Post-earthquake numerical assessment and reinforcement of St James Church, New Zealand

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SUMMARY:
This paper presents a numerical study on the seismic assessment and reinforcement of St James Church, affected by the 2011 New Zealand Earthquake. Numerical analyses were performed using a finite element model including the structural damage of the Church. The numerical model was calibrated against experimental results obtained from the dynamic identification tests carried out in situ. Nonlinear pushover analyses were performed in order to understand the structural behaviour of the damaged Church. The analysis of the results suggests that the damaged structure is not safe according to the new national specifications. Therefore, some reinforcement measures are needed to improve the seismic behaviour of the structure. A reinforcement solution is proposed taking into account the historical heritage value of the building, trying to maintain as much as possible the original aspect of the Church. Pushover analyses were also performed in the reinforced numerical model, proving its effectiveness.

Keywords: Earthquake; Masonry; Numerical Analysis; Reinforcement; Seismic Assessment

1. INTRODUCTION

Unreinforced masonry construction is predominant in many urban areas world-wide, particularly in the form of impressive historical buildings as landmarks of ancient cultures (Vasconcelos & Lourenço 2009). Nevertheless, considering past and recent earthquakes, it has been recognized that unreinforced masonry buildings do not respond in a satisfactory way to this type of dynamic loading, being even referred to as one of the most vulnerable forms of construction in this context (Abrams 2001). In fact, these natural catastrophes have always represented the main cause of damage and loss of cultural heritage, also identified as one of the major threats to life safety (Abrams 2001; Lagomarsino 2006). According to Magenes (2010), most of masonry constructions that undergo seismic action lead to collapses and casualties due to inadequate performance of unreinforced masonry (URM) buildings.

In the last few years, due to an increasing awareness of the preservation of heritage buildings as way to protect our own culture, there has been a major advance in the techniques for assessing the seismic capacity of buildings. In fact, for countries with a long story of civilization, the seismic protection of masonry buildings also includes the issue of protecting the cultural heritage. Due to the referred vulnerability of masonry to earthquakes, research on the seismic behaviour of masonry structures is nowadays almost entirely dedicated to existing buildings with the aim of evaluating and reducing their seismic vulnerability (Magenes 2006).

Several methods of analysis and computational tools are currently available for the assessment of the mechanical behaviour of historic structures which can be successfully used in masonry structures (Antoniou et al. 2004; Betti & Vignoli 2008; Binda et al. 2009; Chopra & Goel 2001; Mendes & P. Lourenço 2010; Penna et al. 2004; Lourenco et al. 2007). During the last few years, displacement-based methodologies, such as the pushover analysis, are being more and more recognized as practical and suitable tools for the evaluation of the seismic response of existing structures (Magenes 2000;

Salonikios 2003). Pushover analysis can be an effective alternative to traditional methods of linear seismic analysis considering the difficulties related to non-linear time-history analysis (Augenti & Parisi 2009). The complexity and computational demand required by nonlinear dynamic analysis led to the development of new methods for the seismic assessment based on a simplified mechanical approach. These have been consolidated during the 1990s, as the capacity spectrum method (Freeman 1998) and N2 method (Fajfar & Eeri 2000) and were considered within modern regulations both for designing new structures and assessing existing ones (EC8 2004; FEMA 2000; OPCM 2003). According to these methods, the comparison between seismic demand and capacity is made in terms of displacements and plotting a force-displacement curve, resulting in the capacity curve of the structure.

St. James Church, located in Christchurch, New Zealand is one of the many structures that have suffered structural damage with the recent earthquake in 22th February 2011 and subsequent aftershocks. In this paper, nonlinear finite element modelling (FEM) analyses are performed in order to assess the seismic vulnerability of the damaged Church and a posterior strengthening solution is proposed. For this purpose, DIANA software (DIANA 9.4 2009) was used. According to ICOMOS guidelines (ICOMOS 2005), a suitable analysis of a masonry building should include the numerical modelling of its structure with constitutive laws accurately describing the mechanical behaviour of the material. Afterwards, the model must be calibrated against experimental results so as to achieve the necessary reliability in the results. As such, a detailed in situ inspection was carried out (visually, by coring and by dynamic identification), allowing the model updating of the heavily damaged structure. Subsequently, the seismic assessment of the Church by means of pushover analysis is presented. Moreover, according to the obtained results, a reinforcement solution is proposed considering the historical and cultural value of the Church, attempting to maintain its original aspect as much as possible. Finally, pushover analyses were carried out on the reinforced structure. The main purpose of this study is to find out how the reinforcement option can really increase the seismic resistance of a damage structure and if it is indeed effective.

2. ST JAMES CHURCH

2.1. General Description

The construction of St James Church was started in 1923, designed by the architects Alfred and Sidney Luttrell, to provide a permanent centre for Anglican worship. The Church has a simpler early English Gothic style (as can be seen in Figure 1) and is a place of worship because it was built as a memorial to the men of Riccarton, in Christchurch, who had died in the First World War.

Figure 1. General view of St. James Church

The church has rectangular shape with a long nave that constitutes the body of the church, with 27.0 m long and 10.3 m wide. There are also other secondary compartments that define the exterior shape of the building, as the entrance, the vestry and the tower, and a sort of side chapel where the organ is located, see Figure 2. The building has a large number of buttresses in all façades and corners, which seems to indicate a clear earthquake concern. The two naves that compose the Church have different heights, the main nave with 10.5 m height and the altar nave with 9 m height. The tower is the tallest element of the Church with a height of 15 m.

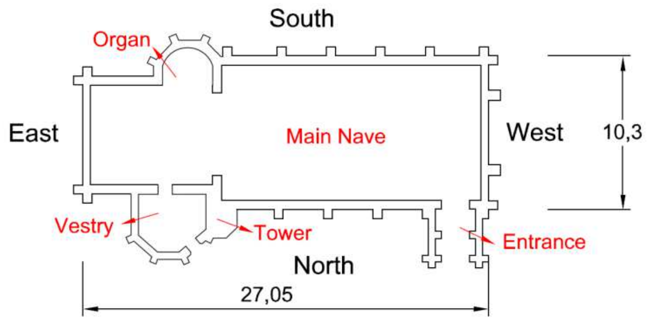

Figure 2. Plan view of the Church with the areas that compose the building and its main dimensions (in meters)

The Church is usually referred to as an unreinforced masonry structure, coated with Halswell stone, with facings of cream Oamaru stone and plastered brick on the inside (Ross 1999). During the in situ inspection, specimens were taken from the walls in order to know their constitution. The wall specimens indicate that three layers of materials are present, brick in the inside leaf of the wall, weak concrete in the inner core and stone in the outside façade.

3.1. Visual Inspection

The Church has suffered structural and non-structural damage a consequence of 22th February 2011 earthquake (Wilby 2011). A visual inspection was carried out and the main damage visible in the structure identified. The most damaged areas are the transversal façades, east and west, and the triumphal arch located in the middle of the two naves. It can be observed that the areas more damaged are aligned with the height at which the buttresses end, which indicates an inadequate design with a strong geometrical discontinuity. It is easily noticeable that the horizontal cracks that develop in the transversal walls are accompanied by spalling of the limestone facings, while simultaneously large areas of plaster have fallen from the chancel arch, as well as the east and west gable walls. In conclusion, damage is mostly due to out-of-plane movements, even if some diagonal cracks due to in-plane movements are also observed. Furthermore, the extensive images collection and the damage observed in situ indicate that the roof structure is not tightly linked to the gable walls, and it is not stiff enough to adequately tie the longitudinal walls.

3. NUMERICAL MODELLING

A numerical model of the structure of St. James Church was developed in order to properly simulate the structural behaviour of the building. The model was prepared taking into consideration the geometrical data and the structural damage found in the building. The model configuration attempts to reproduce the structural behaviour of the Church, while adopting the necessary simplifications.

The 3D finite element model prepared includes eight node quadrilateral continuum shell elements (CQ40S) or triangular 6 nodes elements (CT30S) to simulate walls, buttresses and floors, depending on the geometry. The timber trusses of the roof were modelled using three node beam elements (CL18B). The roof sheeting and transverse beams between the timber trusses were also simulated with shell elements. The mesh was defined taking into consideration a compromise between efficiency concerning the computation time and accuracy. All elements have quadratic interpolation and full Gauss integration, resulting in a mesh with 29.648 nodes and 10.588 elements, see Figure 3. As referred, the model aims also at reproducing the structural damage present. As such, due to the extensive damage observed in the triumphal arch and gable walls, one can assume that the roof structures are not rigidly connected to all transversal walls.

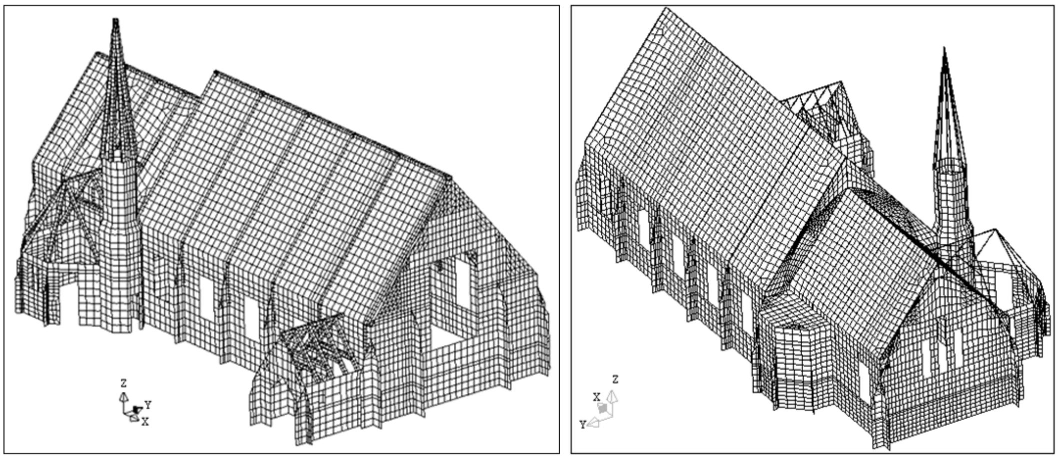

Figure 3. Numerical model

3.1. Calibration of the model

Besides the visual inspection, experimental dynamic identification tests were carried out in the Church in order to understand its structural behaviour, achieving the main vibration modes and the natural frequencies of the structure. With regard to the elastic properties of the materials, a Poisson’s ratio of 0.2 was used for all materials. An elastic modulus value of 30 GPa was used for concrete, while a value of 12 GPa was assumed for wood. For masonry, the value of the elastic modulus is to be obtained from the model updating using the dynamic identification information. The numerical model attempts to reproduce the general structural behaviour of the church considering the existing damage. Therefore, the calibration of the model was performed both according to the dynamic identification results and the damage identification resulting from visual inspection. The optimized parameter for the elastic modulus of masonry is equal to 9.43 GPa, which seems acceptable due to the good quality of the masonry and the inner concrete core.

4. SEISMIC ASSESSMENT OF THE DAMAGED STRUCTURE

As recommended in Lourenço et al. (2011), a proportional mass pushover approach was carried out on the model in order to assess the seismic capacity of the damaged structure of St James Church. The structural reinforcement is then proposed based on the results of this pushover analysis. The reinforcement solution proposed is also analysed in order to evaluate the improvement in the structural behaviour of the Church.

As most of the non-linearities are expected to concentrate in the masonry, only this material was considered with non-linear behaviour. The inelastic constitutive laws selected were an exponential relationship for tension and a parabolic relationship compression, that have been used successfully in many applications in complex masonry structures (Ramos & Lourenço 2004; Lourenco et al. 2007). The nonlinear properties for the masonry constitutive model were calculated based on recommendations from Lourenço (2009), according to the elastic modulus previously calibrated, see Table 1. Here f_c is the masonry compressive strength, G_c is the fracture energy in compression, f_t is the masonry tensile strength, G_t is the tensile fracture energy and β is the shear retention factor. The solution procedures used the regular Newton-Raphson method and an energy convergence criterion, with a tolerance of 0.001.

Table 1. Nonlinear mechanical properties of masonry

E (GPa)	υ (-)	f_c (N/mm^2)	G_c (N/mm)	f_t (N/mm^2)	G_t (N/mm)	β (-)
9.43	0.2	11.7	18.7	0.15	0.01	0.1

4.1. Pushover Analysis

Pushover analyses proportional to the mass were carried out for the two principal directions. As expected, the direction perpendicular to the transversal walls presents the weakest resistance values, confirming previous considerations originated from both experimental results and visual inspection. For this reason, this study focuses mainly on the referred direction. Figure 4 depicts the capacity curve for the gable walls, with a maximum load coefficient around 0.2 g, including loss of capacity of the east wall with a maximum displacement equal to 3.8 cm. It can be also noticed in the pushover curve that masonry behaves as linear until approximately 0.07g and shows a post peak softening behaviour.

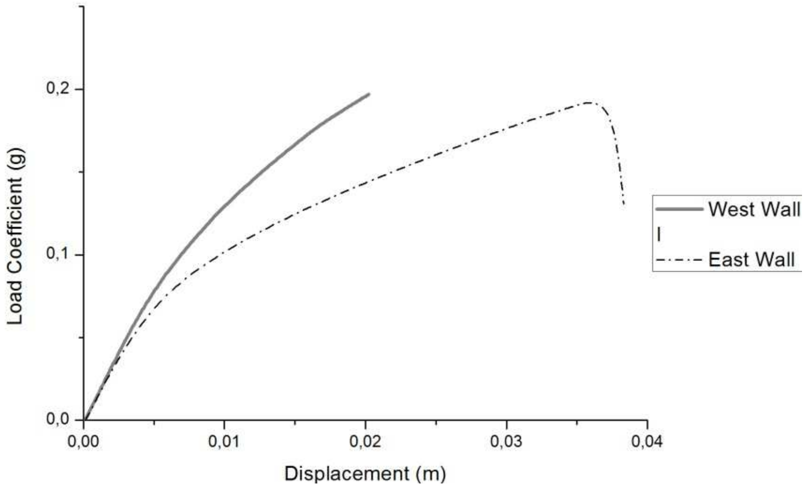

Figure 4. Pushover curve (unreinforced model)

For a better understanding of the structural behaviour, the principal tensile strains are plotted as an indicator of damage (Figure 5). Strains concentrations occur on the top of the tympanum of the east wall. A possible interpretation of the collapse mechanism is a local failure in the east wall with the detachment of the top of the tympanum, which could then lead to the out-of-plane collapse of this part of the structure due to the pre-existent horizontal crack. As such, the results of the pushover analysis revealed that the global capacity of the building is determined by the low capacity of the east wall due to the pre-existing damage. The PGA requirement for the area was fixed at 0.3g after the earthquake, leading to the conclusion that the Church requires repair or strengthening.

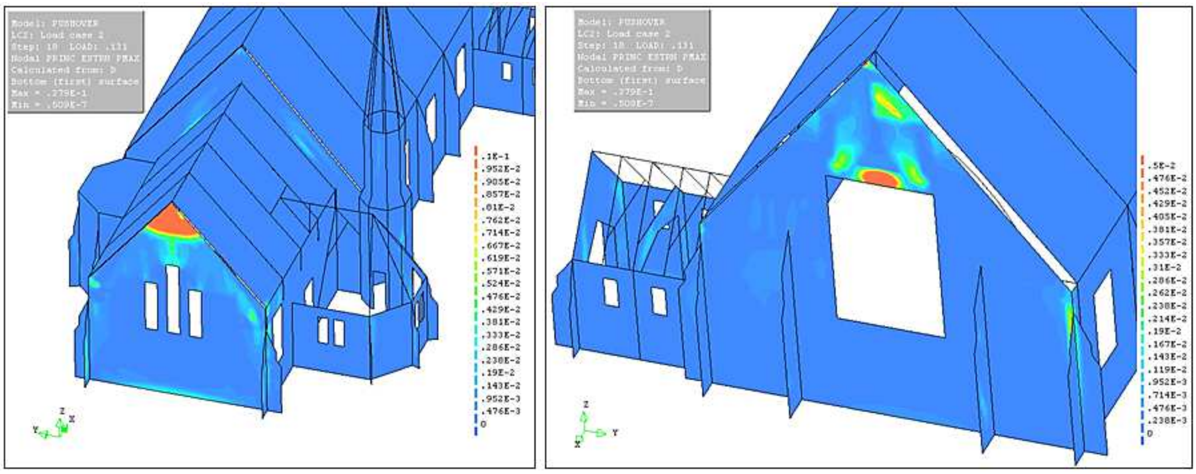

Figure 5. Maximum principal strains (collapse load)

5. REINFORCEMENT

The main goal of this study is not to provide a review of a set of reinforcement techniques for masonry buildings, but only to present a strengthening solution for a real damaged building and study the reinforcement option chosen. In this context and bearing in mind the high asset value of the Church, a possible structural intervention strategy will be proposed, analysed and discussed in order to mitigate the seismic vulnerability of the Church.

5.1. Reinforcement Solution

The deficient connection between the roof structure and transversal walls, provided by visual inspection and also observed numerically, is compromising the global capacity of the Church. As such, the building is not being mobilized as a single structure in this direction. Considering the collapse mechanism previously defined, the use of steel tie bars is proposed at the roof level in order to connect the transversal walls with the triumphal arch, trying to effectively and efficiently attach the roof structure to the walls on both sides, thus allowing the structure to be fully mobilized. Figure 6 presents the distribution of the steel bars (12 mm of diameter) in plan and elevation views (east façade), as an attempt to limit the out-of-plane displacements.

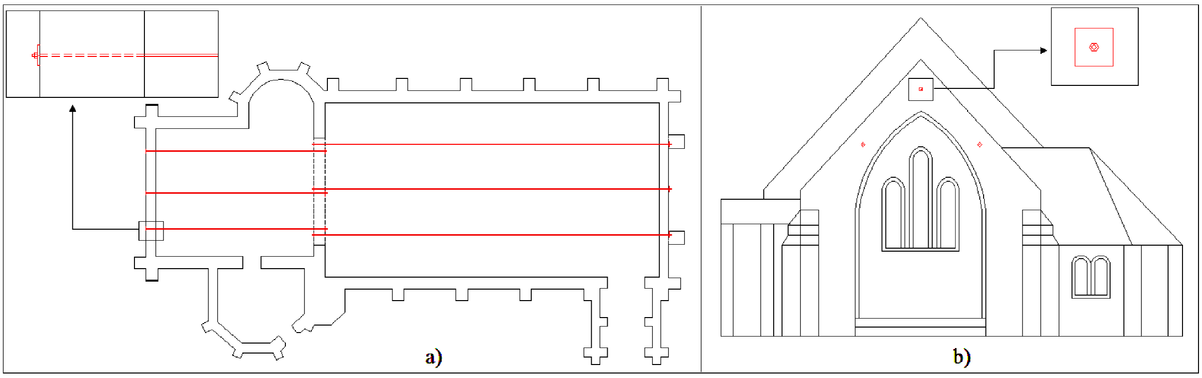

Figure 6. Introduction of steel bars: (a) plan view; (b) elevation view (east wall)

When the span is too long, which is the case in both situations, bars can suffer significant longitudinal deformations, either due to its self-weight or due to temperature. In this context, with the aim to control the deformations of the bars, some non-structural elements were predicted to be attached to the existent trusses. These metallic non-structural elements act as a kind of support for the bars at their intersection with the trusses, limiting the vertical displacement, but leaving the bar free to have small displacements in the other directions (see Figure 7).

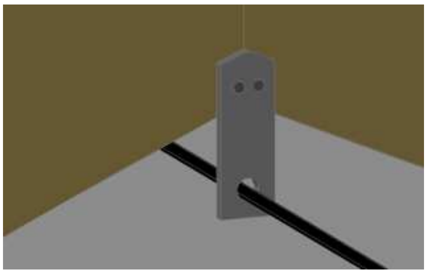

Figure 7. Detail of the metallic element that connects the steel bar to the truss

Moreover, as the steel bars might undergo relaxation, it is important to have a system that allows periodical adjustments so that bars can be effective in its action, bracing the walls and arch. Besides, in order to maintain the initial features of the building, the steel bars to be applied on the roof level should go unnoticed as much as possible, or at least not cause visual contrast (see Figure 8).

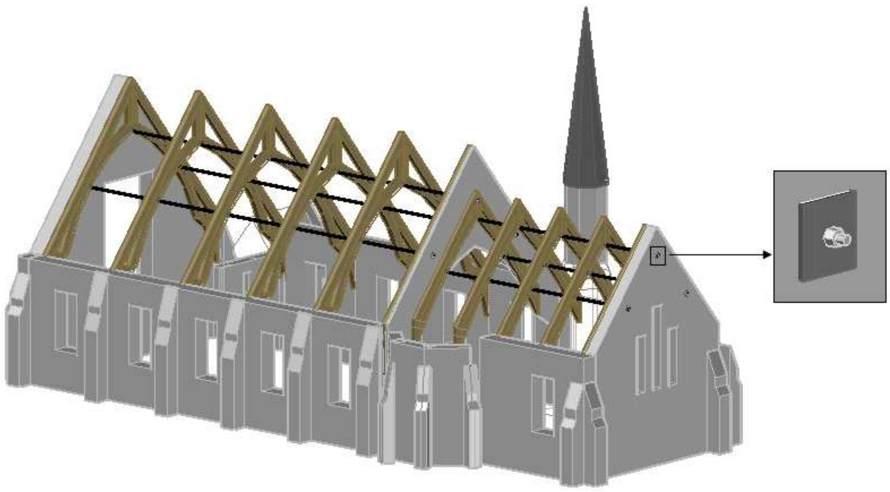

Figure 8. General Overview of the reinforcement solution proposed

Concerning the existing damage in the transversal walls with regard to the cracks, the ones that have more influence in the structural behaviour of the building are the horizontal cracks, so special attention will be given to these. Among the most common techniques in rehabilitation and structural strengthening of masonry walls, the injection of grout or resins can be highlighted mainly because it guarantees that the original aspect of the wall remains unaltered and it recovers its original undamaged resistance, being usually used in buildings of recognized artistic or architectural value, when is necessary to both intervene and preserve the original appearance (Roque & Lourenço 2003). As such, this technique is indicated to restore the initial conditions of the transversal walls and triumphal arch, maintaining its original aspect. However, more than recover the initial strength in the affected areas, it is necessary to increase the resistance of the transversal walls, so as to avoid the formation of new cracks should the building experiment the effects of future earthquakes. In this context, a complementary technique is proposed and presented in Figure 9. In the damaged areas, metallic elements are placed in the masonry joints after removing the mortar, maintaining the original aspect of the wall. Hence, not only does the proposed technique aim to restore the initial capacity of the wall through mortar injections, it also provides greater load capacity to the wall.

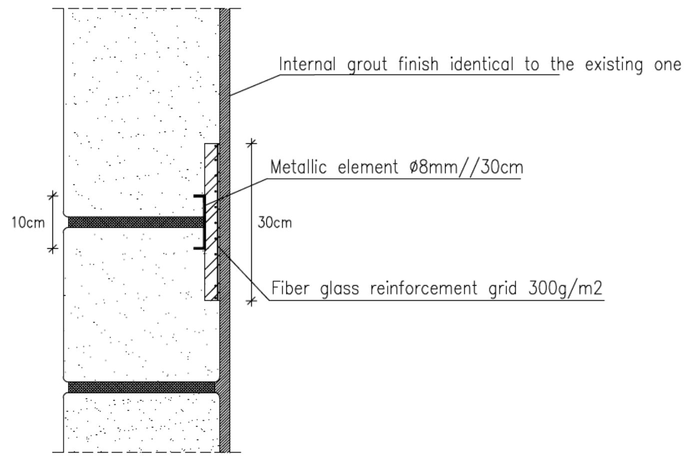

Figure 9. Detail of the reinforcement in the wall section

5.2. Pushover Analysis – Reinforced Structure

According to the reinforcement strategy presented above, the numerical model of the Church was adjusted in order to incorporate these modifications in the structure. The steel tie bars were modelled as beam elements (CL18B), while the walls were considered to be undamaged. Pushover analysis was carried out in this new model and the capacity curves obtained for both the damaged and reinforced model are presented in Figure 10, allowing for a better understanding of the structural behaviour of this new reinforced model, as well as its comparison with the damaged one.

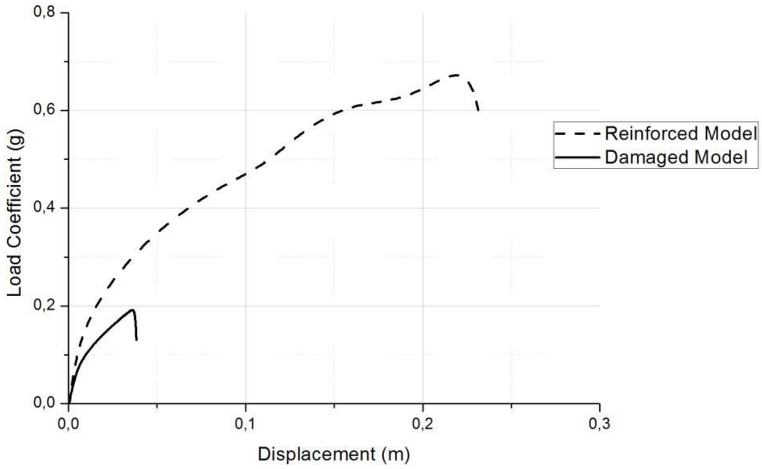

Figure 10. Comparison between the Pushover Curves of the damaged and the reinforced model

Figure 10 shows that the reinforced structure reaches a maximum load coefficient of 0.71g, instead of 0.20g achieved by the initial one (increase of 255%). Furthermore, the maximum displacement that the reinforced structure experiments at failure is around 23 centimetres, which is significantly higher than the 4 centimetres attained by the damaged structure. Therefore, the reinforcement solution proposed presents a significant increase in the structural capacity of the Church, leading to acceptable results both concerning the maximum horizontal load and the displacement capacity attained.

Aiming at a better understanding of the structural behaviour of the new model and its comparison with the damaged, the principal strains distribution is presented in Figure 11 and Figure 12. It can be noticed that this distribution is more widespread for the reinforced model, as expected. The purpose of the reinforcement strategy was achieved, in the sense that the steel bars were able to connect the transversal walls to the horizontal elements of the roof, transmitting the internal loads throughout the structure. It is also important to note that the levels of principal strains that the structure can sustain are considerably higher than the ones achieved by the damaged model. The new structure has a global collapse mechanism instead of a local one.

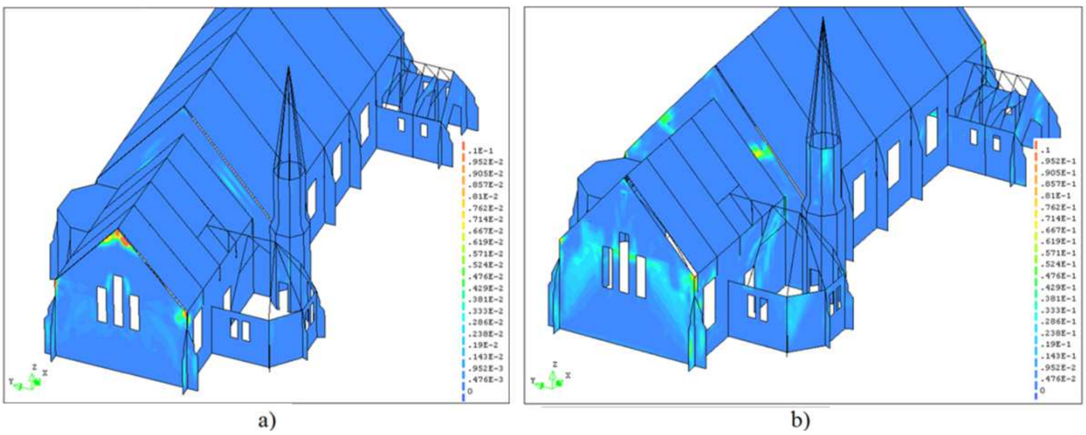

Figure 11. Maximum principal strains distribution for the collapse load (east/north view): (a) Damaged model; (b) Reinforced model

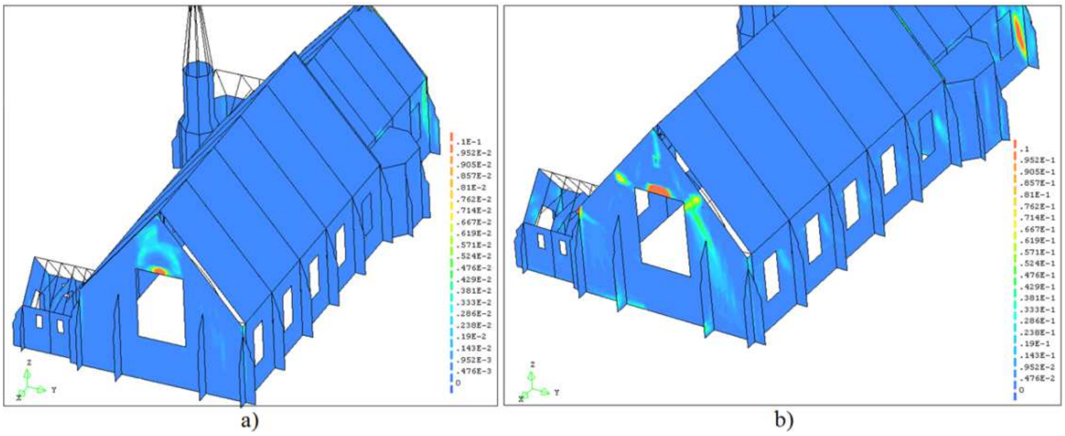

Figure 12. Maximum principal strains distribution for the collapse load (west/south view): (a) Damaged model; (b) Reinforced model

CONCLUSIONS

The seismic assessment of St James Church damaged after the 2011 Christchurch earthquake was presented. A numerical model was constructed using the finite element approach, including the existing damage and calibrated according to the dynamic identification tests and the visual inspection carried out in situ, thus increasing the reliability of the subsequent analysis. Afterwards, pushover analyses were performed and the results show that the capacity of the Church is only 0.20g, which is insufficient taking into account the local seismic hazard, meaning that repair is needed. As such, a reinforcement strategy was proposed and analysed by means of pushover analysis, including the redefinition of the numerical model.

The numerical results of the reinforced model demonstrate that the introduction of the steel bars at the roof level and the convenient repair of the cracks can improve the global behaviour of the Church, reaching a horizontal load capacity of 0.71g. The reinforcement solution allowed the structure to behave as monolithic, as proved by the numerical analysis, with the internal forces being transmitted through all the elements of the structure, increasing the seismic capacity of the Church. The present study demonstrates that a non-intrusive technique, almost without changing the initial features of the Church, can be applied and good results can be achieved.

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