Hadley & Robinson Ltd.
Consulting Civil & Structural Engineers

Report

Dunedin City Council
Fortune Theatre

Likely Performance in Earthquake

Report prepared for City Property
July 2011

Director L M Robinson BE (Hons), NZCE (Dist), FNZSEE, MIPENZ (Geotechnical, Structural), CPEng, IntPE
469 George Street, PO Box 6068, Dunedin, Phone (03) 477 8923, Fax (03) 477 0608
Email solutions@hadleyrobinson.co.nz Website: www.hadleyrobinson.co.nz

ACENZ

19 July 2011

DUNEDIN CITY COUNCIL—FORTUNE THEATRE
Likely Performance in Earthquake

Summary

- In the Building Act 2004 and its regulations, a building is defined as earthquake prone if it would collapse in a moderate earthquake, which in turn is defined as one that produces effects at the site one-third those assumed for the design of a new building.

- Analyses of this building for earthquake effects show that it is not earthquake prone.

- It has been determined that the building could sustain an earthquake with two-thirds the earthquake shaking that would be assumed for the design of a new building on this site.

Introduction

City Property of the Dunedin City Council commissioned this report to determine if occupants of this building would be safe in earthquake. For that purpose, the threshold of unacceptable danger has been taken as that associated with earthquake proneness as defined in the Building Act 2004.

Assessments therefore start at the present configuration, with any deferred maintenance that might affect performance in earthquake first completed.

If the building is assessed as earthquake prone, then no improvement is required by legislation (unless there is a change of use) but improvement might be sought nevertheless.

The building would then normally be assumed to be incrementally improved, first by improving interconnection of its elements and then by insertion of additional structure. That exercise for this building is not necessary, as the performance of the building as is would be satisfactory.

Construction and Condition

Drawings and other documents describing the construction of the building were obtained from City Property. Testing of the building materials and details supplemented this limited information.

Exterior walls are stone throughout. Interior walls are variously stone and brick.

Floors are timber. Joists and beams seat into pockets in the masonry. They are not mechanically connected.

The roof is timber framed. Trusses are connected to the exterior walls, but connection of the purlins and rafters is notional only, with simple carpentry connections (nails and the like). However, there are through rods at each gable, connecting through the roof parallel to the ridge to opposite gables, adding considerably to the security. Roof planes are sarked with tongue and grooved boarding.

Foundations are stone and concrete.

The condition of the building is reasonable, standards of maintenance not having been high during recent years. Any deferred maintenance is assumed completed for the purposes of this report.

Legal Provisions: Earthquake Proneness

The provisions of the Building Act with respect to earthquake prone buildings are noted in Appendix A. In brief, a building is defined as being earthquake prone if it would collapse in a moderate earthquake. A moderate earthquake is defined as one that would produce shaking at the site one-third as great as would be assumed for the design of a new building at the site. This is often referred to as one-third new building standard, which is imprecise but perhaps useful in understanding earthquake proneness.

If a building is assessed as being earthquake prone, the Territorial Authority may require the danger presented by that condition to be removed by demolition or by improvement of performance in earthquake within a specified time.

The Dunedin City Council Policy states that assessments would be required whenever the cost of alterations exceeds 25% of the capital value of the building, but for any buildings of this age, by 2014 in any event. The time provided to complete improvement would vary from 15 years or more, depending on the level of assessment.

That policy is presently being reviewed. The revised Policy is unlikely to require higher standards or less time to effect improvement.

Methods of Assessment

The assessment methods used in the preparation of this report are those outlined in the document "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes", which the New Zealand Society for Earthquake Engineering prepared for the Department of Building and Housing.

The NZSEE document is intended primarily for the assessment of earthquake prone buildings and the improvement of performance of a building to the extent that it would not then be earthquake prone. However, the methods are also useful in assessments for other purposes, such as improving performance to higher standards, for example to meet requirements for changes of use.

The analyses reported here were supplemented by a modal response spectrum analysis, and by displacement based approaches.

Analysis in General

The building is constructed with unreinforced masonry walls, which together dominate the mass and stiffness of the building.

The floors and the roof are not well connected to the walls. While connection would allow these elements to act as diaphragms, demands on diaphragms would be high due to the large mass of the masonry. Indeed, the rather massive walls suggest that they might be capable of resisting significant earthquake motions without the assistance of diaphragms.

A three-dimensional model of the building was prepared. It is shown in Figures 1 and 2. The roof and floor can be incorporated into the building model or excluded from it. In either case, the roof and walls can be excluded from the display to keep the presentation clutter-free. Figures 3 and 4 illustrate this.

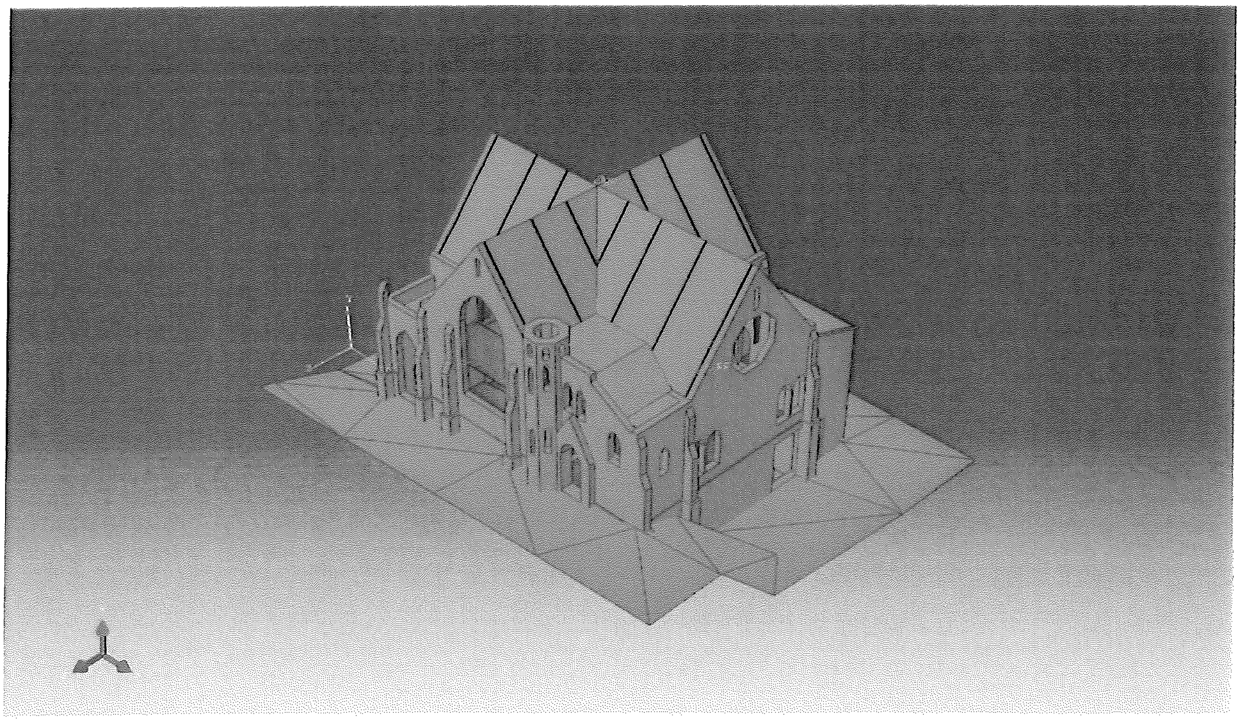

Figure 1
Isometric view of the three-dimensional model used in this study, viewed from the northwest. The roof of the turret is not shown in this view.

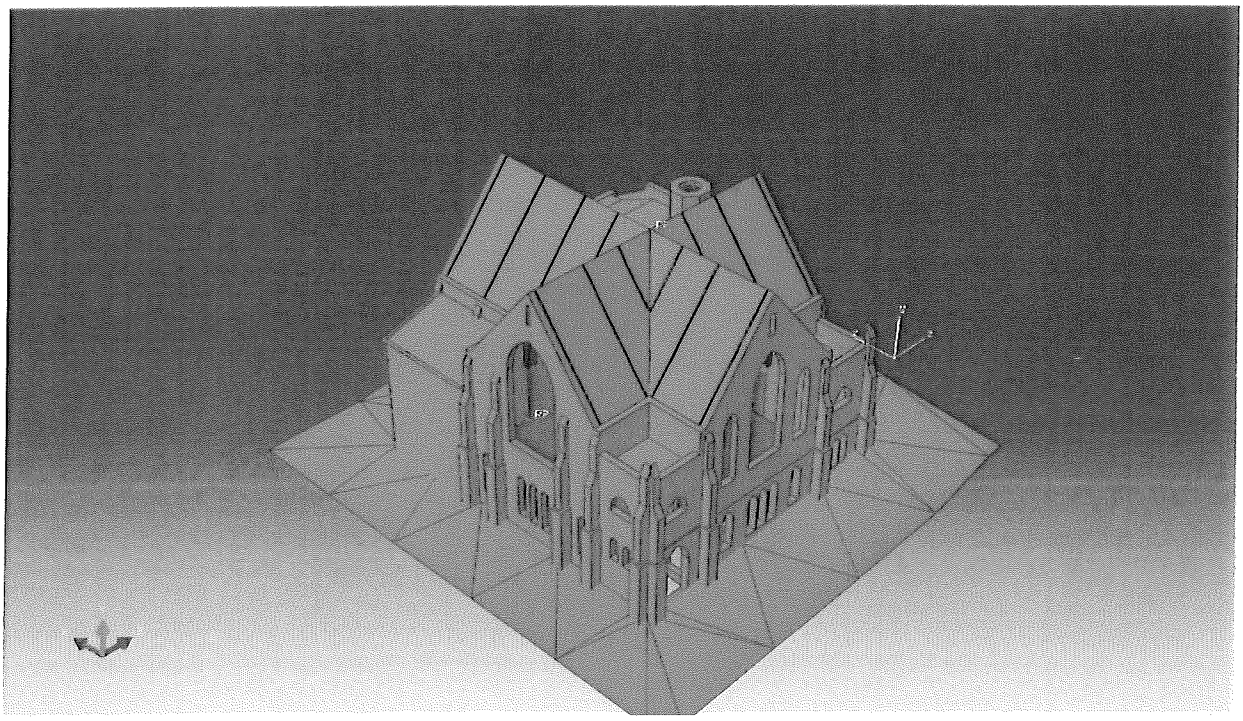

Figure 2
Isometric view of the three-dimensional model used in this study, viewed from the southeast. Not all finials are shown in their entirety in this view.

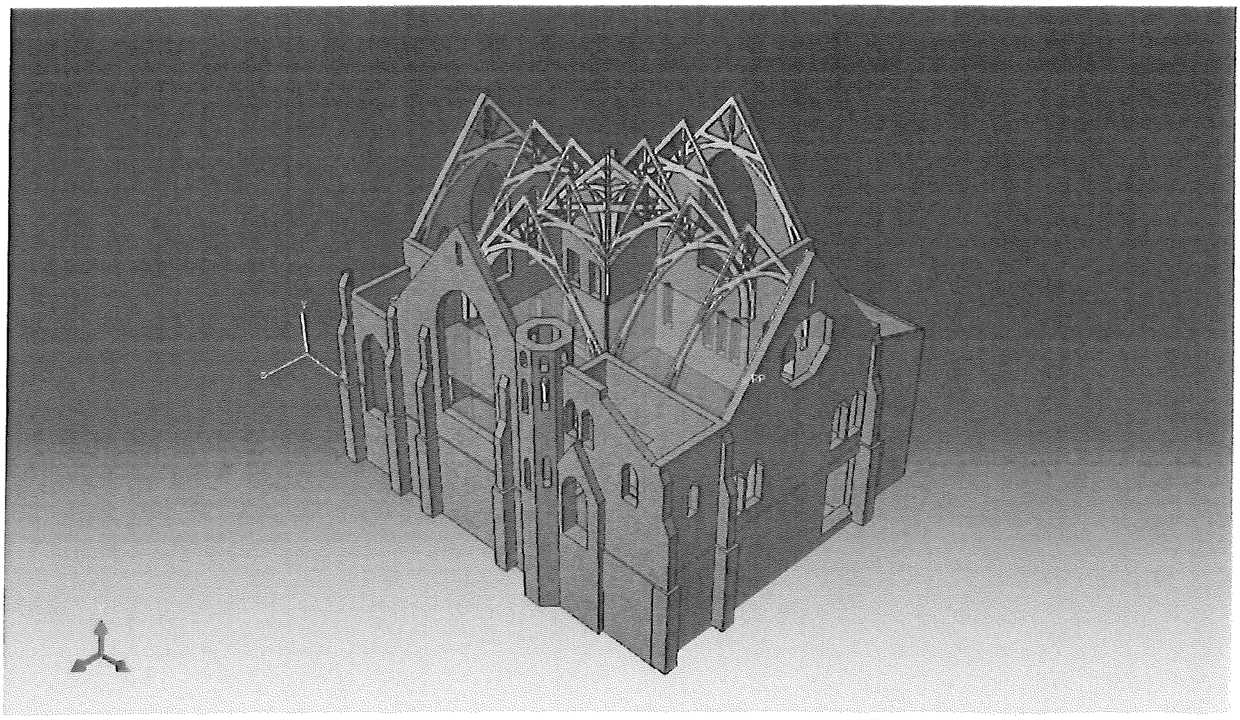

Figure 3
Isometric view of the model with the ground removed, the main roof removed, retaining the main trusses and the floors (greyed and transparent) and the walls (green).

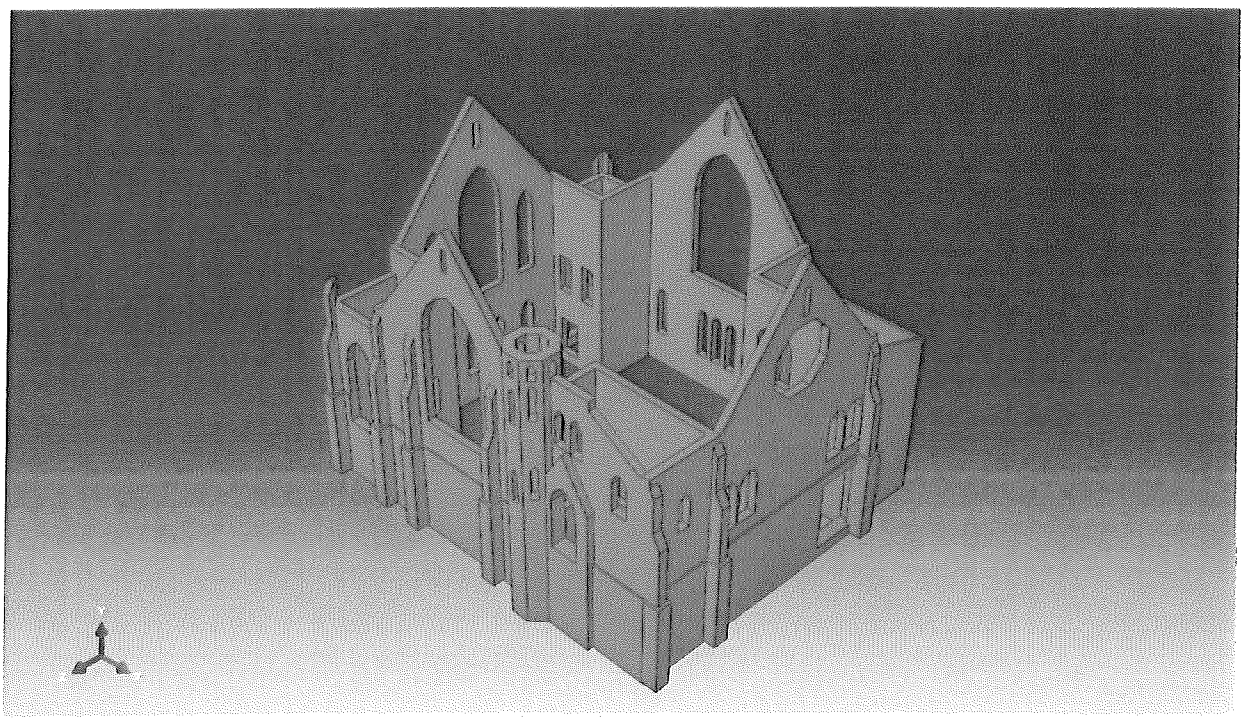

Figure 4
Isometric view of the model with the roof and all floors removed.

The purpose of the model is to explore the earthquake prone potential of the building, and to analyse incremental improvement measures—though, as subsequently described, it is scarcely necessary for that purpose.

In the analyses, the strength of the masonry was set using a Mohr-Coulomb approach as in the NZSEE Guidelines. However, to overcome known deficiencies associated with excess dilatancy, actual implementation was carried out using a Drucker-Prager approach.

Relationships are provided in Appendix C. For this building mid-range values were input consistent with the state of the bricks and mortar. For example, for failure along horizontal planes (courses), the friction angle was input at 22 degrees (coefficient of 0.4) and cohesion of 100 kPa. Friability of the mortar was taken into account by assessing progressive damage. Similarly cautious values were input for alternative failure planes at 45 degree intervals from the bedding planes.

Design Parameters

As a precursor to the analysis for earthquake effects, certain basic parameters are assessed as in Appendix B.

For the equivalent static procedure, the seismic coefficient is about *0.20 g*. It should be noted that strict application of NZS 1170.5 for earthquake analysis of this building would prohibit use of the equivalent static procedure. Accordingly the modal responses spectrum method would need to be used. However, the NZSEE Guidelines soften the restrictions somewhat. Coefficients for the modal response spectrum method are given in Appendix B.

The test for earthquake proneness need only assume 1/3 the levels stated in Appendix B (*0.065 g* for the equivalent static procedure, $0.043\ C_d(T)$ for the modal response spectrum procedure).

It is sufficient compliance with the legislation to upgrade to meet the test for earthquake proneness if it were reapplied. Where a higher level of upgrading is be considered, such as for a change of use, or just from considerations of additional safety or property protection, there is a question of what constitutes a "reasonably practicable" level of shaking to assume, taking into account heritage values of the building and how these might be jeopardised by inappropriate intervention into the existing fabric. It is often assumed that 67% is a reasonable level for this type of building, and for this report that is what is targeted.

Overall Analysis

The first analyses were conducted on the building as it presently is. Stability was assumed provided by intersecting and return walls and from the roof ties at gables. Floors were not assumed tied. No contribution to stability from any additional structural elements was assumed.

Preliminary analyses suggested that the building is not earthquake-prone. It was therefore decided to move directly to twice the test loads for earthquake proneness, i.e. to 2/3 the intensity of earthquake assumed for the design of new buildings.

Only results from a static pushover analysis are reported here. Results are shown in the following figures for each of the two principal directions. Only walls are included in the output. Three quantities are output in these figures, the areas of cracking at the end of the run, the displacement magnitudes, and the maximum shear stresses in the appropriate planes. Note that the earthquake excitation for the Z direction is assumed to follow directly on from an earthquake in the X direction. Hence, there may be an apparent lock-in of deflections.

Output is plotted on the deformed geometry. Undeformed geometry is shown for reference.

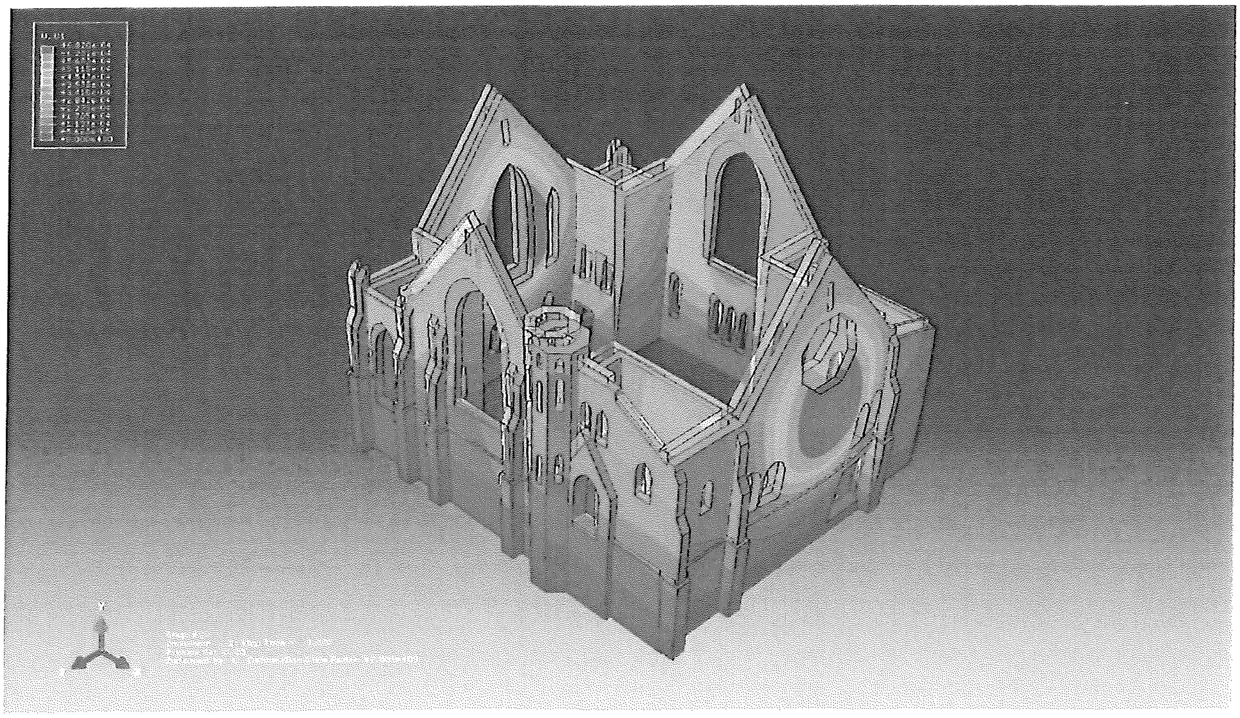

Figure 2.1
This figure shows the magnitude of deflections when the earthquake effects are in the X direction.

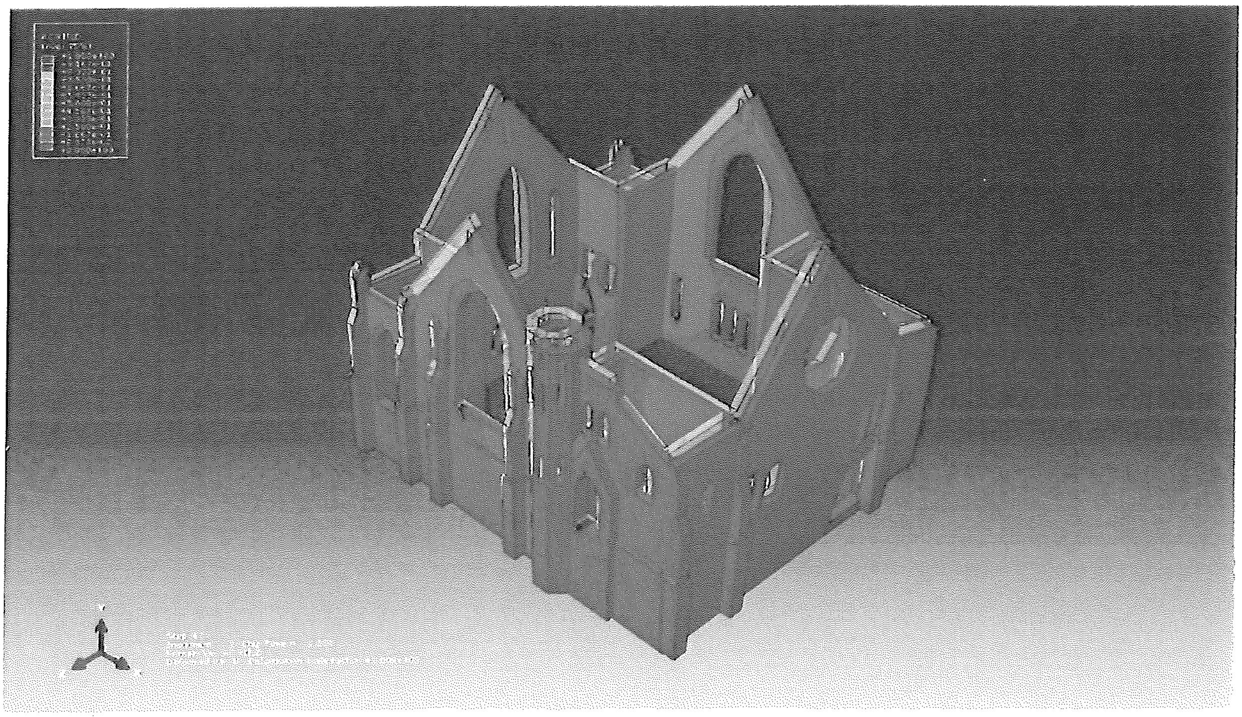

Figure 2.2
This figure shows the areas where the masonry courses open up due to the X-direction earthquake.

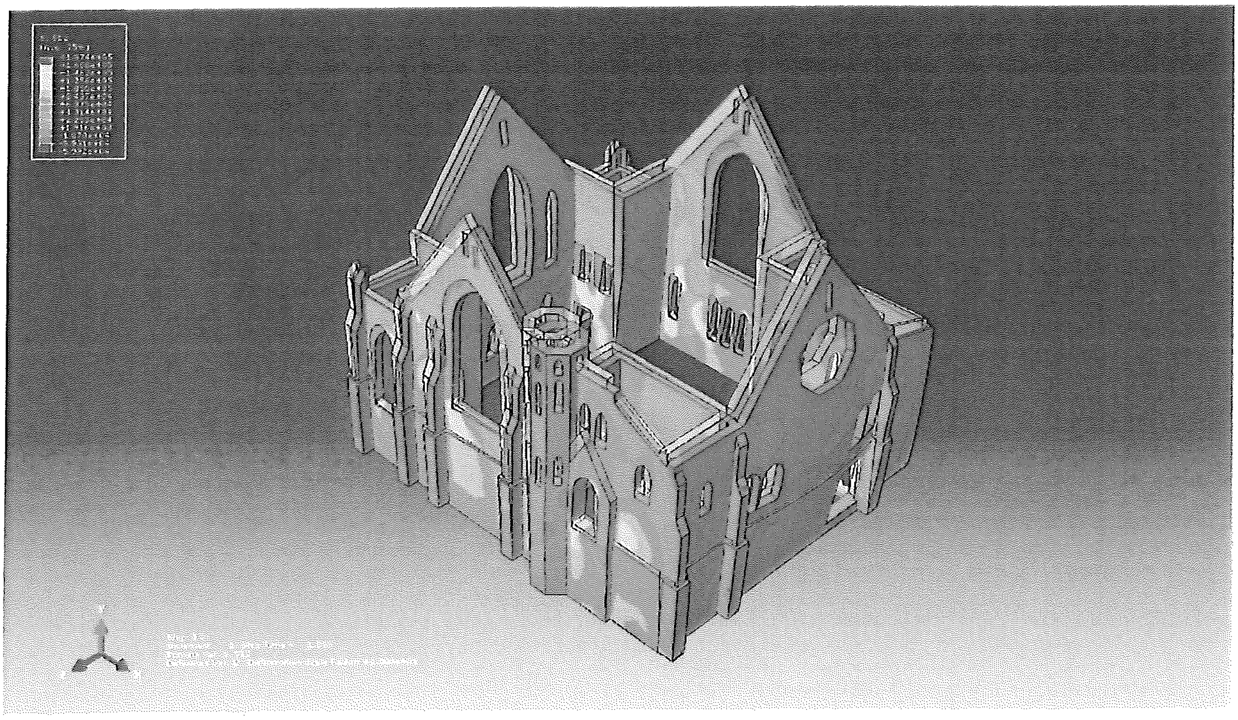

Figure 2.3
This figure shows the shear stresses S12 in the masonry (maximum 0.2 MPa) X-direction earthquake.

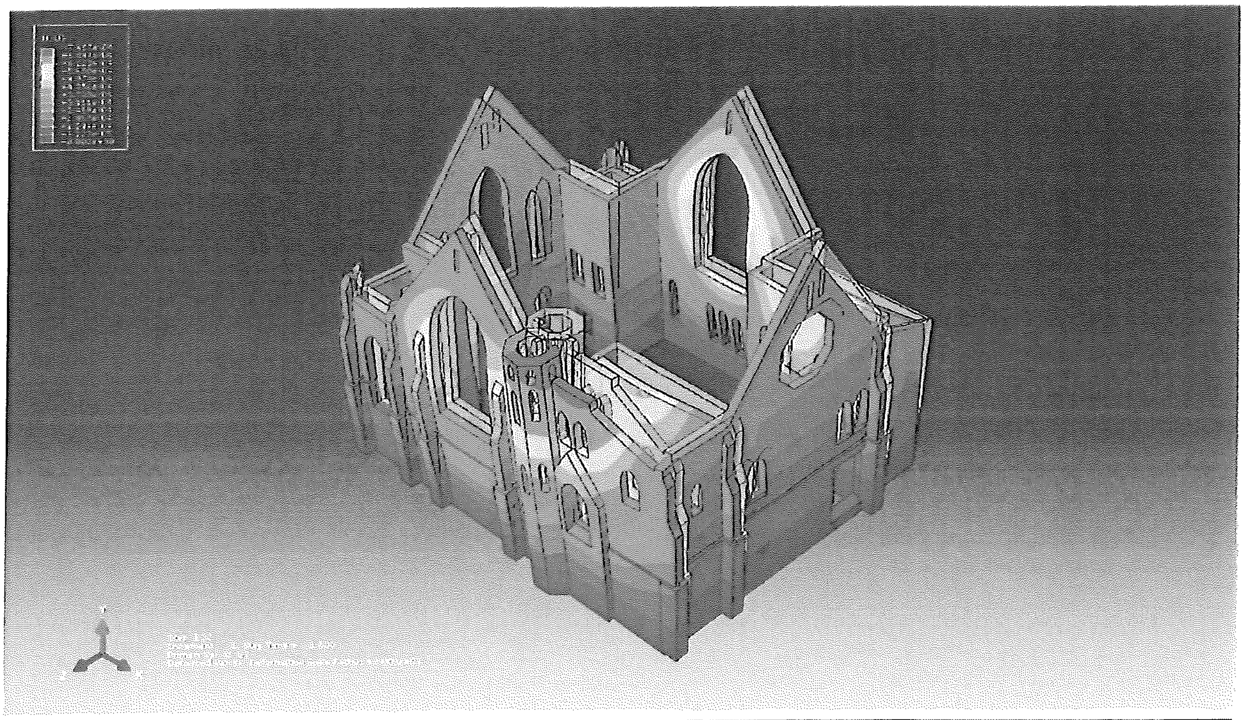

Figure 3.1
This figure shows the magnitude of deflections when the earthquake effects are in the Z direction.

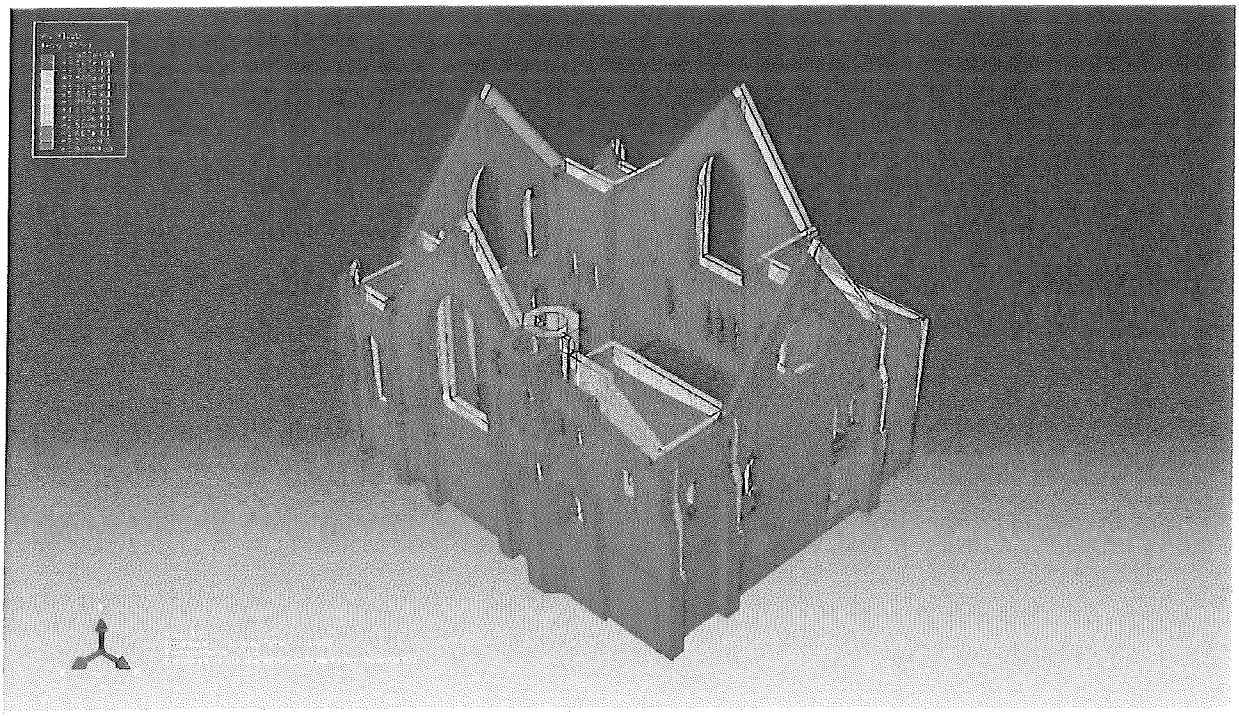

Figure 3.2
This figure shows the areas where the masonry courses open up due to the Z-direction earthquake.

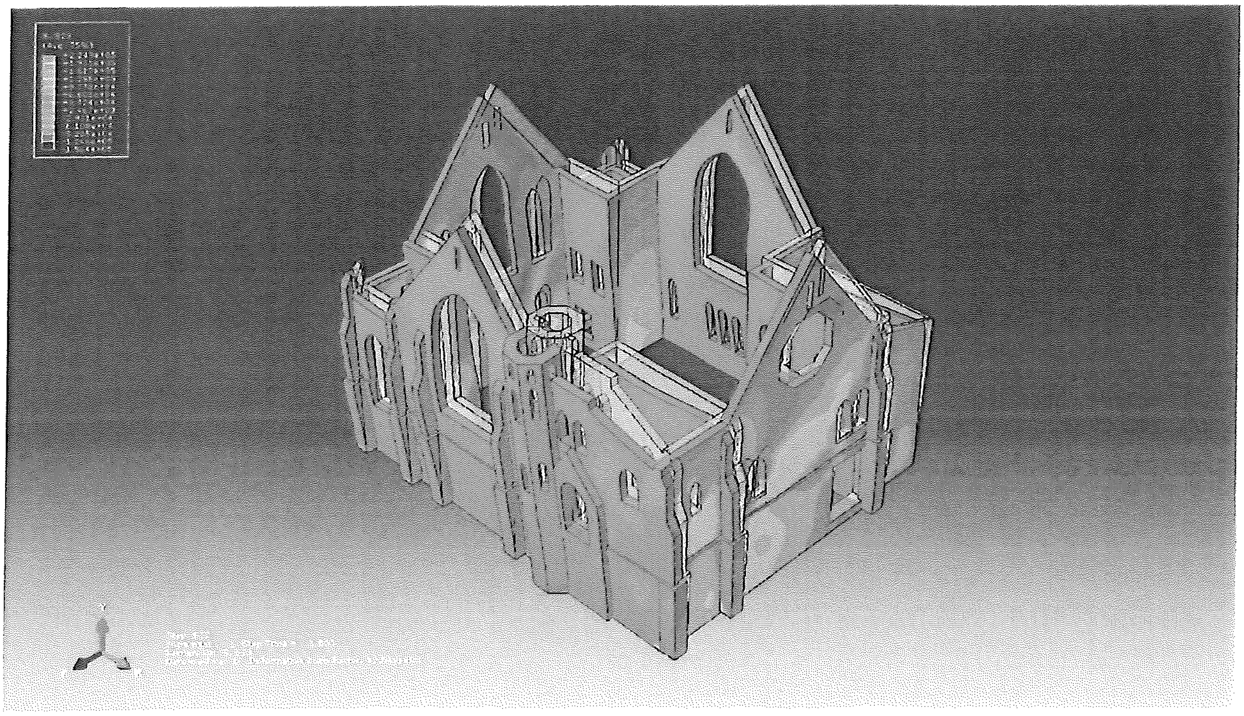

Figure 3.3
This figure shows the shear stresses S23 in the masonry (maximum 0.2 MPa) Z-direction earthquake.

It is evident that the stresses are modest. It should be noted that the results shown in the figures are under an earthquake producing 2/3 the intensity of shaking assumed for the design of new buildings. It is evident from the cracking that occurs that the building is not able to sustain greatly more than this, but the result is nevertheless most satisfactory.

The foregoing analyses use a force-based approach. As has been well established in recent years, a displacement-based approach will produce more reliable results. Alternative analyses based on that approach and developed from the NZSEE Guidelines confirm that the building can be expected to remain stable under 2/3 the intensity of earthquake assumed for the design of new buildings.

LM Robinson

DIRECTOR
Hadley & Robinson Limited
Consulting Engineers
DUNEDIN

Appendix A: Earthquake Proneness

Legal Requirements—earthquake proneness

The test for earthquake prone buildings is defined in section 122 of the Building Act 2004, and in associated regulations.

122 Meaning of an earthquake-prone building

(1) A building is earthquake prone for the purposes of the Act if, having regard to its condition and the ground on which it is built, and because of its construction, the building—

- *(a) will have its ultimate capacity exceeded in a moderate earthquake (as defined in the regulations); and*
- *(b) would be likely to collapse causing—*
  - *(i) injury or death to persons in the building or to persons on any other property; or*
  - *(ii) damage to any other property.*

(2) Subsection (1) does not apply to a building that is used wholly or mainly for residential purposes unless the building—

- *(a) comprises 2 or more storeys; and*
- *(b) contains 3 or more household units.*

The regulations referred to in s122 were promulgated in 2005/32 on 21 February 2005. Regulation 7 defines a moderate earthquake.

7. Earthquake-prone buildings: moderate earthquake defined

For the purposes of section 122 (meaning of earthquake-prone building) of the Act, moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is the same duration as, but is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used for the design of a new building at that site.

Appendix B: Earthquake Design Parameters

As a precursor to the analysis for earthquake effects using either the equivalent static or the modal response spectrum methods, eigenvalue extraction analyses were performed.

In the design of new buildings using NZS 1170.5, the seismic coefficient is derived as follows:

$$C_d(T) = \frac{C(T)S_p}{k_\mu} \geq (Z/20 + 0.02)R_u \geq 0.03R_u$$

Where

$$C(T) = C_h(T)ZRN(T_1, D)$$

In these expressions, T is the period of vibration in any mode. For the equivalent static procedure, only the first mode is considered, and T is then replaced with T_1.

For a short-period building in Dunedin, assumed with 5% of critical damping, the hazard spectrum has the following values:

Period, T, seconds	Hazard spectral value, $C_h(T)$, g
0.0	1.33
0.1	2.93
0.2	2.93
0.3	2.93
0.4	2.36
0.5	2.00

For the equivalent static procedure, a short-period building is taken as having a period of 0.4 seconds. With the site remote from an active fault and composed of shallow soils,

$$C_h(T_1) = 2.36 \qquad Z = 0.13 \qquad R = 1.0 \qquad N(T_1, D) = 1.00$$

Hence,

$$C(T_1) = 0.307$$

In this building, there will be energy dissipation by two principal mechanisms:

1. Dissipation by ductile yielding, especially yielding of nails in the timber diaphragms. However, this is ignored in this assessment. Accordingly,

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 = 1.00$$

and

$$S_p = 1.00$$

2. Dissipation by viscous damping especially related to the masonry. NZSEE Guidelines, Section 10, suggests that 15% equivalent viscous damping may be assumed. This is taken into account by the additional factor

$$k_\xi = \left(\frac{7}{2+\xi}\right)^{0.5} = 0.64$$

Hence, finally, for the equivalent static method

$$C_d(T_1) = 0.196$$

In accordance with the requirements specified for earthquake prone buildings, only one-third this level of shaking needs be assumed.

Where a higher level of upgrading is be considered, such as for a change of use, or just from considerations of additional safety or property protection, there is a question of what constitutes a "reasonably practicable" level of shaking to assume, taking into account heritage values of the building and how these might be jeopardised by inappropriate intervention into the existing fabric. It is often assumed that 67% is a reasonable level for this type of building, but for this report 100% is targeted.

For the modal response spectrum method, use is made of the hazard spectrum Tabled above to extract earthquake effects in each mode. The other parameters (Z, N, R, S_p, k_μ and k_ξ) apply as for the equivalent static procedure. However, for the MRS analysis implemented in this study, the modal damping values can be stated directly. Therefore, the NZS 1170.5 spectrum at the 5% level is used. The design spectrum is therefore given for the assessment by:

$$C_d(T) = 0.130 \times C_h(T)$$

This is the appropriate spectrum for the modal response spectrum method for the building as at present.

For additional structure, there will be energy dissipation by ductile yielding, especially yielding of reinforcement. Large ductility capacity is not expected, with cross-braced steel frames or blockwork shear walls being selected for much of the resistance. Accordingly, a ductility factor of 3.0 might be adopted, with

$$k_\mu = \frac{(\mu - 1).T_1}{0.7} + 1 = 2.14$$

and

$$S_p = 0.7$$

Hence, finally,

$$C_d(T) = 0.043 \times C_h(T)$$

Appendix C: Material Failure Criteria

Unreinforced masonry is commonly assessed using a modified Mohr-Coulomb failure criterion. For the implementation of finite element work, the Mohr-Coulomb criterion shows excess dilatancy. For that reason the Drucker-Prager criterion is often used. The two can be directly related if simplified assumptions are made. For plane-strain conditions, cohesion, d, and the friction angle, β, for the Drucker-Prager criteria are related to c and φ for Mohr-Coulomb as follows.

For associated flow:

$$\tan\beta = \frac{\sqrt{3}\sin\phi}{\sqrt{1+\frac{1}{3}\sin^2\phi}}$$

$$d = \frac{\sqrt{3}\cos\phi}{\sqrt{1+\frac{1}{3}\sin^2\phi}}c$$

For non-dilatant flow (flow parallel to the shear plane only):

$$\tan\beta = \sqrt{3}\sin\phi$$

$$d = \sqrt{3}\cos\phi.c$$

The Drucker-Prager yield criterion is given as:

$$\alpha J_1 + \sqrt{J_2^{'}} = k^{'}$$

Where J_1 is the first stress invariant (three times the mean hydrostatic stress) and $J_2^{'}$ is the second deviatoric stress invariant, which is related to the Mises stress as follows:

$$\sqrt{J_2^{'}} = \frac{S}{\sqrt{3}}$$

The other terms are given by the following relations:

$$\alpha = \frac{2\sin\varphi}{\sqrt{3}(3+\sin\varphi)}$$

$$k^{'} = \frac{6c.\cos\varphi}{\sqrt{3}(3-\sin\varphi)}$$

For low axial stresses, the first term of the yield criterion can be ignored, so that the yield criterion approximates to the following form:

$$S = \frac{6c.\cos\varphi}{3-\sin\varphi}$$