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Performance of the Christchurch, New Zealand Cathedral during the M7.1 2010 Canterbury earthquake

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PERFORMANCE OF THE CHRISTCHURCH, NEW ZEALAND CATHEDRAL DURING THE M7.1 2010 CANTERBURY EARTHQUAKE

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Abstract. *The Catholic Cathedral is classified as a category 1 listed heritage building constructed largely of unreinforced stone masonry, and was significantly damaged in the recent Canterbury earthquakes of 2010 and 2011. In the 2010 event the building presented slight to moderate damage, meanwhile in the 2011 one experienced ground shaking in excess of its capacity leading to block failures and partial collapse of parts of the building, which left the building standing but still posing a significant hazard. In this paper we discuss the approach to develop the earthquake analysis of the building by 3D numerical simulations, and the results are compared/calibrated with the observed damage of the 2010 earthquake. Very accurate records were obtained during both earthquakes due to a record station located less than 80 m of distance from the building and used in the simulations. Moreover it is included in the model the soil structure interaction because it was observed that the ground and foundation played an important role on the seismic behavior of the structure. A very good agreement was found between the real observed damage and the nonlinear dynamic simulations described through inelastic deformation (cracking) and building's performance.*

1 INTRODUCTION

The Roman Catholic Cathedral (Figure 1a) is located in Christchurch, New Zealand (NZ) and is considered as a Protected Building Category 1 by the NZ Historic Places Trust (NZHPT). The structural form is primarily limestone used as an internal and external layer keyed into concrete poured in between. The building is widely recognized as one of the finest examples of this type of architecture in Australasia. In the seismological context, NZ is a country under high seismic hazard due to its location in the Pacific Ring of Fire. This seismic area is well known for the presence of large and destructive earthquakes (EQ) due to the interaction between plates, also named interplate EQs.

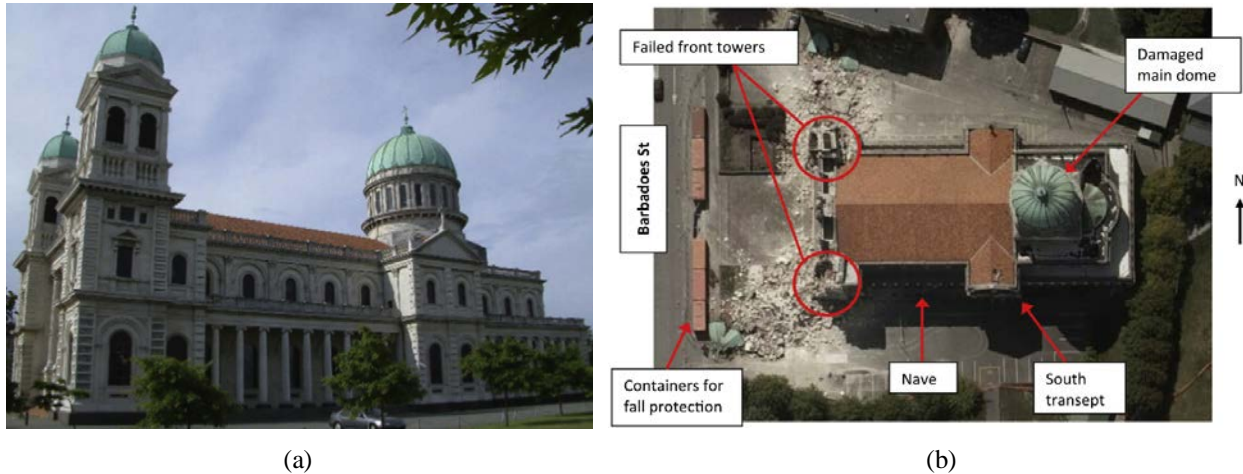


Figure 1. Christchurch Cathedral; (a) general view and (b) 2011 EQ damage [1]

The Christchurch Cathedral was significantly damaged in the recent Canterbury EQ of 2010 and 2011. The building experienced ground shaking in excess of its capacity leading to block failures and partial collapse of parts of the building, which left the building standing but still posing a significant hazard. Compared to the 4 September 2010 seismic event M7.1 (epicenter 35 km away), the EQ of 22 February 2011 M6.3 was the most damaging of all the EQs at this site to date (Figure 1b). This M6.3 event was centered 7 km from the site, at a depth of 5 km, and produced severe shaking across Christchurch city with the maximum recorded horizontal acceleration of 1.41 g at the Heathcote Valley site (1 km from the epicenter). The ground shaking experienced at the site was severe, even though the high acceleration was only for a short period of time. [1] affirms that the magnitude of accelerations that the building experienced was significantly greater than the previously evaluated building capacity for a low period structure and similar to the design level for an Importance Level. The M7.1 2010 event caused some damage to the Basilica, which was generally limited to minor damage to the facing stone, damage that was noted consisted of minor to moderate cracking to the underside of the first floor diaphragm all around the nave and the main dome, minor cracking and displacement of stone in the west wall of the sacristy, and evidence of out-of-plane movement of the middle column of the north bell tower eastern elevation. Access to the building was restricted following this event until engineers could assess the damage and evaluate the remaining seismic capacity. The M6.3 2011 event caused severe damage to the building, inducing a complete collapse of the top section of the South and North bell tower above the main roof level, and an unstable wedge of the western wall above the

first floor level left leaning outward, but still attached and significant damage to the main dome drum, including the entire keystone section of the supporting north arch having dropping out and significant cracking to the south arch. Moreover the collapse of a large section of the first floor roof around the main dome on the north side.

2 METHODOLOGY

The Cathedral suffered minor to moderate damage due to the M7.1 2010 EQ and strong damage after the M6.3 2011 seismic event. Due to the state of damages and the last damaging EQ, the seismic capacity of the building was exceeded. The building was secured with cargo containers to avoid future collapses of weak elements out-of-plane and as contention walls in case of defragmentation due to collapse. The access to the public in general is forbidden and the authorities pretend to keep the building as safe as possible in the meantime and to analyze the possibility of retrofitting the complete Cathedral or leaving some parts of the building as Relic with form of “L” (leaving the front façade and one lateral) by removing unsecured elements and roof and retrofitting the Relic to allow the access of people to the ruins. This project is integrated by Opus International Consultants Ltd. and the University of Auckland in order to analyze the performance of the Cathedral in both 2010 and 2011 EQs and the retrofitting possibilities of the Relic by 3D numerical simulations. This project covers three main stages regarding the seismic protection of this cultural heritage ranked as number 1 in NZ and are summarized as follows:

- 3D Numerical simulations of seismic damage and performance due to the M7.1 2010 EQ of the complete Cathedral
- 3D Numerical simulations of seismic damage and performance due to the M6.3 2011 EQ of the complete Cathedral
- Analysis of most suitable seismic retrofitting of the “L” shaped Relic and capacity

Due to the complexity and extension of the three stages, here is presented only the results of the first stage of this research corresponding to the full 3D numerical simulations of seismic damage and capacity due to the 2010 M7.1 seismic event. The 3D model of the full Cathedral is developed as accurate as possible using the finite element method (FE) and calibrated in the static and dynamic field by linear elastic analyses and key-behavioral characteristics of historical masonry cathedrals of similar size. It is included the interaction between soil and foundation by adding linear elastic springs of different stiffness for both the soil and foundation. The damage analyses of the masonry building are developed by means of full dynamic nonlinear simulations combined with a material model included in ABAQUS[®] for plain concrete and very accurate EQ records obtained at a station located in front of the building under study.

3 CONSTRUCTION OF THE 3D FE MODEL OF THE CATHEDRAL

Firstly, all the non-structural elements are eliminated from the 2D plans and the axis of all identified structural elements is drawn by means of single lines over the 2D plans. It is worth noting that for developing the 3D geometrical model is strongly needed CAD elevations and sections, all the 2D plan views complemented with the photographic survey, field inspections and structural engineering experience. The final geometrical model includes all walls, bell-towers, dome and cover system and is presented in Figure 2. The second stage on the generation of the FE model consists on importing the 3D CAD geometrical model in a pre-processing software such as GID[®]

for assigning surfaces to all structural elements to be meshed by selecting element sizes and types as depicted in Figure 2. It is worth noting that the 3D FE model of the Cathedral was meshed with a non-structured mesh. It is well known that 4-nodes quadrilateral elements are more efficient in terms of numerical approximation. However, due to the geometrical complexity of the structure, it was decided to use an automatic meshing of triangular elements to simplify the process. Moreover, this meshing technique is easily developed and adapted to irregular geometry and openings. It is important to take into account that for linear elastic analysis the size of the mesh elements does not have a huge impact on the results and nor in the computational time. The main issue is during the nonlinear analyses with refined meshes, where convergence problems and calculation time are substantially increased. However, based on own expertise, the size of FE elements in nonlinear analysis does not play an important role since it is enough to identify generalized zones of plastic deformation to determine crack patterns or the final failure modes. One recommended FE size in literature and most used is 1x1 m. Refined meshes are mainly used in small structures, macro-elements or contact areas in linear elastic analyses.

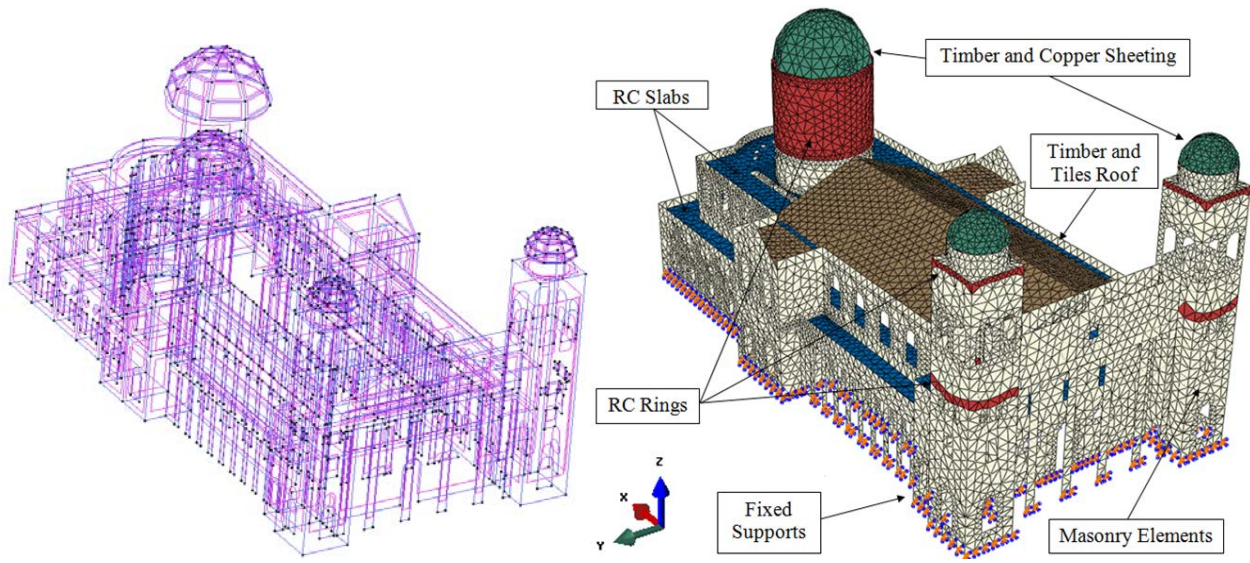


Figure 2. Full 3D geometrical and meshed model including the 2002 retrofitting scheme

3.1 Description of the 3D FE model

Once the input/batch file containing all the geometrical information and coordinates and connectivity of the FE has been defined in the commercial software ABAQUS[®], it is assigned the different materials and model's boundary conditions. Afterwards, linear elastic analyses consisting on vertical loading and dynamic analysis are carried out to identify possible zones of tension (cracking) and characterize the natural frequency and shape modes. These analyses are helpful to verify the model by comparing the sum of vertical forces at the supports and comparing the results with the total vertical load of the structure. Since there is no dynamic information of the building in original state before being strongly damaged by the 2011 EQ (see Figure 1b), becomes more difficult the dynamic field tests because the building has been divided into macro-elements by the strong cracking and most of the main structural elements have been removed or collapsed such as towers and dome, moreover parts of cover and other damaged elements. Due to the lack of real dynamic data of the building in original conditions, the FE model has been con-

structured as reliable and accurate as possible by including all the different materials and thicknesses of all structural elements even those that are not of interest in the damage assessment as timber roof and upper parts of domes and towers. To increase the reliability of the model it was implemented the former retrofitting measures applied in 2002 to address key building areas identified to be most vulnerable to EQ damage (Figure 2 on the right).

Table 1. Material parameters and thicknesses for the different structural elements

Material	Density [kg/m ³]	E modulus [MPa]	Poisson's ratio	Compressive strength [MPa]	Tensile strength [MPa]
Masonry (thk= 50 cm)	2000	2000	0.15	8	0.8
RC slabs (thk= 25 cm)	2400	41000	0.20	40	4
RC rings (thk= 20 cm)	2400	41000	0.20	40	4
Timber and copper (thk= 10 cm)	700	11000	0.30
Timber and tiles (thk= 15 cm)	700	11000	0.30

The selected FE for all structural elements is a general purpose triangular shell (S3R), which has three nodes and three thicknesses with six degrees of freedom (DOF) at each node. This element could represent the in-plane and out-of-plane behavior and has plasticity capabilities. The 3D model (see Figure 2) is integrated by 25024 shell elements and 14577 nodes with 87462 DOF and a total mass of 6922 Tons (vertical force of 67911 kN). In the generation of the FE model the following main assumptions were considered: as a first approach all base nodes are assumed as fixed, the materials and thicknesses are described in Table 1, as well as their respective linear elastic parameters and strengths. The main mechanical properties of the three layers masonry were proposed taken into account an equivalent layer with average properties.

4 VALIDATION OF THE 3D FE MODEL BY LINEAR ELASTIC ANALYSES

As a first approach and aimed at obtaining significant progress on the seismic risk assessment of the Christchurch Cathedral without the convergence problems related to nonlinear analyses, static and dynamic linear evaluations such as vertical loading and modal are developed. This first approach based on linear principles permits to determine the presence and magnitude of tensile and compressive stresses in the masonry structure generated by vertical loading, as well as the dynamic properties (frequencies and vibration modes) in the modal analysis. In the generation of structural models of real historical constructions there are many assumptions and uncertainties regarding the determination of geometry, material properties, support and boundary conditions. In this case, the linear analyses could be used to calibrate (or up-date) the initial model with the experimental data by adjusting geometry, material properties and interaction with adjacent buildings. This permits to obtain models more representative of the structure under study, and with this, a reliable EQ performance evaluation. In the case of the Christchurch Cathedral the first natural frequencies are compared with those of other Cathedral located in Mexico with similar size, structural elements, materials and soil characteristics.

4.1 Simulation and analysis of load bearing capacity

This approach is often used to assess the vertical load carrying capacity of a building under self weight and service conditions. In the case of historical masonry structures the vertical load plays an important role in the seismic behavior, because these structures were constructed by empirical rules only to withstand their self weight. These massive structures are extremely heavy, presenting important inertia forces amplifications during an EQ that could generate strong damage. Therefore it is important for the seismic analysis to firstly evaluate the distribution of vertical stresses at different parts of the structure and to verify that they are not higher than the intrinsic strengths. Analyzing the vertical distribution of stresses at the Cathedral model is worth noting that most of the masonry elements are under compressive conditions such as walls, dome and both towers presenting the maximum values in the order of 1.02 MPa. However, the values of compressive stresses are lower than the intrinsic strength of masonry. Some of the RC slabs present tensile stresses due to the fact that they are supported by masonry architraves which are very vulnerable to suffer inelastic deformation (cracking). It has been identified another large area with tensile stresses at the top slab close of the heavy and tall dome and at the base of it. This is in agreement with real behavior observed in similar domes. Moreover the top slab presents the maximum compressive stresses in the order of 3.39 MPa. It could be concluded that the Cathedral is stable to satisfactory resist at least its self weight since the compressive stresses are lower than the intrinsic strength and most of the tensile stresses are present at the retrofitted parts by RC elements. To corroborate the results, a sum of vertical forces is developed at all supports and compared to the dead load of the Cathedral, which is equal to 67911 kN.

4.2 Simulation and analysis of vibration characteristics

To obtain a first estimation of the vibration response of the Christchurch Cathedral, the linear investigations are extended to dynamic analysis. As in the case of the vertical loading analysis, the modal evaluations of FE models are relatively fast due to the progress of recent decades on computational tools. The fundamental vibration modes and their respective natural frequencies (in Hz) and periods (in sec) are illustrated in Figure 3. It is worth noting that the global dynamic behavior of the building is governed by a lateral vibration in the transversal direction (N-S) and longitudinal (E-W) respectively. The observed dynamic behavior on this building by following the theory of small displacements in the linear range is characteristic of massive masonry Cathedrals. To compare the dynamic characteristics of this historical building under study, the Cathedral of Colima, Mexico may serve as an example because has similar dimensions, materials, distribution and structural elements such as two towers and a heavy and tall dome as described in [2] and [3]. The only difference is on the top roof system which is integrated by masonry vaults. This Cathedral was seriously damaged after the 2003 EQ (M7.5) and presented similar failure mechanisms like those observed at the Christchurch Cathedral in 2011. The Cathedral of Colima was also strengthened with RC rings at towers and RC on slabs and replacement of damaged units as rehabilitation. Afterwards was subjected to detailed investigations both numerically (3D limit analysis and 3D FE) and experimentally in-situ regarding dynamic behavior by assessing the first natural frequencies with vibration equipment (accelerometers) and taking into account the excitation induced by the ambient and traffic. The first experimental and numerical natural frequency was measured at the dome level and was 2.200 Hz, which is in good agreement with the one obtained numerically at the Christchurch Cathedral of 2.105 Hz.

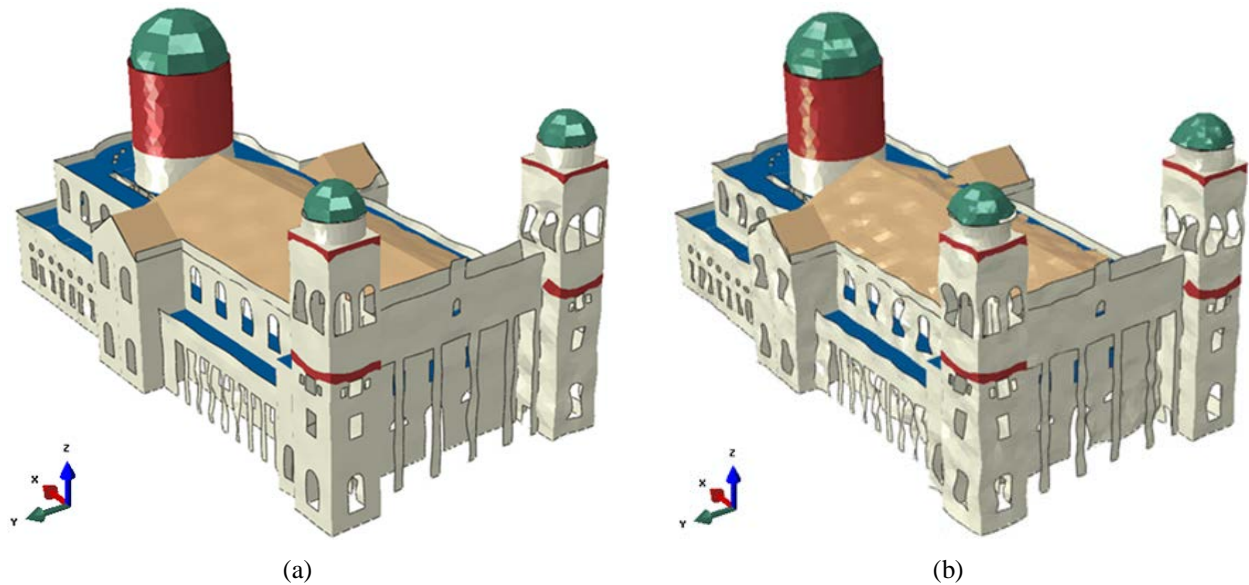


Figure 3. Fundamental vibration modes and natural frequencies of the 3D FE model: (a) N-S $f= 2.105$ Hz ($T= 0.475$ s) and (b) E-W $f= 3.145$ Hz ($T= 0.318$ s)

5 LINEAR DYNAMIC ANALYSES OF THE CATHEDRAL

Before developing a detailed and time consuming nonlinear full dynamic analysis (also named time-history) for assessing performance and failure modes, results very important to manage the seismic input that will be used and to develop a series of linear dynamic analyses to verify that the EQ is correctly applied and the structural response is characteristic. Also it is verified the suitability of applying a complete EQ for all the directions (two orthogonal and vertical components) acting at the same time on a large scale model like the 3D Christchurch Cathedral model. Other researchers usually apply only one of the three components of the EQ and divide large structures in macroelements to simplify the analyses but disregard the overall structural behavior, being the results in a certain way unreliable and non-representative of the real response.

5.1 Simulation of soil-structure interaction

When a certain structure is subjected to an EQ, its base (foundation) tends to follow the movement of the terrain and the mass of the structure tends to oppose to that movement, leading to the transmission of important inertia forces. These forces generate at the base of the structure shear forces and moments. In the framework of the Soil-Structure interaction philosophy, the seismic response of a building is dictated by the interaction between the structure with the foundation and the underlying soil/rock. This interaction is commonly simulated by the addition to the numerical model of linear elastic springs of constant stiffness to include the damping effect provided by the foundation and soil. Vertical springs represent the soil and the horizontal ones the foundation, both groups with different stiffness and damping characteristics. In previous stages of this research, the 3D model of the Christchurch Cathedral was modeled as fixed to the base disregarding the soil-structure interaction (see Figure 2). After the 2010 and 2011 EQ sequence it was observed that the foundation and the soil characteristics played an important role on the damage scenario presented by the building. Due to this and to build a more accurate and realistic model it was decided to implement a uniform distribution of vertical and horizontal linear elastic springs

to represent the soil-structure interaction. The sum of vertical spring/dashpot elements (SPRING2/DASHPOT2) for the vertical component (Z) corresponds to 453 and a total of 906 (453 in X component and 453 in Y) for both horizontal components. For defining the spring stiffness (118.68 kN/mm) of the foundation was used the value proposed by [2] to simulate the interaction induced by neighbor masonry buildings and the spring soil stiffness (83.5 kN/mm) was defined taking into account the proposed by [4] and [5].

5.2 The 4th of September 2010 Canterbury earthquake as seismic input

The magnitude Mw 7.1 Canterbury EQ on the 4th September 2010 (4:35am NZST) had resulted in widespread liquefaction, land damage, buildings and infrastructure damage in the Canterbury region. The EQ epicenter was approximately 35 km west of the Christchurch (Ōtautahi) Central Business District (CBD). While the EQ impact was widespread and a number of unreinforced masonry buildings suffered significant damage there was no loss of life [6]. The 4th of September 2010 EQ was triggered by a sequence of four faults' rupture, resulting in a large magnitude EQ [7]. The recorded peak ground horizontal accelerations were up to 0.15g at the Catholic Cathedral College Christchurch (CCCC) recording station (see Figure 4). The CCCC station is located about 35 km away from the 2010 EQ epicenter and this station is very close of the Christchurch Cathedral, just in the opposite street (80 m).

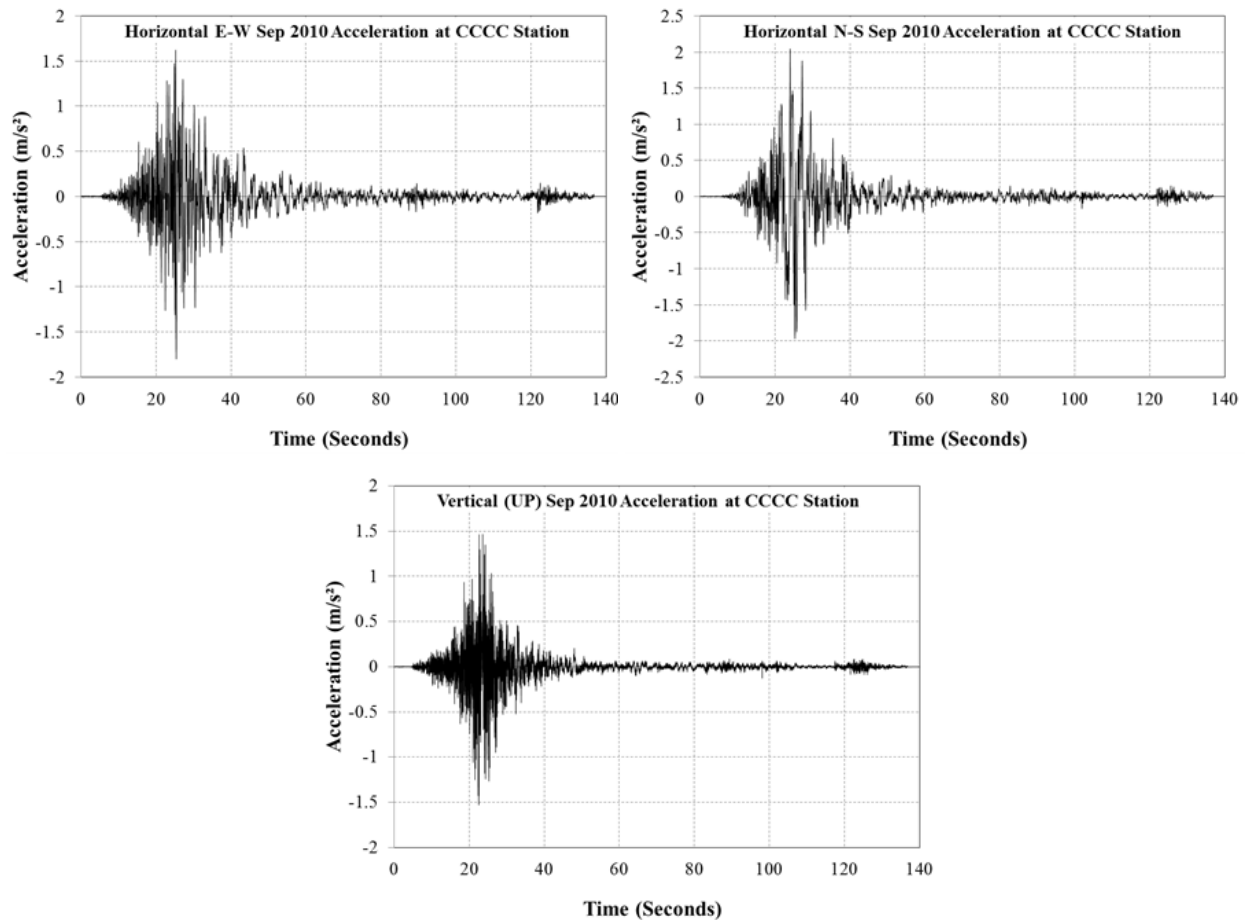


Figure 4. The 4th of September 2010 acceleration ground motion measured at the CCCC Station

A common procedure to include the seismic input into the 3D FE model for seismic analysis is by selecting the intensive phase of the EQ, disregarding the initial and final parts where the acceleration starts and ends. This procedure is also named EQ window and is developed to reduce the computational time and to focus on the response of the structure subjected to the important part of the EQ. The total duration of the record is of 140 seconds and peak ground acceleration (PGA) higher than 0.15g as illustrated in the three components of acceleration in Figure 4. For subjecting the Christchurch Cathedral to initial linear dynamic analyses it was selected the three components (E-W, N-S and Up) of the September 2010 EQ in terms of acceleration. The selected EQ window has a duration of 20 seconds and is worth noting that the strong motion record presents large impulses in the range of 0.10g to 0.20g.

5.3 Seismic input verification and analysis of response at the 3D FE model

The 3D seismic input based on acceleration (see Figure 4) was applied at the base (foundation and soils springs) of the FE model of the Cathedral as shown in Figure 5a. After developing the linear dynamic analysis, it was verified that the seismic input was correctly applied by verifying that all the components of acceleration at the springs base are the same as the applied. As a second stage it is verified that the model behaves as a structure under EQ conditions by presenting the characteristic deformation and transmission of inertia forces. As a final verification the structural seismic response in terms of acceleration is measured at a control node at the highest part of the building corresponding to the bell towers. The aim of measuring the acceleration at a top node is to verify that the acceleration is higher (amplified) than the one of the terrain (see Figure 5b).

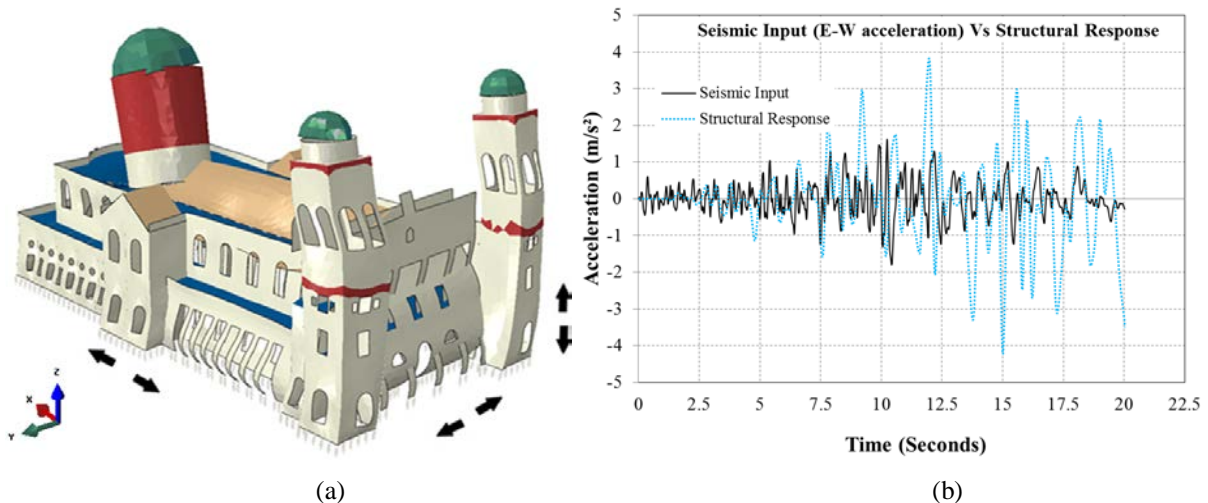


Figure 5. Seismic input verification and analysis of FE model response; (a) 2010 M7.1 EQ applied in three directions (E-W, N-S and Up) and (b) EQ vs. structural response at a control top node of bell-tower

[8] affirms that due to the natural bending behaviour and the high inertia forces of masonry towers generate that the structure vibrates in a different way of that of the terrain, leading to top amplifications of about 2.3 times. This trend could be observed in the comparative between seismic input and structural response of Figure 5b, where the top left bell-tower presented a maximum acceleration of 0.43g. By comparing the PGA (0.17g) of the acceleration input (E-W) with the response, it is worth noting that the tower presented an amplification factor of 2.5 which is in

very good agreement with the aforementioned. Analyzing the state of vertical stresses at the point of the analysis where the structure presented the maximum response, it was observed the elastic deformation (see Figure 5a) of vertical elements as columns and even more obvious the presented the bell-towers. The positive values at the contour represent tension and are present in large parts of the structure. This could be interpreted as zones of cracking due to the fact that masonry structures have zero resistance to tensile stresses. By means of the analysis of a recorded video it is possible to visualize the complete behavior of the Cathedral during the intensive phase of the EQ corresponding to 20 seconds.

Due to the flexibility of towers they presented large out and in-plane deformation as well as torsional effects. The huge central dome is also very excited due to the nature of the lateral and vertical seismic input, presenting the typical behavior of this kind of structures with strong damage at the base and supporting arches. Because of the height and natural bending behavior of bell-towers, these structures are less excited when subjected to the vertical component of an EQ. the opposite occurs for domes.

6 2010 M7.1 EARTHQUAKE PERFORMANCE OF THE CATHEDRAL

The construction of the 3D FE model and the nonlinear numerical assessments are developed with the commercial software ABAQUS[®] and its concrete damaged plasticity model. The selected modelling technique is the macromodelling approach, which is the most recommended in literature for large historical masonry structures. In this technique, the material conformed by units and interfaces is treated as a composite. The linear elastic parameters are described in Table 1 and the damping and inelastic parameters are those suggested by other authors to describe the nonlinear behavior of plain concrete. As a first approach the building will be subjected to nonlinear dynamic investigations to assess its behavior and failure modes due to the 2010 EQ. The results are analyzed and compared with the real observed damage.

This comparison between simulated and real behavior is quite helpful to calibrate the model. Figure 6 depicts the results of the full nonlinear dynamic analysis of the Cathedral due to the 2010 EQ. The acceleration was applied in 3-directions, including the soil-structure interaction represented by vertical and horizontal springs. The springs were proposed due to the poor soil conditions, which played an important role on the seismic amplification and observed damage. It is worth noting in the crack pattern of Figure 6 that the damage is limited and corresponds to the described in previous paragraphs, minor to intermediate damage with some cracks at the cover at the back part of the main dome and concentration of cracks at both lateral façades at the concrete floor level. Both bell-towers did not present damage due to the absence of inelastic deformation and reduced lateral displacement of 20 mm, practically remaining in elastic conditions. This lateral displacement may be compared with the different damage states in similar bell-towers investigated by [2]. The arches that support the dome presented large inelastic deformation and rotation. During the intensive phase of the EQ, the main dome presented a maximum lateral displacement of 300 mm, which explains the large damage at the arches and surrounding concrete slab.

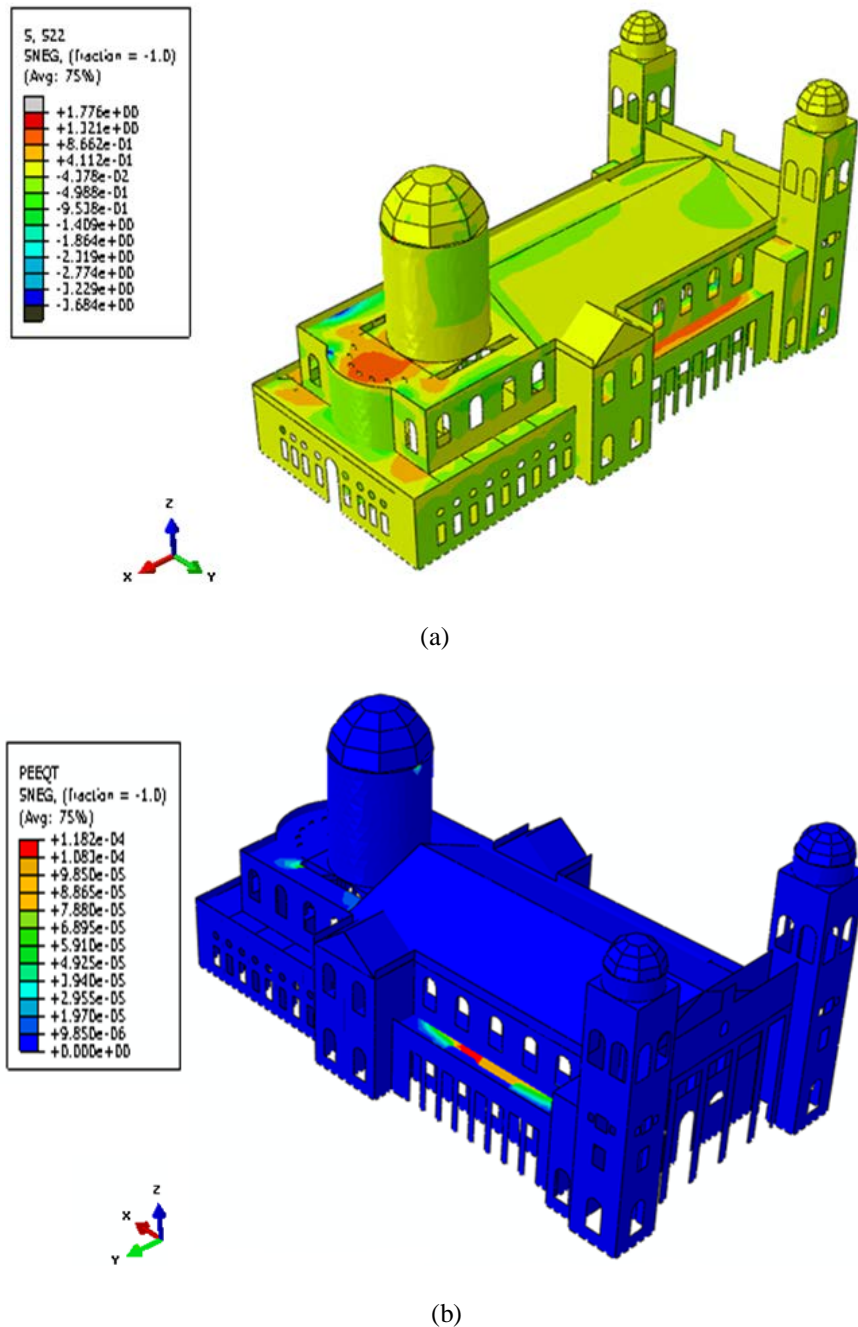


Figure 6. Crack pattern after the 2010 M7.1 EQ ranging from slight to moderate damage during the intensive phase; (a) distribution of vertical stresses and (b) inelastic deformation (cracking)

7 CONCLUSIONS

The first stage of this research was satisfactorily concluded, corresponding to the numerical simulations of seismic damage and performance due to the M7.1 2010 Canterbury EQ. The modelling technique, soil-structure interaction, calibration of the FE model and very accurate EQ records obtained at some meters from the Cathedral played an important role in simulating the 2010

damage successfully. A very good agreement was found between the real observed damage and the dynamic simulations (3-directional) described through inelastic deformation (cracking), which served to assess the building's performance. These results were very helpful to conclude that the constructed/calibrated 3D FE model of the Cathedral is accurate and representative of the real structure. However, more research is needed regarding the interaction between soil and foundation, which was represented by linear elastic springs of different stiffness. In the second stage of this research, the springs' stiffness will probably be calibrated and also the material parameters in order to reach the observed failure mechanisms in the Cathedral due to the most damaging EQ occurred in 2011 (M6.3). This EQ was more damaging due to the near source effect (7 km from epicenter), which was also represented by higher PGA at the Cathedral's site. In a final stage of this extensive research, the seismic safety of the relic (L shape) will be assessed in original state and moreover by including the most suitable retrofitting.

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