Performance-based assessment and seismic strengthening of the Christ Church Cathedral

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ABSTRACT

The Christ Church Cathedral is a significant landmark for New Zealand. It was severely damaged in the 2011 Canterbury earthquakes – the Tower and Rose Window/Western Façade of the Cathedral suffered partial collapses while the remainder of the building sustained extensive cracking and displacement to its unreinforced masonry superstructure. As a result, the Cathedral was declared a Dangerous Building and fenced off from the public. Following a 2017 decision by the Anglican synod to proceed with stabilisation and reinstatement of the Cathedral, Christ Church Cathedral Reinstatement Ltd. (CCRL) was formed and charged with delivering the project. Holmes have provided structural engineering services to CCRL, including design of a strengthening/retrofit scheme to bring the building's seismic rating up to 100% of new building standard (NBS) for an Importance Level 3 building. To achieve this, the Cathedral will be retrofitted with base isolation to reduce seismic demands on the building and therefore, the amount of strengthening required to the superstructure. This paper focuses on Holmes' use of non-linear time history analysis (NLTHA) to inform and validate the design of the earthquake repair and strengthening scheme. The use of an NLTHA model during the project's concept phase enabled the scope of seismic strengthening to be refined, thereby minimising disruption to the building's heritage fabric. The NLTHA procedure that was adopted is described, including element types, post-elastic degradation characteristics, acceptance criteria and their application, and modelling of soil-structure interaction effects.

1 INTRODUCTION

The Christ Church Cathedral is an Anglican cathedral which is of outstanding historical and cultural significance to New Zealand. It defines the centre of Christchurch City and is considered to be an excellent example of the Victorian mid gothic architecture style (ICOMOS, 2015). The building was constructed in various stages, between 1865 and 1904 and has Heritage New Zealand Category I rating, the highest rating possible for a building in New Zealand.

 

Figure 1 Photograph illustrating the Christ Church Cathedral from the southwest perspective (left) and 3D Revit model of the same view (right)

The building has undergone a series of alterations, particularly related to earthquake damage and earthquake strengthening, over its history. More recently, between 1999 and 2002, seismic strengthening addressed identified weaknesses in the building's lateral strength. Strengthening elements added at that time included reinforced concrete overlay walls with stone facing to the nave and west transept, continuous capping beams throughout the building at eaves level, steel cross-bracing in the side aisle roof of the nave, and various steel strengthening around the clerestory walls.

Earthquake damage to the Tower spire occurred in 1881, 1888, 1901, 1922, and-during the more recent 2010/2011 Canterbury earthquakes and aftershocks. This latest event resulted in severe structural damage to the building including collapse/demolition of the Tower and Rose Window/Western façade. In 2017 the Anglican synod made a decision to proceed with stabilisation and reinstatement of the Cathedral. Christ Church Cathedral Reinstatement Ltd. (CCRL) was formed and charged with delivering the project. Holmes have provided structural engineering services to CCRL, including the design of a seismic strengthening/retrofit scheme.

2 BUILDING DESCRIPTION

The main Cathedral building was constructed in stages with the first stage consisting of the Nave and Tower completed in 1881, followed by the West Porch in 1894, and the chancel, transepts and apse in 1904. Vestry additions to the north-east and south-east were added in 1960 and are integrated with the main Cathedral. The building is generally constructed from unreinforced masonry (URM) wall elements with a stone plinth around the base. The building sits over alluvial soils and is supported on unreinforced concrete and stone footings bearing on the upper sandy-gravels.

The walls of the Cathedral are predominantly rubble stone masonry with timber roof trusses and sarking supporting a steeply pitched slate roof. The original bell tower was located directly adjacent to the main Cathedral and was connected via the north-west corner of the nave of the Cathedral without structural separation.

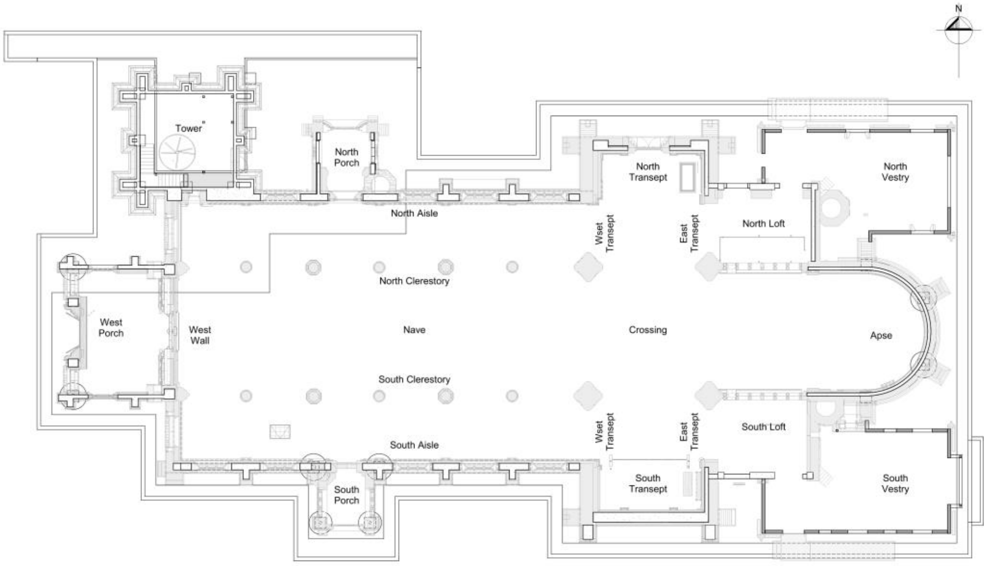

Figure 2 Ground floor plan with key names of the various areas

3 PERFORMANCE OBJECTIVES

The performance of the strengthened building was defined by two performance levels. The first of these performance levels (Life Safety) is used in conjunction with acceptance criteria in order to define the performance of various elemental/material behaviours. The second (Collapse Prevention) was taken from ASCE 41-17 (ASCE, 2017) for reference only:

- Life Safety (LS) Performance level. This is equivalent to the Ultimate Limit State (ULS) as defined in the Loadings Standard NZS1170.5 (SNZ, 2004). The functional requirements for this performance level are assumed to be met if:
    - People within, and adjacent to the structure are not endangered by the structure or part.
    - There is no loss of structural integrity in either the structure or part.
- Collapse Prevention (CP) Performance level. This performance level is not defined in the Loadings Standard NZS1170.5 (SNZ, 2004). It is the ASCE 41-17 (ASCE, 2017) benchmark for building performance beyond Life Safety and is defined as the post-earthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin against collapse.

4 SEISMIC ASSESSMENT AND STRENGTHENING DESIGN OVERVIEW

4.1 Performance-based design

Performance-based design via non-linear time history analysis (NLTHA) was adopted for the project. Although more intensive and computationally demanding compared to conventional linear-elastic techniques, the methodology had several benefits:

1. unreinforced masonry failure modes defined in Section 6 were captured using non-linear panel elements; a more accurate and realistic approach than simplifying the response of the URM by using linear-elastic elements Oliver et al. (2018).
2. retrofit/strengthening components such as the base-isolation system, post-tensioning and reinforced concrete overlay walls can be explicitly assessed.
3. seismic demands on secondary and non-structural elements can be accurately assessed using floor response spectra generated directly from the NLTHA model.

4. the stiffness of the URM walls relative to the flexible timber roof diaphragms can produce wide-ranging local modal behaviour. NLTHA provides the best means for capturing and assessing this localised behaviour.

Experience with previous projects (including Chambers and Kelly (2004), Oliver and Mackenzie (2011) & Oliver et al. (2018)) has demonstrated the adoption of performance-based design/assessment using NLTHA results in seismic strengthening schemes that supplement, rather than replace, the existing lateral load resisting system. Typically, this results in less strengthening and more cost-effective seismic retrofit solutions.

4.2 Macro modelling approach

The model resolution (i.e. the level of model refinement), influences how non-linear behaviour is distributed through a cross section and length of a given structural component. In this instance, a macro modelling approach was adopted, whereby the structure was idealised using the fewest number of finite elements possible. An example is given below in Figure 3 – where a single finite element is used to represent each wall component.

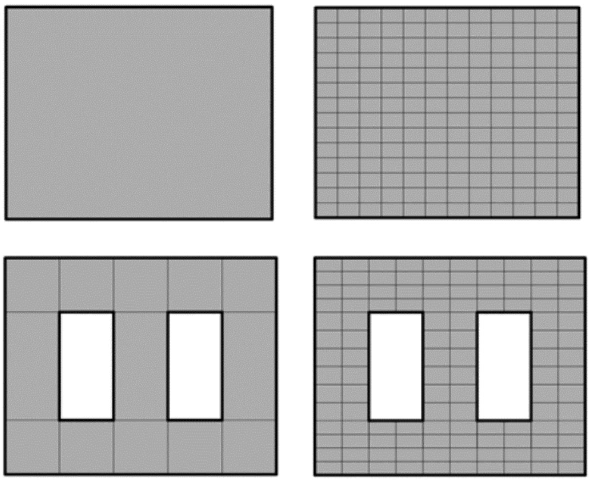

Figure 3 Comparison of finite element arrangements for macro modelling (left) and micro modelling (right) of two different wall panels

Element plasticity was concentrated in zero-length hinges with moment-rotation or shear-displacement backbone relationships (refer Figure 3). These backbone curves are derived through calibration of experimental test data. Building standards and guidelines such as ASCE 41-17 (ASCE, 2017), EN 1998-3 (CEN, 2005) and NZSEE Guidelines (NZSEE, 2017) provide standardised backbone relations which can used.

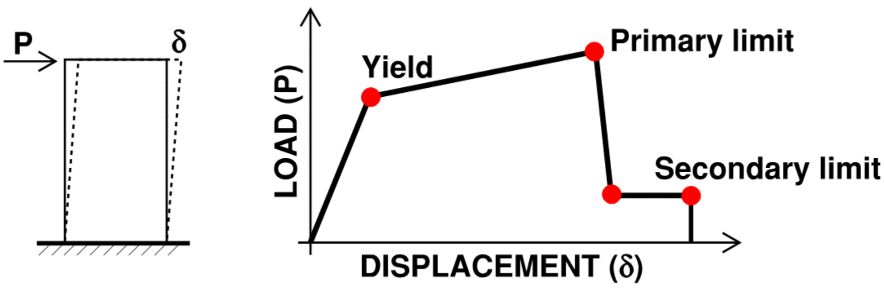

Figure 4 Idealised back bone curve for a wall panel

Macro-modelling elements have relatively condensed numerically efficient formulations and this, coupled with a greatly reduced number of finite elements needed per analysis model, results in significantly shorter analysis run times when compared with micro-element building models. Macro elements have an added advantage of typically being more numerically stable than an equivalent assemblage of micro-elements and can therefore often be more readily used to assess structures that have low residual strength and stiffness. They are also typically easier to debug

4.3 Seismic strengthening strategy

Base isolation was adopted for this project. The isolation system enabled the project performance objectives to be achieved while minimising disruption to the heritage fabric of the building. It consisted of a combination of Lead-Rubber Bearings (LRBs) and flat-plate pot-bearing PTFE sliders. Nominal effective periods, T_{eff}, of the isolation system for the ULS and CP performance limit states were 2.6 seconds and 2.9 seconds respectively and are based on a maximum rattle space target of 700mm. This target was set by the movement limit at the Northeastern corner of the site, where the building is closest to Cathedral square road.

Above the isolation plane, new structural elements associated with the seismic strengthening includes:

- pinning and grouting of URM walls,
- a new ground floor reinforced concrete transfer grillage to resist the overturning actions generated by the isolators,
- selected reinforced concrete overlays to mitigate brittle failure of unreinforced masonry elements,
- new ply roof diaphragms,
- various steel bracing/collector elements and
- post-tensioning of the west porch wall.

The Tower, West Porch and Vestries are to be re-built entirely.

Below the isolation plane, a new foundation raft is required to support the strengthened building.

These new structural elements are all shown below in Figure 4, which contains two 3D perspectives of the retrofit and new/re-build elements.

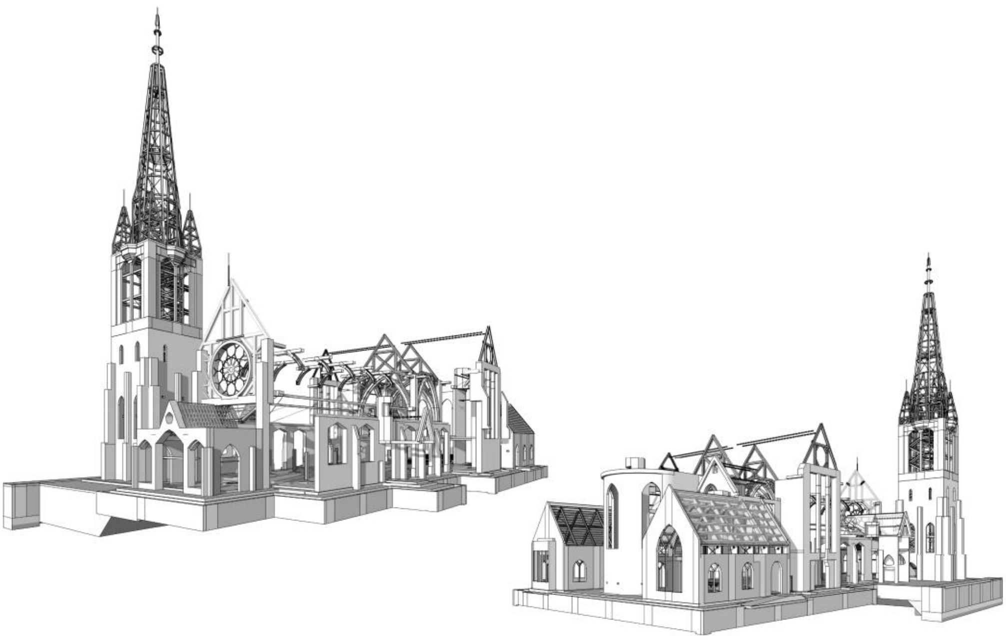

Figure 5 Strengthening (structure only) southwest (top-left) and northeast perspectives (bottom-right)

4.4 Soil structure interaction

To capture the effects of soil structure interaction (SSI), elastic springs, representative of the vertical stiffness of the supporting soil, were modelled below the isolators. Three scenarios were considered:

1. Rigid ground (i.e. the isolators were fixed rigidly at their base)
2. 100% ULS shaking on ground without liquefaction
3. 80% ULS shaking on ground with liquefaction

For scenarios 2 and 3 the spring stiffnesses were determined from a foundation analysis in RAM (Bentley, 2022) using lower bound soil stiffnesses.

5 SEISMIC INPUT

5.1 Target spectrum

The parameters used to define the target spectrum are based on the Loadings Standard NZS1170.5 (SNZ, 2004) and the NZSEE Draft BI Guidelines (NZSEE, 2019). The site soil class (Class D) and estimated site period (0.6 s) were based on a memorandum provided by Geotech Consulting Ltd (Geotech Consulting Ltd, 2019).

Analysis runs were carried out at the two performance levels to assess the seismic performance of different components of the building, specifically: superstructure (building above the isolation plane), and isolator stability and rattle space (moat). Parameters used for the different analysis runs are summarised in Table 1.

Table 1: Project analysis parameters

Criteria Name	**Limit State**	**Isolator Properties**	**Structural Performance Factor, S_p**
Superstructure	ULS	Upper Bound	1.0
Isolator – Stability & Rattle Space	CALS	Lower Bound	1.0

5.2 Time-history records

The earthquake time history records used in the analysis were generally scaled in accordance with ASCE7-16 (ASCE, 2016) with some modifications as recommended by the NZSEE Draft BI Guidelines (NZSEE, 2019).

ASCE7-16 requires a minimum of eleven earthquake ground motion records to be used for NLTHA. Each of the records is required to have a seismological signature (i.e. magnitude, source characteristic – including fault mechanism and source-to-site distance) the same as, or reasonably consistent with, that of the site under consideration. Ground motion selection was based on the recommendations from an extensive study of ground motion ensembles undertaken by Bradley Seismic Limited (Bradley and Tarbali, 2017). The vertical component of the ground motions were included in analysis to investigate the effects of vertical acceleration on the performance of the isolation system.

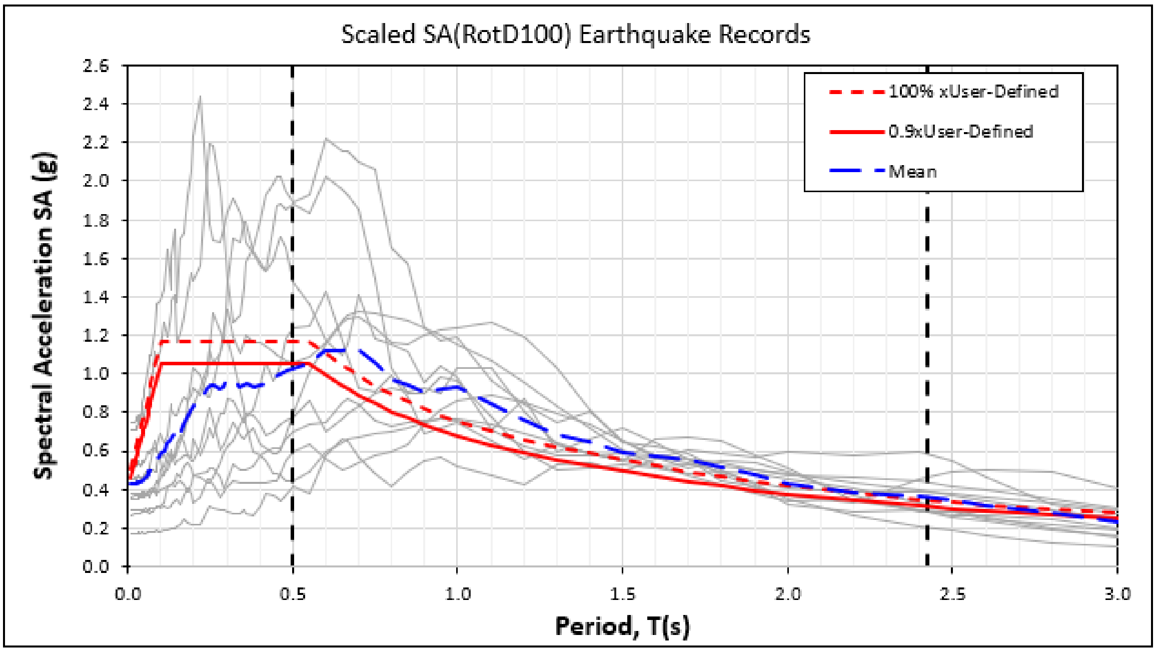

Figure 6 NZS1170.5 Hazard spectrum for the Christ Church Cathedral site (1/1000 year return period) and scaled earthquake records (shown in grey); the upper and lower bounds of the scaling range are indicated by the vertical dashed lines

6 EVALUATION PROCEDURE AND ACCEPTANCE CRITERIA

The structure was analysed using the Holmes nonlinear analysis procedure developed by Kelly (2014). This is based on a linked series of modules:

1. MODELA, an Excel spreadsheet for preparing input data and writing input files
2. ANSR-II, a general purpose non-linear analysis program (Mondkar and Powell, 1979) to analyse the structure.
3. PROCESSA, an Excel spreadsheet to process output files and import envelope results.

The seismic performance of existing building components was assessed using the 2017 NZSEE Engineering Assessment Guidelines (NZSEE, 2017). This document provides standardised backbone curves and acceptance criteria which were applied to the non-linear model. Base isolation systems are not addressed by the Assessment Guidelines and so we have referred to the NZSEE Draft BI Guidelines (2019) for the design of the isolation system. Actions and deformations in components were assessed against the prescribed deformation limits.

6.1 Unreinforced masonry walls and piers

The in-plane behaviour of unreinforced masonry (URM) wall and pier elements was represented by the bed joint sliding shear strength relationship provided in the Assessment Guidelines (NZSEE, 2017). This failure mode was chosen because, in the case of the Christ Church Cathedral, the mortar strength is low relative to the strength of the stone. URM walls and piers were modelled using a combination of compression only gap elements and nonlinear membrane elements. Gap elements were provided at the top and bottom of pier elements where in-plane rocking was anticipated. Nonlinear membrane elements, which include cyclic stiffness and strength degradation, were used to model the bed-joint sliding failure mode.

6.1.1 Masonry modelling parameters

While rubble stone masonry as a construction material falls within the scope of the NZSEE Assessment Guidelines, C8.1.2 acknowledges the need for additional requirements to be considered in assessment. To appropriately capture the characteristics of rubble stone masonry, guidance has been sought from NTC (NTC, 2008). Specific direction is provided within NTC for material strength improvements via grout injection to the central rubble core and pinning of inner and outer wythes with stainless steel anchor rods. Un-retrofitted rubble stone masonry is susceptible to delamination, limiting its ability to act homogeneously. The proposed rubble stone masonry remedial of grout injection to the central rubble core and installation of tie rods at regular centres aims to mitigate the delamination risk and allow the walls to act homogeneously.

The estimated rubble masonry strengths (compressive strength, cohesion, and friction) derived from NTC 2008 were lower than those given by the NZSEE Assessment Guidelines (refer Table 2). As such, the values output by the two documents were used as a set of bounds – whereby the analysis model was run with both sets of properties and the results enveloped.

Table 2 Comparison of probable masonry strength properties

Probable Material Properties	**NTC 2008 Typology B**	**Correction Factors for Mechanical Properties**	**NTC 2008 Subject to Improvements**	**NZSEE 2017 Clay Brick Masonry**
Compressive Strength, f'_m	2.5 MPa	1.4 x 1.5 x 1.7 = 3.57	8.9 MPa	14 MPa
Cohesion, c (MPa)	0.043 MPa	1.4 x 1.5 x 1.7 = 3.57	0.15 MPa	0.50 MPa
Coefficient of Friction, μ_f	0.40	-	0.40	0.65

6.1.2 Bed-joint sliding

Bed joint sliding capacity of unreinforced masonry piers and walls was assessed in accordance with the NZSEE Guidelines using Equation 1:

$$v_s = 0.7(c + \mu_f f_a) \quad (1)$$

where, $c =$ masonry bed-joint cohesion, $f_a =$ axial compression stress in panel element due to gravity loads and $\mu_f =$ masonry coefficient of friction.

Bed-joint sliding is considered to be a desirable failure mode as it has substantial deformation capacity past initial cracking. The standardized backbone curve for bed-joint sliding and related shear hysteresis as implement in the ANSR II model are illustrated in Figure 7 below. Shear capacity of the element degrades linearly to the frictional component when a shear strain of 0.004 is reached.

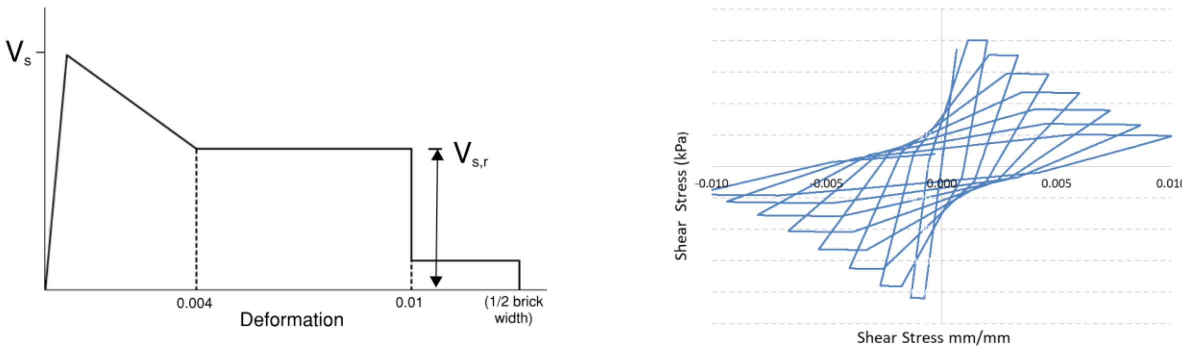

Figure 7 Bed-joint sliding standardised backbone curve and shear hysteresis for URM walls and piers

6.1.3 In-plane rocking

In-plane rocking capacity of unreinforced masonry piers and walls was explicitly accounted for within the analysis model by non-linear gap elements. The gap elements are defined as compression only elements and allow uplift to occur when the uplift force exceeds the gravity force. In-plane rocking is also considered to be a desirable failure mode as it is characteristic of a stable structural response up to modest inter-storey drift demands.

6.1.4 In-plane acceptance criteria

Acceptance criteria for the in-plane response of unreinforced masonry piers and walls are provided in Table 3 below. The acceptance criteria were derived from the NZSEE Guidelines.

Table 3 Acceptance criteria for the in-plane response of unreinforced masonry piers and walls

Failure Mode	**ULS Acceptance Criteria (drift ratio mm/mm)**
Bed-joint sliding	0.075
In-plane rocking	Min ($0.003h_{eff}/L_w$, 0.011)

6.2 Unreinforced masonry spandrels

In-plane strength of unreinforced masonry spandrels was taken as the lower of the assessed flexural and shear strengths. Unreinforced masonry spandrels were modelled using nonlinear membrane elements.

6.2.1 Flexural capacity

Peak and residual flexural capacities of unreinforced masonry spandrels was assessed in accordance with the NZSEE Guidelines using Equations 2 and 3 respectively:

$$v_{fl} = (f_{t,eff} + p_{sp})\frac{h_{sp}}{3l_{sp}} \tag{2}$$

$$v_{fl,r} = \frac{p_{sp}h_{sp}}{l_{sp}}(1 - \frac{p_{sp}}{0.85f_{hm}}) \tag{3}$$

where, $f_{t,eff}$ = equivalent tensile strength of masonry spandrel, p_{sp} = axial stress in the spandrel, h_{sp} = height of spandrel excluding depth of timber lintel if present, l_{sp} = clear length of spandrel between adjacent wall piers and f_{hm} = compression strength of masonry in the horizontal direction.

6.2.2 Shear capacity

Like walls and piers, the unreinforced masonry spandrels are comprised of masonry with mortar that is weak relative to the strength of the stone. As such, the peak and residual shear capacities of spandrels was assessed using Equations 4 and 5 respectively (NZSEE, 2017):

$$v_s = \frac{2}{3}(c + \mu_f p_{sp})h_{sp}b_{sp} \tag{4}$$

$$v_{s,r} = \frac{11}{16}p_{sp}\frac{h_{sp}}{l_{sp}} \tag{5}$$

where, b_{sp}= spandrel breadth.

6.2.3 In-plane standardised backbone curve and acceptance criteria

Experimental testing has demonstrated that unreinforced masonry spandrels have substantial deformation capacity past initial cracking (Beyer & Dazio, 2012). The standardised NZSEE backbone curve and typical shear hysteresis implemented in the ANSR II model for unreinforced masonry spandrels are illustrated in Figure 8 below. Acceptance criteria for the ULS limit state were derived from the NZSEE Guidelines and are summarised in Table 4.

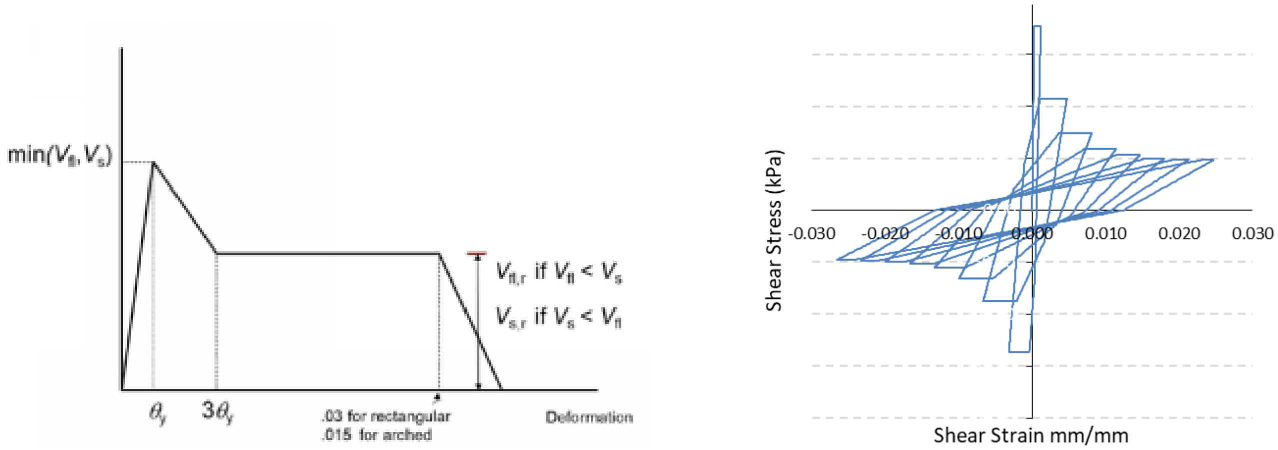

Figure 8 Standardised backbone curve and shear hysteresis for unreinforced masonry spandrels

Table 4 Acceptance criteria for the in-plane response of unreinforced masonry spandrels

Spandrel Type	**ULS Acceptance Criteria (drift ratio mm/mm)**
Rectangular	0.02
Arched	0.01

7 ANALYSIS RESULTS

The three scenarios used to capture soil structure interaction (SSI) described in Section 4.4 were each evaluated independently. The outputs from the three scenarios are generally similar and therefore following results are provided for just one of the scenarios – the rigid ground case (scenario (1)).

Unless otherwise noted, all building response quantities presented are average values. They are obtained by enveloping the building response output from the four mass-eccentricity runs (±5% accidental eccentricity in accordance with NZSEE BI Guidance, and then averaging the response over the eleven ground motions.

7.1 Global building response

Table 5 provides an overall summary of the average global building response quantities from the NLTHA at the considered performance levels ULS (R=1.3) – superstructure, and CALS (R=1.95) – isolation, on rigid ground.

Table 5 Summary of average building response as reported from NLTHA

Global response (mean)	**Ultimate Limit State (ULS) – NTC masonry**	**Ultimate Limit State (ULS) – NZSEE masonry**	**Collapse Avoidance Limit State (CALS)***
X Displacement (mm)	267.5	198.7	582.1

Z Displacement (mm)	433.4	230.0	642.2
X Base Shear Coefficient (g)	0.17	0.18	0.19
Z Base Shear Coefficient (g)	0.16	0.16	0.20
Acc. – Side Aisle Roof (g)	0.31	0.31	1.61
Acc. – Clerestory (g)	0.51	0.43	1.35
Acc. – Nave Roof (g)	0.90	0.78	1.01

*The Collapse Avoidance Limit State (CALS) has been run with NZSEE masonry properties.

Referring to the elastic design spectrum illustrated in Figure 6, it is clear that the isolation system effectively reduces the seismic demands in the superstructure by a factor of 5 or more.

7.2 Component response

Figure 9, Figure 10, and Figure 11 provide a visual summary of the building component performance at ULS for the upper and lower bound masonry properties considered.

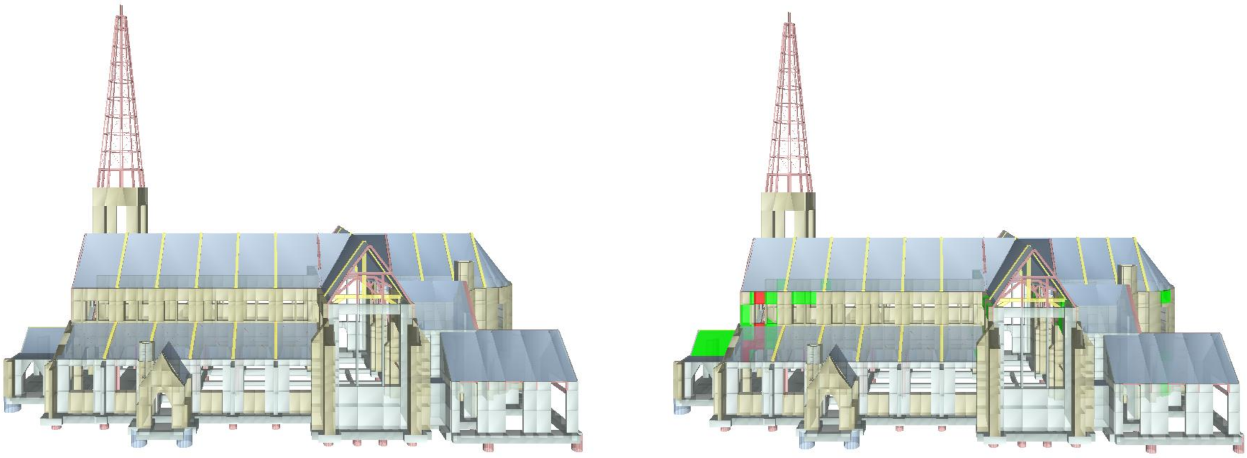

Figure 9 Illustration of component performance (south elev.), NZSEE masonry (left), NTC masonry (right)

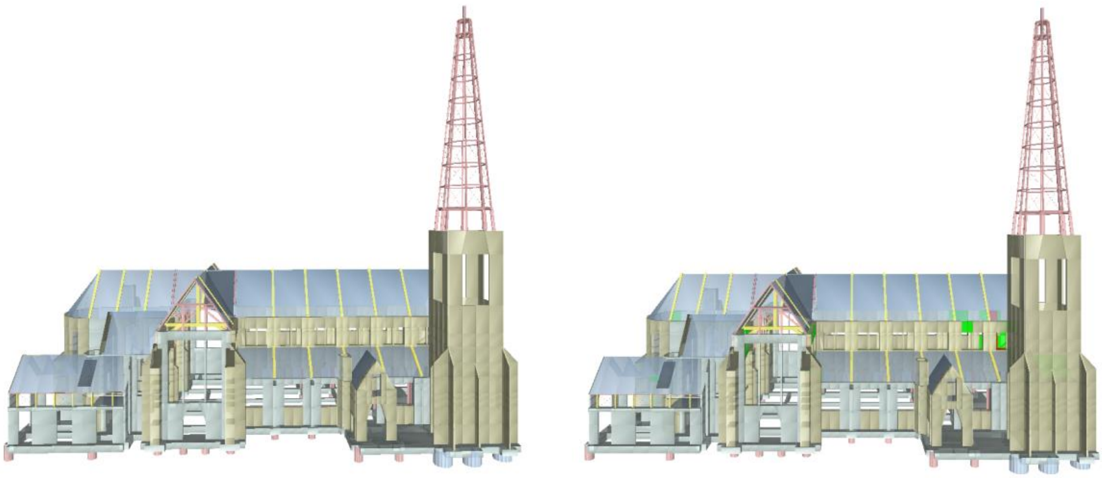

Figure 10: Illustration of component performance (north elev.), NZSEE masonry (left), NTC masonry (right)

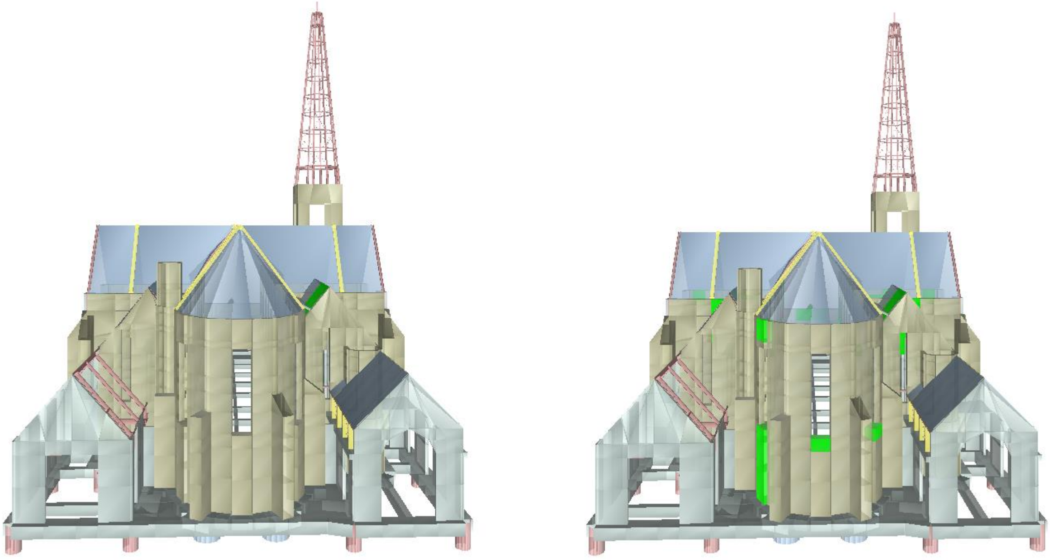

Figure 11: Illustration of component performance (east elev.), NZSEE masonry (left), NTC masonry (right)

Elements that are shaded green are those that exceed the ASCE 41-17 immediate occupancy (IO) deformation limits, while elements that are shaded red, are those that exceed the NZSEE ULS deformation limits. Generally, deformations were observed to be less than the allowable NZSEE ULS limits – with two exceptions:

1. The two Clerestory arch spandrels, directly east of the Western Façade. When the lower bound (NTC) masonry properties were used in the analysis, the seismic demands on these spandrels exceeded the ULS deformation limits. However, when the upper bound (NZSEE) masonry properties were used, the spandrels performed well – staying below the ASCE 41-17 IO deformation limits. In-situ masonry shear testing was completed to assess the strength of the improved masonry representative of the Clerestory. Results from this testing indicated that the masonry was stronger than the assumed lower bound strengths, and the model was re-run with updated strength values, inferred from the test data. Results from the updated model confirmed the seismic demands on the Clerestory arch spandrels did not exceed the ULS deformation limits and therefore strengthening of these spandrels was not required.
2. A single spandrel wall element at the top of the North organ loft wall. When the lower bound (NTC) masonry properties were used in the analysis, the seismic demands on this spandrel exceeded the ULS deformation limits. However, failure of this element would not result in the loss of gravity load carrying capacity and localised damage to this spandrel alone is unlikely to affect the global building performance at the ULS criteria. As such, no further strengthening was required.

8 CONCLUSIONS

The Christ Church Cathedral reinstatement project successfully utilised performance-based assessment and macro-modelling finite element analysis to validate a seismic strengthening solution that minimises disruption to the heritage fabric of the building while still achieving the project performance objectives. Details of the NLTHA were provided, including the way in which soil structure interaction was accounted for, and the modelling parameters adopted for the rubble stone masonry elements. The project provides further evidence that performance-based assessment and NLTHA are valuable tools for consulting engineers looking to strengthen/retrofit URM buildings.

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