POST-EARTHQUAKE SEISMIC ANALYSIS OF ST JAMES CHURCH, NEW ZEALAND

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Abstract: *The paper presents a numerical study of the seismic assessment of the St James Church in Christchurch, New Zealand affected by the most recent 2011 earthquake and subsequent aftershocks. Dynamic identification tests, as well as a careful visual inspection, were performed in the Church in order to understand its structural behaviour after the seismic action. The seismic assessment of the Church has been done using the finite element modelling technique, in which the nonlinear behaviour of masonry has been taken into account by proper constitutive assumptions. A model updating according to the experimental results was carried out in order to validate the numerical model, which includes the simulation of the damage identified in the structure. Nonlinear pushover analyses were carried out demonstrating that, as a result of the seismic action, the Church can no longer be considered safe.*

Another numerical model was analysed, this time considering the intact structure. The pushover analysis results of this undamaged model show good agreement with the visual inspection performed in situ, which further validates the model used.

Moreover, since the analysis proved that the structure is no longer safe, a reinforcement strategy is proposed and analysed, proving its effectiveness.

1. INTRODUCTION

A vast majority of the existing cultural heritage buildings is made up of masonry. Considering past and more recent earthquakes, it has been recognized that masonry buildings, particularly if inadequately tied, are very vulnerable to earthquakes [1–3]. As such, the architectonic and cultural world heritage is at permanent risk due to the threat that this natural catastrophe represents. In recent years, many archival sources and in situ surveys carried out after seismic events proved that many heritage buildings have been damaged. Several examples of studies regarding masonry constructions affected by past earthquakes can be found in literature, e.g. [3–6].

The increasing awareness regarding the preservation of heritage buildings as a way to protect this patrimony for future generations leads to the need for the study and assessment of masonry buildings in historical centres around the world, like in Portugal, Italy, Greece, Turkey, Peru, Mexico, etc. [4]. In fact, for countries defined by ancient civilizations, the seismic protection of masonry buildings must also take into account the issue of protecting the cultural heritage. Furthermore, considering that seismic events are also a threat to human live, it is imperative to gain knowledge on how masonry buildings may collapse, so as to minimize casualties and injuries to people in their interior [3]. Due to the vulnerability of masonry to earthquakes, research of seismic behaviour of masonry structures is nowadays almost entirely dedicated to existing buildings with the aim of evaluating and reducing their seismic vulnerability [7].

The present study is focused on the seismic assessment of a masonry building affected by the recent 2011 New Zealand earthquake. The 6.3 magnitude earthquake was a powerful natural event that severely damaged New Zealand's second-largest city, Christchurch, causing 185 known casualties and resulting in widespread damage throughout the city buildings. St. James Church, located in Christchurch, has suffered extensive structural damage with the February earthquake and subsequent aftershocks. As such, numerical analyses were performed using a finite element model (FEM) calibrated with experimental results obtained by dynamic identification tests, with the aim of understanding the structural behaviour of the Church as well as evaluate its safety. The evaluation of the seismic vulnerability of masonry structures depends on reliable numerical simulation of their seismic response. As such, the model updating proposed in this study presents a relevant contribution. Two numerical models were constructed, the first one considering the damage on the building and the other considering the undamaged structure. Both models were subject to pushover analyses, each with a different purpose. On the damaged one the intention was, first and foremost, the calibration of the numerical model according to the visual inspection and the experimental results carried out in-situ, followed by the safety evaluation of the Church. On the other hand, the model associated with the undamaged structure was used as a reference when comparing its results with the crack pattern currently observed, as well as to also allow for a clear understanding of the seismic behaviour of the Church. Afterwards, according to the obtained results, a reinforcement solution is proposed, taking into account the role of historical heritage of the building, trying to maintain as much as possible the original aspect of the Church. The software DIANA 9.4 [8] was used as the main resource both for the construction of the numerical models and to perform the pushover analysis.

2. NUMERICAL MODELLING

2.1. Church description

St James Church is located in Christchurch and was constructed in 1923 by the architect brothers Alfred and Sidney Luttrell, to provide a permanent centre for Anglican worship. The Church was built in a simpler early English Gothic style, as a memorial to the men of Riccarton, in Christchurch, who had died in the First World (see Figure 1).

Figure 1: General view of St. James Church

Regarding the church geometry, it has nearly rectangular shape with a long nave that constitutes the body of the church as well as other secondary compartments that also 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. Furthermore, the building structure includes a large number of buttresses in all façades and corners, which seems to indicate a clear earthquake concern. 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 [9]. During the in-situ inspection, specimens were taken from the walls in order to know their constitution. Three layers of materials are present, namely: brick on the inside of the wall, weak concrete in the inner core and stone in the outside façade. Thus, the main material is masonry both in the walls and buttresses, and wood beams on the roof structure.

2.2. Damage identification

As a consequence of the 22th February 2011 earthquake and subsequent aftershocks, the Church has suffered both structural and non-structural damage [10]. In order to identify the main damage in the structure, two different approaches were adopted: a detailed visual inspection and dynamic identification tests. The visual inspection carried out in-situ allowed the identification of the main visible damage. As a result, the transversal walls (east and west) and triumphal arch were the elements with more perceptible structural damage. Indeed, in the east and west wall is evident the formation of a significant horizontal crack located in the alignment where the corner 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. With regards to the longitudinal walls, these do not have much earthquake-related damage, presenting only some thin diagonal cracks on the windows base. As such, the visual inspection allowed the verification that the damage is concentrated in the transversal walls and is mostly due to out-of-plane movements, even if some diagonal cracks due to in-plane movements were also observed.

The second approach used to identify the main damage on the Church was the dynamic identification tests, allowing for further characterization of its structural behaviour. The extensive images collection as well as the damage observed in-situ allied to the dynamic tests results, 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.

2.3. FEM Model

The issues concerning the study and seismic assessment of historical masonry buildings are reviewed in Lourenço [11], such as the constitutive laws that characterize the masonry behaviour; the numerical modelling approaches available for the study of these structures, the seismic analysis methods, among others. The finite element continuum modelling (FEM) has been widely and successfully used in the study of heritage masonry buildings, e.g. [12–14]. Thus, for the study and assessment of St James Church a 3D finite element model was constructed aiming the proper simulation of its structural behaviour. Eight node quadrilateral (CQ40S) and six node triangular continuum shell elements (CT30S) were used to simulate walls and three node beam elements (CL18B) were employed to model the timber trusses of the roof [8]. The first numerical model was prepared taking into consideration the structural damage identified in the building. With regards to the elastic properties of the materials, a constant Poisson's ratio ν of 0.2 was used for all materials, while a value of E equal to 12 GPa was assumed for wood. For masonry, the value of the elastic modulus was obtained from the model updating using the dynamic identification results, reaching an optimized parameter equal to 9.43 GPa, which seems acceptable due to the good quality of the masonry and the inner concrete core. Afterwards, another numerical model was prepared considering the intact structure maintaining the calibrated parameters.

3. SEISMIC ASSESSMENT

A set of proportional mass pushover analyses was performed for both numerical models with different purposes. On one hand, the simulation regarding the damaged model was analysed with the aim of assessing the current seismic capacity of St James Church. On the other hand, pushover analysis was carried out in the intact structure model in order to discuss if a repair strategy, e.g. to reinstate the original capacity of the building, would be enough to comply with the code requirements. In addition, this analysis allows further validation of the model, by comparison between the new results and the crack pattern obtained after the 22th February earthquake.

As expected, the direction perpendicular to the transversal walls is proven to present 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. Concerning the pushover analysis in the damaged model, the principal tensile strains are plotted as an indicator of damage (see Figure 2). In this analysis damage is concentrated in the transversal walls as a result of out-of-plane deformations. Moreover, it can be easily noticeable that the maximum concentration of strains occurs on the top of the tympanum of the east wall, limiting the capacity of the Church. 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 can then lead to the out-of-plane collapse of this whole part of the structure due to the pre-existent horizontal crack.

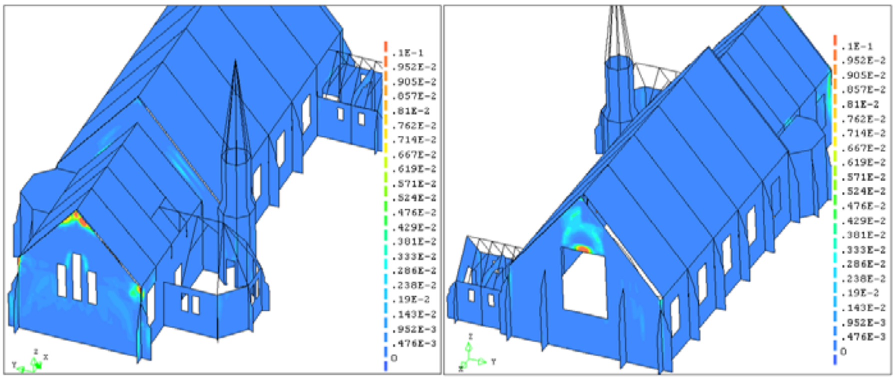

Figure 2 : Principal maximum strains distribution on the damaged model

The analysis results of the undamaged model were evaluated and compared not only with the crack pattern due to the earthquake, but also with the previous analysis. As presented for the model with existing damage, the principal strains distribution resulting from pushover analysis is exposed in Figure 3. The damage obtained in the pushover analysis is in reasonable agreement with the damages observed in the structure, i.e. the horizontal crack formation in the east wall or the developing of diagonal cracks on the longitudinal walls.

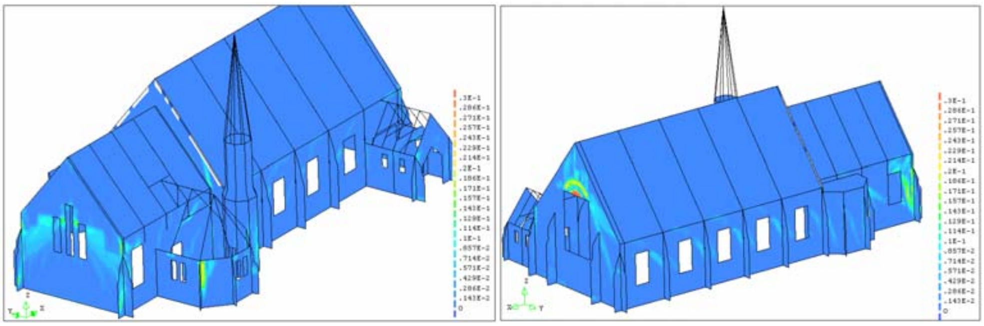

Figure 3 : Principal maximum strains distribution on the undamaged model

The analysis of the capacity curves depicted in Figure 4 shows differences between the maximum load coefficient values in both numerical models. For the damaged structure, the capacity is determined by the out-of-plane collapse of the gable wall reaching 0.20g. Nevertheless the maximum load supported for the undamaged structure was around 0.40g, with the seismic capacity being determined by the transverse direction, which means that the earthquake reduced the capacity to the half. Both pushover curves present similar shape though with different scales, beginning with a linear development, followed by a clearly non-linear path until the maximum load is reached. The curves end with a post peak softening behaviour, consisting of a gradual decrease in the mechanical resistance of masonry under a continuous increase of deformation. Overall, considering the results related to the Church's seismic assessment and pushover analyses, a conclusion should be made: due to its extensive damage, the capacity of the Church is now relatively low (0.20g). A reinforcement solution is proposed aiming to increase the seismic capacity of the Church.

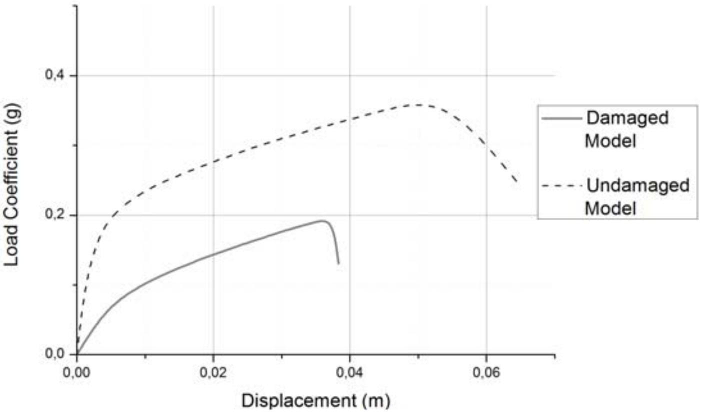

Figure 4 : Comparison between the pushover curves of both numerical models

4. REINFORCEMENT STRATEGY

As revealed by the results provided by pushover analysis, St James Church is no longer safe due to the earthquake damage, as such the structure requires repair or strengthening. A reinforcement and repair solution is proposed, bearing in mind the high patrimonial and cultural value of the Church, and then analyzed numerically.

With regards to the damage identification, provided by the experimental results, visual inspection and the pushover analysis results, it should be highlighted the deficient connection between the roof structure and transversal walls, which is compromising the global capacity of the Church. In order to connect the transversal walls with the triumphal arch, steel tie bars are introduced at the roof level to attach the roof structure to the walls on both sides, allowing the structure to be fully mobilized. The longitudinal deformation of these bars is controlled and reduced with the introduction of some non-structural metallic elements attached to the existent trusses. Furthermore, the existing damage in the transversal walls with regards to the cracks should be also taking into consideration. The injection of grout or fluid resins for the repair of cracks was proposed because it guarantees that the original aspect of the wall remains unaltered and it allows to recover its original undamaged resistance [15]. According to the reinforcement strategy presented below, the numerical model of the Church was adjusted in order to simulate these modifications and pushover analysis carried out. The results show a more widespread damage distribution for this structure, as expected. It is also important to note that the levels of principal strains that the structure can sustain are considerably higher than the ones achieved for the damaged model. Moreover, the capacity curve achieved for this model reaches a maximum load coefficient of 0.71g and a quite peak displacement of 23 mm, significantly higher than the initial values of 0.20g and 4 mm, respectively (see Figure 5). The initial stiffness of the structure increases slightly. The initial goal was thus achieved in the sense that the building is now able to behave monolithically, transmitting the efforts throughout the structure. The reinforced structure has a global collapse mechanism instead of a local one.

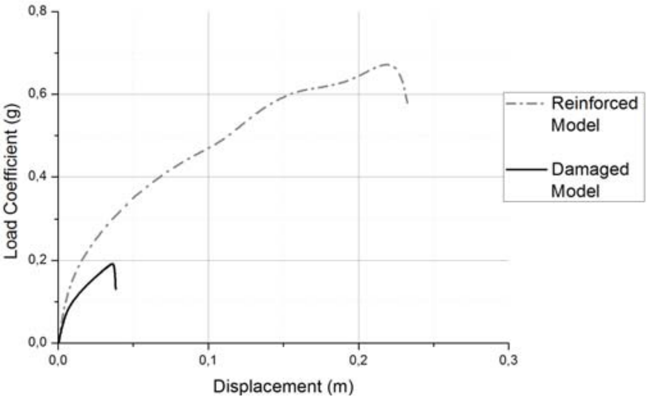

Figure 5 : Comparison between the pushover curves for the reinforced and damaged models

5. CONCLUSION

The seismic assessment of St James Church, after the Christchurch earthquake in New Zealand, was presented as well as the numerical study of the undamaged structure. A reinforcement solution was also proposed and analysed by means of pushover analysis. Furthermore, the dynamic identification of the damaged church was carried out allowing the calibration of a finite element model with extensive damage, thus increasing the reliability of the subsequent analysis.

The results related to the seismic assessment of the Church affected by the 2011 earthquake proved that the structural capacity of this building is 0.20g. With regards to the numerical study carried out on the undamaged structure, it reveals that the Church had twice the structural capacity (0.40g) before the earthquake, which seems consistent when compared with the previous one. Furthermore, the damage obtained in the pushover analysis is in reasonable agreement with the crack pattern observed in the structure after the earthquake, which further validates the model used. Concerning the reinforced model analysis, the results revealed that the applied technique, if properly employed, could lead to a significant increase on the global capacity of the building reaching 0.71g, which is significantly higher than the local seismic hazard. 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.

6. REFERENCES

[1] Abrams, D. P. “Performance-based engineering concepts for unreinforced masonry building structures,” *Progress in Structural Engineering and Materials*, vol. 3, no. 1, pp. 48-56, Jan. 2001.

[2] Lagomarsino, S. “On the vulnerability assessment of monumental buildings,” *Bulletin of Earthquake Engineering*, vol. 4, no. 4, pp. 445-463, Sep. 2006.

[3] Furukawa, A. and Kiyono, J. “Casualty estimation in the collapse of masonry structures due to earthquakes,” *Protection of Historical Buildings - PROHITECH 09*, pp. 423-429, 2009.

[4] Sturm, T. Astroza, M. and Saragoni, G. R. “Vulnerability analysis of two centennial buildings that survived undamaged the two large Valparaiso earthquakes of 1906 and 1985,” vol. 1647, pp. 1547-1552, 2009.

[5] Dogangun, A. Sezen, H. and Livaoglu, R. “Performance evaluation of historical masonry monumental structures during the 1999 Turkey earthquakes,” *Protection of Historical Buildings, PROHITECH 09*, no. 1, pp. 1465-1470, 2009.

[6] Corradi, M. Borri, A. and Vignoli, A. “Strengthening techniques tested on masonry structures struck by the Umbria–Marche earthquake of 1997–1998,” *Construction and Building Materials*, vol. 16, no. 4, pp. 229-239, Jun. 2002.

[7] Magenes, G. “Masonry Building Design in Seismic Areas: Recent Experiences and Prospects from a European Standpoint,” in *1st European Conference on Earthquake Engineering and Seismology*, 2006, no. September, pp. 1-22.

[8] DIANA 9.4, “DIsplacement method ANAlyser, release 9.4, User’s Manual.” 2009.

[9] Ross, J. “Parish of Riccarton-St James” *Riccarton St James’ History*, 1999. [Online]. Available: http://www.riccartonstjames.org.nz/history.html. [Accessed: 14-Jun-2011].

[10] Wilby, G. “St James Riccarton Report - Post 22 February 2011 Earthquake Structural Assessment,” Christchurch, 2011.

[11] Lourenço, P. “Computations on historic masonry structures,” *Progress in Structural Engineering and Materials*, vol. 4, no. 3, pp. 301-319, Jul. 2002.

[12] Annecchiarico, M. Portioli, F. and Landolfo, R. “FE simulation of masonry wall samples of Mustafa Pasha mosque by homogeneous continuum models : Analysis and calibration,” *Protection of Historical Buildings, PROHITECH 09*, pp. 95-100, 2009.

[13] Roca, P. “Studies on the structure of Gothic Cathedrals,” *Historical Constructions*, pp. 71-90, 2001.

[14] Cuadra, C. Karkee, M. and Tokeshi, K. “Earthquake risk to Inca’s historical constructions in Machupicchu,” *Advances in Engineering Software*, vol. 39, no. 4, pp. 336-345, Apr. 2008.

[15] Roque, J. A. and Lourenço, P. B. “Técnicas de Intervenção Estrutural em Paredes Antigas de Alvenaria,” *Construção Magazine*, vol. 7, pp. 4-10, 2003.