Post-earthquake damage simulation of two colonial unreinforced clay brick masonry buildings using the equivalent frame approach

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Abstract

In New World colonial countries and territories such as Australia, New Zealand, and the West Coast of North America, the predominant form of unreinforced masonry (URM) construction is relatively modest one or two storey buildings composed of clay brick masonry. Whilst the equivalent frame (EF) approach for modelling the in-plane seismic response of URM buildings is gaining popularity, particularly in Mediterranean countries, the procedure is currently relatively unknown elsewhere and the validity of the procedure when applied to simple and prevalent New World clay brick URM construction has not yet been demonstrated in a meaningful way. To address this deficiency and provide a novel evidence base for use of the procedure by the professional structural engineering community, modelling of the seismic response of two unreinforced clay brick masonry buildings that were damaged during the 2010/2011 Canterbury earthquake sequence is reported. Static and dynamic nonlinear analyses were undertaken by applying the equivalent frame (EF) approach, and time-history records attained during the Canterbury earthquake sequence were used to conduct dynamic analyses and facilitate direct comparison to observed building damage. The effect of diaphragm stiffness on the global response of the two case-study buildings was also studied. It is shown that use of the EF method enabled prediction of the seismic response of the two case-study clay brick URM buildings with a high level of accuracy.

Keywords: *URM buildings, nonlinear analyses, seismic response, equivalent frame model, damage simulation*

26 **1 Introduction**

27 A number of modelling strategies are available to assess the in-plane and out-of-plane seismic response of
28 unreinforced masonry (URM) buildings. These strategies span from the application of continuous models such
29 as used in finite element analyses, to discrete models such as used for masonry micro-modelling, and with
30 some strategies allowing analysis of both the in-plane and out-of-plane response of unreinforced masonry walls
31 to be undertaken whilst other strategies address only one of the two response modalities. An overview of
32 possible modelling strategies for existing URM buildings is given [1]. The study reported herein focused
33 specifically on global in-plane response by using the equivalent frame (EF) approach to model the simulated
34 seismic behaviour of two earthquake-damaged clay brick URM buildings. These two buildings were subjected
35 to the 2010/2011 Canterbury earthquakes, and hence provided a novel opportunity to investigate the validity
36 of the EF modelling approach when applied to relatively modest and yet prevalent historic clay brick
37 unreinforced masonry construction as encountered in many New World colonial countries such as New
38 Zealand, Australia, and the West Coast of North America (including the states of California, Oregon, and
39 Washington in USA, and British Columbia in Canada).

40 The current study of two comparatively simple and regular clay brick URM buildings sits alongside previous
41 investigations that incorporated the EF approach applied to the post-earthquake response of a New Zealand
42 historic complex clay brick URM building having extensive ornamentation, steeply-pitched gabled walls, and
43 conical turrets that was damaged in the Canterbury earthquakes [2], to the post-earthquake assessment of an
44 Italian three-storey unreinforced clay brick masonry residential building damaged in the 2012 Emilia
45 earthquake [3], to post-earthquake assessment of the response of stone masonry buildings in the Azores
46 Archipelago following the 1998 earthquake that struck the island of Faial [4], to the post-earthquake
47 assessment of an historical unreinforced stone masonry building damaged in the 2014 Napa earthquake [5],
48 and to the post-earthquake response of a 13th century European URM tower after the 2012 Emilia earthquake
49 [6]. The reported post-earthquake study also augments previously reported laboratory-based EF simulations
50 that sought to replicate the shaking table response of two full-scale stone masonry building tests undertaken in
51 Italy [7], and the simulated seismic response of two modern European URM clay brick masonry mock-ups that
52 were tested on a shaking table in Portugal [8]. Recognising that the equivalent frame modelling approach has
53 recently been promulgated into the New Zealand national guidelines for the detailed seismic response of simple

54 clay brick masonry buildings [9], the research significance of the reported work was to provide unique
55 justification for the validity of applying the EF procedure to this class of comparatively simple clay brick URM
56 buildings as a consistent national methodology.

57 In the EF approach, each lateral load resisting URM wall is subdivided into a set of URM panels (piers and
58 spandrels), where deformation and nonlinear response are concentrated and connected by rigid portions (or
59 joints). Thus, the EF approach requires a limited number of degrees of freedom and allows nonlinear static and
60 dynamic analyses of complex three-dimensional models of URM structures to be undertaken with reasonable
61 computational effort. In addition, EF idealization of a structural system allows the possibility of introducing
62 other structural elements such as reinforced concrete (RC), steel or wood beams, and columns. Because the
63 modelling procedure requires fewer elements when compared to more complex tools (e.g., for finite element
64 (FE) models), the EF approach is becoming recognized as a useful engineering practice tool and its use has
65 been explicitly proposed in recent guidelines as ASCE/SEI 41-13 [10], Eurocode 8-1 [11], Norme Tecniche
66 per le Costruzioni [12], and NZ guidelines [13]. Comparison with more refined models, such as the finite
67 element approach, provided promising results about the reliability and effectiveness of EF modelling [14, 15],
68 but a systematic validation and standardisation of the rules for its use are still lacking.

69 The following main drawbacks are recognized in the EF modelling approach for URM buildings: 1) restriction
70 of the number of degrees of freedom, 2) idealization of URM walls into ‘equivalent frames’, and 3) neglecting
71 of out-of-plane wall response. These drawbacks introduce uncertainty regarding whether the EF method is
72 applicable for the analysis of complex buildings with/without flexible diaphragms. The issue 2) becomes
73 particularly relevant in case of walls characterized by irregular layout of openings, as testified by recent
74 research works [16, 17].

75 For the reasons explained above, numerical simulations of experimental campaigns and simulations of the
76 actual response of existing buildings during earthquakes constitute a valuable resource to verify results from
77 the EF method and corroborate its use. Shaking table experimental testing [18, 7] offers the advantage of
78 providing rich datasets (i.e., time-history registrations, material properties, force-displacements relationship)
79 that can be compared with the results of numerical analyses. However, it is difficult to simulate full-scale
80 and/or complex buildings in the laboratory and the high experimental cost reduces the number of experiments

that can be undertaken. To test the reliability of applying the EF approach to complex buildings, an alternative approach is to numerically model existing buildings that have experienced strong earthquakes and compare the simulated damage with observed damage. However, uncertainties in actual seismic input and/or the characterization of mechanical properties and structural details remain, that increase the modelling and interpretation difficulties.

The method and the software adopted, Tremuri [19, 20], for the study reported herein have been previously validated, for both experimental shaking table tests [8] and for regular existing buildings subjected to earthquake-induced shaking [21, 3]. The aim of the research presented herein was to investigate the reliability of the EF modelling approach by attempting to simulate the observed seismic response of two irregular case-study clay brick URM buildings that were significantly damaged during the 2010/2011 Canterbury earthquake sequence and subsequently demolished [22, 23], namely (1) Avonmore House (AH) located in Christchurch and (2) Royal Hotel (RH) located in Lyttelton. References on general performance of URM buildings during the earthquakes and demolition issues are available in [24]. RH and AH were standalone structures and clearly exhibited a prevailing in-plane global response during earthquake-induced shaking and were hence selected as suitable exemplar cases for investigating the reliability of the EF approach. Prior to and during building demolition, sustained damage was well documented and detailed photographic records were collected. In addition, construction and alteration drawings were acquired from Christchurch City Council property records and reviewed in detail. The case-study buildings and the damage they incurred during the Canterbury earthquake sequence are described in the subsequent sections.

2 Description of case-study buildings

2.1 Avonmore House (AH)

AH was a heritage building located at 203 Hereford Street, at the corner of Latimer Square, in the Christchurch Central Business District. The building was located on sand and silty-sandy soil that is at least 23 metres deep (from available borehole data). It was a three-storey building with a height of approximately 11.2 metres (excluding parapets) and a plan area of approximately 380 square metres, and was constructed of external clay-brick URM walls on three sides and an unreinforced concrete wall on the north and had timber floor

107 diaphragms supported by the external walls and internal steel frames. Wall thicknesses were estimated from
108 photographic evidence collected during demolition.

109 The building had significant plan irregularities (**Fig. 1** and **Fig. 2**) due to the heavily perforated south and east
110 façades. The west façade was an unperforated clay brick masonry wall with additional stiffness provided by
111 the URM return walls of the elevator shaft, which were four leaves thick (470 mm). The north wall was
112 constructed of unreinforced concrete and was assumed to be non-loadbearing based on photographic evidence
113 of an independent loadbearing steel frame supporting the floor gravity loads in the vicinity. For numerical
114 simulation, this steel frame was considered to be linked to the unreinforced concrete wall. The north wall was
115 initially constructed of unperforated unreinforced concrete with openings subsequently added as part of
116 alterations during the lifetime of the building, and had RC lintels that were installed above and beneath the
117 openings. The timber floor and roof diaphragms spanned between the perimeter URM walls, with intermediate
118 support provided by seven internal steel columns having an assumed cross section of 0.26 metres that were
119 located beneath steel beams and are thought to have been 0.5 metres in height (shown as dashed lines in **Fig.**
120 **2a**). These steel elements were modelled as elasto-plastic with a yield strength of 235 MPa. The dimensions
121 of the columns and beams were based on photographic evidence. In Tremuri the connections between the steel
122 beams and the URM walls were modelled as simple supports.

a) The south-east corner

b) The north-west corner

123 **Fig. 1.** Avonmore House following the 2010/2011 Canterbury earthquake sequence

a) Plan of the ground floor (measurements in metres)

b) 3D view of 3Muri [20] model

Fig. 2. Geometry of AH. 3D numerical model developed with Tremuri software commercial version (3Muri) [20].

Parapet height was estimated to have been 2.5 metres on the south and east (street facing) façades and 1.2 metres on the north and west façades, and the parapet thickness was estimated to have been four leaves (470 mm). It was assumed that concrete ring beams on the south and east façades consisted of three $\frac{3}{4}$ -inch (approximately 19 millimetres) bars for longitudinal reinforcement, without transverse reinforcement.

A lightweight partial extension at the roof level was not modelled in the EF model, but its weight was considered. AH was seismically retrofitted in 1994, with each floor diaphragm having been overlaid with 20-millimetre particleboard. URM perimeter walls and the north unreinforced concrete wall were connected to the floor and roof diaphragms using a combination of adhesive anchors and steel equal angles. Steel parapet restraints were added at the roof level around the building perimeter.

2.1.1 Material properties

Lumantarna [25] reported the mortar compressive strength determined from samples extracted from AH during demolition. Cohesion was based on Lumantarna's tests, and the values suggested in [13] for cement-based mortars were used to determine friction characteristics. While the clay bricks were not tested, surveys made during demolition deemed the brick units to be strong and difficult to scratch. Based on these observations and the recommendations in [13], an average brick compressive strength of 26 MPa was adopted for modelling. Moreover, the equations proposed in [13] were used to establish: 1) probable masonry compressive strength, 2) Young's modulus of the masonry, and 3) masonry shear modulus. **Table 1** reports the results. For Young's

142 modulus of the masonry, values of 500 times instead of 300 times the masonry compressive strength were used
 143 because for strong cement mortars, this relationship better matched the value suggested in [11].

144 **Table 1:** Summary of the masonry mechanical properties assumed for Avonmore House

Brick compressive strength	26 MPa	From Table 10.3 in [13]
Brick tensile strength	3.1 MPa	From Table 10.3 in [13]
Mortar compressive strength	18 MPa	From tests on mortars in [25]
Cohesion	1.2 MPa	Assumed from tests on mortars in [25]
Friction	0.8	From Table 10.4 in [13] for strong mortars
Masonry compressive strength	20.6 MPa	Using equation 10.1 in [13]
Secant (cracked) Young modulus*	5.14 GPa	500 times masonry compressive strength
Secant (cracked) shear modulus*	2.06 GPa	0.4 masonry Young modulus
Masonry density	18 kN/m ³	From Table 10.6 in [13]

145 * Initial elastic stiffness was assumed to be double as suggested in [11].

146

147 2.1.2 Damage analysis

148 The 4 September 2010 Darfield earthquake [22] was the first in the 2010/2011 Canterbury earthquake sequence
 149 to strike the AH building. While no photographic evidence was found regarding the condition of the building
 150 after this earthquake, a rapid assessment form compiled on 5 September 2010 reported neither moderate nor
 151 severe damage apart from the risk of an overhead falling hazard (due to broken glass) and marked the building
 152 as safe. The 22 February 2011 event was the third-largest earthquake to strike AH in less than six months, but
 153 this smaller magnitude event had an epicentre located closer to the AH building and therefore generated greater
 154 shaking intensity at the site. The region experienced an extensive number of aftershocks, the strongest of which
 155 occurred on 26 December 2010. While no photographic evidence was found regarding the state of the building
 156 after this ‘Boxing day’ earthquake, the rapid assessment form completed on 27 December highlighted a severe
 157 risk of “collapse/partial collapse” and “overhead falling hazard” but included no description of which elements
 158 were damaged. The form indicated that fencing was required for safety and recommended that an engineering
 159 assessment be undertaken. Following the 22 February 2011 Christchurch earthquake, AH experienced
 160 extensive damage, particularly to the south façade (Fig. 3), with large cracks exhibited in piers and spandrels.
 161 The piers were most severely damaged at the top storey, and the spandrels incurred a significant increase in
 162 the level of earthquake damage at all storeys, with both flexural and shear failures triggered.

Fig. 3. The south façade of AH after the 22 February 2011 earthquake (small cracks marked in red for visibility)

The east façade had remarkable yet less extensive damage, with the external piers and a few spandrels incurring the main damage from the earthquake (**Fig. 4**). Both the north and west façades had less extensive damage compared with the other façades (**Fig. 2b**), which was likely due to the fewer openings on these façades.

a) General view; cracks highlighted in red for clarity

b) Damage to a corner pier

Fig. 4. The east façade of Avonmore House after the 22 February 2011 earthquake

2.2 Royal Hotel (RH)

RH was located at 34 Norwich Quay in Lyttelton (**Fig 5**). Although the parapet above the main entry was inscribed with “Estd 1851”, it is likely that the masonry building was constructed after this date. Indeed, there is mention of a wood tavern located on the site of RH that was destroyed in an 1870 fire. However, there is photographic evidence that the building was constructed prior to 1910 [26].

a) West view

b) North view

Fig. 5. Royal Hotel after the 22 February 2011 earthquake

RH was a two-storey building with external clay brick URM walls, internal steel frames (likely only on the ground storey), and timber floors. The building had a height of approximately 7.2 metres (excluding the parapets) with a plan area of approximately 290 square metres at the ground storey and 250 square metres at the first storey (excluding small rooms on the ground storey that were added after initial construction). Drawings indicating wall thickness were not found, such that the estimated dimensions were based on limited photographic evidence taken during building demolition. The assumed dimensions are reported in Section 3.2. With regard to the internal steel frames, the cross section of columns and beams were assumed from photographic evidence and set equal to 0.24 m and 0.50 m, respectively and were modelled as elasto-plastic elements with a yield strength of 235 MPa. The timber floor diaphragms spanned between perimeter URM walls and, at least on the ground storey, received intermediate support from two internal steel frames (shown by dashed lines in **Fig. 6a**).

a) Plan of the ground floor, measurements in metres

b) 3D view of the 3muri [20] model

Fig. 6. Geometry of RH. 3D numerical model developed with Tremuri software commercial version (3Muri) [20].

The estimated parapet height above the section of the building shown in hatched lines in **Fig. 6a** was 1.2 metres, with a height of 1.0 metre above the remaining sections. Both sections of the parapet had a thickness of three leaves (350 millimetres). From photographic evidence, concrete ring beams were identified at the ground and first storeys. Based on this evidence, steel reinforcement was identified only for the concrete ring beam located on the first storey. However, based on observed earthquake damage, it was assumed that reinforcement was also present in the ground-storey ring beam.

2.2.1 Material properties

Material tests were not carried out for RH. From the photographs and observations made during site inspection following the earthquakes, the clay bricks appeared soft and the lime mortar was identified as that used in the original construction. Therefore, the procedure outlined in [13] was used to estimate material properties (reported in **Table 2**).

Table 2.
Summary of masonry mechanical properties assumed for RH

Brick compressive strength	14 MPa	From Table 10.3 in [13]
Brick tensile strength	1.7 MPa	From Table 10.3 in [13]
Mortar compressive strength	2 MPa	From Table 10.4 in [13]
Cohesion	0.3 MPa	From Table 10.4 in [13]
Friction	0.3	From Table 10.4 in [13]
Masonry compressive strength	6.68 MPa	Using equation 10.2 in [13]
Secant (cracked) Young's modulus*	1.67 GPa	Assumed to be 500 times the masonry compressive strength
Secant (cracked) shear modulus*	0.67 GPa	Assumed to be 0.4 masonry Young's modulus
Masonry density	18 kN/m ³	From Table 10.6 in [13]

* Initial elastic stiffness was assumed to be double as suggested in [11].

2.2.2 Damage analysis

As for AH, the 22 February 2011 earthquake was the most severely intense earthquake to hit RH in less than six months. However, no information was found regarding the level of damage that the building incurred after the two earthquakes that occurred prior to 22 February 2011. In contrast to AH, damage to RH was concentrated mainly in the piers (**Fig. 7** and **Fig. 8**). **Fig. 8** depicts the façade along Norwich Quay and shows

208 more extensive damage on the first storey than observed on the ground storey. The damage did not extend to
209 the slender piers, which appear to have experienced almost no cracking. Finally, small portions of the parapet
210 failed out-of-plane.

a) The façade on Canterbury road

b) Flexural damage of the corner pier at first floor

211 **Fig. 7.** North-west façade of RH after the 22 February 2011 earthquake (cracks highlighted in red for visibility)

a) Flexural damage of the corner pier and the adjacent spandrel

b) The façade on Norwich Quay (cracks highlighted in red for visibility)

212 **Fig. 8.** South-west façade of Royal Hotel after the 22 February 2011 earthquake

213 3 Development of the numerical models

214 3.1 Principles of the equivalent frame approach

215 Seismic response of the case-study buildings was simulated by nonlinear analyses (both static and dynamic)
216 through the EF modelling strategy. This strategy is based on discretization of the walls into a set of masonry
217 panels (piers and spandrels), in which the nonlinear response is concentrated, connected by rigid areas called
218 nodes. The equivalent frame idealization of 3D models was generated by using the 3Muri software [20], which
219 is a commercial version of the Tremuri research software [19] that specifically allows nonlinear monotonic
220 static analyses to be performed. The research version of Tremuri was then used to undertake the nonlinear
221 static and dynamic analyses by using multi-piece constitutive laws implemented with the specific aim of also
222 being able undertake cyclic analyses [27]. In accordance with beam theory, the elastic response phase of the
223 masonry pier and spandrel panels is described by defining the initial Young's (E_m) and shear (G_m) moduli of
224 masonry and then approximating progressive degradation using a secant stiffness. The elastic values are
225 defined by multiplying the secant stiffness by a coefficient, which in this case was equal to 2 (as suggested in
226 [11]). The maximum shear strength is defined on the basis of common criteria proposed in the literature as a
227 function of different failure modes examined (either flexural or shear) and the progression of nonlinear
228 response is defined through subsequent strength decay (β_{Ei}), which corresponds with assigned drift limits (δ_{Ei}).
229 The drift limits are associated with the achievement of reference damage levels DL_i (with $i=1,5$, where DL_5
230 is associated with “collapse” of the panel, representing the state in which the panel loses the capacity to support
231 horizontal loads), as shown in **Fig. 9a**.

232 **Table 3** summarizes the values adopted to describe progressive nonlinear response for both spandrels and
233 piers, based on experimental campaigns such as [28], [29], [30] and [31]. An accurate description of the
234 hysteretic response necessary to conduct cyclic analyses is also included in **Table 3**, based on the
235 phenomenological approach described in [32]. In **Fig. 9 b)** and **c)** the hysteretic response of masonry piers
236 when subjected to prevailing shear (b) and flexural (c) failure modes are depicted and in **Fig. 9 d)** the hysteretic
237 response of spandrels is shown. More information on the parameters which these laws are founded upon is
238 provided in [27].

239

a) Multi-pieewise linear constitutive law of URM panels
(adapted from [27])

b) Example of hysteretic response of piers in the case of shear
failure mode

c) Example of hysteretic response of piers in the case of
flexural failure mode

d) Example of hysteretic response of spandrels

Fig. 9. Details of the multilinear constitutive law adopted for the unreinforced masonry panels

Despite distinct classification of the hysteretic response for shear and for flexure as shown in **Fig. 9**, it is evident from both post-earthquake damage observations and from experimental campaigns [33, 34] that mixed deformation modes are common. Mixed failure modes are considered within the modelling strategy by interpolating the values assigned for basic failure modes, where the two strength criteria associated with shear response and with flexural response provide similar predictions. Strength decay values for piers reflect results reported in experimental campaigns on piers characterized by brick and mortar masonry (e.g., [28], [29]). For spandrels, the drift limits and the corresponding shear decay values proposed in **Table 3** were derived from the experimental testing of spandrels with URM arches [30] and with RC beams [31]. For spandrels with URM

arches an equivalent tensile strength contribution was considered for the flexural behaviour, as adopted in [13] and proposed in [30] and [35].

Table 3: Summary of the thresholds used for piers and spandrels [28, 29].

[%]	δ_{E3}^*	δ_{E4}^*	δ_{E5}^*	β_{E3}^*	β_{E4}^*
Piers**	0.6; 0.3	1.0; 0.5	1.5; 0.7	0 – 30	15; 60
Spandrels	0.2	0.6	2	40	40

* δ_{Ei} drift limits, β_{Ei} strength decays shown in Fig. 9a

**The first value is assumed for prevailing flexural behaviour, the second value for shear behaviour.

The final part of the modelling strategy was to account for diaphragms by modelling them as finite horizontal orthotropic membrane elements. These elements are identified by a principal direction (floor spanning orientation), with Young's modulus E_1 (direction of the joist in the case of a timber diaphragm) and E_2 (along the perpendicular direction), Poisson's ratio ν , and shear modulus G . The moduli of elasticity E_1 and E_2 represent the normal stiffness of the membrane along the two perpendicular directions and account for the effect of the degree of connection between the walls and the horizontal diaphragm. The moduli provide a link between the in-plane horizontal displacement of the nodes belonging to the same wall-to-floor intersection and hence influence the axial force computed in the spandrels. The most important diaphragm parameter is G , which influences the tangential stiffness of the diaphragm and the horizontal force transferred among the walls, in both the linear and the nonlinear phases [19].

3.2 Modelling assumptions for the case-study buildings

To develop a numerical model of a building it is necessary to know the building's geometry to a high level of accuracy. This task is relatively easy when access to the building is possible, but for the buildings under analysis this task was difficult because both buildings had already been demolished at the time when this study began. For this reason, necessary information was collected from the city council archives for the jurisdictions in which the buildings were located. When uncertainties remained, it was necessary to carefully review available photographs and sometimes use basic strategies such as counting the number of bricks to define the thickness or length of walls. The most common size of clay bricks used in masonry buildings in New Zealand, and the size adopted for simulation of the two case-study buildings, is 230 x 110 x 70 millimetres [13]. Table 4 summarizes the approximate thickness values of the walls.

274 The next step in developing the numerical models was to define the loads acting on the buildings. For URM
 275 buildings, the most relevant contribution to the stress acting on their elements is their self-weight. This property
 276 depends on masonry density, and for both buildings a value of 18 kN/m^3 was used. From photographs taken
 277 during demolition it was possible to observe that both buildings had timber diaphragms and lightweight timber-
 278 framed internal partitions (also confirmed by building plans). The values adopted for diaphragm weight are
 279 summarized in **Table 5**, with the imposed load values taking into account the reduction coefficient ($\Psi_2 = 0.3$)
 280 suggested in [11] and [2] for seismic analysis.

281 **Table 4:** Approximated wall characteristics of the case-study buildings

Model		Element	Ground storey	First storey	Second storey
AH	AH	Wall thickness (mm)	710 (6 leaves)	590 (5 leaves)	470 (leaves)
		Inter-storey height (m)	3.70	3.95	3.50
RH	RH	Wall thickness (mm)	470 (4 leaves)	350 (3 leaves)	-
		Inter-storey height (m)	3.57	3.60	-

282

283 **Table 5:** Adopted loads for the case-study buildings

Element		Description	Permanent load	Imposed load
Timber floor and roof diaphragms		Framing, floorboard, and ceiling; superimposed dead load	0.5 kPa	0.60 kPa
Clay brick masonry		External loadbearing walls	18 kN/m^3	Not applicable
Concrete elements		Ring beams at each storey	25 kN/m^3	Not applicable
Internal partitions		Timber stud framing, plasterboard lining	0.65 kPa	Not applicable

284

285 After defining the geometry of the building and the weights of elements it was necessary to determine the
 286 mechanical properties of the construction materials. This step was more difficult than for the previous task,
 287 and it is noted that even for scenarios where the building remains, it may not be permissible to collect
 288 specimens because this exercise could compromise the building's stability, or because it is a heritage building.
 289 Accordingly, the literature and many international codes (i.e., [13]) suggest numerous procedures for how to
 290 collect as much information as possible while causing only minimal damage to buildings. If it is not possible
 291 to collect specimens, then values for the mechanical properties are suggested in codes and guidelines [10, 3,
 292 9, 12, 13] and in the technical literature (see e.g., [36]). **Table 6** summarizes the information needed to develop
 293 a model in Tremuri. It should be noted that the values used for friction and cohesion are not those reported in

294 **Table 1** and **Table 2**, but that instead the equivalent values in **Table 6** were used because the masonry was
295 modelled according to Mann and Müller's proposal [37], which defines equivalent cohesion and friction.

296 As reported in Section 2, the north wall of AH was constructed of unreinforced concrete but no information
297 was available on the concrete or the type of reinforcement used for the RC lintels that were installed above
298 and beneath the openings. From the photographs taken during demolition, diffuse signs of segregation were
299 observed, likely because the concrete was composed mainly of large gravel pieces. For this reason weak
300 concrete with a cylindrical compressive strength of 20 MPa was used for the numerical modelling.
301 Reinforcement was assumed in the north wall, comprised of ½-inch (14 millimetres) bars spaced at 200 mm
302 c/c in both the vertical and horizontal directions, in order to simulate the condition of a weakly reinforced wall
303 with nonlinear behaviour. The north wall of the building originally had no openings, with the first drawings
304 that depict the presence of openings being dated 1940. The drawings indicate that two ½-inch (14 millimetre)
305 and three ¾-inch (19 millimetre) bars were used for the upper and bottom parts of the lintels, respectively.

306 For the two case-study buildings the small openings in the URM walls were not explicitly modelled, but their
307 presence was considered. A corresponding thickness was applied to the piers so that the cross sectional area
308 of a pier was equal to the sum of the cross sectional areas of the two original piers when subtracting the
309 presence of the small window opening. This procedure was adopted in order to avoid discretization of the wall
310 into piers, spandrels, and rigid zones that resulted in an unrealistic mesh, when recognising that the introduction
311 of small openings may cause the presence of unrealistic large rigid nodes and correspondingly squat piers that
312 may lead to the forecasting of premature wall failures arising from large drift values being associated with
313 small lateral displacements. This concern arises because in the EF models, the failure of masonry panels is
314 governed by fixed values of failure drift capacity being reached. Consequently, squat panels will reach their
315 failure drifts at small lateral displacements, thus potentially affecting the global ductility of the system and
316 consequently the outcome of the seismic verifications [16].

317 For arched openings, a depth up to 2/3 of the radius of an arch was used for beams. Note that only the in-plane
318 response was considered, recognizing that apart from a small portion of the parapet, no out-of-plane collapse
319 occurred in either building during the Canterbury earthquakes.

320 **Table 6:** Summary of the mechanical properties of masonry adopted in the numerical models

	AH	RH
Young's modulus*	5.14 GPa	1.67 GPa
Shear modulus*	2.06 GPa	0.67 GPa
Compressive strength	20.55 MPa	6.68 MPa
Equivalent cohesion**	0.67 MPa	0.23 MPa
Equivalent friction**	0.44	0.23

321 *For secant (cracked) stiffness values, initial stiffness was assumed to be double.
322 **Values of cohesion were modified according to Mann and Muller's proposal [37].
323

324 The mechanical properties adopted for the timber diaphragms are summarized in **Table 7**. The commonly
325 adopted values for the Young's moduli of timber diaphragms were used, whereas various values were used for
326 the G modulus. The reference number is the 350 kN/m shear stiffness proposed in [13] for timber diaphragms
327 in good condition. This value is equivalent to 8.75 MPa if the equivalent membrane has a thickness of
328 40 millimetres (reported in **Table 7** as G x1). As previously described, the diaphragms of AH were retrofitted
329 in the 1990s and thus these diaphragms were expected to have higher shear stiffness. To account for this retrofit
330 and to study the influence of this type of retrofit, a parametric analysis with various values for stiffness of the
331 diaphragms was undertaken (**Table 7**).

332 **Table 7:** Summary of the mechanical properties of timber diaphragms adopted for the numerical models

Young's modulus in the joist direction (E_1)	22.5 GPa
Young's modulus in the direction perpendicular to E_1	10 GPa
Shear moduli used for the parametric analysis G:	
	x1 8.75 MPa
	x10 87.5 MPa
	x100 875 MPa
	Rigid 13000 MPa
Thickness of the equivalent membrane	40 mm

333 **4 Records used for the NLTH analyses: 22 February 2011 Christchurch earthquake**

334 The Canterbury region of the South Island of New Zealand was hit by a series of strong earthquakes that began
335 on 4 September 2010 with the Darfield earthquake of magnitude M7.1. While another remarkable earthquake
336 was recorded on 26 December 2010, the Christchurch earthquake that was recorded on 22 February 2011 with
337 magnitude M6.3 caused the most damage. Although of a smaller magnitude, this earthquake produced peak
338 ground accelerations (PGAs) in the Christchurch region that were substantially greater than those measured
339 during the 4 September 2010 earthquake. However, in addition to PGA, other parameters are often used to
340 measure earthquake intensity, such as the spectral acceleration for a significant period of vibration or the

341 maximum spectral displacement [38], [39]. Then, in addition to the two earthquakes mentioned above, two
342 large aftershocks occurred on 13 June 2011, although the two case-study buildings had already demolished at
343 that date because they had both been declared unsafe after the Christchurch earthquake on 22 February 2011.

344 The New Zealand GeoNet project [22, 23] enabled the identification of two time-history accelerograms that
345 were recorded at stations located in the vicinity of the case-study buildings. For AH, the strong motion data
346 that was recorded by the Christchurch Cathedral College (CCCC) station at a distance of approximately 900
347 metres from the building was used, whereas for RH the strong motion data that was used in the analyses was
348 recorded by the Lyttelton Port Company (LPCC) station at a location of approximately 800 meters from the
349 building.

350 Only records from the 22 February 2011 earthquake were used for the nonlinear dynamic analyses because: 1)
351 there was little information or photographic evidence regarding the condition of the buildings after the prior
352 earthquakes that could be used to investigate the potential effects associated with damage accumulation; and
353 2), no relevant differences were detected in the response predicted in the numerical simulations by either
354 applying only the 22 February earthquake or by applying both the 4 September 2010 earthquake followed by
355 the 22 February earthquake. This choice was also justified by recognising that the epicentre of the Darfield
356 earthquake was farther away from the two case study buildings than was the epicentre of the Christchurch
357 earthquake, with the Darfield earthquake epicentre being located approximately 37.5 km from AH and 44 km
358 from RH, whereas the Christchurch earthquake epicentre was only 7.5 km and 2.7 km from AH and RH
359 respectively. However, a comparison is provided for the dynamic response of AH when applying only the time
360 history records from the Darfield earthquake and when applying the records from both the Darfield earthquake
361 and Christchurch earthquake.

362 **Fig. 10** depicts the acceleration response spectra of the accelerograms for the two horizontal components (e.g.,
363 N-S, E-W). For nonlinear dynamic analysis of AH, component N90E was applied in the X direction and
364 component N00E was applied in the Y direction (see **Fig. 2**). For RH, component N10W was applied in the X
365 direction and component S80W was applied in the Y direction (see **Fig. 6**). The vertical lines drawn on the
366 spectra indicate the fundamental periods of the models for the considered direction. Four lines are shown for

each direction, with one line provided for each value of the diaphragm shear stiffness introduced during the parametric analyses. **Table 8** provides a summary of the fundamental periods of the models.

Fig. 10. Acceleration response spectra and fundamental modes in X direction (continuous black line) and in Y direction (dashed red line)

Note that the CCCC recording station is situated on a type of soil similar to that where AH was located [40]. However, the LPCC station is situated on rock while RH was situated in the centre of the valley where Lyttelton is located, indicating that RH sat on at least 12 meters of sediment. It is thus expected that the RH building was subjected to some site effects not identified by the strong motion recorder. Furthermore, RH was located closer to the epicentre than was the recording station. For these reasons, in the dynamic analyses of RH the PGA values derived from the time histories were increased by 30%. Although a change in the soil’s mechanical properties is expected to cause a transformation of the spectra, because no specific study was undertaken this aspect was neglected. However, codes such as [11] and [12] suggest using amplification factors to take into account poorer soil conditions. The numerically predicted level of damage when adopting the 30% increase in PGA values was more similar to the actual level of damage than was numerically forecast when PGA amplification was not applied. **Table 9** summarizes the PGA values adopted for the nonlinear dynamic analyses. Two analyses were undertaken for each time history: 1) a “positive” analysis in which PGA values in **Table 9** were applied with their own sign and 2) a “negative” analysis in which the PGA values of the horizontal components were applied with the opposite signs.

385 **Table 8:** Summary of modal analysis results

AH	T _{1x} [s]	m _x – m _y [*] [%]	T _{1y} [s]	m _x – m _y [*] [%]	RH	T _{1x} [s]	m _x – m _y [*] [%]	T _{1y} [s]	m _x – m _y [*] [%]
G x1	0.29	34.5 – 0.00	0.14	0.70 – 47.1		0.20	33.1 – 0.01	0.19	2.55 – 24.7
G x10	0.28	48.2 – 0.00	0.14	3.21 – 48.6		0.20	50.6 – 0.02	0.19	0.00 – 25.1
G x100	0.20	61.9 – 0.00	0.13	0.00 – 54.7		0.19	38.9 – 7.92	0.17	37.5 – 17.6
rigid	0.15	63.0 – 3.53	0.11	9.41 – 61.8		0.19	31.5 – 10.6	0.16	51.2 – 11.2

386 * mass participation in X and Y direction respectively.

387

388 **Table 9:** PGA values adopted for nonlinear dynamic simulations

	PGA x [m/s ²]	PGA y [m/s ²]	PGA vertical [m/s ²]
AH	3.71	- 4.06	6.78
RH	-9.96	-11.23	5.24

389

390 5 Numerical simulations through nonlinear analyses

391 This section reports the results of the NonLinear TimeHistory (NLTH) dynamic and static analyses. NLTH
392 analyses were undertaken for the purpose of comparing actual damage with the damage determined by
393 numerical simulation. Actual damage was used as reference and to define which option for the diaphragm
394 shear stiffness was the most reliable for simulating actual response. The nonlinear dynamic analysis results
395 were compared with pushover analysis results that were generated by applying a load pattern proportional to
396 the building mass (referred to as “uniform”). From a preliminary analysis carried out by using different load
397 patterns (i.e. the uniform and the inverted triangular load pattern), the uniform load pattern was shown to be a
398 reliable load pattern for this type of buildings when compared with the results achieved from NLTH.

399 5.1 Avonmore House

400 **Fig. 11** depicts the damage state of the south and east façade walls of the numerical model of AH after
401 conclusion of the nonlinear dynamic analysis. In particular, **Figs. 11a** and **11b** show the walls of the model
402 with the diaphragm shear stiffness as proposed in [13] (G x1), and **Figs. 11c** and **11d** show the walls of the
403 model when adopting a diaphragm shear stiffness that is ten times higher (G x10). **Fig. 9a** provides a legend
404 of the damage obtained from the numerical model.

405 As described in Section 2, the south façade of AH was heavily damaged after the earthquake, with the spandrels
406 and piers having incurred extensive cracking, especially those in the top storey. This behaviour was captured

by the Tremuri model that incorporated $G \times 10$ diaphragm shear stiffness. This finding is unsurprising because, as mentioned in Section 2, the diaphragms of the building were retrofitted, which justified much higher stiffness values than those proposed in [13]. The south façade of the model with $G \times 1$ diaphragm shear stiffness showed a much lower increase of damage, which supports the greater consistency achieved by the stiffened diaphragm model compared with the actual building response observed following the earthquakes.

Fig. 11. Comparison between actual damage of AH and that obtained from the numerical simulations (the pink shade indicates that the element was also subjected to tensile forces during analysis).

The east façade presented a lower extension of pre-existing damage when compared with the south façade, with the numerical simulation slightly overestimating the actual damage to the spandrels. Because the spandrels were rather slender, it is possible that they had a lintel underneath that resulted in reduced damage. However, because no information was found about such an element, a lintel was not added to the numerical model. For the east façade, changing the diaphragm stiffness resulted in insignificant differences in the predicted damage (Fig 11b-d).

For both the south and east façades the numerical model captured the actual behaviour of the building with an adequate level of accuracy, especially when comparing the results for each storey. On the south façade, the

423 piers at the top storey incurred the most damage (shown in *purple* in **Fig 11c**), with this level of damage
424 correlating closely with the actual earthquake damage that was observed following the earthquakes. A similar
425 observation was made for the external piers, where in the numerical model the piers on the east façade exhibited
426 a flexural mechanism, consistent with actual damage. After the time-history analysis the majority of piers
427 showed a *green* level of damage, with some *pink* exceptions that indicated that the piers were not active because
428 they were subjected to tensile forces. In Tremuri, elements subjected to tensile forces are not considered as
429 loadbearing, but at the end of the analysis these elements showed flexural damage, which was consistent with
430 actual damage. The *green* colour refers to the first *plateau* of the multilinear constitutive law (**Fig. 9a**), which
431 has a differing plateau length depending on the panel stiffness. For the spandrels in the top storey, the presence
432 of the ring beam at this storey in the numerical model resulted in reduced damage to this element for both
433 façades when compared to damage to the storey beneath. However, there was some discordance between the
434 numerical forecast and actual observed damage, especially for the east façade. Therefore, it is worth
435 mentioning that the drift limits of the multilinear constitutive law for the spandrels (**Table 3**) are derived from
436 limited experimental evidence.

437 It is noted that the reported study specifically addressed global response only, with this decision being justified
438 based on the specific selection of two case study buildings that showed no activation of relevant out-of-plane
439 mechanisms. However, when flexible diaphragms are present within a URM building it is relevant to account
440 for out-of-plane wall mechanisms in conjunction with the global analysis. An example of this issue is given
441 by the dynamic response of the internal steel frames.

442 The displacement time history at the top of frame 10 (identified in **Fig. 2a**) is shown in **Fig. 12a** as a function
443 of diaphragm stiffness, where it can be seen that the frame displacement demand decreased as the diaphragm
444 stiffness increased. The results suggest that the Gx1 diaphragm stiffness would result in a frame 10
445 displacement demand of approximately 130 millimetres. Therefore the east and west façade walls were
446 expected to be subjected to an equivalent mid-span out-of-plane displacement demand (**Fig. 12b**). **Fig. 12b**
447 shows the maximum displacement attained at the top of all internal frames during the time-history analysis. It
448 can be observed that: 1) with increasing diaphragm stiffness the scatter between the displacements of individual
449 frames decreases, and 2) with a diaphragm stiffness of Gx100 the results match those for rigid diaphragms.
450 Note that the stiffness needed for “rigid” behaviour depends on the stiffness of the walls and thus it is not

451 always necessary for a diaphragm to have the stiffness of an RC slab. Note also that in **Fig. 12b** the maximum
 452 displacement of frame 10 is higher than the maximum displacement from the NLTH analyses reported in **Fig.**
 453 **12a**. This behaviour is observed because **Fig. 12b** reports the maximum displacement from both the positive
 454 and negative analyses (see **Section 4**) whereas **Fig. 12a** refers only to the results when negative PGAs were
 455 applied.

a) Displacement of the top level of frame 10 for variations in diaphragm shear stiffness

b) Maximum displacement registered at the top level of the internal frames of AH

Fig. 12. Displacements of the internal steel frames in the NLTH analysis.

457 **Fig. 13a** compares the hysteresis loops of the dynamic analysis compared with the shear-displacement curves
 458 from static analyses, for the case of Gx10 diaphragm stiffness, with **Fig. 13b** showing the effect of diaphragm
 459 stiffness on the pushover curves. In both graphs, on the vertical axis the base shear is normalized to the weight
 460 of the building, whereas the horizontal axis reports the average displacement of the roof (computed using
 461 weightings dependent on the mass at each node) normalized to the overall height of the building ("building
 462 drift"). The control node in the static analyses was situated on the more significant and/or flexible walls: wall
 463 2 for analyses in the Y direction and wall 3 for the analyses in the X direction (for their positions, see **Fig. 2a**).
 464 Comparison of the static and dynamic analyses confirmed the highlighted damage observations, i.e. during the
 465 NLTH the displacements reached values corresponding to the post-peak phase of the pushover curve, mainly
 466 in the X direction. (**Fig. 13a**). Global displacement demand did not change significantly with changing
 467 diaphragm stiffness, but global shear capacity did increase due to the redistribution of actions. As discussed
 468 previously, only the time history accelerograms recorded during the Christchurch earthquake were applied to
 469 the case study buildings, because this earthquake was responsible for causing the major extension of damage.
 470 The relative insignificance of the Darfield earthquake is also illustrated by the graphs in **Fig. 14**, where it is

471 possible to observe that the dynamic response when both the Darfield and Christchurch earthquakes were
 472 applied (**Fig. 14 b**) is very similar to that when only the Christchurch earthquake was applied (**Fig. 13 a**).

473 Another consequence of increasing diaphragm stiffness was a change in shear demand among the walls (**Fig.**
 474 **15**). For each diaphragm stiffness, **Fig. 15** depicts the ratio of the maximum shear received by a wall during
 475 dynamic analysis compared to the total building base shear in the same direction. Increasing diaphragm
 476 stiffness caused shear demand to decrease for wall 2 (after an initial increase) but to increase for wall 3.
 477 Therefore, for retrofit projects an aspect that should not be neglected is to assess the influence of diaphragm
 478 stiffening on the change in shear force distribution to individual walls (in both the linear and nonlinear phases).

Fig. 13. Comparison of pushover analysis and nonlinear time-history analysis results for AH.

a) Only 4 September 2010 Darfield earthquake applied

b) Both Darfield and 22 February 2011 earthquake applied

Fig. 14. Comparison of dynamic response of AH when applied only the Darfield earthquake or both the Darfield and the Christchurch earthquakes (X direction).

Fig. 15. Ratio of maximum wall shear to total base shear attained during dynamic analyses with changing diaphragm stiffness.

5.2 Royal Hotel

Fig. 16 shows the damage pattern obtained from the numerical model of RH after NLTH analysis. Overall, building RH incurred less earthquake related damage than did AH, which was also illustrated in the numerical model. This observation was confirmed by comparing the hysteresis loops with the pushover curves (**Fig 17a**).

Fig. 16. Comparison of the observed damage and numerical analysis for RH (the pink shade indicates that the element was also subjected to tensile forces during analysis).

Indeed, the maximum displacement determined by dynamic analysis was smaller than that which corresponds to pushover base shear decay. In this case, the actual damage was concentrated in the piers, especially at the first storey. This damage distribution was likely due to the presence of an irregularity in elevation (see **Fig. 7a**) that caused the first storey to be less stiff than the ground storey. The numerical model was also able to predict the type of damage. The first-storey piers of the façade along Canterbury Road mainly incurred actual damage through a flexural mechanism, which was also the case in the numerical model. For the façade along Norwich Quay, the piers showed both flexural and mixed flexural-shear damage, which is consistent with the numerical model. The model with the lowest shear diaphragm stiffness (G x1) had the highest correlation with actual observed earthquake damage. However, the RH models were not as sensitive as the AH models when diaphragm stiffness was changed. For all analyses, the extent and distribution of the damage did not vary significantly. Changing the diaphragm stiffness did not significantly change the maximum displacement and slightly increased the base shear (see **Fig. 17b**).

Fig. 17. Comparison of pushover analysis and nonlinear time-history analysis results for RH.

It should be noted that both the actual building and the numerical model were subjected to very high PGAs in all three spatial directions (**Table 9**) but did not present heavy damage.

6 Conclusions

Two irregular clay brick URM buildings that were damaged during the Canterbury earthquake sequence were analysed using the EF approach. These buildings were representative of the predominant form of URM constructions present in New Zealand and in other New World colonial countries and territories such as Australia and the West Coast of North America. The particular features of this type of buildings are: a) a significant plan irregularity due to the presence of heavily perforated façades on the main street frontages and with side and rear walls having few openings; b) only the external walls being composed of URM, whereas internally the floors are supported by steel frames (contrarily to common European URM buildings); c) light timber diaphragms that may not be considered as rigid in their plane.

The results demonstrated that with a good level of knowledge of the construction details, the EF modelling approach can predict damage with an adequate level of accuracy. Some differences between actual damage and the damage determined by numerical simulation were observed. In the case of AH, the extensive damage incurred by the building was slightly overestimated by the numerical analysis, mainly in the spandrels of the east façade. This overestimation was attributed to a lack of knowledge regarding the lintels and highlighted

522 the challenge previously reported in the literature associated with accurate modelling of spandrel/lintel
523 behaviour. Evidence from experimental campaigns specifically addressing these spandrel/lintel issues has
524 emerged over the last decade, but such studies remain limited in number. For RH, nonlinear dynamic analysis
525 was able to predict actual damage when the recorded PGA values were increased to account for site effects.
526 Moreover, for both buildings, it was also possible to observe an adequate agreement of results, in terms of both
527 stiffness and base shear, between the dynamic and static nonlinear analyses, with the latter being widely used
528 at engineering practice level.

529 Given the reliability of the numerical simulations conducted, the modelling approach based on EF idealization
530 can be considered an effective method. The approach has the following main advantages and potentialities:

- 531 • a URM building can be modelled with fewer elements (if compared with more complex tools such as
532 the finite element method), thus reducing computational burden and enabling nonlinear dynamic
533 analyses to be conducted in a short amount of time;
- 534 • more control of the parameters under analysis is possible. The required mechanical properties are
535 characteristics such as the strength and elastic moduli of elements. This information can be collected
536 from site samples and interpretative models proposed in standards, which in turn enable the discovery
537 of additional reference values in the literature. More refined methodologies, such as the finite element
538 method, also require knowledge of more complex properties to determine parameters such as the
539 material fractural energy and post-peak behaviour.

540 Many issues should be further explored in future research (e.g., the validity of EF idealization for irregular
541 patterns of openings, the role of the flange effect, and interaction with out-of-plane response). While the
542 simulations presented herein cannot be considered conclusive for all configurations, the EF approach is a
543 promising method for engineering practice for the seismic assessment of existing and historical URM
544 buildings.

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