS).

Findings of the assessment are:

- An analysis of the building based on the available information prior to the earthquake was carried out. The building was found to have a capacity of approximately 20%NBS (New Building Standard).

- Following the Feb 2011 earthquake the Rose Chapel sustained severe amounts of earthquake damage. An assessment of the current capacity of the building has been carried out to determine the extents of the repair/strengthening scheme. The capacity of the current building was found to be between 20-40%NBS

- To determine an accurate seismic capacity of the building, and considering its historic importance we strongly recommend that material testing is carried out. This will facilitate two actions:

  1. Strengthening and repair scheme that will minimise impact on the existing fabric of the building.

  2. Determine an accurate %NBS for the strengthening scheme and the existing structure.

- Once the material testing is carried we will be able to carry out a more accurate analysis to determine precise capacities of the building and enable a efficient strengthening scheme to be developed

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of The Rose Chapel, located at 866 Colombo Street, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the building is classed as being Earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

2 Background Information

2.1 Building Description

The Rose Chapel was opened in 1911, it is a single storey masonry structure located on the edge of the red zone in Christchurch CBD, see Figure 2. It is classified as Category II building by the New Zealand Historic Places Trust.

The building is constructed from approximately 600mm thick walls with an inner Wythe consisting of brick and the facade made up of Oamaru and blue stone. The cavity between wythes is filled with no fines concrete. The roof is composed of principal trusses, supporting purlins and rafters clad with slate tiles. An ornate tiled floor, finishes the ground bearing concrete slab. In the late 1990's the building was seismically strengthened. The strengthening included a number of modifications which were, but not limited to creating a roof diaphragm, concrete beam to the head of the north and south walls, concrete bond beam to the nave area and tying the gable walls into the roof structure.

2.2 General

Following the February earthquake the chapel suffered severe seismic damage and was given a red placard by others. The building served as a chapel for weddings and ceremonies however, it is now un-occupied. Currently, the building has been stabilised to facilitate access for contractors and engineers.

Photograph 1 - Before February 2011 earthquake

Photograph 2 - After February 2011 earthquake

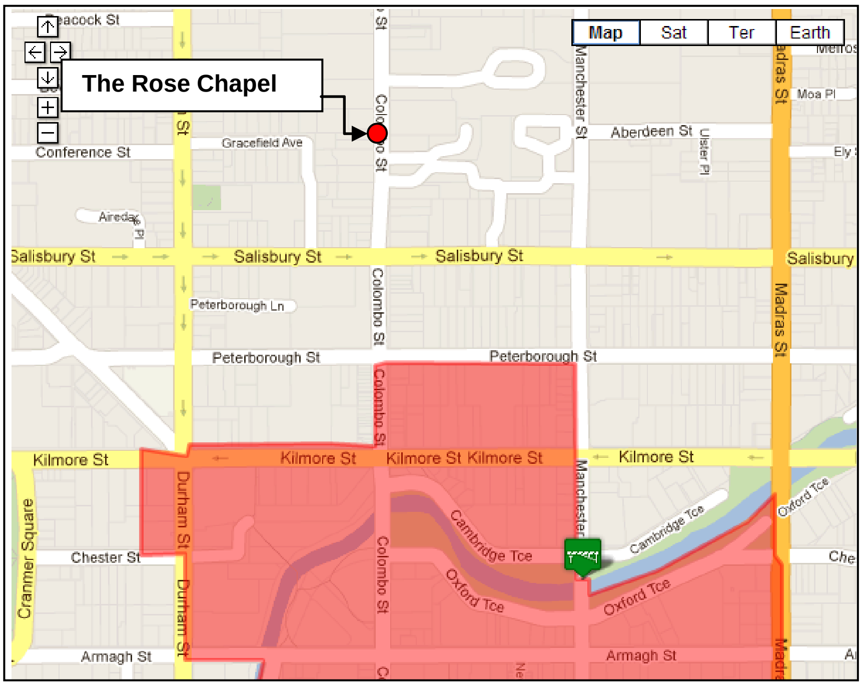

Figure 1 – Location of Rose Chapel

Opus has carried out an overall damage assessment of the church following the December 2011 earthquake. Some reference to non-structural damage has been mentioned however, this is not extensive, and does not represent a full condition report of non-structural items.

2.3 CBD Red Zone Cordon

Following the Lyttelton Earthquake of 22 February 2011, the central business district (CBD) suffered major damage to a large proportion of its building stock and so a central area of the city was cordoned off and closed to the public, forming what is known as the red zone. Some outskirts of the red zone cordon have now been lifted and The Rose Chapel is currently on the perimeter of the red zone. The red zone extent, as of 18th May 2012, is displayed below in Figure 2.

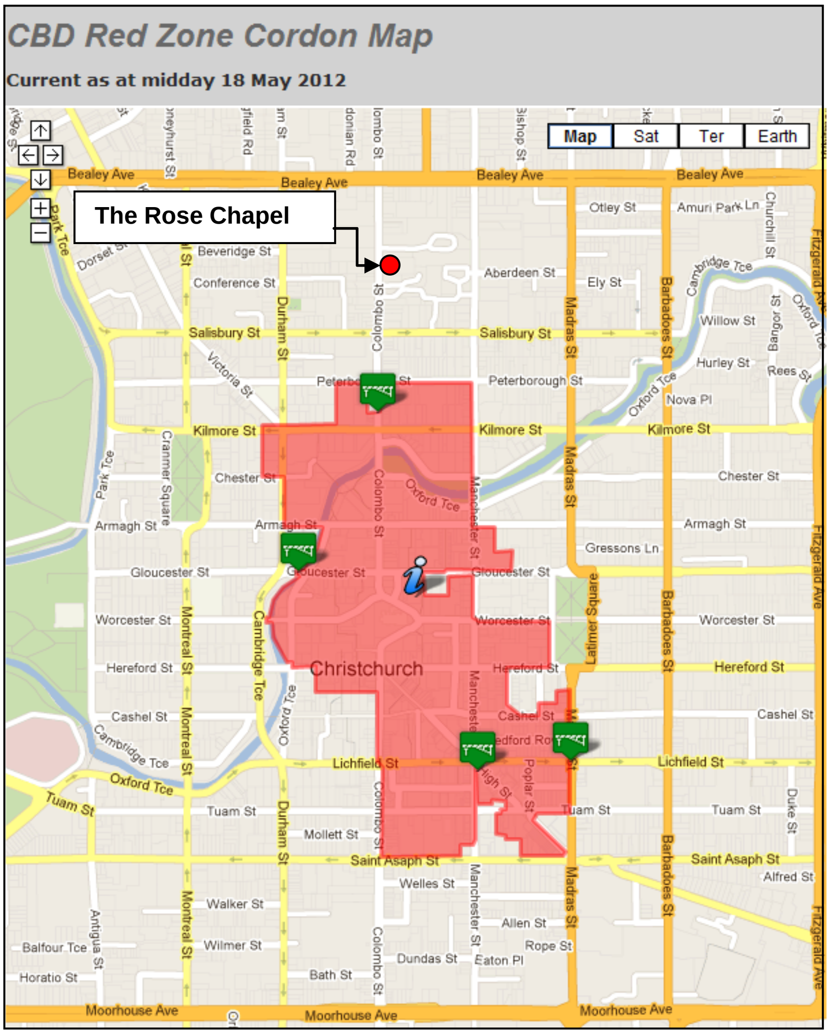

Figure 2 – Current CBD Red Zone Cordon current

as midday 18th May 2012

2.4 Survey

2.4.1 Post 22 February 2011

Following the February Earthquake a survey was carried out by others and the building was given a red placard. Opus carried out a damage assessment on the 23rd March 2012 our observations are recorded in the damage assessment report found in Appendix 2.

2.5 Original Documentation

Copies of the following construction drawings were provided by Insight Unlimited on March 2012:

- Holmes Consulting Seismic Strengthening Scheme dated February 1998
- Skews Hey Ussher Architects, Architectural drawings of the proposed strengthening scheme April 1997

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

The original Structural and Architectural drawings were not located. No original design calculation or specifications have been provided to assist the assessment of the existing building.

3 Structural Damage

A damage assessment report has been carried out to identify the extent of the damage on the Rose Chapel which is attached in Appendix 2.

4 General Observations

The building performed similar to other buildings of this construction and age. In particular, the failure mechanism of the gable end walls was common for this type of seismic retrofit. The gable end restrained by resin anchors into the roof structure is likely to have failed due to the connections pulling out of the gable end wall and/or failure of the perimeter roof truss. The overall structure of the south and north walls has fared well, with minimal structural damage to the walls, and a reasonable amount of residual seismic capacity.

It is likely from the observed damage that strengthening in the roof could not resist the applied seismic loads. This is evident from the visual damage to the timber trusses. In general, the connections between the purlins and trusses are showing signs distress and the connections can be observed to be pulling away from one another.

It is expected that the mezzanine floor would have stayed intact if the gable wall had not collapsed onto it.

5 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2012 guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

5.1 Qualitative Assessment Summary

An initial qualitative assessment of the buildings was undertaken in accordance with the DEEP guidelines and involves a desktop review of existing structural and geotechnical information, including existing drawings and calculations, and some non-intrusive site investigation, see Appendix 1 for Qualitative report. The purpose of the assessment was to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, to confirm the required scope of the Quantitative assessment, and to make an initial assessment of the likely building strength in terms of % NBS.

5.2 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building. During the initial qualitative stage of the assessment the potential CSW's were identified for each of the buildings and have been considered in the Qualitative analysis see Appendix 1 for Qualitative report.

7.3 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included below:

An equivalent static linear analysis has been carried in accordance with NZS1170.05 Structural Design Actions Code. This analysis used spectral values established by this code with an updated Hazard Factor of Z=0.3. The analysis was used to determine the applied actions on the existing structure. These results were used to determine the existing capacity of the structure.

The wall capacity of the Nave was determined following the NZSEE Detailed Assessment of Unreinforced Masonry Buildings 2012 guidelines. The existing capacities for the in and out-of-plane direction were compared with expected demand of current building code to provide a percentage NBS.

7.4 Review of Critical Structural Weaknesses

Most of the critical structural weaknesses identified in the qualitative assessment (see Appendix 1) will have an effect on the capacity of the building. These have been considered in the assessment Table 4.

7.5 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its current state.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Details of connection to determine dependable capacity and material composition of elements such as the no fines concrete filled walls.
- Assessments of material strengths based on limited drawings, specifications and site inspections
- The normal variation in material properties which change from batch to batch.

5.3 Seismic Coefficient Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170.5:2004
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B
- Return period factor R_u = 1.0 from table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life.

5.4 Expected Ductility Factor

Based on our assessment of the structural drawing, our initial estimates for the expected maximum structural ductility factors for the main seismic resisting systems are:

- μ_{max} = 1 for the un-reinforced masonry walls in both the east-west and north-south directions.

7.6 Assessment

A summary of the structural performance of the building is shown in the following tables. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements. This will be considered further when developing the strengthening options.

Table 4: Summary of Seismic Performance – Original Building, μ = 1.0

Structural Element/System	**Failure Mode, or description of limiting criteria based on displacement capacity of critical element.**	**Critical Structural Weakness and Collapse Hazard**	**% NBS based on calculated capacity**
Nave walls – In plane	Potential for rocking failure within the walls. However this has not been a commonly seen failure mechanism.	No	100%
Nave walls – Out of plane	Flexural failure, there is evidence of this from observed high level horizontal cracking however, the existing resin anchors and high level concrete ring beam provide some additional restraint.	No	20-40%
Existing roof diaphragm *(Pre February 2011 Earthquake)*	Insufficient capacity to carry the longitudinal seismic loads to the nave walls. Assuming it carries gable walls	No	10-20%
Existing roof diaphragm *(Post February 2011 Earthquake)*	due to absence of gable wall loads and in current structural state	No	100%
Existing roof diaphragm connection to nave walls *(Pre February 2011 Earthquake)*	Shear failure of the connections between the roof diaphragm and the nave walls.	Yes	33%
Existing roof diaphragm connection to nave walls *(Post February 2011 Earthquake)*	Shear failure of the connections between the roof diaphragm and the nave walls.	Yes	100%

6 Remedial options

The building requires some rebuild and strengthening, with a proposed seismic performance to meet at least 67%NBS. Our concept strengthening scheme to achieve this would include:

- Locally strengthen existing nave roof diaphragm at each end bay, to reduce the flexibility of the existing roof.
- Reliable connector beam for nave walls.
- Create roof diaphragms located to the east and west ends of the church.
- Re-build the collapsed gable walls from reinforced concrete with finishes to match existing whilst retaining as much as practically possible of the existing fabric.
- Shotcrete some of the internal faces of the existing walls.
- Strengthen nave walls for out of plane actions.

7 Conclusion

a) The seismic performance of the original building was governed by the existing nave roof diaphragm. The connection between the roof and the top of the gable wall is calculated to have had a capacity of 10-20% NBS. These elements failed, resulting in the collapse of the gable walls during the February Earthquake. The building in its original form is considered to be earthquake prone in accordance with the Building Act 2004.

b) The assessed current capacity of the building post February 2011 is 20-40% NBS, which is governed by the out of plane capacity of the nave walls.

c) The performance of the building is governed by the flexibility of the main nave roof diaphragm and its ability to transfer loads to the nave walls.

d) An assessment of the nave walls has been carried out however; this has been based on no material testing and computer modelling. we suggest that material testing is carried out to obtain more precise material properties thus reducing un-certainties in the analysis.

8 Recommendations

a) Material testing should be undertaken to provide detailed information for the material properties. This would enable a more thorough examination of the masonry walls to be carried out and allow an accurate value of %NBS to be determined.

b) Computational analysis of the nave walls using actual material properties, may show that the capacity is higher than the present calculations, which would reduce the scope of the strengthening works required.

c) A strengthening works scheme be developed to increase the seismic capacity of the building to at least 67% NBS, this will need to consider compliance with accessibility and fire requirements. Moreover, be sympathetic to the historical characteristics of the existing structure.

d) A quantity surveyor is engaged to determine the costs for strengthening the building

9 Limitations

a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

b) 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.

c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

10 References

[1] NZS 1170.5: 2004, *Structural design actions, Part 5 Earthquake actions,* Standards New Zealand.

[2] NZSEE: 2012, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix 1

QUALITATIVE REPORT

The Rose chapel

Detailed Engineering Evaluation

Stage One Qualitative Report

Christchurch City Council

Christchurch City Council

The Rose Chapel

Detailed Engineering Evaluation Qualitative Report

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Executive Summary

Christchurch City Council (CCC) appointed Opus International Consultants to carry out a detailed seismic assessment of the Rose Chapel, Christchurch. The key outcome of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards. Opus were also asked to provide conceptual strengthening options to improve the building's seismic performance, with a target of meeting at least 67% of the new building standard (%NBS).

Findings of the assessment are:

(a) a number of Critical Structural Weaknesses and structural deficiencies have been identified.

(b) the overall building is deemed to be earthquake prone due to the critical structural weakness identified

(c) the remaining structure has reasonable residual capacity against seismic forces.

(d) conceptual strengthening scheme to bring the building up to 67% and 100% NBS has been developed.

Our recommendations are:

- a quantitative analysis is undertaken in order to confirm the seismic capacity of the building, taking into account the identified potential critical structural weaknesses.
- Repair and strengthening scheme be developed to repair damage and increase seismic capacity to not less than 67% NBS.

1 Background

Opus International Consultants Limited (Opus) has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Rose Chapel, located at 866 Colombo Street, Christchurch.

This report is a Stage One qualitative assessment of the building structure, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

A qualitative assessment involves a desktop review of existing structural and geotechnical information, including existing drawings and calculations, and undertaking some non-intrusive and intrusive site investigation. The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (% NBS).

At the time of this report, no intrusive site investigation, and detailed analysis or modelling of the building structure have been carried out.

2 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.

2.1 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 to 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 (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

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

2.2 Building Act

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

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 the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) 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 CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

2.2.1 Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

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 loads 33% of those used to design an equivalent new building.

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.

Section 131 – Earthquake Prone Building Policy

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

2.3 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 on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

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.

2.4 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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

3 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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented 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
Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

Figure 1: NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exeedance 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.

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

Table 1: %NBS compared to relative risk of failure

4 Building Description

4.1 General

The Rose Chapel was opened in 1911, it is a single storey masonry structure located on the edge of the red zone in Christchurch CBD, see Figure 2. It is classified as Category II building by the New Zealand Historic Places Trust.

Following the February earthquake the chapel suffered severe seismic damage and was given a red sticker. The building served as a chapel for weddings and ceremonies however, it is now un-occupied. Currently, the building has been stabilised to facilitate access for contractors and engineers.

Photograph 1 - Before February 2011 earthquake

Photograph 2 - After February 2011 earthquake

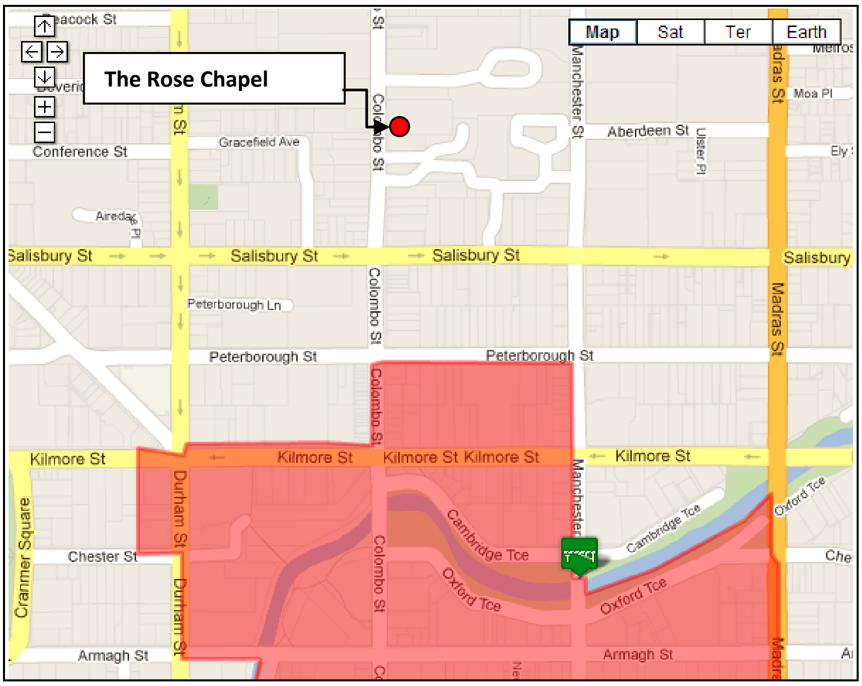


Figure 2 – Location of Rose Chapel

The building is constructed from approximately 450mm thick walls with an inner wythe consisting of brick and the facade made up of Oamaru and blue stone. The cavity between wythes is concrete filled. The roof is composed of principal trusses, supporting purlins and rafters clad with slate tiles. In the late 1990's the building was seismically strengthened. The strengthening included a number of modifications including but not limited to creating a roof diaphragm, Concrete beam to the head of the north and south walls, concrete bond beam to the nave area and tying the gable walls into the roof structure.

Opus has carried out an overall damage assessment of the church following the December 2011 earthquake. Some reference to non structural damage has been mentioned however, this is not extensive and does not represent a full condition report of non structural items.

4.2 Gravity Load Resisting System

The gravity loads from the roof are transmitted in the timber wall plates through the principal trusses to the reinforced concrete beam located at the top of the nave walls. Where the concrete beam is not installed the gravity loads are transmitted to the existing timber wall plates.

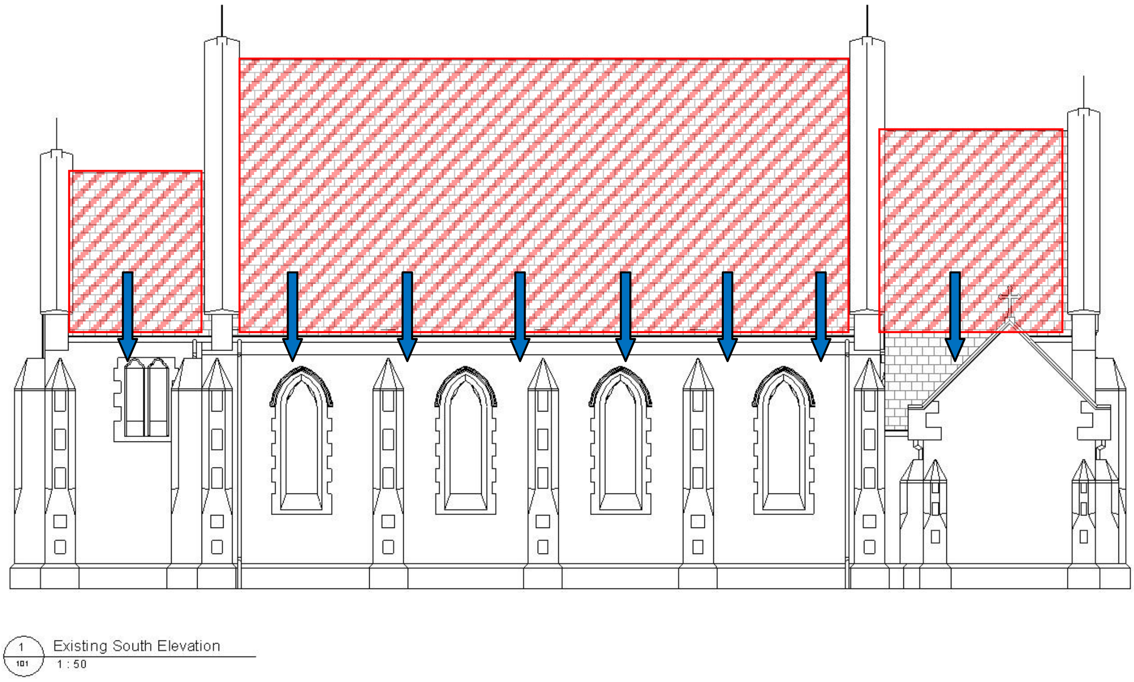


Figure 3 – Gravity system

The loads from the timber wall plates are picked up by the load bearing 450mm thick masonry wall below, and transmitted to the ground through the assumed existing strip footings, see Figure 3. The internal gable walls act as an arch and the vertical loads are transmitted through the piers either side of the arches to the strip footings below.

The internal spiral staircase was supported off the timber mezzanine floor. The loads from the mezzanine floor are transmitted to the load bearing walls below to the assumed existing strip footings.

4.3 Seismic Load Resisting System

Following the seismic retrofit in the late 1990's a number of major alterations were made to the existing structure, see below:

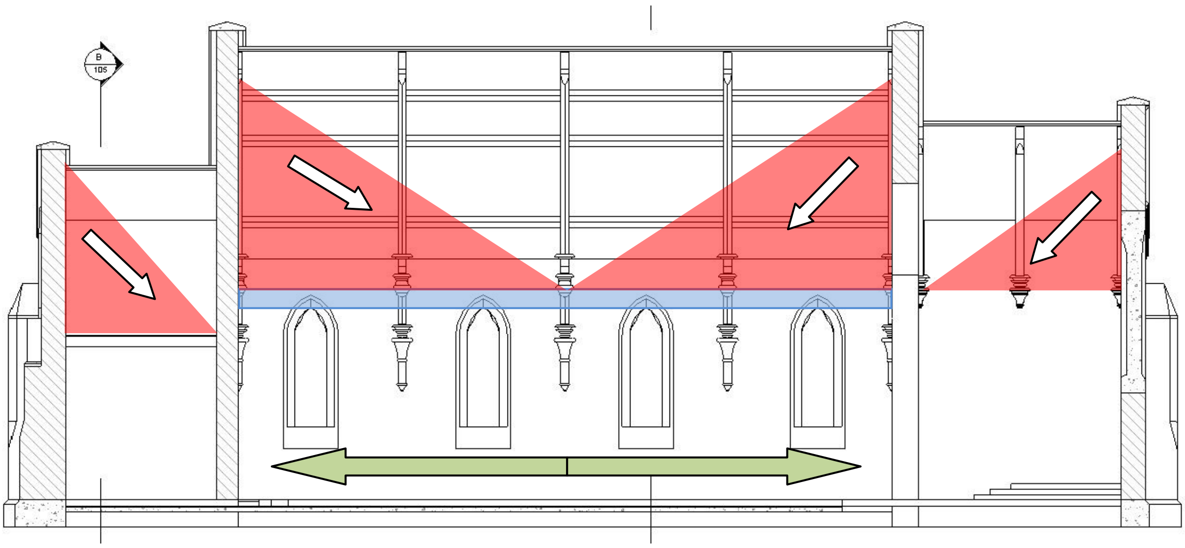


Figure 4 – seismic load resisting system longitudinal

4.3.1 Longitudinal seismic restraint

Restraint to the gables in-plane and out-of-plane was achieved by tying the gable walls with resin anchors to the roof structure. The roof structure has been strengthened with steel angle sections at each connection between the purlins and trusses. This provides some form of load path for seismic forces to be distributed down to the concrete bond beam via a flexible diaphragm see Figure 4. The bond beam then transmits the longitudinal forces into the masonry walls which resist the seismic forces in the plane of the wall.

A mid-height mezzanine floor at the western end of the chapel was strengthened with angle brackets and plywood to create a diaphragm facilitating seismic load transfer to the walls.

4.3.2 Transverse seismic restraint

The transverse seismic loads are transmitted through the flexible roof diaphragm to the wall plates and concrete beam located over the main walls. The loads are taken in-plane by the buttress walls and shear walls located at the external gables.

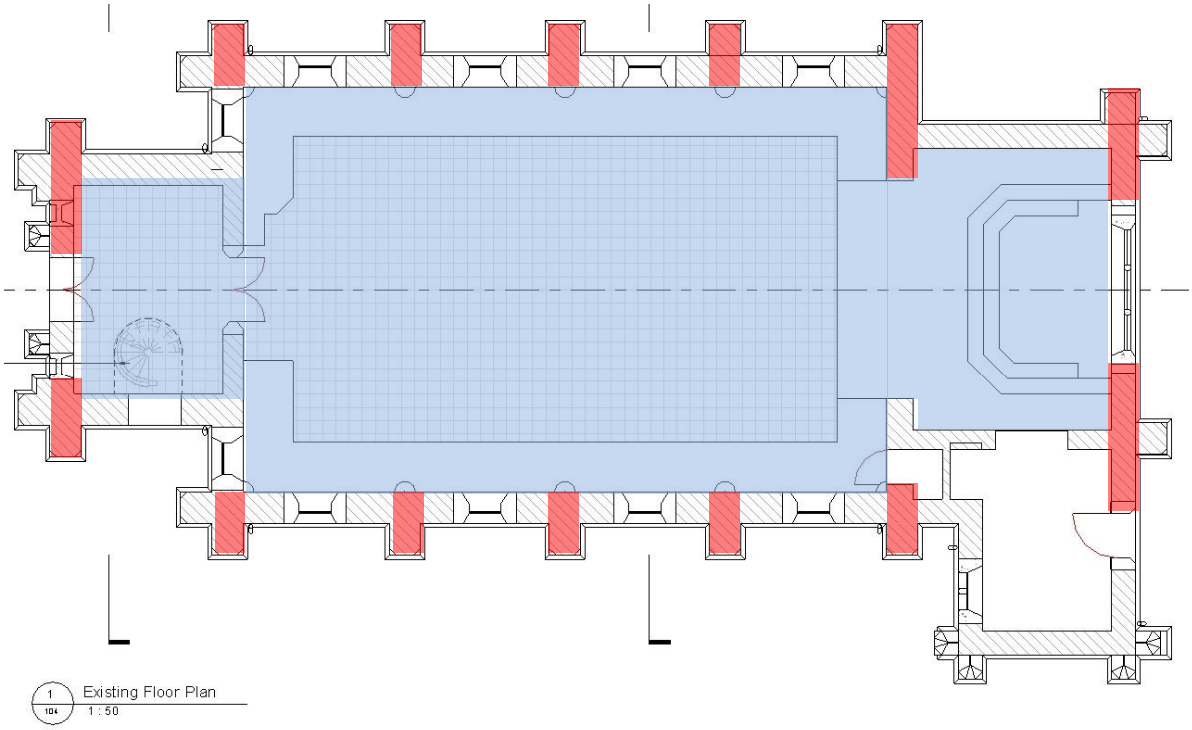


Figure 5 - seismic load resisting system transverse

The blue areas highlight where the roof diaphragm is located. The transverse loads from the roof diaphragm are transmitted to the red areas denoting the external buttress walls and shear walls.

5 Survey

A structural assessment of the building was undertaken on 23rd March 2012 by Opus International Consultants. The whole building was assessed during this inspection. The above investigations included external and internal visual inspections of all structural elements above foundation level, and of areas of damage to structural and non-structural elements.

Copies of the following construction drawings were provided by the Architect:

- Structural sketch drawings of the late 1990's seismic retrofit scheme; and
- Original architectural drawings.

These drawings were used to confirm the structural systems, investigate potential critical structural weaknesses (CSW's) and identify details which required particular attention.

No copies of the design calculations or specification have been obtained as part of the documentation set. A damage assessment of the structure was carried out and is attached in Appendix A of this report

6 General Observations

The building performed similar to other buildings of this construction and age. In particular, the failure mechanism of the gable end walls was common for this type of seismic retrofit. The roof was resin anchored into the end gable wall and it is likely to have failed due to the connections pulling out of the gable end wall. The overall structure of the south and north walls has fared well, with minimal structural damage to the walls, and a reasonable amount of residual seismic capacity.

It is likely from the observed damage that strengthening in the roof could not resist the applied seismic loads. This is evident from the visual damage to the timber trusses. In general, the connections between the purlin and trusses are showing signs of distress and the connections can be observed to be pulling away from one another.

It is expected that the mezzanine floor would have stayed intact if the gable wall had not collapsed onto it.

7 Geotechnical

A visual inspection of the site was carried out on the 23rd March 2012 the findings from the inspection showed no surface expression of liquefaction or settlement of the surrounding ground at the time of our inspection. However, a levels survey would need to be carried out to determine whether there has been a significant change in ground levels.

A desktop geotechnical assessment has been performed based on information obtained from borehole records surrounding the site. No site specific testing has been carried out.

The interpreted ground conditions from the previous investigations are as follows: Silts and sands from ground surface to 2.5m in depth; peat between 2.5m and 5.0m in depth; silts and sands between 5.0m and 22.0m, and the Riccarton Gravels below 22.0m. However, a detailed geotechnical investigation will need to be carried out to confirm the exact soil properties surrounding the site.

8 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 19th July 2011 , the term 'Critical Structural Weakness' (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of the building. We have identified the following potential CSW's for the building:

8.1 Gable walls

The connection between the gable wall and the roof provides a system of restraining the gable end walls. It is our opinion that the capacity of the connection between the gable wall and roof diaphragm was inadequate.

9 Remedial Works Scheme

Two conceptual strengthening schemes have been developed and are shown in Appendix C and D. The conceptual schemes aim to strengthen the building to 67% and 100% NBS respectively. An overview of the conceptual structural schemes are explained below.

Note the schemes are conceptual and are based on the visual inspection and engineering judgement. No calculations or detailed analysis has been carried out to develop the scope of the remedial works. As this is an indicative scheme, an appropriate allowance in the cost estimate should be made to account for the outcomes of a detailed engineering design.

9.1 Remedial works scheme 67% NBS – Appendix C

- Assume that the existing capacity of the nave roof diaphragm has enough capacity to carry seismic loads. This needs to be confirmed with a quantitative assessment.
- Create roof diaphragms located to the east and west of the church
- Assume the existing roof diaphragm has enough capacity to carry some of the seismic forces to the concrete bond beam
- Re-build the collapsed gable walls from reinforced concrete with finishes to match existing
- Shotcrete some of the internal faces of the existing walls

9.2 Remedial works scheme 100% NBS – Appendix D

- All works to be carried out as the 67% NBS scheme
- Strengthen the existing main roof diaphragm with plywood
- Locally shotcrete the north and south Nave walls

10 Initial Capacity Assessment

10.1 General

The initial strength assessment has been completed by using the initial Detailed Engineering Evaluation (DEE) procedure. No original calculations have been located so the original seismic coefficient is based on the knowledge of the structure and engineering judgement.

10.2 Seismic Coefficient Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170.5:2004
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B
- Return period factor R_u = 1.0 from table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life.

10.3 Expected Ductility Factor

Based on our assessment of the structural drawing, our initial estimates for the expected maximum structural ductility factors for the main seismic resisting systems are:

- μ_{max} = 1 for the un-reinforced masonry walls in both the east-west and north-south directions.

10.4 Estimated Structural Capacity

Based on the performance of the structure following the February 2011 earthquake the buildings failure was shown to be equivalent to a structure of 33% NBS and below. This is evident of the nature of the gable wall collapses.

A number of structural deficiencies have been identified below:

- Gables
- Roof diaphragm capacity and flexibility
- Transfer elements between walls and diaphragms

The residual capacity of the remaining structure is assumed to be greater than 33% NBS. An initial investigation of the north and south walls has shown the structure to have good residual capacity. Most of the structural deficiencies identified above have been a contributing factor to the failure of the existing structure. Therefore, we suggest a quantitative assessment is carried out to determine the capacity of the remaining structure and proposed strengthening scheme.

10.5 Discussion of Results

The majority of the original structure is estimated to have a seismic capacity of approximately 33% NBS. The end wall connection to the diaphragm are estimated to have a seismic capacity of less than 33% NBS. Hence, the building would have been assessed as an Earthquake Prone Building.

Based on the DEE assessment, the remaining building has an estimated seismic capacity of approximately 33% NBS. Therefore, strengthening works may be required to improve the building capacity such that it exceeds 67% NBS as required by the CCC Earthquake Prone Buildings Policy.

11 Conclusions

(a) Following the February 2011 earthquake the building sustained significant structural damage and partial collapse.

(b) Although the building had been strengthened, the seismic performance of the building is assessed at less than 33%NBS therefore, the building is deemed to be earthquake prone.

(c) A number of Critical Structural Weaknesses and structural deficiencies have been identified.

(d) We have developed a conceptual strengthening scheme to bring the building up to 67% NBS

12 Recommendations

It is recommended that:

(a) a quantitative analysis is undertaken in order to confirm the seismic capacity of the building, taking into account the identified potential critical structural weaknesses.

(b) Repair and strengthening scheme be developed to repair damage and increase seismic capacity to not less than 67% NBS.

13 Limitations

(a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is mentioned but this is not intended to be a comprehensive list of non-structural items.

(b) Our investigations have been visual and non-intrusive, no linings or finishes were removed to expose structural elements. Calculations have been limited to simple assessments and comparisons of seismic coefficients. No other analyses have been performed.

(c) 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 the time.

(d) This report is prepared for the CCC to assist with assessing remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

Appendix A – Damage Assessment Report

1 Background

Opus International Consultants were asked to carry out a damage assessment report of the Rose Chapel, Colombo street, Christchurch following the Dec 2011 Earthquake. A non invasive inspection was carried out on 23rd March 2012. The findings from this inspection are described in this report. It should be noted that this was only a visual inspection and no intrusive works were carried out.

2 Existing strengthening works

The general scope of strengthening works carried out included the following:

1. Tying the gable end walls into the roof truss, and subsequently transferring the load through the roof structure to the load bearing walls below.
2. Providing a substantial connection between the purlins and the trusses to facilitate transfer of seismic forces to the load bearing walls.
3. Concrete ring beam was installed at the head of the existing north and south walls. The beam is to provide a mode of transferring seismic loads to the walls and footing under.
4. The cavity wall has been filled with concrete however, it is unclear how much of the cavity is filled we suggest bore holes will need to be drilled to confirm the makeup.
5. Parapet strengthening.

3 General damage observations

Findings from the inspection have shown that the remaining building has not deteriorated significantly since the February 2011 earthquake. However, we have highlighted our main observations and mechanisms of failure throughout the structure, these are shown below:

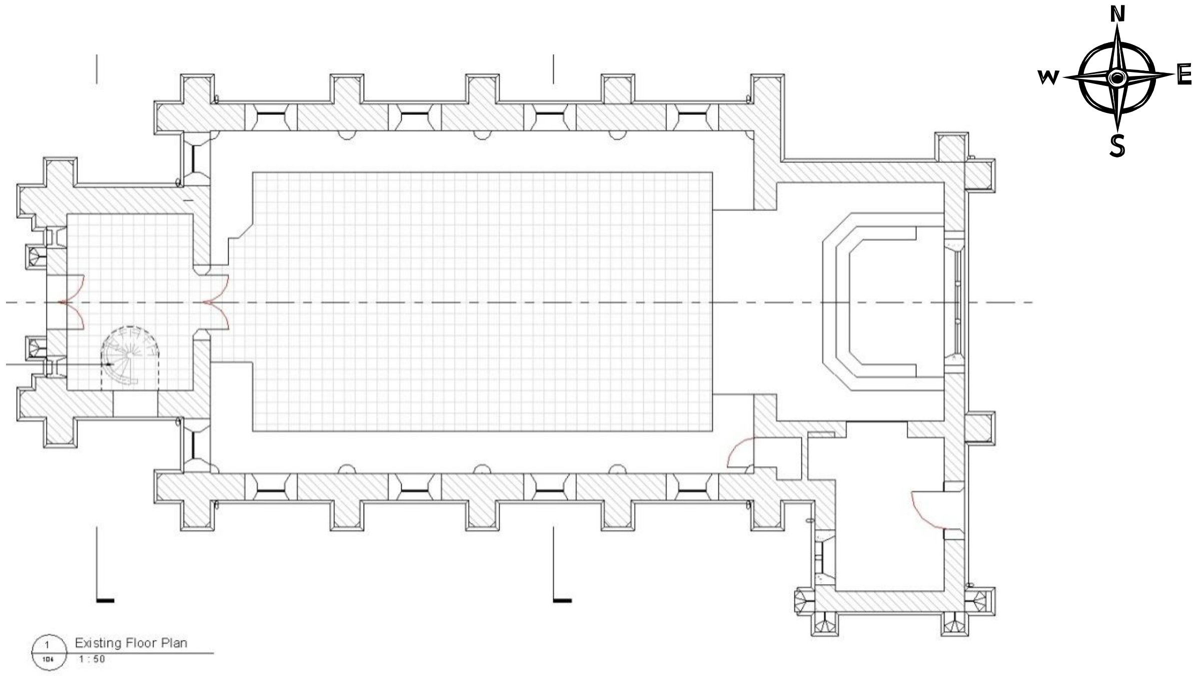


Figure 6 – Orientation of building

3.1 Gable end walls:

Three main stone gable walls have completely collapsed from the level above the nave walls out of plane. The mode of failure is likely to be the pulling out of the resin anchors from the gable walls, furthermore, a lack of mobilisation of the entire gable wall.

The gable at the entrance to the church chancel appears to have been deconstructed

The gable wall of the extension to the south of the church is showing signs of movement out-of-plane above the eaves level.

3.2 Roof collapse:

The roof in the porch located at the western end of the chapel, has suffered a roof collapse. This is a result of the gable wall above falling on it.

The mezzanine timber floor of the choir loft located at the western end of the church has failed in the mid-span due to the roof and gable wall above collapsing onto it.

3.3 Timber trusses:

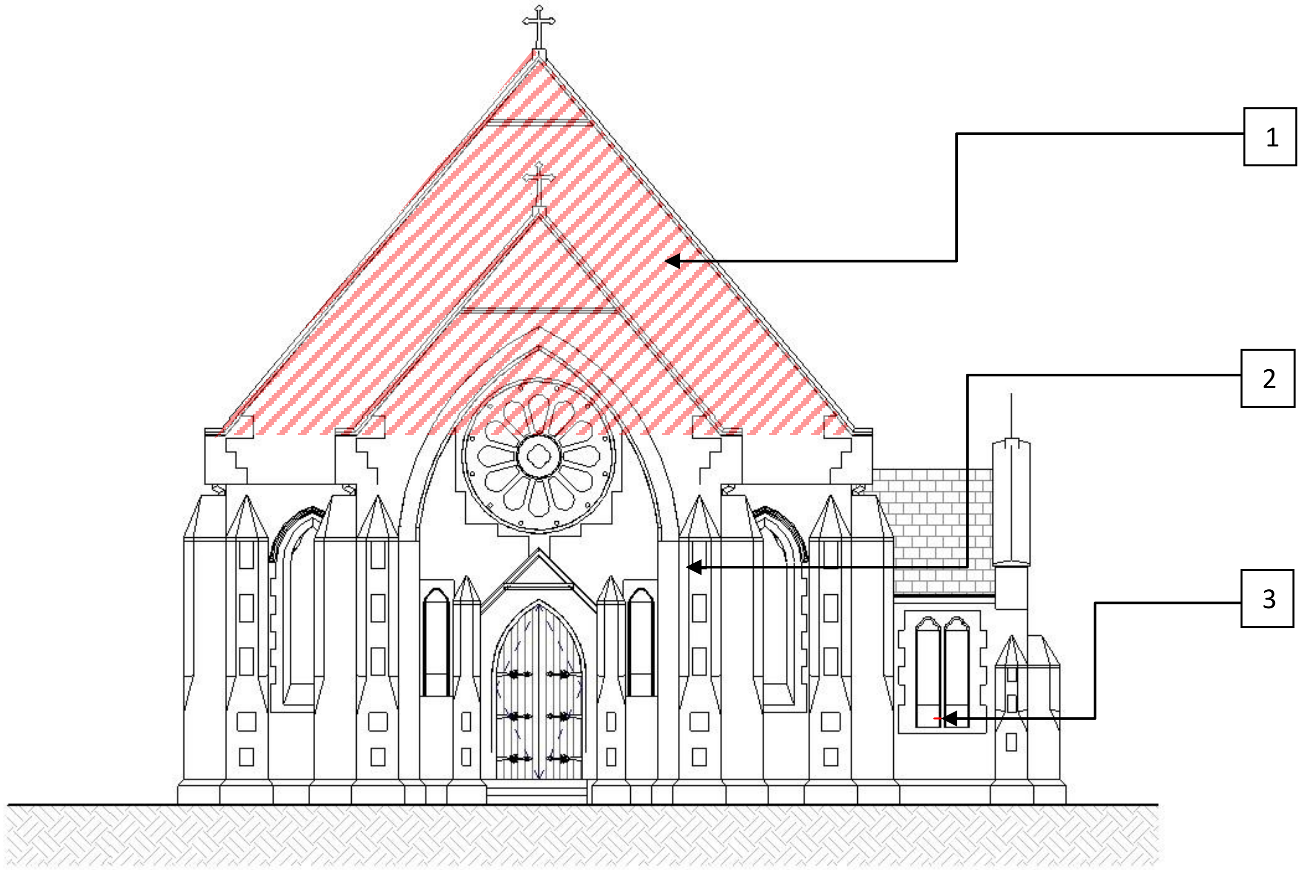
There has been a noticeable damage to the existing roof structure; the purlins, in particular have started to pull away from the trusses. The truss nearest the west end of the structure has failed at some of the connections and at the apex.

3.4 Walls:

No significant seismic damage was observed externally. Internally there are some horizontal cracks in the render, which are likely to be from horizontal seismic loads. These cracks are relatively minor ranging in size from 1mm to 5mm typically

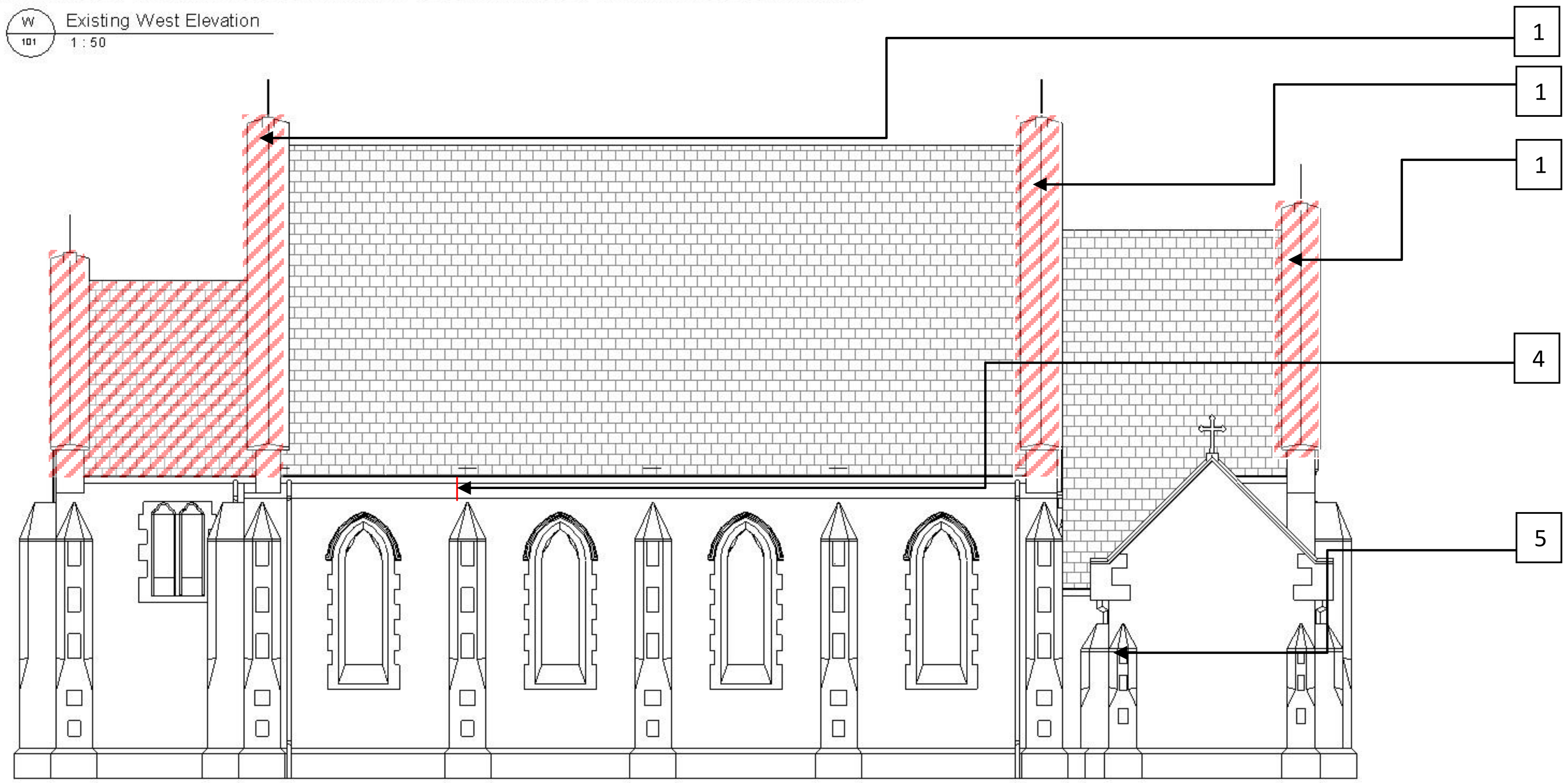
3.5 Ground conditions:

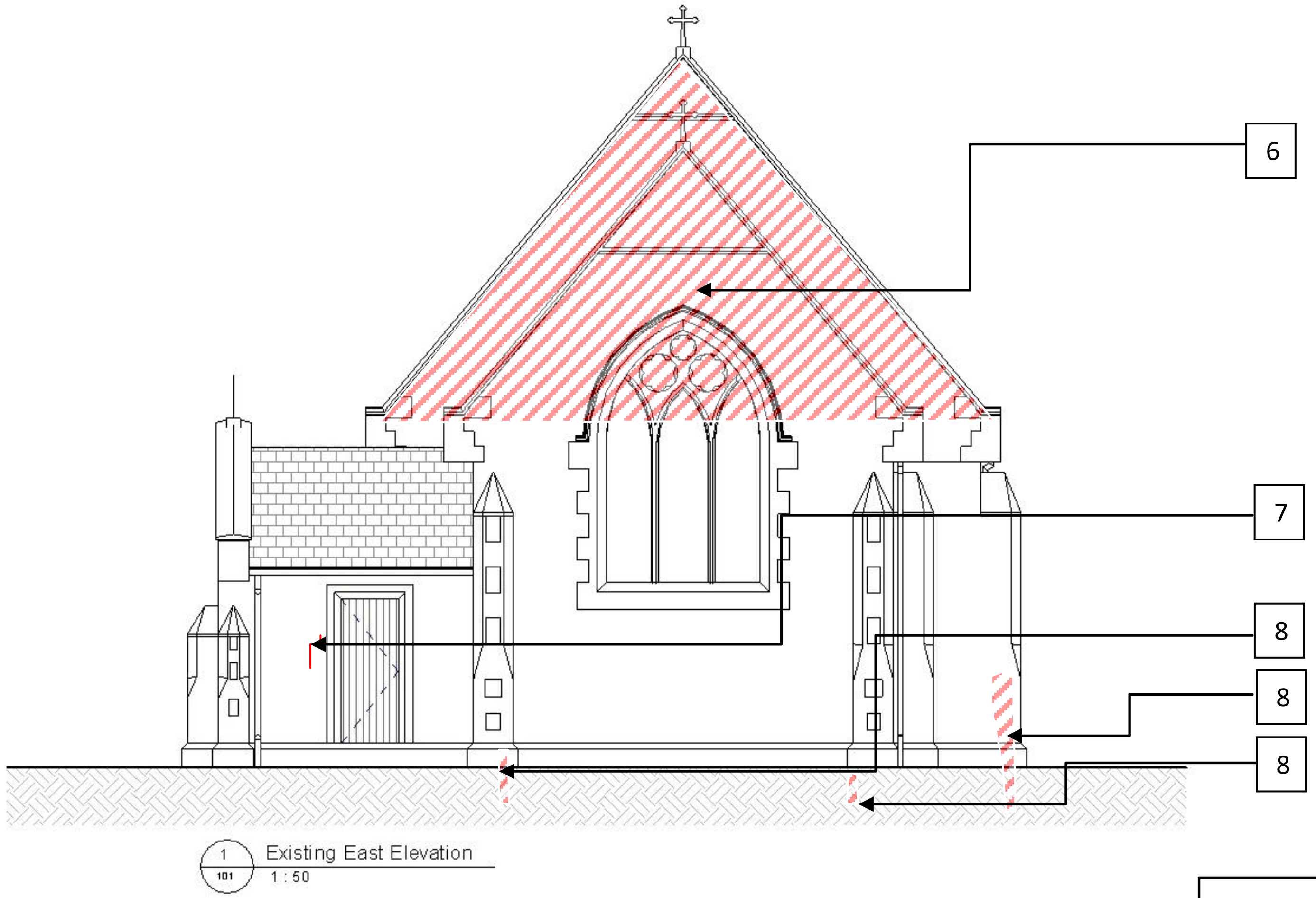
There are signs of some settlement near the front entrance of the church, but no surface expression of liquefaction were observed at the time of our inspection.



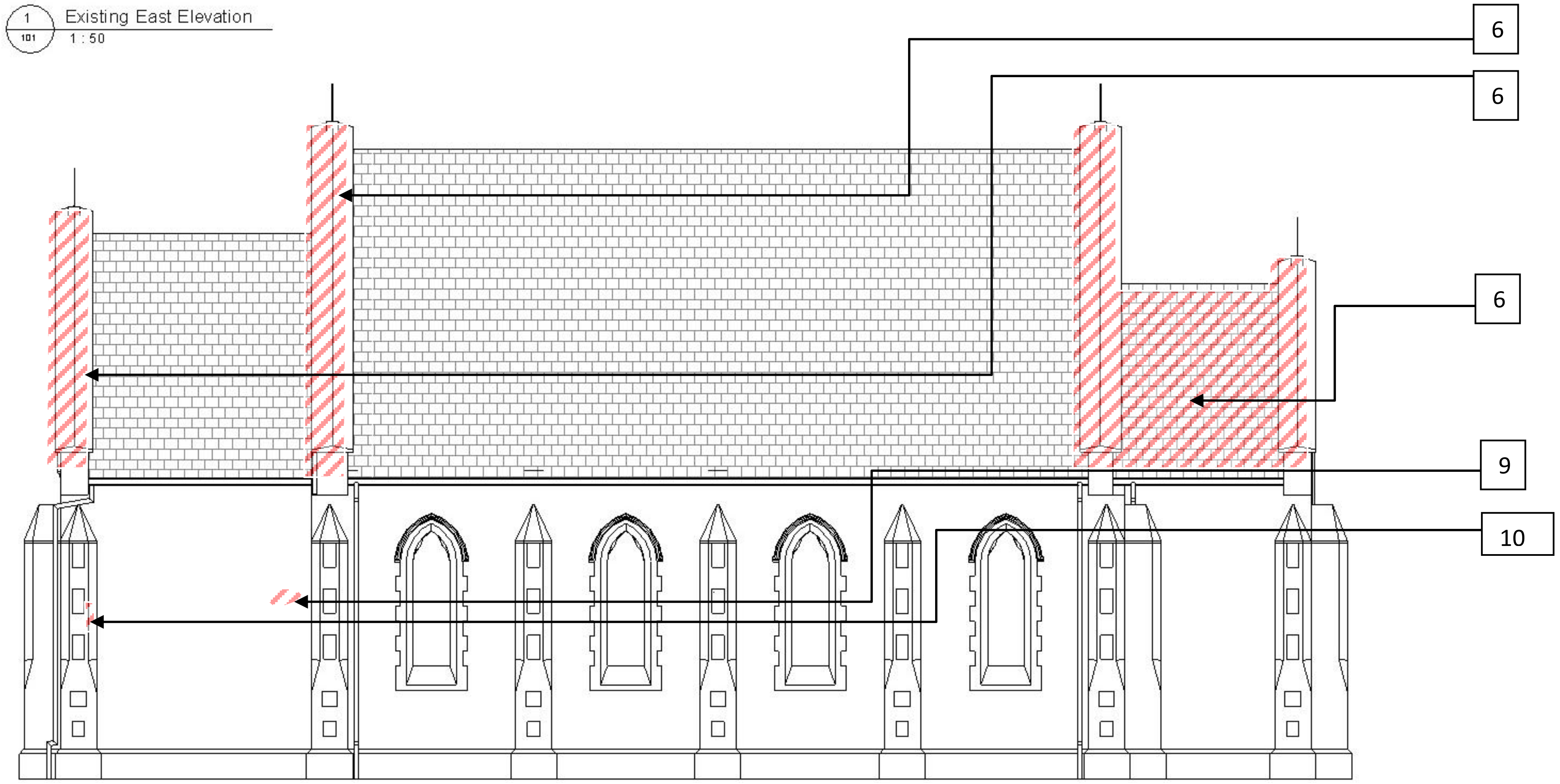
W 101 Existing West Elevation
1 : 50

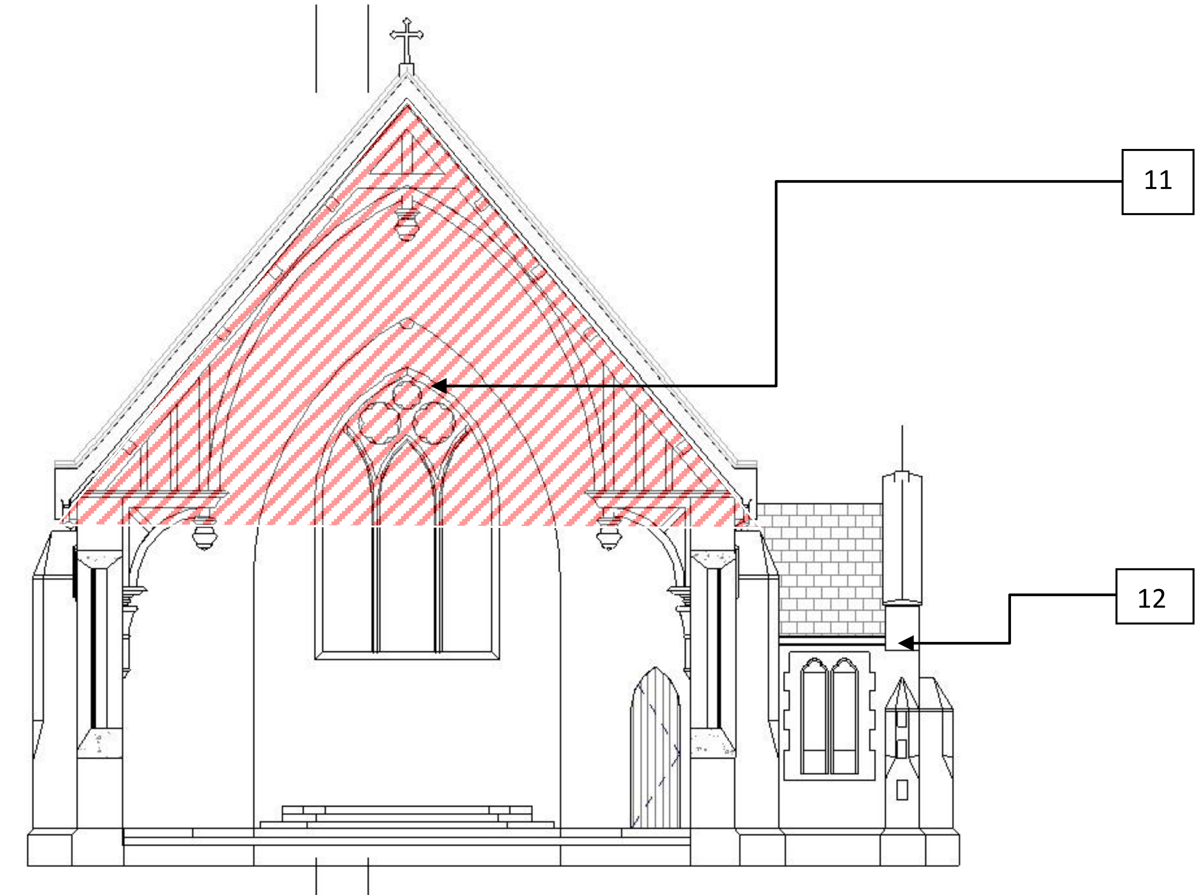
Schedule No.	**Description**	Photo Ref.
1	Indicates areas of the building that have collapsed due to the earthquake	*Fig 7*
2	Cement rendering beginning to peel off areas of recent repair	*Fig 8*
3	Damage to base of stone column	*Fig 9*
4	Vertical cracking has appeared between the cornicing	*Fig 10*
5	Damp ingress from guttering above, damp noted on the inside walls	*Fig 11*





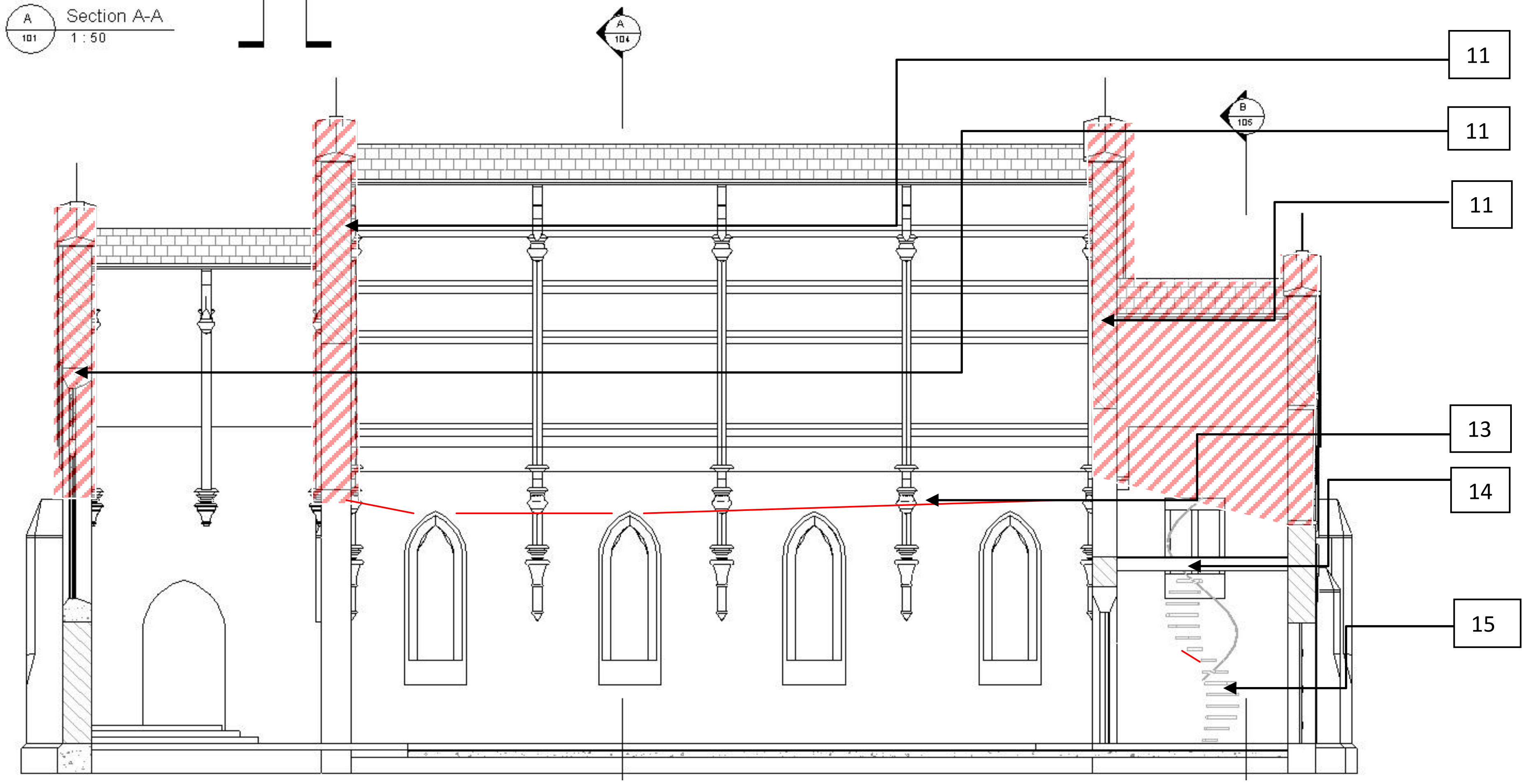
Schedule No.	Description	Photo Ref.
6	Indicates areas of the building that have collapsed due to the earthquake	
7	Vertical shear crack 1-3mm located around door lintel	Fig 12
8	Loss of facing stone	Fig 13
9	Pocket in wall, facing stone removed exposing brick	Fig 14
10	Loss of stone to top of buttress	Fig 15

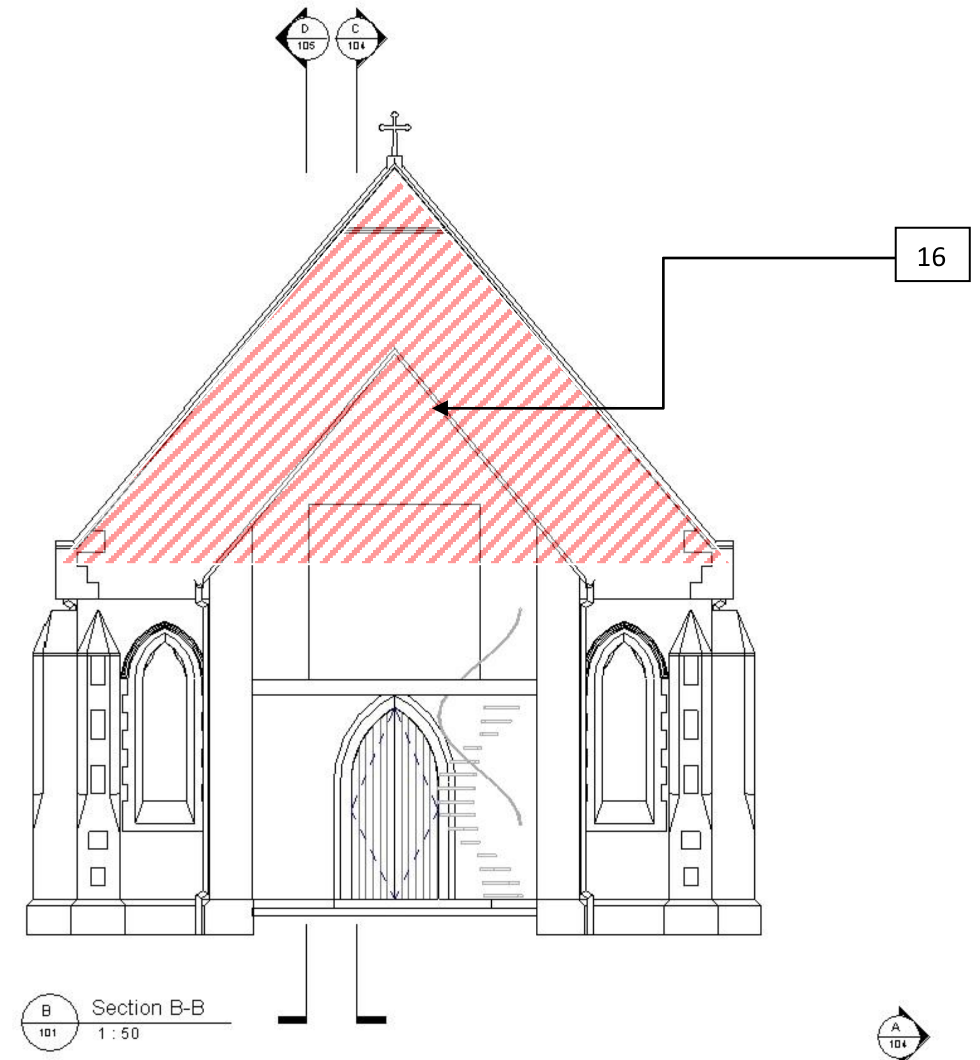




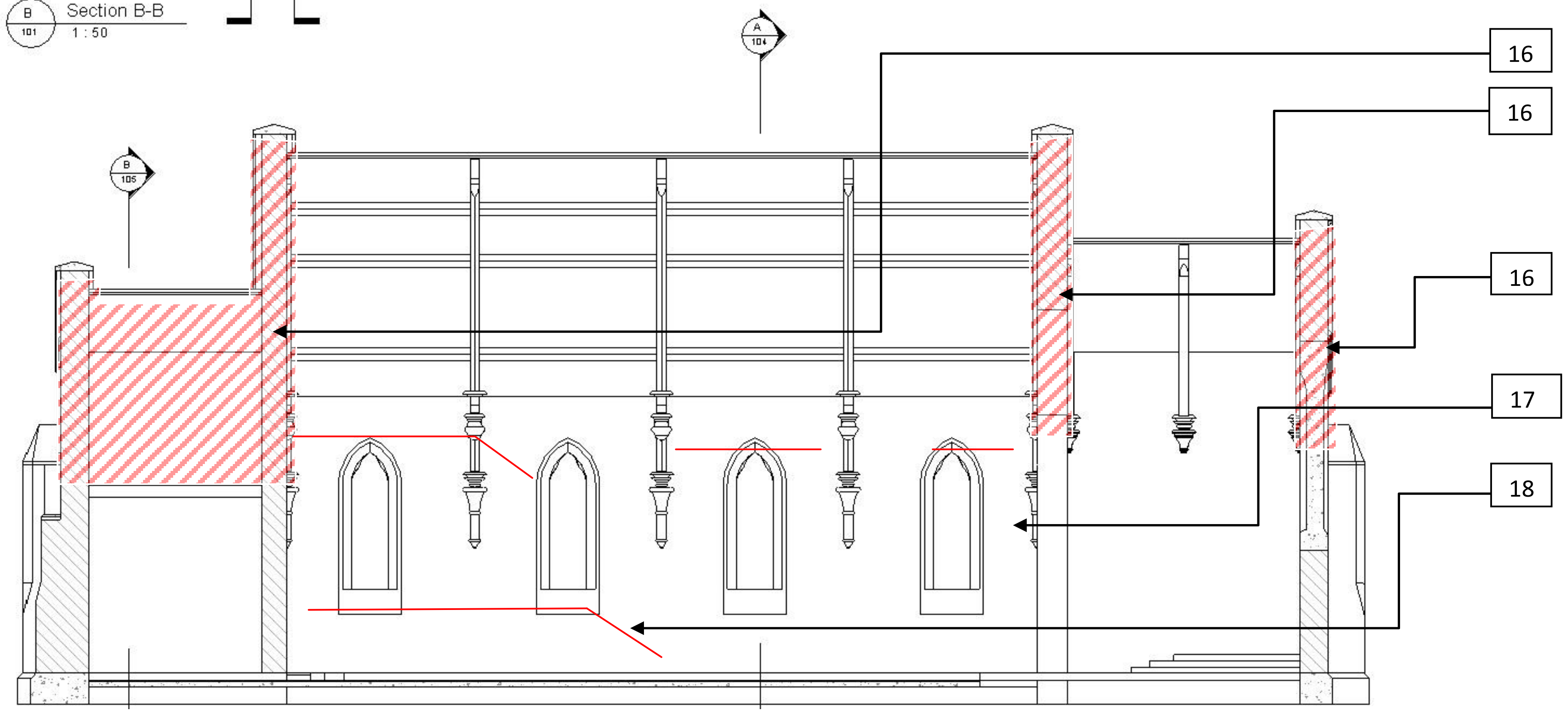
Section A-A
1 : 50

Schedule No.	Description	Photo Ref.
11	Indicates areas of the building that have collapsed due to the earthquake	
12	Gable wall showing signs of a 2˚ lean out of plane towards the toilet block. Damage to some of the internal rafters	*Fig 16*
13	Horizontal crack approx 1mm spanning the length of the church located above the existing window heads	*Fig 17*
14	Shear cracks in most of the timber floor joists due to the roof collapsing onto the joists	*Fig 18*
15	Spiral stair case removed previously	*Fig 19*

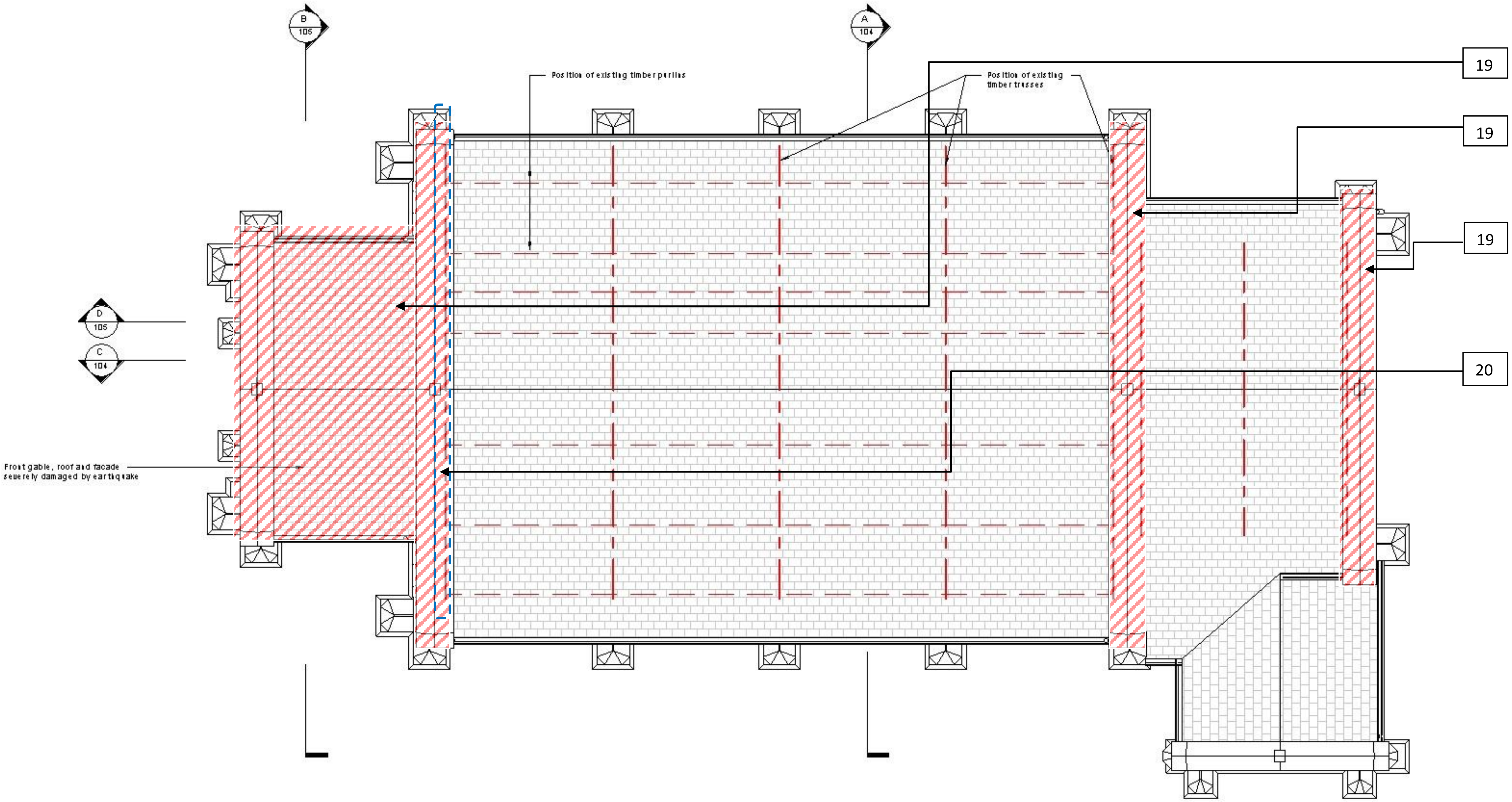




Schedule No.	Description	Photo Ref.
16	Indicates areas of the building that have collapsed due to the earthquake	
17	Horizontal crack approx 1-2mm spanning the length of the building	*Fig 20*
18	Horizontal crack approx 1mm spanning the length of the church located at the base of the windows	*Fig 21*



Schedule No.	**Description**	Photo Ref.
19	Indicates areas of the building that have collapsed due to the earthquake	
20	Truss severely damaged at the apex and the mid height splice connection	*Fig 23 & 24*
21	Generally connections into the gable walls have failed due to pull out and purlins pulling away from the truss	*Fig 25 & 26*



Appendix B – Photographs

Figure 7 – General front Elevation

Figure 8 – General South Elevation

Figure 9 – New concrete render peeling away from existing stone

Figure 10 – Horizontal crack and stone pulling away from central column

Figure 11 – Vertical joint opened up in between cornicing likely to be historic

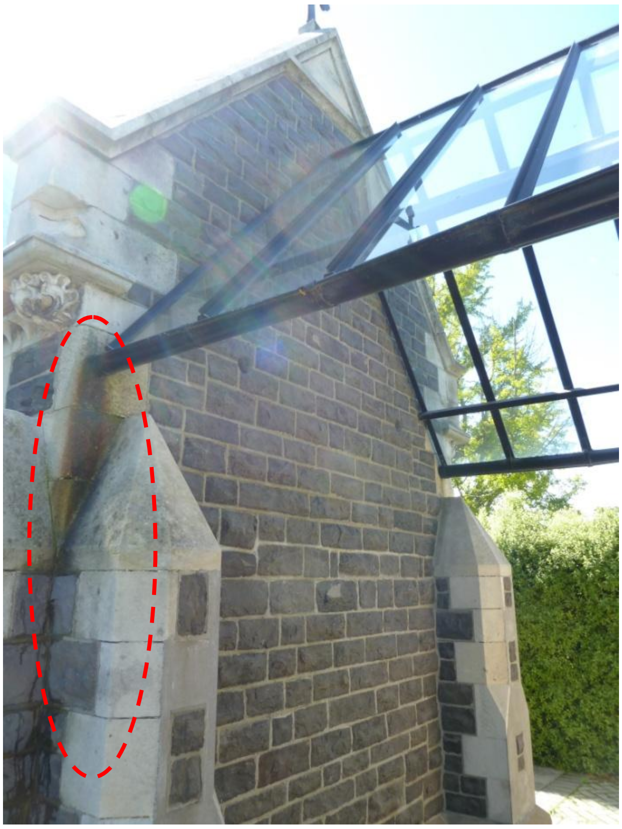


Figure 12 – water ingress from failing guttering

Figure 13 – Internal water ingress from failing guttering

Figure 14 - Shear cracks forming between the door lintel and masonry wall

Figure 15 - Pocket in wall appears to be historic

Figure 16 – Top of buttress suffered from damage and removal of facade stone

Figure 17 – Gable wall showing signs moving out of plane

Figure 18 – High level horizontal cracking

Figure 19 – Internal timber floors failure in Nave

Figure 20 – Main Entrance to church

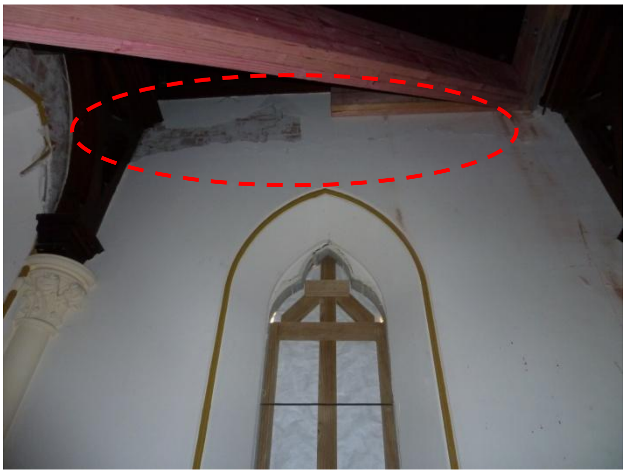


Figure 21 – High level horizontal cracking North Elevation

Figure 22 – Low level horizontal cracking North elevation

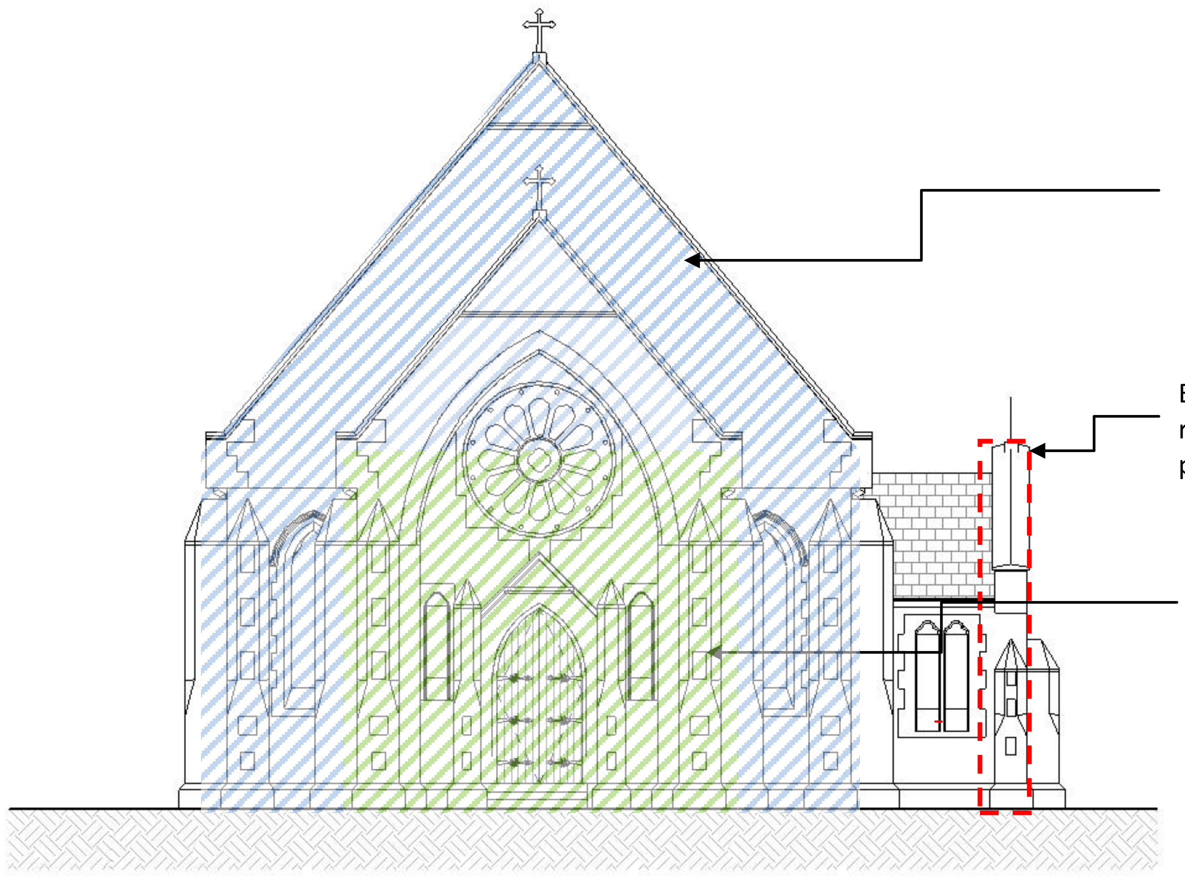
Figure 23 – Shear crack in roof truss connection

Figure 24 – shear failure at the apex of timber truss roof

Figure 25 – Purlin connections pulling away from main truss

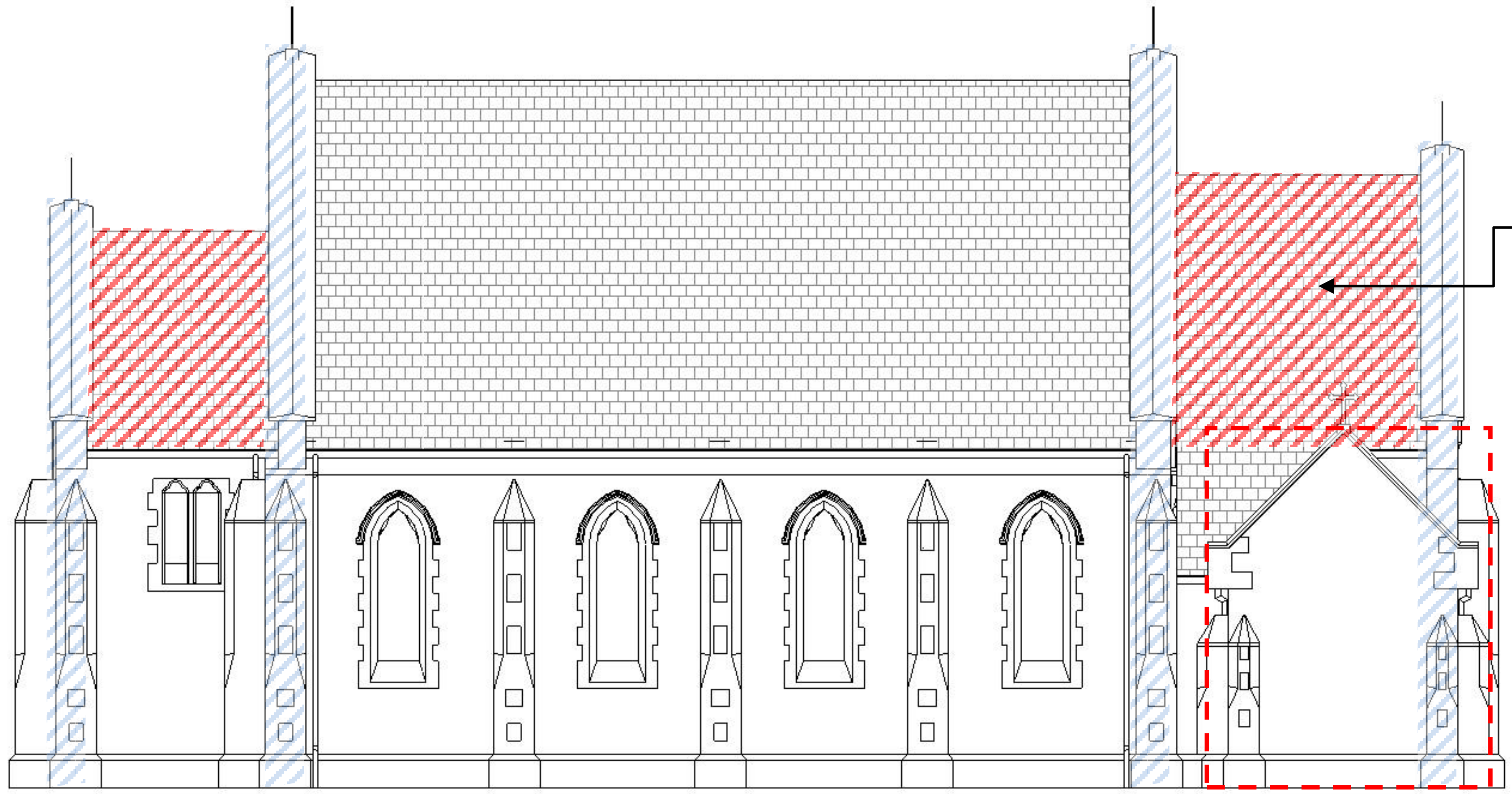
Figure 26 – Gable end ties failed in pull out

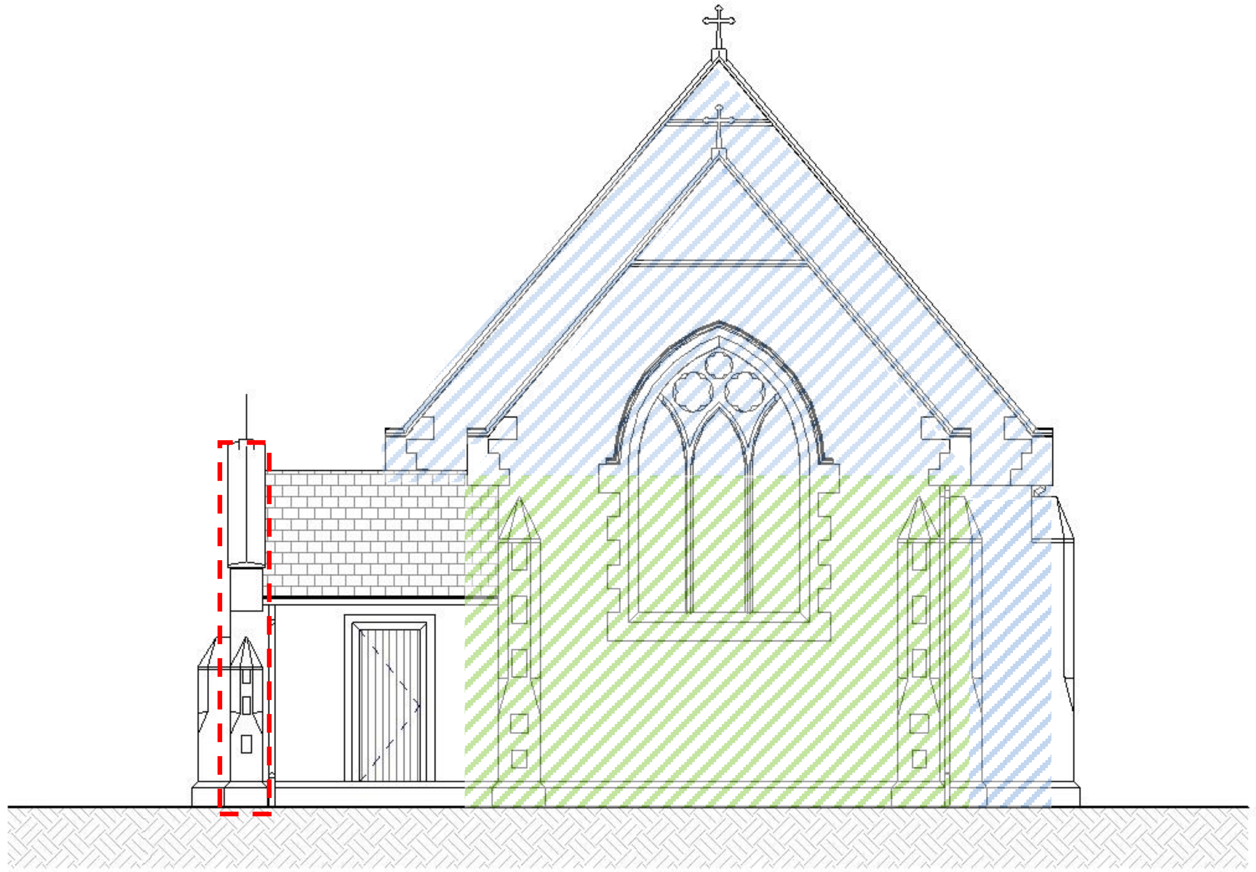
Appendix C – Structural concept strengthening 67% NBS



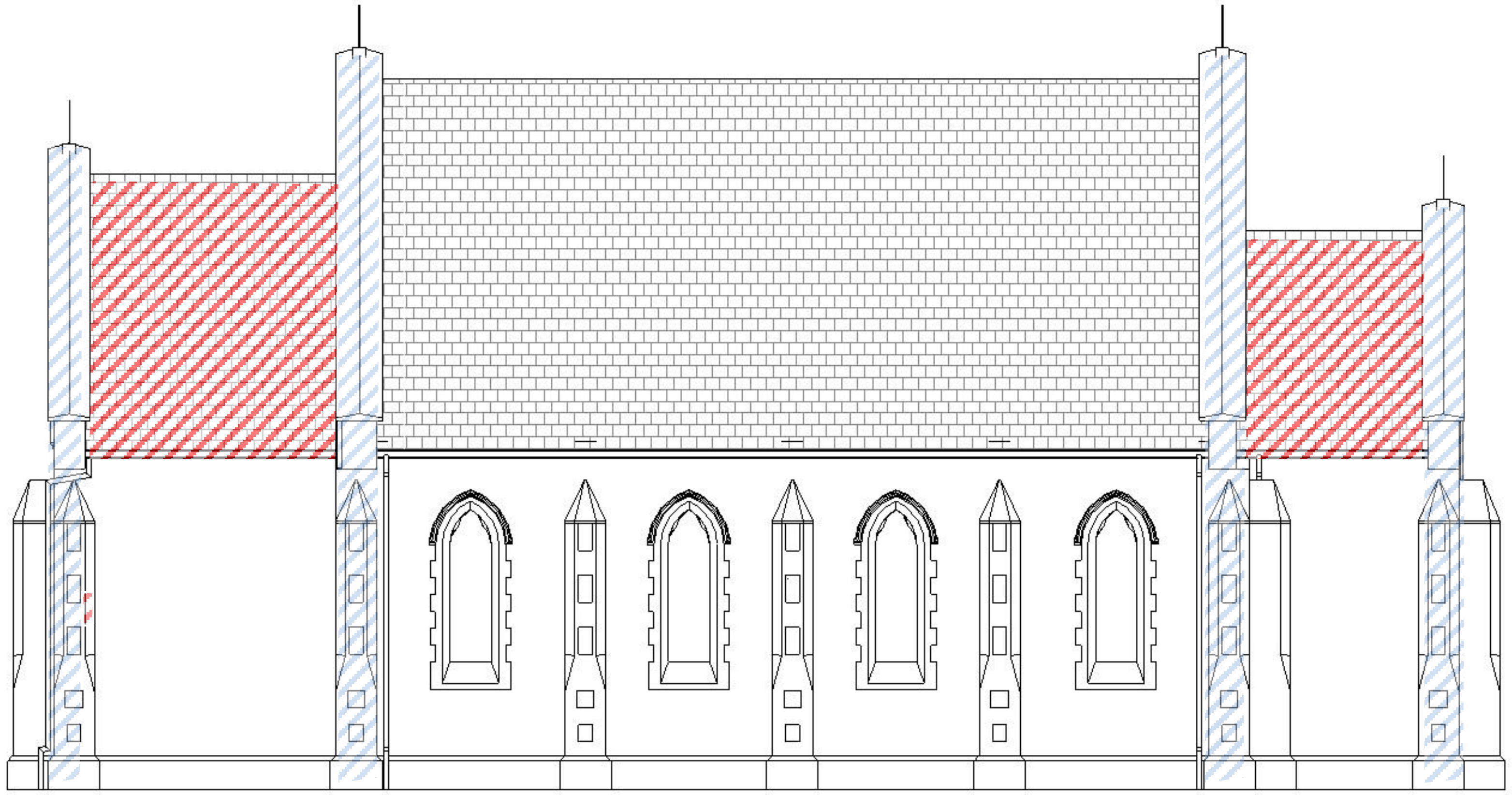
W 101 Existing West Elevation 1 : 50

Key	**Description**
	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possible existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
	Denotes proposed timber ply diaphragm constructed from 150x150 chords with grade F22 21mm thick ply top and bottom secured to the gables and fixed to the proposed RC walls
	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels. Inner 200mm of brick to be removed to allow installation

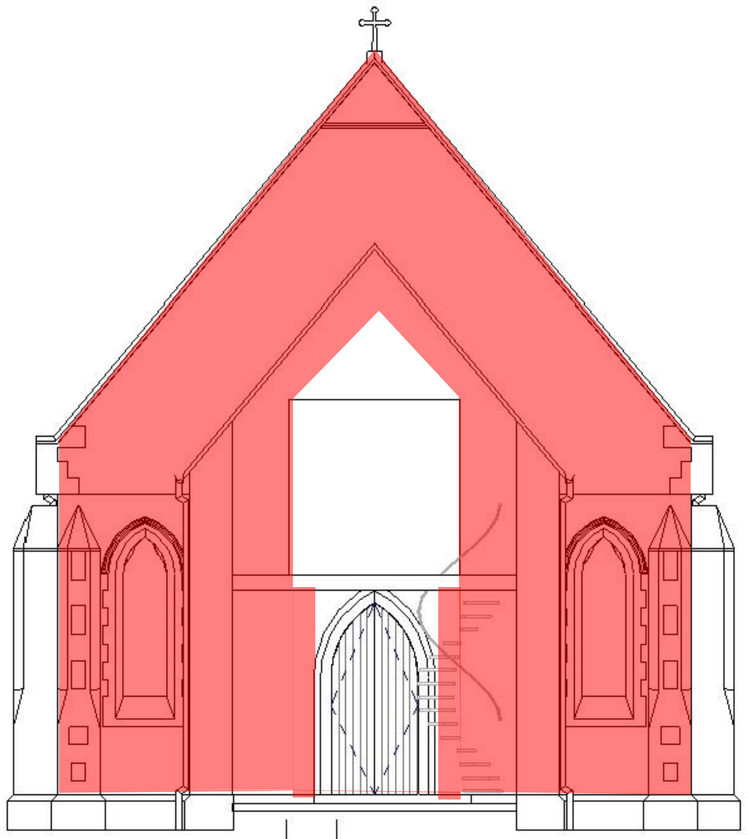




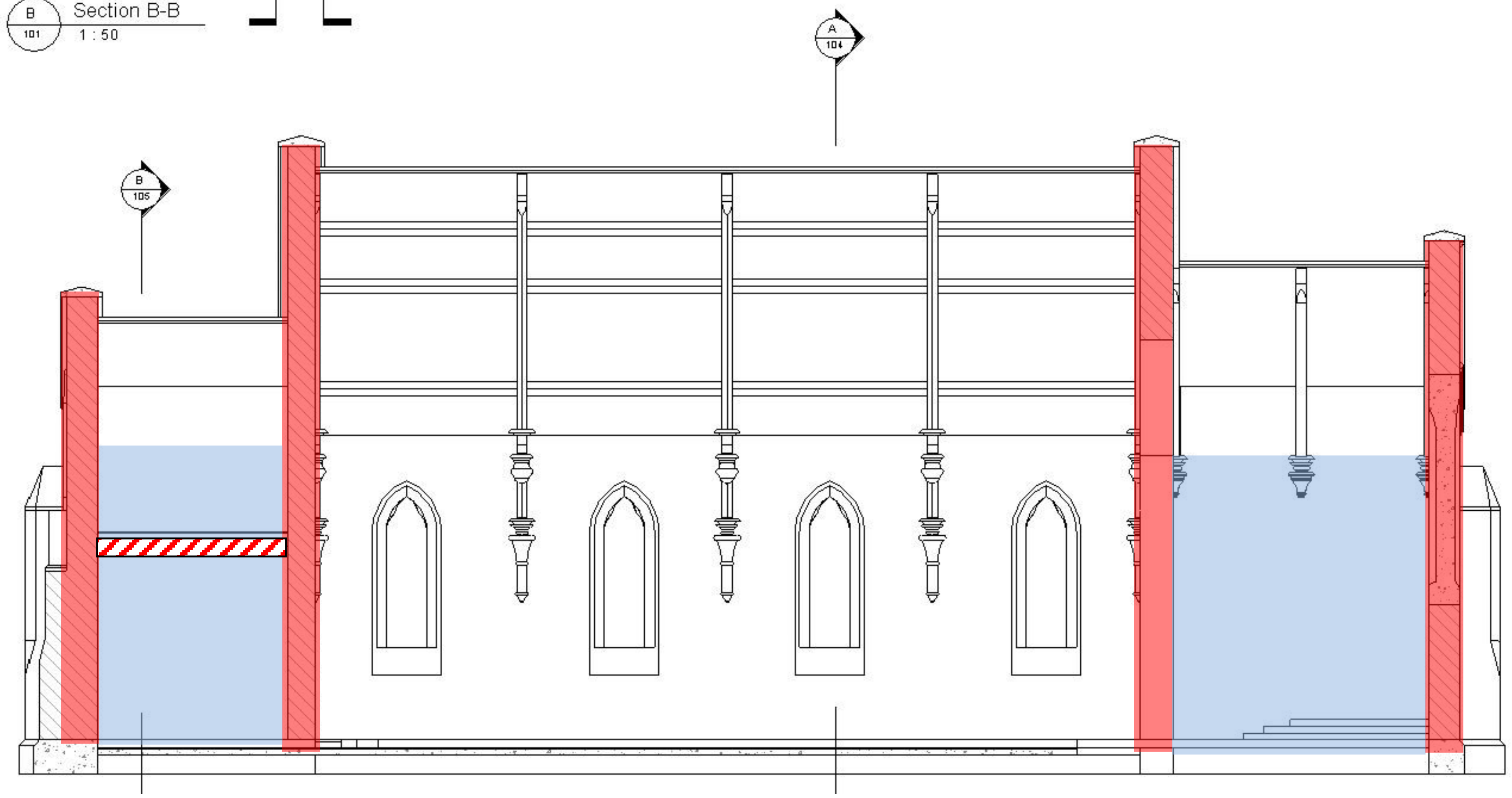
1 / 101 Existing East Elevation 1 : 50



Key	**Description**
(blue hatch)	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possibly existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
(green hatch)	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels. Inner 200mm of brick to be removed to allow installation
(red hatch)	Denotes proposed timber ply diaphragm constructed from 150x150 chords with grade F22 21mm thick ply top and bottom secured to the gables and fixed to the proposed RC walls



Section B-B
1 : 50



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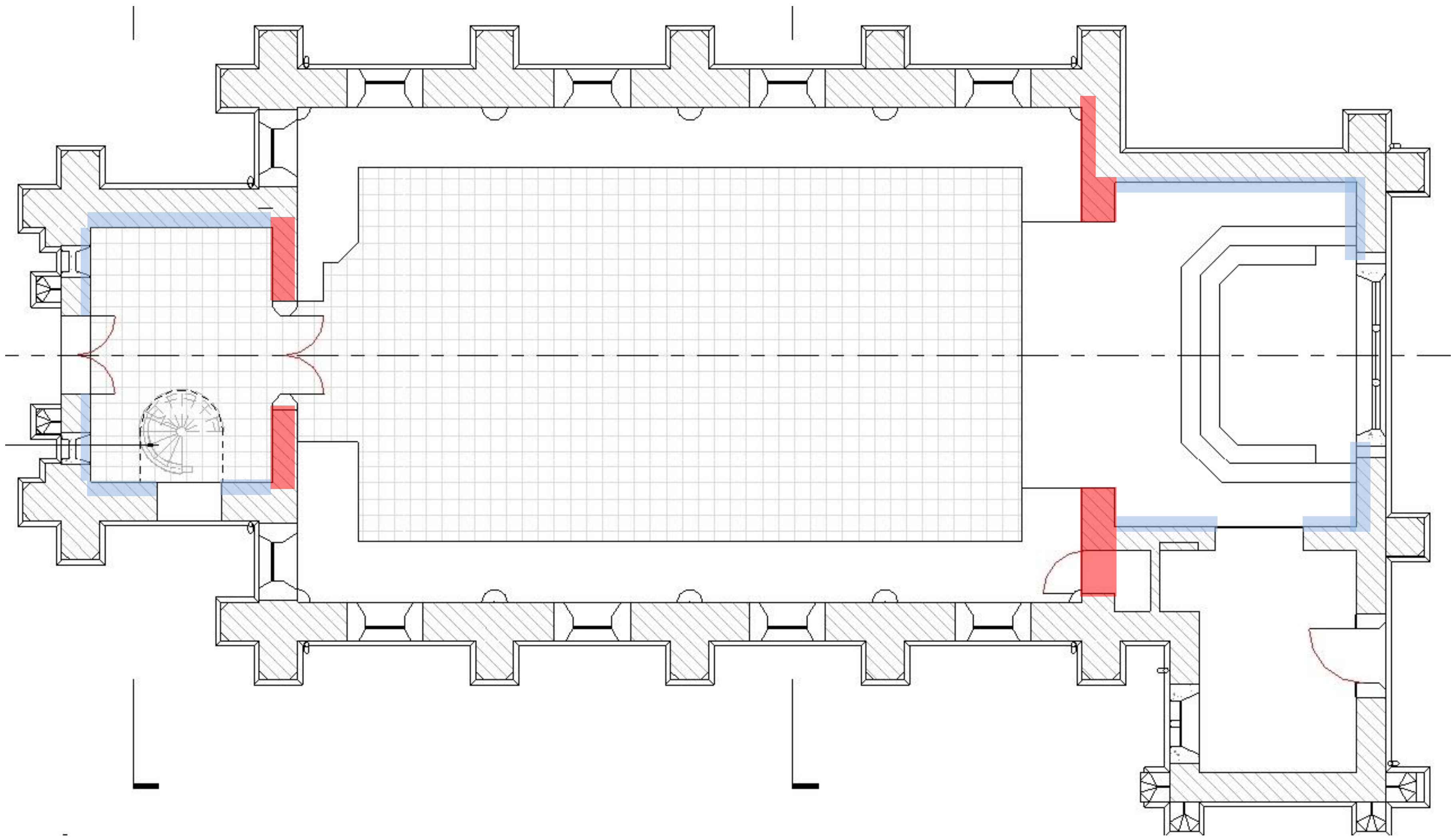
	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.
	Denotes proposed timber ply diaphragm and steel bracing floor to be tied into gable end walls

Key	**Description**
(red)	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possible existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
(blue)	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.



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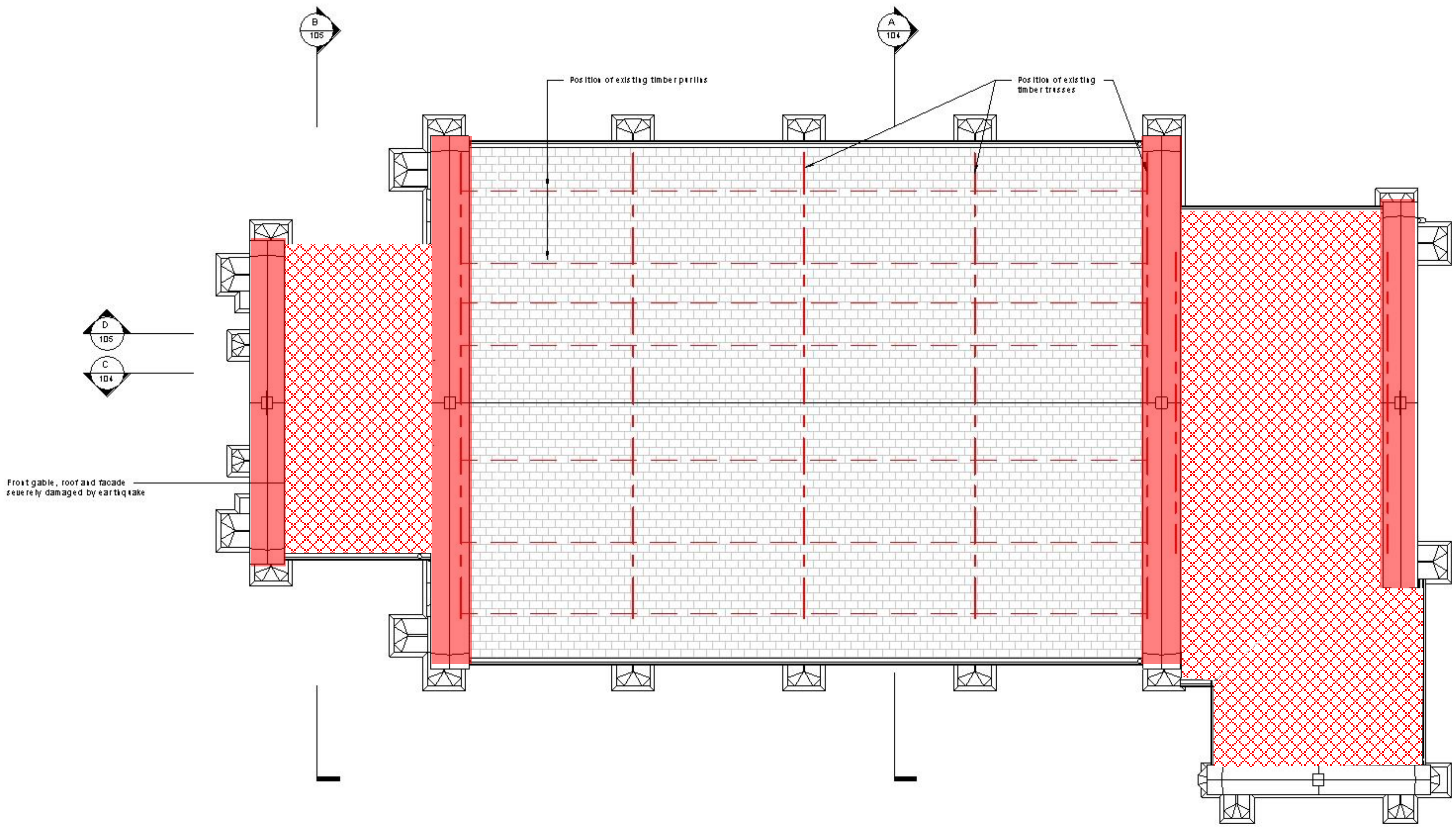
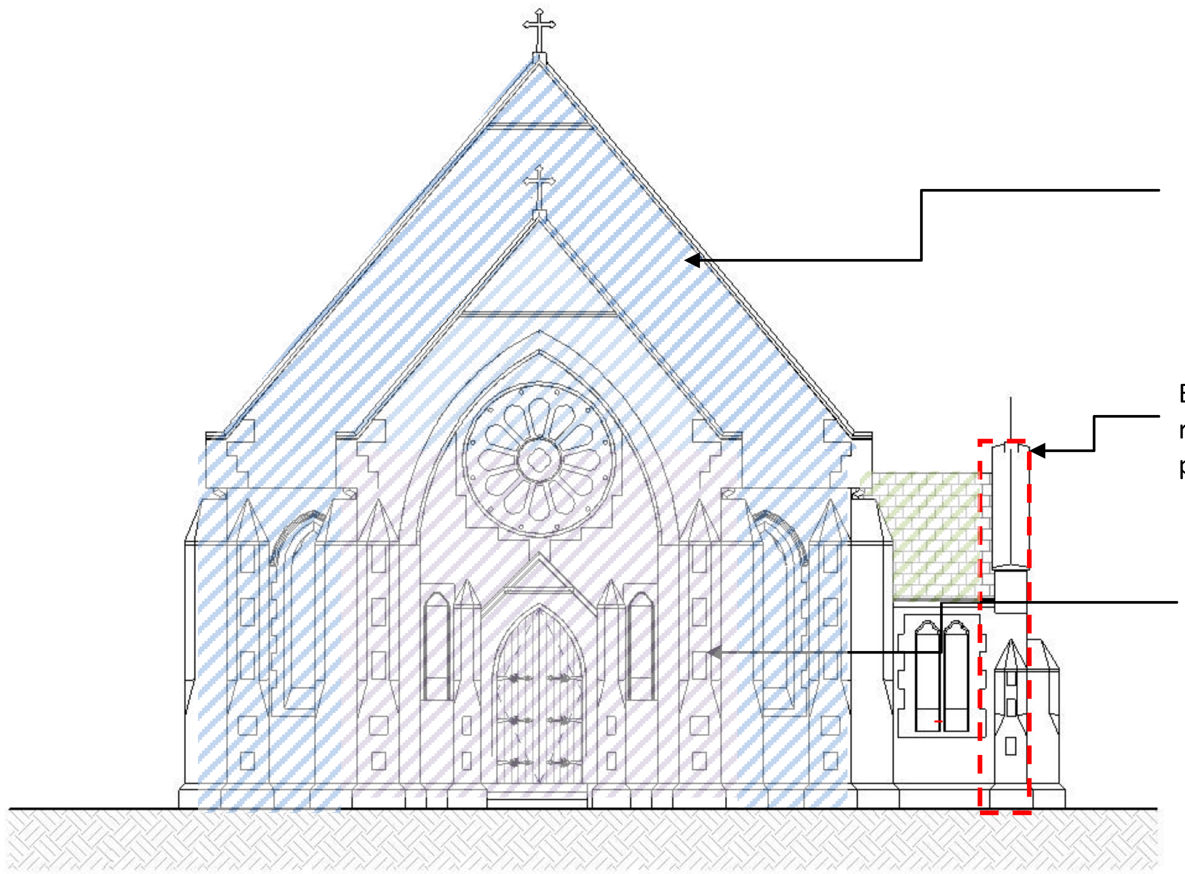


Appendix D – Structural concept strengthening 100% NBS



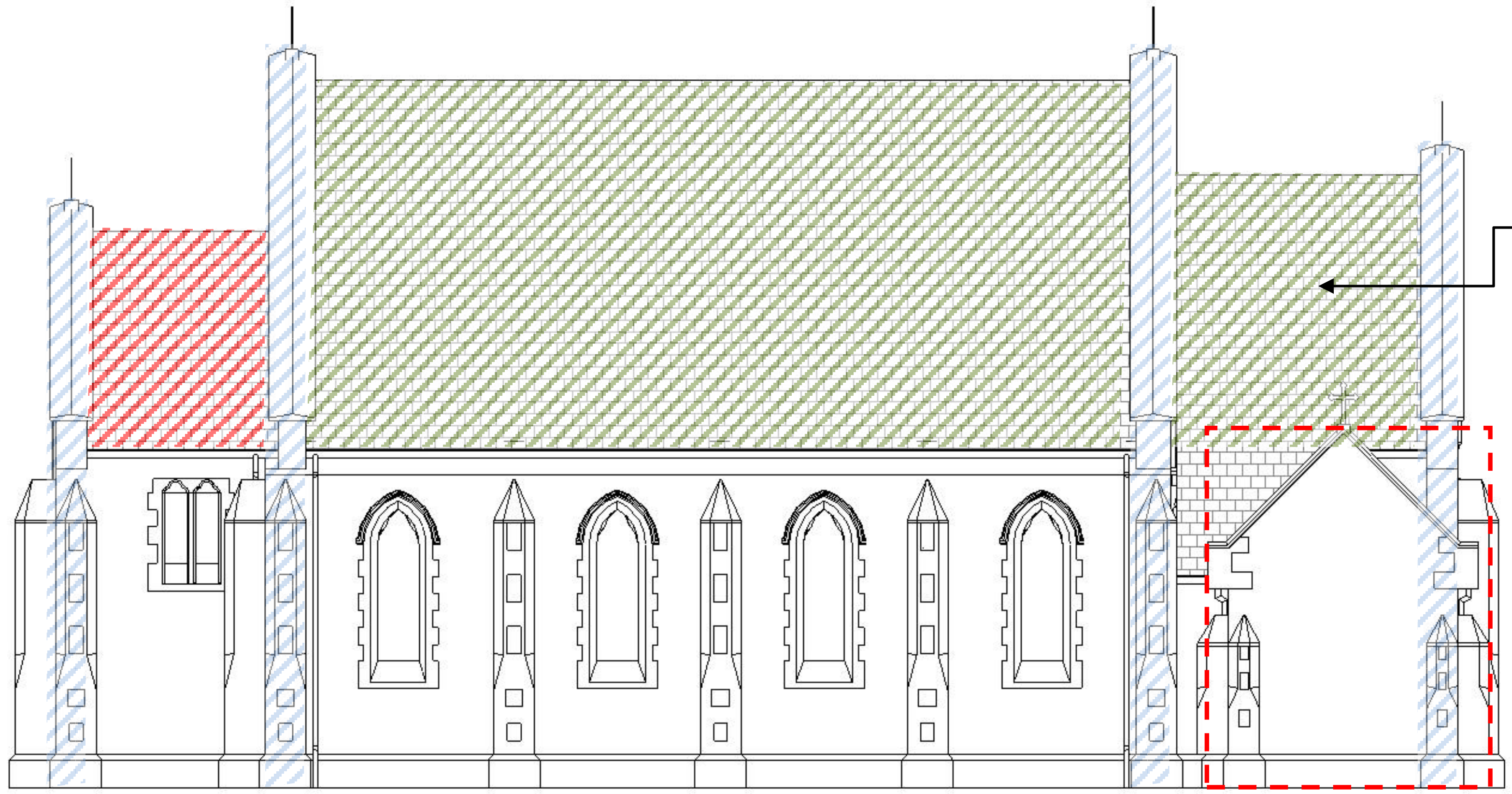
Existing West Elevation
1 : 50

Gable wall to be constructed from new RC wall and finished to match existing

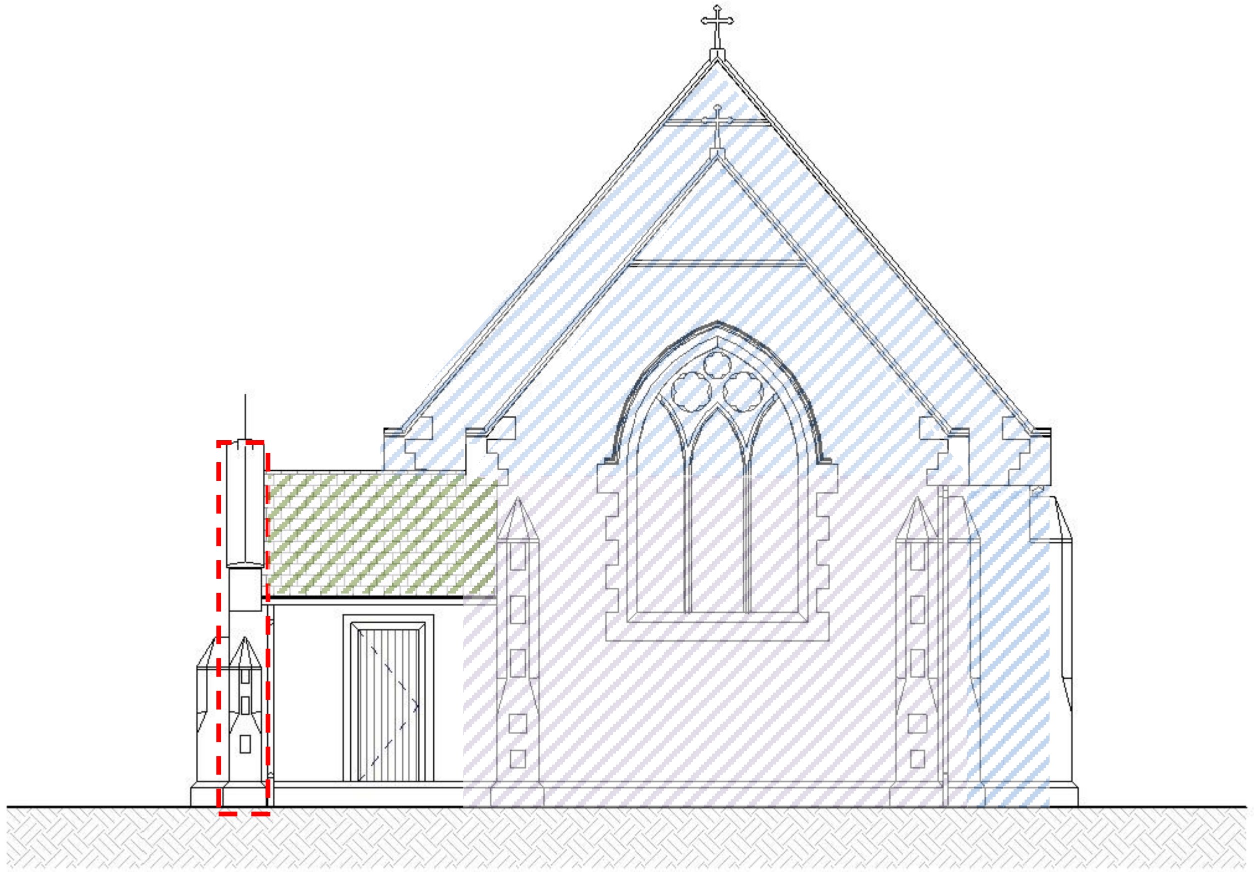
Existing gable wall to be strengthened with resin anchors grouted into the wall and roof purlins to be tied into the gable end

Existing wall to be retained and faced with an internal RC wall

Key	Description
	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possible existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
	Denotes proposed timber ply diaphragm constructed from 150x150 chords with grade F22 21mm thick ply top and bottom secured to the gables and fixed to the proposed RC walls
	Denotes Proposed roof diaphragm constructed from 21mm ply secured to rafters. Slate tiles to be removed and plywood installed
	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.



Roof to be constructed to form new Timber plywood diaphragm



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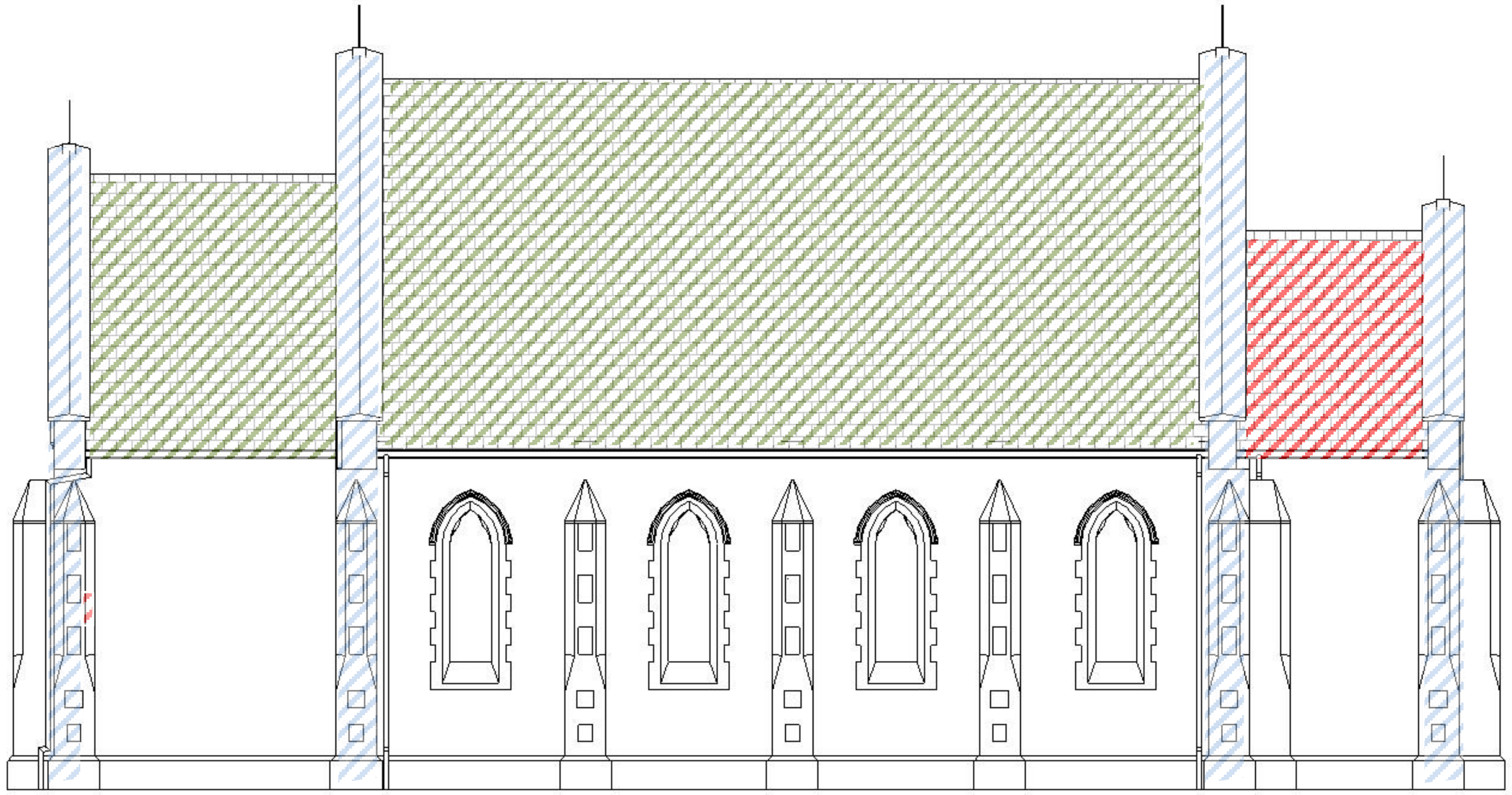
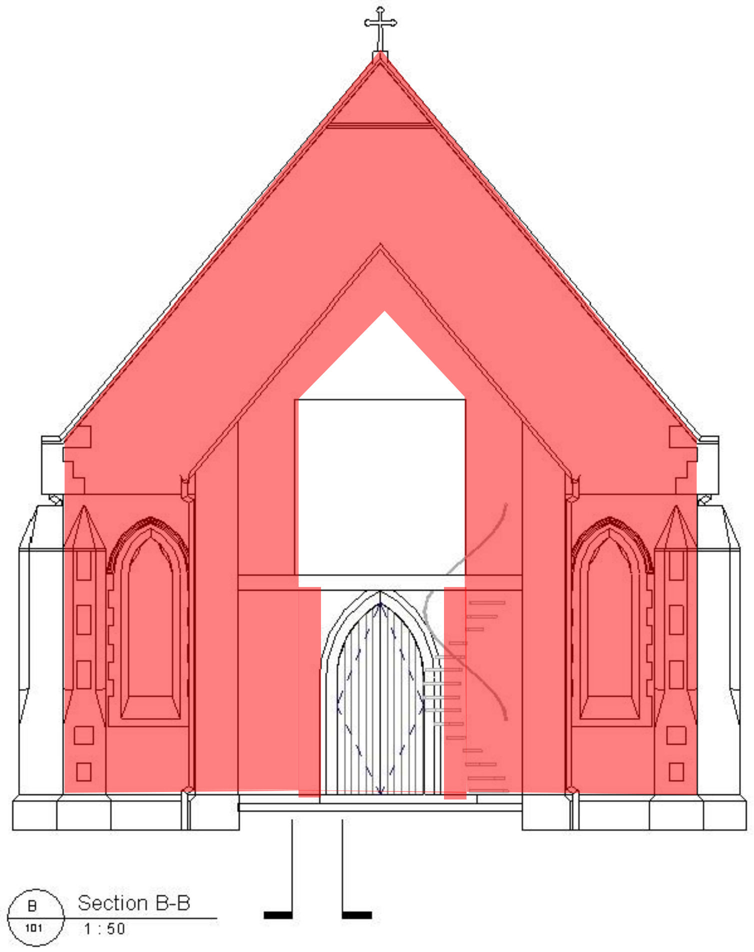
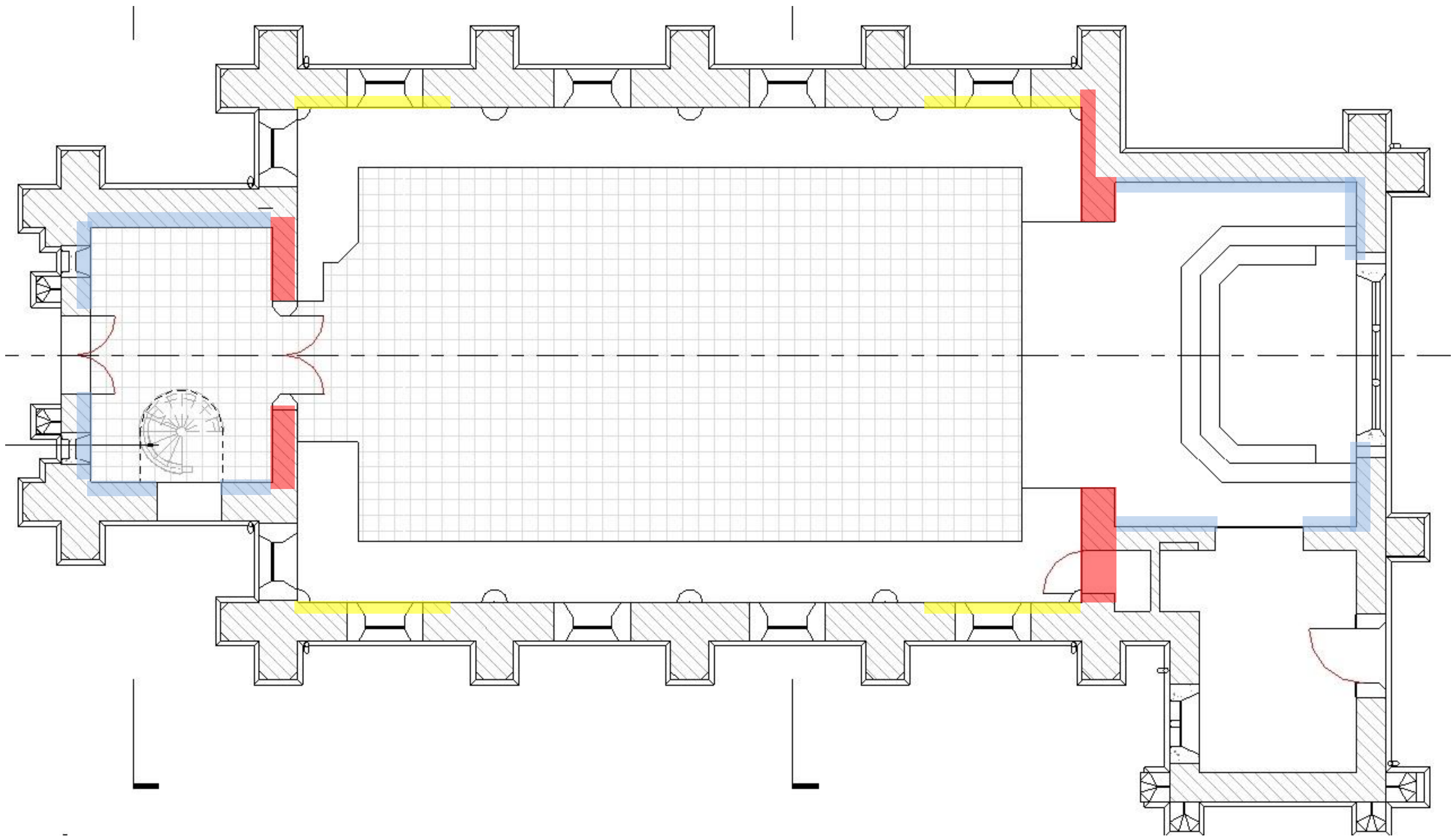
(purple hatch)	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.
(red hatch)	Denotes proposed timber ply diaphragm constructed from 150x150 chords with grade F22 21mm thick ply top and bottom secured to the gables and fixed to the proposed RC walls
(green hatch)	Denotes Proposed roof diaphragm constructed from 21mm ply secured to rafters. Slate tiles to be removed and plywood installed





Key	Description
(red)	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possible existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
(blue)	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.
(red hatched)	Denotes proposed timber ply diaphragm floor and steel bracing to be tied into gable end walls
(yellow)	Remove internal layer of brick and replace with 200mm RC shotcrete wall. Finishes to match existing

Key	**Description**
(red)	Denotes areas of existing wall to be constructed from a 250mm thick reinforced concrete wall with H16 re-bar at approx 200c/c. Existing facing stone to be re-used where possible and tied into new RC wall. Finishes are to match existing and where possibly existing stone onsite is to be re-used. Any imported stone is to be approved before used on site.
(blue)	A proposed 200mm thick RC wall is to be installed within the inner face of the existing wall. The existing facade is to be tied into the RC wall. Proposed RC wall is to be secured into the existing footings with steel dowels.
(yellow)	Remove internal layer of brick and replace with 200mm RC shotcrete wall. Finishes to match existing



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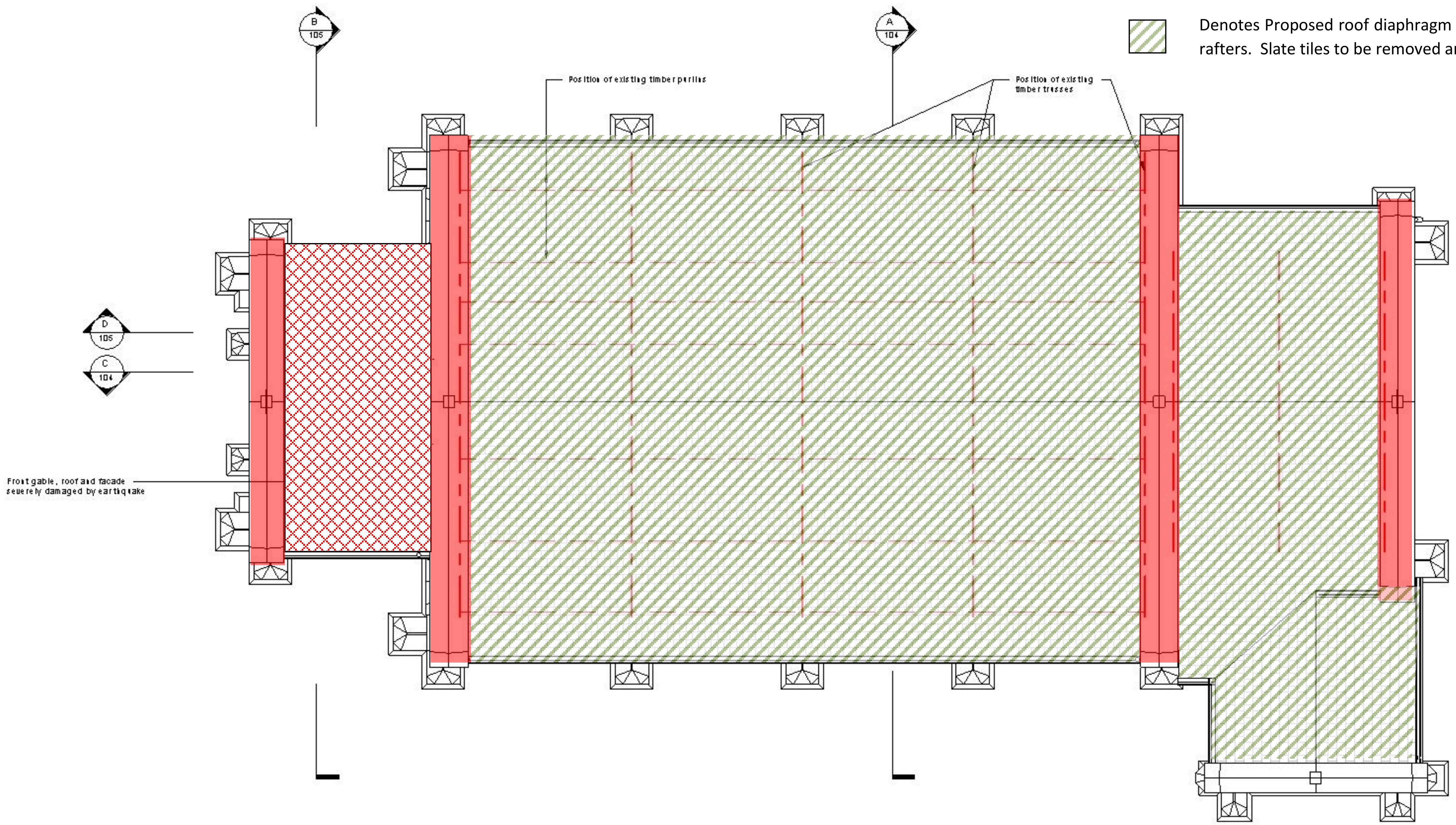
	Denotes Proposed roof diaphragm constructed from 21mm ply secured to rafters. Slate tiles to be removed and plywood installed



OPUS

Appendix 2

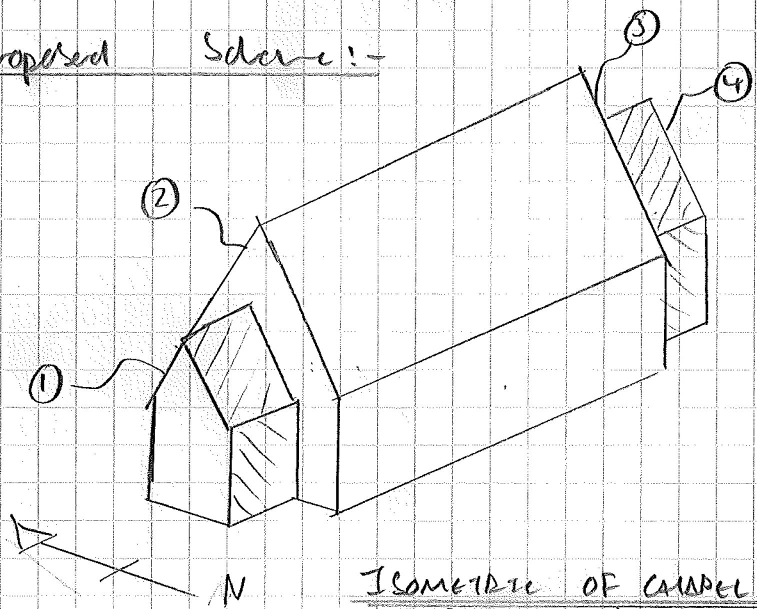
STRUCTURAL CALCULATIONS

Calculation Sheet

Project/Task/File No:	THE ROSE CHAPEL	Sheet No 01	of
Project/Description:	STRENGTHENING SCHEME	Office: CHCH	
		Computed: ADL	27/03/12
		Checked:	/ /

DESIGN THE STRUCTURE TO RESIST THE APPLIED SEISMIC LOADS NZS 1170.5.

Proposed Scheme :-



1. Provide Diaphragm in roof
2. Transmit loads to wall under
3. Design RC SHEAR WALL to resist seismic loads.

ISOMETRIC OF CHAPEL
Proposed Strengthening

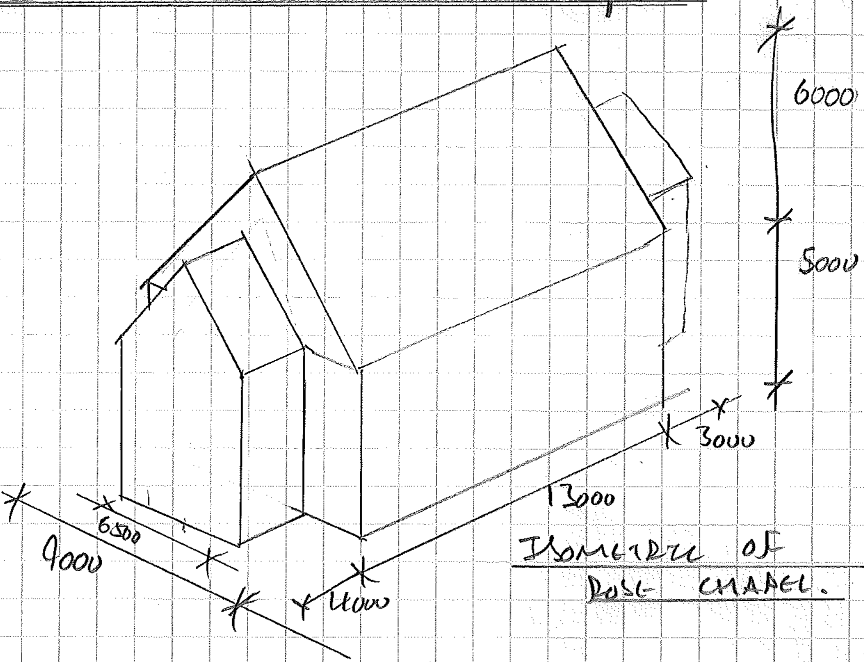
Determine seismic loads assume that new roof diaphragm will need to resist 3 gable walls. determine weight of gables. Determine the existing seismic capacity of the building.

CSF 400 (7/2000)

Calculation Sheet

Project/Task/File No: THE ROSE CHAPEL	Sheet No 02 of
Project/Description: STRUCTURAL CAPACITY CALCULATIONS	Office: CHCH
	Computed: ADL 22/03/12
	Checked: / /

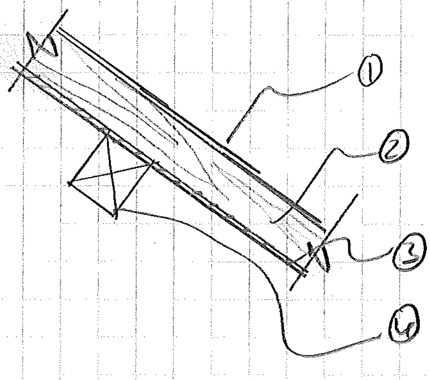
Determine the in & out-plane response of the walls in the rose chapel.



ISOMETRIC OF ROSE CHAPEL.

DETERMINING LOADS ACTING ON AND SELF WEIGHT OF THE CHAPEL ∅ NZS 1170.5/1

Roof :-



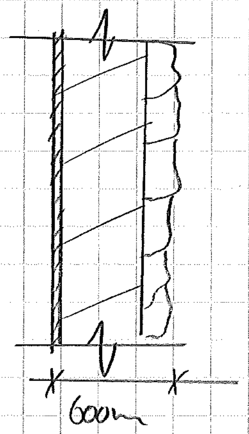
	kN/m²
1 SLATE TILES	: 0.5
2 Rafters	: 0.1
3 T&G BOARDING	: 0.1
4 Purlins	: 0.1
	0.8

CONVERT TO PLAN $= \frac{0.8}{\cos 50} = 1.25\ kN/m^2$

Calculation Sheet

Project/Task/File No:	THE ROSE CHAPEL	Sheet No	03 of
Project/Description:	STRUCTURAL Capacity Calcs	Office:	CHCH
		Computed:	ADR 22/03/12
		Checked:	/ /

WALLS:-



SAY Brick masonry @ 11.4 kN/m^2

CSF 400 (7/2000)

Calculation Sheet

Project/Task/File No:	THE POST CHAPEL	Sheet No 04	of
Project/Description:	SEISMIC WEIGHT	Office: CHCH	
		Computed: ARL	27/03/12
		Checked:	/ /

Determine the Seismic weight of the gable walls:-

GABLE 1 = 17.8 m² - Rose window (3.6 m²)

= 14.2 m²

GABLE 2 = 28.2 m²

GABLE 3 = 28 m²

GABLE 4 = 11 m²

Determine the Seismic weight & the Shear force to be resisted:-

Take density as 20 kN/m³ :-

Dead loads:-

Gable 1 : 20 × 14.2 × 0.45 = 127.8 kN

Gable 2 : 20 × 28.2 × 0.45 : 253.8 kN

Gable 3 : 20 × 28 × 0.45 : 252 kN

Gable 4 : 20 × 11 × 0.45 : 99 kN

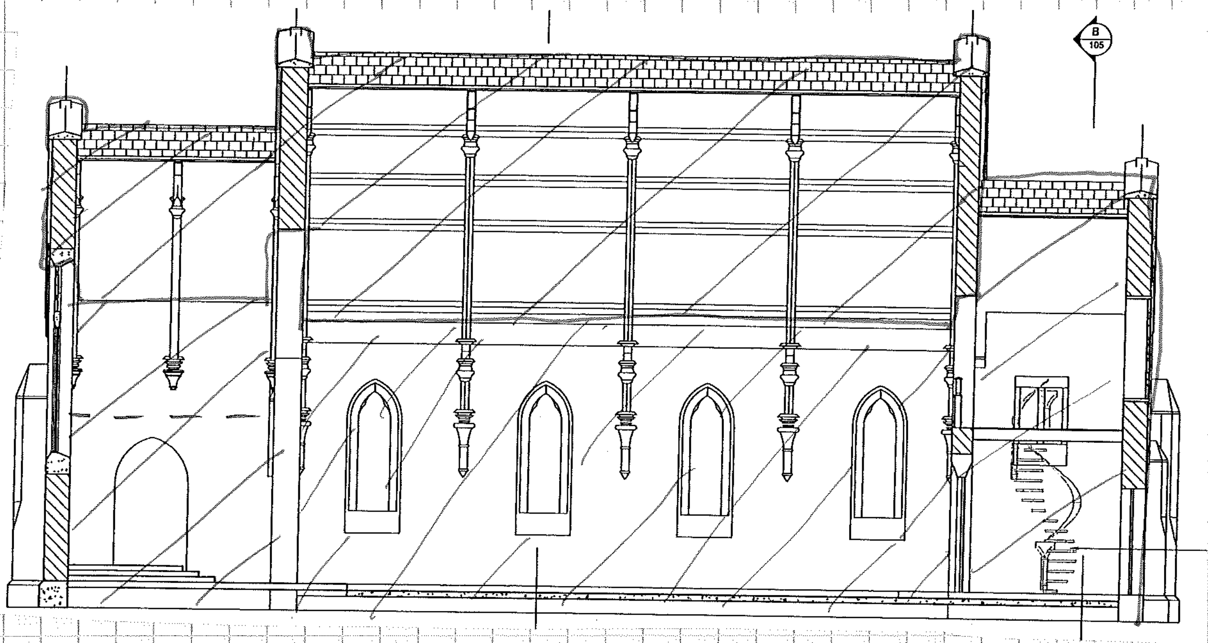
733 kN

OPUS

CSF 400 (7/2000)

Calculation Sheet

Project/Task/File No: THE ROSE CHAPEL	Sheet No 05 of
Project/Description: EQUIVALENT STATIC METHOD	Office: CHCH
	Computed: ADL 28/03/12
	Checked: / /



GENERAL ELEVATION of ROSE CHAPEL

DETERMINE SEISMIC WEIGHT OF ENTIRE STRUCTURE:-

$$W_{t,\,Roof} = \left[(6.5\times 4)+(13\times 9)+(3\times 6)\right]1.25$$
$$= 1691.4\ \text{kN}$$

$$W_{t,\,walls} = .5\left[(13+1.25+1.5)2+(6.5*4+4)+(6+3+3)\right]11.4$$
$$+\ \text{GABLES} = 3306 + 733 = 4039\ \text{kN}$$

Determine the base shear from Spreadsheet:-

$V = 3635$ kN at base.

OPUS

SHEET 06

OPUS

Job Title: *The Rose Chapel*
Job Number: **Member Reference:**
Calcs By: *ARL* **Date:** *25/05/2012 8:14:31 a.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version **1.1**

HORIZONTAL SEISMIC SHEAR (V) TO NZS 1170.5

Input Data
Period (T) = 0.3775 sec
Site Classification D
Equivalent Static Method
Hazard Factor (Z) (See Table 3.3) = 0.3
Importance level of 2
Design Working Life of 50 Years
ULS Ductility (mu) = 1
SLS1 Ductility (mu) = 1.0
ULS Structural Performance Factor (Sp) = 1.0
SLS Structural Performance Factor (Sp) = 0.7
Seismic Weight (Wt) = 4039 kN

ULS Results
ULS Return Period of 1/500
Spectral Shape Factor Ch(T) = 3.000
Return period factor from table 3.5 (Ru) = 1.00
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9000
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.900
Horizontal Seismic Shear = **3635** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.2250
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.158
Horizontal Seismic Shear = **636** kN

Calculation Sheet

Project/Task/File No:	The Rose Chapel	Sheet No 07	of
Project/Description:	look at the in & out plane capacity of Nalls walls	Office: CHCH	
		Computed: ADI	18/05/12
		Checked:	/ /

Determine the existing Masonry & mortar properties

ϕ NZSEE Detailed assessment of unreinforced masonry buildings.
Section 8

Masonry properties:-

Brick compressive strength $= 21.8\,N/mm^2$

Mortar " " " $= 7.4\,N/mm^2$

$\rho_m = 1578 + (5 \times 21.5) + (8 \times 7.4) = 1745\,N/mm^3$

ϕ Section 8.3:- In-plane wall properties:-

$\gamma_m = \rho_m \times 9.81 = 1745 \times 9.81 = 17{,}118.45$

$f'_m = 0.7 \times 21.8^{0.75} \times 7.4^{0.3}$

$= 6.9 \times 1.82 = 12.6\,MPa$

Modulus of elasticity:-

$E_m = 300 f'_m = 3780\,MPa$

Flexural bond strength:-

$f'_{fb} = 0.025 f_j = 0.19\,MPa$

Cohesion, $C = 0.045 f_j = 0.33\,MPa$

Coefficient of friction $= 0.65$

OPUS

Calculation Sheet

Project/Task/File No: The Rose Chapel	Sheet No 08 of
Project/Description: In & out of plane	Office: CHCH
	Computed: ADR 18/05/12
	Checked: / /

Work out the typical in plane response of the Nave walls.

Wall:-
height, h : 5000 mm
t : 200 mm $\quad W_{wall}$: 5 x 2 x 11.4 = 114 kN

Roof:-
Span, S : 4500 mm
length, L : 2000 mm

W_{roof} : 1.25 x 4.5 x 2 = 11.25 kN

Determine horizontal seismic load acting on Nave wall:-

Roof weight : 169.1 kN from previous sheet
Gable walls : 733 kN "
Ring Beam : (13+9)2 x 3.22 = 141.7 kN " / W_t = 1045 kN

Determine shear at top of assessed in-plane wall,

V: 1535 kN ∴ per pier $\frac{944}{10}$ = 94.4 kN

See attached calculation sheet 09 10

% NBS 100

per pier $\frac{419}{10}$ = 41.9 kN

OPUS

CSF 400 (7/2000)

Sheet 09

OPUS

Job Title: *The Rose Chapel*
Job Number: **Member Reference:**
Calcs By: *ARL* **Date:** *25/05/2012 8:19:43 a.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version 1.1

HORIZONTAL SEISMIC SHEAR (V) TO NZS 1170.5

Input Data
Period (T) = 0.3775 sec
Site Classification D
Equivalent Static Method
Hazard Factor (Z) (See Table 3.3) = 0.3
Importance level of 2
Design Working Life of 50 Years
ULS Ductility (mu) = 1
SLS1 Ductility (mu) = 1.0
ULS Structural Performance Factor (Sp) = 1.0
SLS Structural Performance Factor (Sp) = 0.7
Seismic Weight (Wt) = 1045 kN

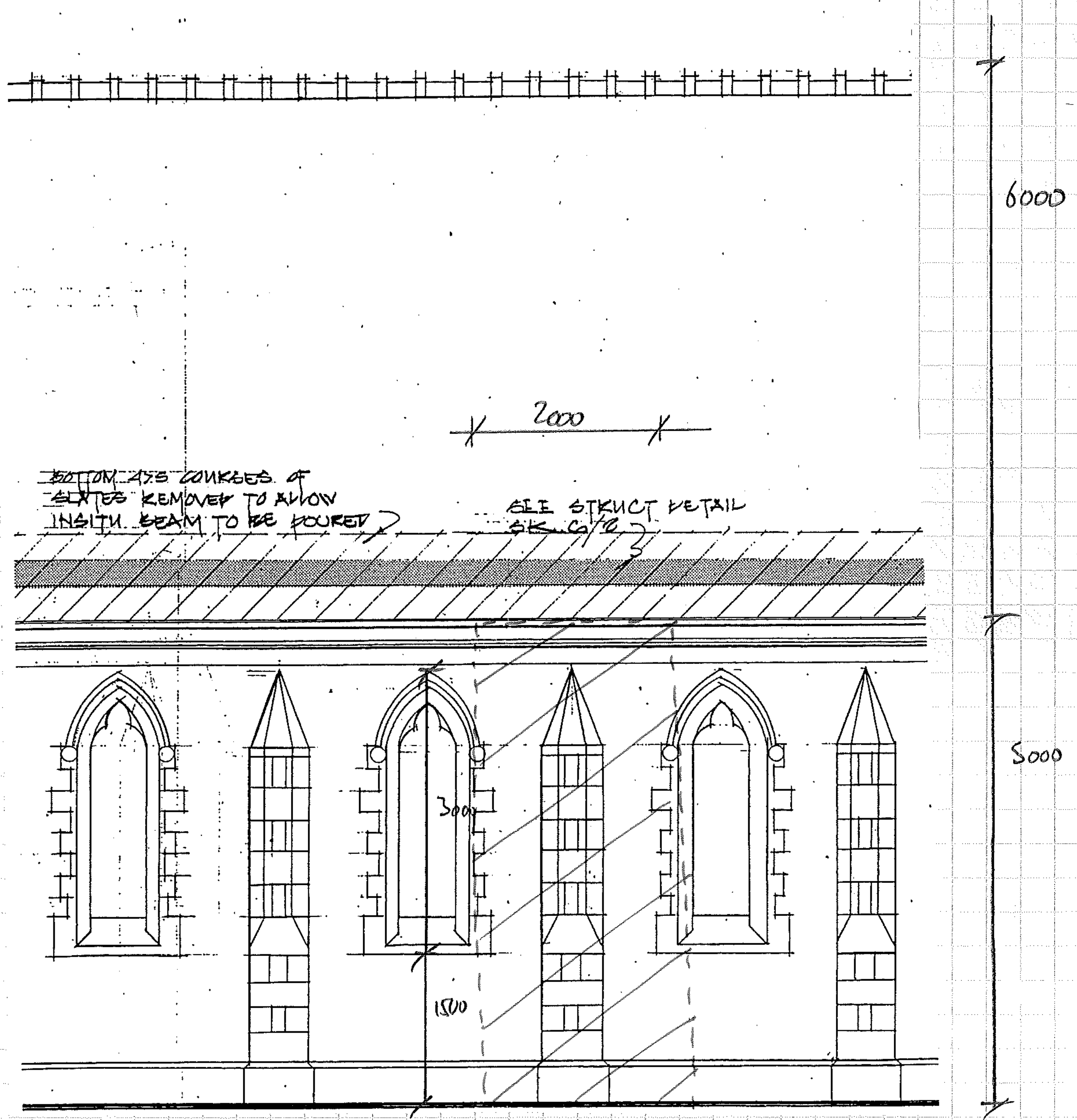
ULS Results
ULS Return Period of 1/500
Spectral Shape Factor Ch(T) = 3.000
Return period factor from table 3.5 (Ru) = 1.00
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9000
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.900
Horizontal Seismic Shear = **941** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.2250
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.158
Horizontal Seismic Shear = **165** kN

Calculation Sheet

Project/Task/File No:	THE ROSE CHAPEL	Sheet No 10	of
Project/Description:	IN & OUT OF PLANE	Office:	CHCH
		Computed: ADL	22/03/12
		Checked:	/ /

IN & OUT OF PLANE WALL STRUCTURE



ELEVATION OF THE ROSE CHAPEL

OPUS

Calculation Sheet

Project No/Reference No:		Sheet No: 11	of
Project:	The Rose Chapel	Office:	Christchurch
Element:	Design Response Of In-Plane Loaded URM	Computed:	ARL 23/05/2012
	In Plane wall calculation	Checked:	

OPUS

Ref:	Calculation:	
	<u>Wall properties</u>	
Uni of Auckland		
	Height of wall, h	3 5 m
	Length of wall, l_w	**2.00** m
cl.8.4.2	Effective length of wall, l_{er}	0.2 m
	Width of wall, b_w	**0.60** m
	Density of masonry, γ_m	**17.11** kN/m^2
cl.8.3.2	Weight of wall, W_w	102.66 kN
	Normal force acting at top of wall, N_t	**13** kN
cl.8.3.4	Normal force acting at bottom of wall, $N_b = N_t + W_w$	115.66 kN
	Cross-sectional area of wall, $A_w = l_w b_w$	1.20 m^2
cl.8.3.7	Average masonry compressive strength, $f'_m = 0.7f'_b{}^{0.75} f'_j{}^{0.3}$	18.6 MPa
	Cohesion, $c = 0.045f'_j$	0.33 MPa
	Coefficient of friction, μ_f	0.65
	<u>Diagonal Tension Strength of Masonry</u>	
cl.8.3.5	$f_{dt} = 1/2\ (c + N_t/A_w \times 0.8\mu_f)$	167.8 kPa
	<u>Distance to centre of intertia of wall</u>	
cl.8.3.6	$a = 0.5l_w$	1 m
	<u>Average compressive stress</u>	
cl.8.3.7	$\sigma = N_t / l_w b_w$	10.8333 kPa
	<u>Capacity in Diagonal Tensile Failure Mode</u>	
cl.8.4.1	$V_{dt} = 0.54 b_w l_w \zeta f_{dt} \sqrt{(1+\sigma_{avg}/f_{dt})}$	168.3 kN
	<u>Capacity in Rocking Failure Mode</u>	
cl.8.4.2	Nominal shear capacity, $V_r = N_b/h\ (a - l_{er}/3)$	See hand calcs ~~21.6 kN~~
	<u>Capacity in Bed-Joint Sliding Failure Mode</u>	
cl.8.4.3	$V_s = l_w b_w c + 0.8\mu_f N_t$	402.8 kN
	<u>Capacity in Toe Crushing Failure Mode</u>	
	Effective length for toe crushing, $l_{etc} = 2N_b / 1.3f'_m b_w$	0.016 m
cl.8.4.4	$V_{tc} = N_b/h * (0.5l_w + 0.33l_{etc})$	23.3 kN
	<u>Nominal Shear Capacity</u>	
cl.8.4	$V_n = min(V_{dt}, V_r, V_s, V_{tc})$	21.6 kN

Calculation Sheet

Project/Task/File No: The Rose Chapel	Sheet No 12 of
Project/Description: In plane wall response	Office: CHCH
	Computed: ARA 21/05/12
	Checked: / /

Determine a_i of Equation 8-6

$$a_i = \frac{(\Sigma w_{fi} \times a_{fi}) + 0.5 \times W_w \times L_w}{\Sigma w_{fi} + W_w}$$

$W_{fi} = 0.6 \times 0.5 \times 17.11 \times 5 = 25.7$ kN

$a_{fi} = 1$m

$$a_i = \frac{(25.7 \times 1) + 0.5 \times 103 \times 2}{25.7 + 103}$$

$= 41.9$

Capacity in Rocking failure mode:-

$$V = 25.72 \times \left(41.9 - 24 \frac{0.2}{1.3}\right) = 107.6 \text{ kN}$$

$$\% NBS = \frac{100}{94} \times 107.6 = 114\%$$

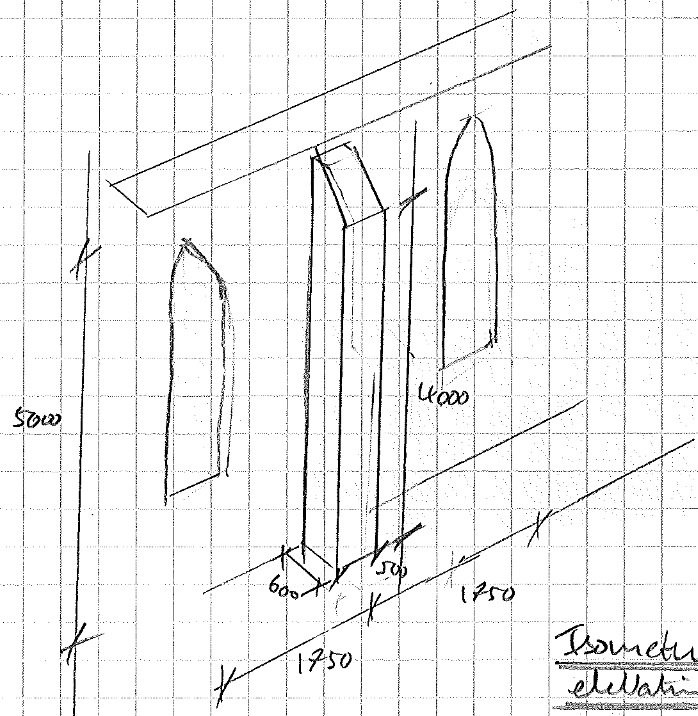
Determine Toe crushing capacity:-

$$\% NBS \quad \frac{100}{41.9} \times 23 = 55\%$$

Calculation Sheet

Project/Task/File No:	The Rose Chapel	Sheet No 13	of
Project/Description:	Determine the out plane capacity of the wall.	Office: CHCH	
		Computed: ADL	22/05/12
		Checked:	/ /

Determine the out of plane capacity of the masonry wall of NZSEE Assessment (A) improvement of URM buildings for EQ Assessment.



Isometric of typical elevation of wall

Determine loads:-

Dead:

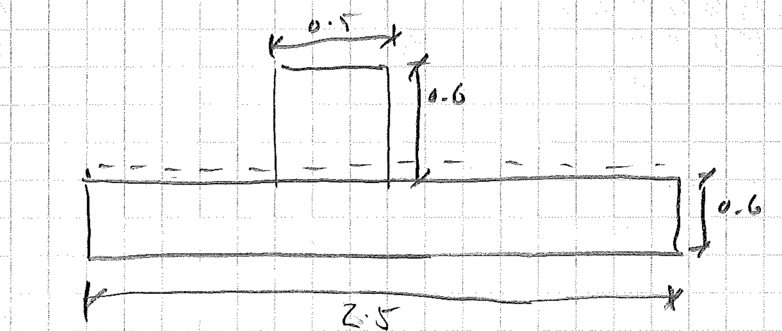
Wall, P = 0.6 x 5 x 3.5 x 17.1 = 180 kN
Buttress = 0.6 x 0.5 x 4 x 17.1 = 20.5 kN
Parapet = 0.6 x 0.2 x 3.5 x 17.1 = 7.18 kN

Roof = $\frac{4.5}{2}$ x 3.5 x 1.25 = 9.84 kN

Calculation Sheet

Project/Task/File No: The Rose chapel	Sheet No 14 of
Project/Description: Determine out of plane capacity of church wall	Office: Chch
	Computed: ABL 25/05/12
	Checked: / /

Determine equivalent width of masonry wall:-



Determine equivalent thickness!-

∴ Width = 615mm.

Wt of wall = $2.5 \times 0.615 \times 5 \times 17 = 131$ kN

See attached spreadsheet Sheet No. 15

$\Delta_m = 584$ mm

take $T = 1.5$ sec for midspan deflection.

$Z = 0.3$

$C_p(T_p) = C(0)\, C_{Hi}\, C_i(T_p)$

$C_0 = 1.5$

$C_{Hi} = \left(1 + \frac{5}{6}\right) = 1.83$

$C_i(T_p) = 2(1.75 - 1.5) = 0.5$

$C_p(T_p) = 1.5 \times 1.83 \times 0.5 = 1.37$

$D_{ph} = 1.14\left(1.5/2\times\pi\right)^2 \times 1.37 \times 1 \times 9.81 = 873.2$

NBS: 48%

OPUS

CSF 400 (7/2000)

Calculation Sheet

Project No/Reference No:	6-G1340.00/330YC	Sheet No: 15	of
Project:	St. Joseph's, Papanui	Office:	Christchurch
Element:	Out-of-plane strength of an unreinforced masonry wall subject to seismic loading	Computed:	JAS 9/12/2010
		Checked:	

OPUS

Ref:	Calculation:			Output:
	Wall parameters ◉ Rectangular panel ○ Gable panel			
NZSEE				
Table 10.3, Figures 10A.1, 10.1	Effective panel height, h	**5**	m	
	Nominal thickness of top part of wall, t_{nom}	**615**	mm	
	Nominal thickness of bottom part of wall, t_{nom}	**615**	mm	
	Self-weight of top part of wall, W_t	**65**	kN	
	Self-weight of bottom part of wall, W_b	**65**	kN	
	Weight acting on top of the wall, P	**16.36**	kN	
	Effective thickness of top part of wall, t	595.8	mm	
	Effective thickness of bottom part of wall, t	595.8	mm	
	Eccentricity of P to top centroid, e_p	**297**	mm	
	Eccentricity of bottom pivot to bottom centroid, e_b	**297**	mm	
	Eccentricity of mid pivot to top centroid, e_t	**297**	mm	
	Eccentricity of mid pivot to bottom centroid, e_o	**297**	mm	
	Height of centroid of W_b from pivot at bottom of panel, y_b	**1250**	mm	
	Height of centroid of W_t from pivot at the top of panel, y_t	**1250**	mm	
NZSEE	**Mid-height deflection**			
cl. 10A.2.6	ψ assumed inter-storey drift	**1**	%	
Eq. 10(8)	b	95030680	Nmm	
Eq's 10(9), 10(21)	a	406800000	Nmm	
Eq. 10(7)	Mid-height deflection, Δ_i	584.0	mm	
cl. 10.3.4 a) 6	Maximum usable deflection, $\Delta_m = 0.6\Delta_i$	350.4	mm	
	Period of the wall			
	Rotational inertia of top part of wall panel, J_{to}	3646989	kg.mm^2	
	Rotational inertia of bottom part of wall panel	3646989	kg.mm^2	
NZSEE	Additional inertia (e.g. Veneers), J_{anc}	**0**	kg.mm^2	
Eq's 10(11), 10(22)	Rotational inertia of wall panel, J	36198202	kg.mm^2	
Eq. 10(10)	Period of the wall, T_p	1.9	s	
NZS1170.5	**Earthquake demand (for a part)**			
	Location	**Arrowtown**		
1170.5 Table 3.3	Z Value	0.3		
1170.0 cl. 3.4	Required Annual Probability of Exceedance	**1/500**		
1170.5 cl. 3.1.6	Near fault factor, $N(T,D)$	**1.00**		
1170.5 Table 3.5	Return Period Factor, R_u	1		
1170.5 cl. 2.2.4	Ductility factor, μ	1.0		
1170.5 cl. 4.4	Structural Performance Factor, S_p	1.0		
1170.5 cl. 3.1.3	Site subsoil class	**D**		
1170.5 cl. 3.1.2	Spectral shape factor, $Ch(0)$	1.12		
1170.5 cl. 3.1.1	Site hazard coefficient, $C(0)$	0.336	g	

Calculation Sheet

Project/Task/File No: The Rose Chapel

Project/Description: Determine the out of plane capacity of the walls wall

Sheet No 16 of

Office: CHCH

Computed: ADL 25 /05 /12

Checked: / /

Check out of plane capacity of wall near the cross walls :-

Take $T = 0.3s$

$C_0 = 3$

$C_{Hi} = 1$

$C_i(T_p) = 0.5$

$\therefore \ C_p(T_p) = 3 \times 1.83 \times 0.5 = 2.75$

$D_{ph} = 1.14\left(1.5/(2\times\pi)\right)^2 \times 1.5 \times 1 \times 9810 = 956$

$\therefore$ NBS = 22% — compared with usable deflection from spreadsheet. see sheet 17

CSF 400 (7/2000)

Calculation Sheet

Project No/Reference No:	6-QUAKE	Sheet No: 17	of
Project:	St. Joseph's, Papanui	Office:	Christchurch
Element:	Out-of-plane strength of an unreinforced masonry wall subject to seismic loading	Computed:	JAS 22/11/2010
		Checked:	

OPUS

Ref:	Calculation:		Output:
	Parapet parameters		
	Effective panel height, h	**5** m	
	Length of panel, L	**2.5** m	
	Nominal thickness of wall, t_{nom}	**615** mm	
	Effective thickness, t	603 mm	
	Self-weight of wall, W	**131** kN	
	Weight acting on top of the wall, P	**16.36** kN	
	Eccentricity of pivot to centroid, eb	302 mm	
	Height of centroid of W_b from pivot, y_b	**2500** mm	
	Instability deflection		
	Instability deflection, Δ_i	301.5 mm	
	Maximum usable deflection, $\Delta_m = 0.6\Delta_i$	180.9 mm	
	Period of the wall		
	Period of the wall, T_p	1.6 s	
	Earthquake demand (for a part)		
	Location	**Darfield**	
	Z Value	0.3	
	Required Annual Probability of Exceedance	**1/1000**	
	Near fault factor, $N(T,D)$	**5.00**	
	Return Period Factor, R_u	1.3	
	Ductility factor, μ	1.0	
	Structural Performance Factor, S_p	1.0	
	Site subsoil class	**D**	
	Spectral shape factor, $Ch(0)$	1.12	
	Site hazard coefficient, $C(0)$	~~2.184~~ g 3	
	Height of the attachment of the part, h_i	**0.00** m	
	Height from the base of the structure to the uppermost seismic weight or mass, h_n	**11.00** m	
	Floor height coefficient, C_{hi}	1.00	
	Part spectral shape coefficient, $C_i(T_p)$	0.5	
	Design response coefficient for part, $C_p(T_p)$	~~1.092~~ g 1.5	
	Participation factor for the rocking system, γ	1.48	
	Part risk factor, R_p	**0.9**	
	Displacement response, D_{ph}	~~925.3~~ mm 956	
	%NBS	$\boxed{\cancel{23.5\%}}$	$1.2 \times \frac{180.9}{956} \times 100$

= 22 %

Calculation Sheet

Project/Task/File No: The Rose Chapel	Sheet No 18 of
Project/Description: Check connection details of roof diaphragm	Office: Chch
	Computed: ADL 23/05/12
	Checked: / /

① Check roof connection to concrete ring beam :-

ϕ SEE SHEET 19 FOR SKETCH

Determine Seismic lateral load :-

Roof W_t : $13 \times 9 \times 1.25 = 140.3$ kN

Gable walls W_t : $253.8 + 252 = 506$ kN

$W_t = 652.3$ kN

Determine horizontal shear from Seismic Spreadsheet :-

V: 587 kN Split between each wall $= \frac{587}{2}$

$= 294$ kN

Determine whether Nails 3.15×60 @ 75 c/c's can resist horizontal load :-

Characteristic Strength of nail : 0.631 kN

∴ 13m @ 0.075m Nail Spacing

No. Nails: $\frac{13}{0.075}$ = 174 Nails.

∴ $174 \times 0.631 = 109$ kN.

% NBS : $\frac{100}{294} \times 109 = 37\%$

CSF 400 (7/2000)

HolmesConsultingGroup

Project Name: SISTERS OF MERCY CHAPEL

Project No: 29899

Calcs By: BAG

Date: 23/2/98

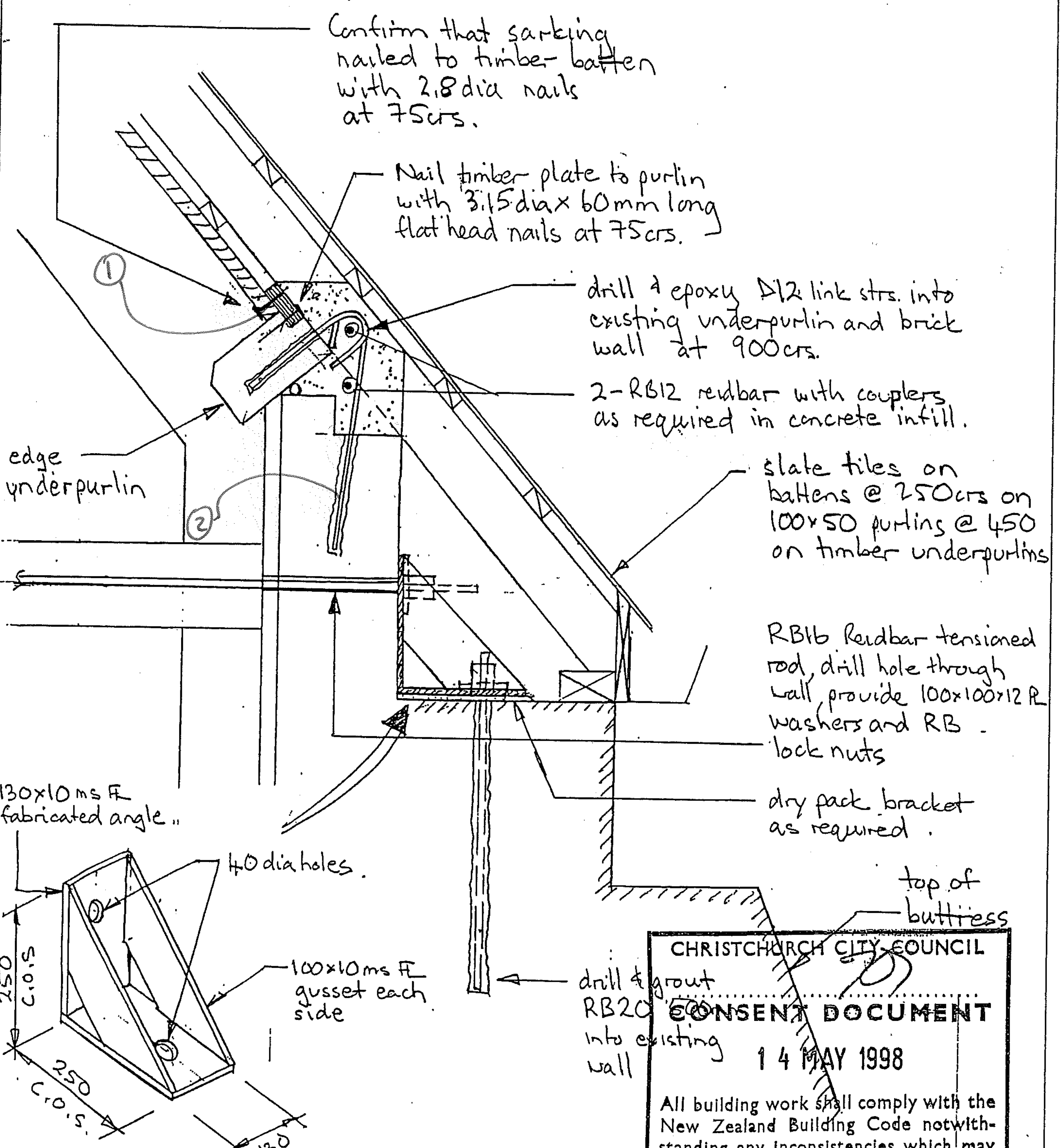
Sketch No: 6

Page No:

Revision: A

SHEET 19

CALCS/SKETCHES



2/SK1 1:10 TYPICAL NORTH AND SOUTH MAIN WALLS

Calculation Sheet

Project/Task/File No:	The Rose chapel	Sheet No: 20	of
Project/Description:	check connection to wall	Office: CHCH	
		Computed: ARL	21/05/12
		Checked:	/ /

Roof + 2 gables = load to be considered for concrete ring beam connection.

∴ Roof + gables 2+3 = (13 × 9 × 1.25) + 254 + 252
= 652.3 kN

∴ Determine Base Shear V: 587 kN.

∴ determine load per fixing. D12's @ 900 c/c

∴ $\frac{587}{30}$ = 20 kN

load per fixing assuming D12 secured with Hilti HIT HY 70 @ 900 c/c = 5 kN) allow for embedment depth.

∴ $\frac{100}{20} \times 5$: 25 % NBS

Determine shear demand of just roof:-

V: 135 kN / 2 : 70 kN/wall.

∴ load/fixing : $\frac{70}{30}$ = 2.3 kN

∴ $\frac{100}{2.3} \times 5$ = 215 %

CSF 400 (7/2000)

Calculation Sheet

Project/Task/File No: The Rose Chapel	Sheet No 21 of
Project/Description: look at diaphragm connection	Office: CHCH
	Computed: ADM 25/05/12
	Checked: / /

Look at connection between purlin and rafter and anchor D12's @ 900 c/c assume M12 Bolt into timber:-

Determine characteristic strength of 1 Bolt:-

depth of timber say 150mm.

$\therefore Q_{sk1} = 10.4\,kN$

$Q_{k1} = k_{11} f_{cj} d_a^2$

$= 2 \times 36.1 \times 12 \times 150$

$= 12.96\,kN$

Take $Q_{sk1} = 10.4\,kN$.

$\therefore$ from previous calculation $\%NBS = \frac{100}{20} \times 10.4$

$= 52\%\,NBS$.

OPUS

Calculation Sheet

Project/Task/File No:	The Rose Chapel	Sheet No	22 of
Project/Description:	Determine the flexibility of the existing roof	Office:	CHCH
		Computed:	[illegible] 24/05/12
		Checked:	/ /

Determine the flexibility of the existing roof diaphragm:-

From ϕ NZSEE Detailed assessment of timber Section 11.3.2

Take strength V to be, V = 6kN/m

① ∴ Shear force transmitted to walls = 6 x 13 = 78kN

load to transfer $\frac{1535}{2}$ = 768kN

∴ %NBS = 10%

② can roof transfer own self weight:-

Base Shear = $\frac{153}{2}$ = 76.5 kN

∴ 100% NBS

③ can roof transfer Roof & 1 gable wall

Base Shear = $\frac{378}{2}$ = 189 kN

∴ NBS = 41%

OPUS

CSF 400 (7/2000)

Appendix 3

COMPLIANCE

11 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.

11.1 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 to carry out a full structural survey before the building is re-occupied.

We understand that CERA 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). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 33% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

11.2 Building Act

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

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 the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case CCC) 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 CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone (EPB) 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 loads 33% of those used to design an equivalent new building.

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.

Section 131 – Earthquake Prone Building Policy

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

11.3 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 on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

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.

11.4 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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

11.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

12 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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2012 [2] is presented in Figure 3.1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)

Improvement of Structural Performance	
Legal Requirement	NZSEE Recommendation
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
	Not recommended. Acceptable only in exceptional circumstances
Unacceptable	Unacceptable

Figure 3: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2012 AISPBE Guidelines

Table 3.1 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).

Table 3.1: %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

12.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

12.1.1 Occupancy

- The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of “dangerous building” to include buildings that were identified as being EPB’s. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

12.1.2 Cordoning

- Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

12.1.3 Strengthening

- Industry guidelines (NZSEE 2012 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

- It should be noted that full compliance with the current building code requires building strength of 100%NBS.

[1] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

12.1.4 Our Ethical Obligation

- In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

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