THE OLD MUNICIPAL CHAMBERS BUILDING – DAMAGED BUT NOT DESTROYED – WILL IT BE THERE IN ANOTHER 125 YEARS?

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ABSTRACT: *The Old Municipal Chambers is a Category I heritage building situated in Worcester Street, Christchurch. It was designed by architect Samuel Hurst Seager in the Queen Anne Arts and Crafts Style and was opened in 1887. The two storey building is constructed in solid brick and incorporates several decorative features, and it is an important part of the cultural heritage fabric of Christchurch. Some securing works were undertaken in 1989 to improve its performance under seismic loads. This paper will outline damage during the earthquake on 4 September 2010, subsequent aftershocks and the 22 February 2011 earthquake which resulted in some localised zones of collapse. The structure of the building has been stabilised externally, to secure or allow retrieval of very significant heritage features. The high cost to repair the building makes the future of the building uncertain despite its very significant heritage value.In response to the building's seismic vulnerability and possible retrofitting, finite element and simplified equivalent frame models were used for pushover analysis, enabling a complementary seismic evaluation from both approaches. The predictions identify the weak parts of the building and its expected failure modes, which are in agreement with the observed damage. The computations appear conservative, because the computed capacity curves provide insufficient capacity of the building to survive the recorded earthquakes. Given that there was uncertainty on the constitution of the floors, they were assumed as unidirectional by default, bidirectional diaphragm floors were also simulated in the simplified model, reflecting the securing works undertaken in 1990. In this case, a significantly better behaviour is observed. This paper will examine the seismic performance of the building, comparing results of analysis including both in-plane and out-of-plane behaviour, with actual damage. It will then consider conceptual scenarios for the future of the building, including comparison of performance and cost of both conventional and base isolation retrofit.*

KEYWORDS: Cultural heritage building, seismic damage, stabilisation works, seismic assessment, retrofit scenarios

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1 INTRODUCTION

As with many of the heritage buildings in Christchurch, the Old Municipal Chambers experienced considerable damage as a result of the 4 September 2010, boxing day, and the 22 February 2011 Christchurch earthquakes. In this paper, the structural damage sustained to the building is identified and the stabilisation works carried out to secure the building will be briefly outlined.

The paper will then overview the structural modelling carried out at University of Minho, Portugal, comparing the analysis results with the actual observed damage. The paper will then discuss the strengthening retrofit options currently being considered.

The building has sustained a substantial amount of damage and the cost to repair the building is high. The future of the building is currently uncertain despite its very significant heritage value. Options considered for strengthening the building would enable repair and protection of the building for use by future generations.

2 BACKGROUND

The Old Municipal Chambers, which is located on Worcester Street in Christchurch, was designed by Architect Samuel Hurst Seager and was opened in 1887. The building is a Category I heritage fabric building, therefore considered to be of international significance.

The building has not been occupied since September 2010 due to the damage sustained and the risk of collapse.

Following the September 2010 earthquake, Opus International Consultants were commissioned to undertake the stabilisation work and subsequently to provide structural engineering consulting services for the strengthening and retrofit scheme.

2.1 Structural Description

The two storey building is constructed with solid brick walls and timber floors and roof. The ground floor plan of the building is presented below in Figure 1.

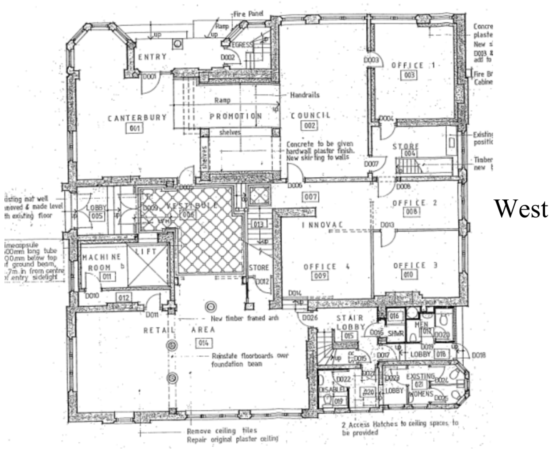

Figure 1: *Ground floor plan*

The building dimensions are approximately 19.5m x19.5m in plan with a height of approximately 14.6m at the highest point of the building.

The external walls are generally 4 courses thick on the ground level reducing to 3 courses on the upper level. The brick walls to the external envelope incorporate the original decorative features. Small terracotta panels have also been used for decorative purposes along with stone window surrounds, sills and balusters. Concrete strip footing supports the brick walls, and brick piers support the ground floor at intermediate locations.

The gravity load is generally resisted by the masonry walls through the timber joists and down into the foundation. The lateral load is also resisted by the masonry structure with the timber floors on the first and second floor transferring the load by diaphragm action to masonry walls acting in plane.

2.2 Previous Strengthening/Securing Works

The building has previously undergone structural strengthening work in 1989 to address the key areas of the building that were at high risk of earthquake damage. The strengthening work carried included:

- Tying the first and second floor diaphragms to the external brick walls (Figure 2).

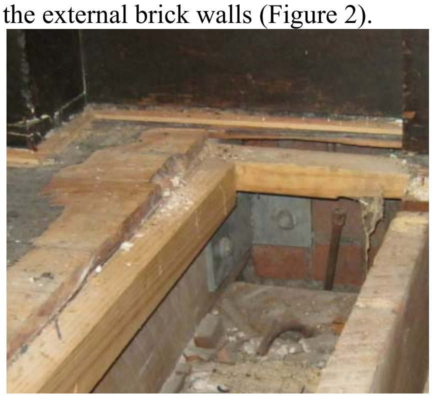

Figure 2: *Typical floor ties to wall*

- Install roof tie above the Old Council Chamber on the first floor.

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- Install new anchor bolts to connect the roof and the brick walls.
- Construct concrete frames to create new openings through internal brick walls and new foundation beam to resist the structure above.
- Construct a concrete lift shaft.
- Construct a new basement level to the south-west corner of the building and strength the room above by providing a 150mm thick reinforced concrete wall lining.
- Steelwork installed to tie the brick chimneys into the building.

3 DAMAGE OBSERVED

3.1 4 September 2010

The damage sustained in the September earthquake was mostly limited to the west wall, NW stair tower, SE Turret and chimneys. Temporary stabilisation works were installed in all these locations.

Internal structural damage included diagonal cracking to longer lengths of internal masonry walls, especially around the main staircase.

Further damage occurred during aftershocks, including the Boxing Day, and further stabilisation measures were taken, especially to the south gable of the council chamber.

3.2 22 February 2011

Significant widespread damage was sustained during the February earthquake which resulted in some localised zones of collapse, as described below for the parts of the building. Shoring measures and details were developed with input from the heritage architect in order to preserve as much building fabric as possible.

3.3 Building elevations

3.3.1 South Elevation

The south facade had several large cracks around the window openings to the Old Council Chamber. The wall was partly separated from the roof and presented large cracks along the returns (corners) of the east and west elevations. The first floor had maintained connection to the wall (Figure 3).

Figure 3: *First floor connection to the wall*

3.3.2 West Elevation

Large cracks to the brickwork were observed around the window openings in the west elevation. Debris of brickwork had collapsed over the windows leaving some of the roof timber exposed. The stone window framing had cracked and displaced, and the chimney above the roof level cracked increasing the risk of collapse.

There was cracking in the brickwork at the north end of the elevation below the eaves level.

Moderate cracking was observed near the ground level which suggested that there had been ground settlement towards the north end of this elevation. This is consistent with the geotechnical findings for the site.

3.3.3 North Elevation

The eastern end of the north wall has diagonal cracks from near the chimney and the wall at first floor level appears to have a slight out of plane bow. The chimney on the east end was cracked and in an unstable state.

3.3.4 East Elevation

The damage sustained on the east elevation was localised in the collapse of the gable at the north end. The other areas remained also relatively undamaged.

3.4 Turret on SE Corner

The roof of the turret had displaced about 300mm horizontally and sections of the masonry walls below were near the collapse. It was recommended to remove the roof to avoid collapse and further degradation of the structure.

3.5 Chimney

Most of the chimneys were severely damaged at roof level due to lack of support to the building.

3.6 Internal structure

There has been limited access into the building following the February earthquake and a detailed damage assessment is yet to be carried out. In general the most damage was observed on the west side of the building where the Old Council Chamber was located on the first floor. The east side had performed reasonable well, most likely a result of the construction of the concrete lift core in the late 1980s. The following observations have been recorded:

Old Council Chamber

This room is located on the west side of the building and due to it being a large space open plan room, it has sustained significant damage which is also reflected on the external face. Although the roof structure appears to have remained intact, large areas of the plaster work had collapsed and the supporting brickwork on all four walls show extensive cracking, which makes the structure of this zone the most vulnerable in the building (Figure 4).

Figure 4: *Damaged zone below the roof structure*

The north-west staircase which was constructed as part of the original construction suffered significant damage, where the west and north perimeter brick walls cracked and displaced. There has been localised collapse of these walls, as presented in Figure 5.

Figure 5: *North-west stairwell*

4 STABILISATION AND SECURING WORKS

Following the September earthquake, considerable effort has been made to prevent further damage to the building fabric, resulting from aftershocks. Given the complex geometry, vulnerability of remaining heritage features, and sometimes lack of ideal materials due to time constraints, a wide range of measures have been used to shore the building.

In general methodologies were employed which utilised the existing structure wherever possible, however in many cases this was not feasible due to damage, heritage considerations, lack of resistance in parts of the existing structure, or inability to provide a viable load path to the existing structural system. Where an external restraint was provided, consideration was given to stiffness compatibility, albeit generally in a simplified way. An example is the resilient pads beneath the base plates of the steel props to the south elevation.

The photos in Figures 6 to 10 give a good representation of the extent and type of shoring measures. With the exception of the NW stair tower (which partially collapsed in February), all the measures put in place have been effective in minimising on-going damage and in some cases preventing collapse.

Figure 6: *Shoring of the South wall*

Figure 7: *Shoring of the South wall in the entryway*

Figure 8: *Shoring of the East wall*

Figure 9: *Shoring of the North wall*

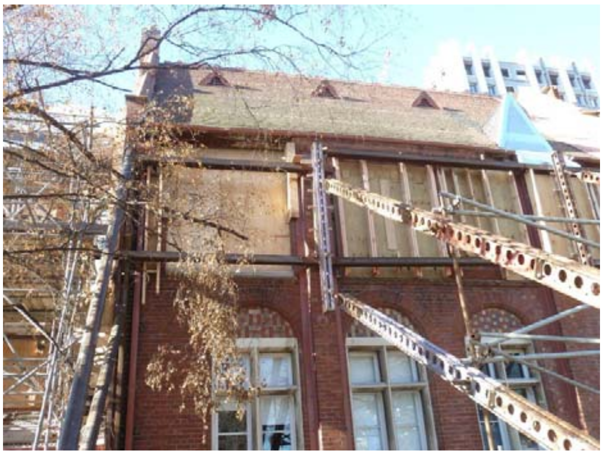

Figure 10: *Shoring of the West wall*

5 STRUCTURAL MODELLING

5.1 General

The research team at the University of Minho was engaged to model the building using a Finite Element Method (FEM) and an Equivalent Frame Model (EFM) to determine the seismic behaviour and identify the failure modes of the structure. Refer to Marques et al. [2012] for full details of the modelling and analysis.

5.2 FEM & EFM Models

The FEM model was built in the DIANA software [TNO, 2009] adopting a total strain crack model, and where the walls are simulated as shell elements. The EFM model was idealised using the TreMuri computer code [Galasco et al., 2009] which adopts a macroscopic integration of the masonry material, and where the masonry panels are modelled as beam elements. The mesh for each model is illustrated in Figures 11 and 12.

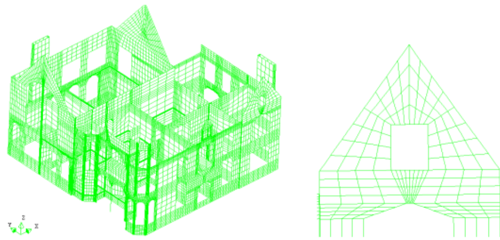

Figure 11: *FEM mesh of the building and model detail of the East gable*

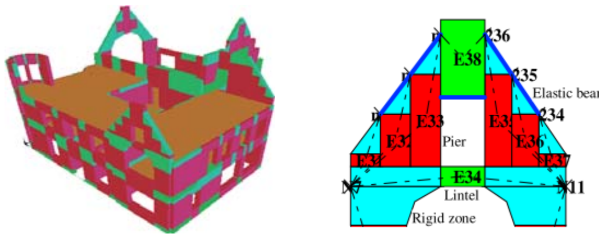

Figure 12: *EFM mesh and modeling of the gable in the East façade*

5.3 Seismic Analysis

The analytical simulation is based on drawings, pictures and in situ observation, and considers the original structure of the building. The subsequent modifications to the original drawings were only incorporated in the simplified model, given the simplicity of the changes and the low running time of the analyses. The strategy defined for the seismic assessment is based on a comparative evaluation of the results obtained from the two modeling approaches, the FEM and EFM.

5.4 Assumptions

The lower ends of the walls at the ground floor are considered fixed to the ground (0.0m level), which is in agreement with the existing deep foundations for the building. A dead load of 2 kN/m2 was considered to simulate the weight of the timber floors, while live loads of 0.9 kN/m2 and 0.6 kN/m2 were assumed respectively for the first and second floors. The roof was not modeled to reduce the complexity of the FEM model, being replaced by linear loads, namely vertical loads of 5 kN/m on the longer walls of the Old Council Chamber, of 4 kN/m on the turret walls and of 3 kN/m on the remaining walls supporting the roof.

The floors are one-way timber joists with wood planks. The wood is assumed with a weight of 6.5 kN/m3 and an elastic modulus of 12,000 MPa. The floors are assumed to be made with wooden beams of section 0.15 × 0.3 m2 spaced of 0.5m. The floors are considered spanning along the smallest dimension of each room. In the case of the FEM the floors were modeled as a distribution of concentrated masses along the edges of the supporting walls. Note that this modeling approach is conservative, as it is neglecting the connection and diaphragm effects of the floors, known to be important to the seismic response of the building, particularly concerning the prevention of out-of-plane mechanisms.

The properties for the masonry are presented in Table 1 and were derived from experimental local data in Christchurch, the Italian code [NTC, 2008] and the recommendations given in Lourenço [2009].

Table 1: *Properties of the masonry*

f_m	Compressive strength	5.0 MPa
τ_0	Pure shear strength	0.1 MPa
w	Weight	*18.0 kN/m3*
E	Elastic modulus	2,500 MPa
G	Shear modulus	417 MPa
δ_f	Flexural limit drift	0.006
δ_s	Shear limit drift	0.004
f_t	Tensile strength	0.15 MPa
G_c	Compressive fracture energy	8 N/mm
G_f^I	Mode I-fracture energy	0.012 N/mm

5.5 Procedure

The definition of the geometric model was based on a CAD drawing, used for both models. The nonlinear static (pushover) analysis was selected for evaluation of the seismic response of the building. The analysis is made using an incremental-iterative procedure, which allows to prediction of the base shear-displacement response (capacity curve) and to simulate the damage evolution in the individual elements. The structure is in a first stage submitted to the vertical loading, and then the analysis proceeds with horizontal loading replicating the seismic load, for which a mass-proportional pattern is assumed.

The pushover analysis is often linked to displacement-based seismic assessment, using a methodology such as the N2 method [Fajfar and Fischinger, 1988], where the expected displacement demands in design earthquakes are computed by means of a response spectrum analysis of an equivalent single degree-of-freedom system. Afterwards, these displacements are compared with the displacement capacities at given performance levels. In the present study, pushover analysis is used, through the prediction of deformation and damage, to identify the weak features of the structure and to design the strengthening that will be implemented in the building.

In the pushover analysis the building response is mainly determined by the in-plane response of the walls, with minor contribution from the out-of-plane resistance. Note, however, that the prevention of out-of-plane collapse mechanisms is possibly the most important action in seismic retrofitting actions, as the global seismic response of a building can only be exploited if early out-of-plane damage is avoided. In this work, the assessment of out-of-plane mechanisms is not directly made for the EFM model, where critical zones are only identified based on global pushover analyses and local mechanisms must be prevented by improving connections, but it is made in the FEM model.

6 ANALYTICAL RESULTS

The results obtained in the analyses are now presented, for the FEM and EFM models, and also for the original and modified building structures. Four analyses are always carried out, covering the two main directions of the building with positive and negative loading signs. The results presented next include the predicted damage in the structural elements, the deformed shape of the building and the capacity curves.

6.1 Results Corresponding to the Original Design

First, results from the pushover analyses applied to the models consistent with the original building drawings by Samuel Seager in 1886 are shown, for the FEM model. This is a rather interesting analysis, as the original design was based only on prescriptive rules for earthquake resistance, as is the case of many traditional masonry buildings around the world.

6.1.1 FEM model

Figures 13 and 14 show the deformed shapes and the damage level for the four analysis, +X (S-N), –X (N-S), +Y (E-W) and –Y (W-E), representing the two earthquake directions and the left-to-right and right-to-left actions. Damage is measured by the maximum principal strain, which is associated with the crack width.

In the case of the +X direction it is possible to see in Figure 13a the collapse of the South gable, with cracks appearing on the top of the NE and NW edges and on the left side of the North gable (Figure 13b). In the –X direction the SW corner of the building collapses in Figure 13c, and cracks appear in the North façade and interior walls of the Council Chambers (Figure 13d).

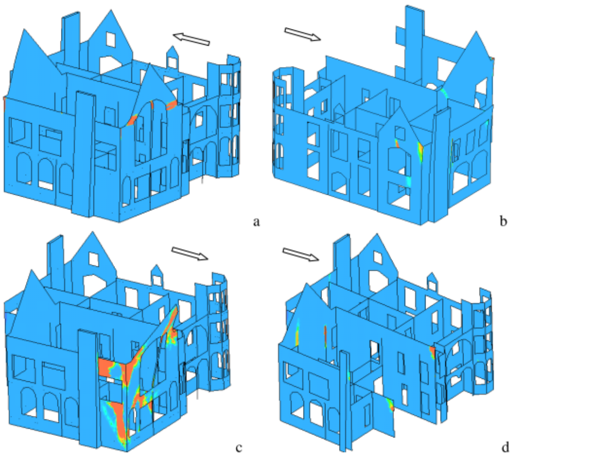

Figure 13: *Deformed shape and damage for +X: (a) collapse of South gable and (b) damage without South gable, and –X: (c) collapse of SW corner and (d) damage without SW corner*

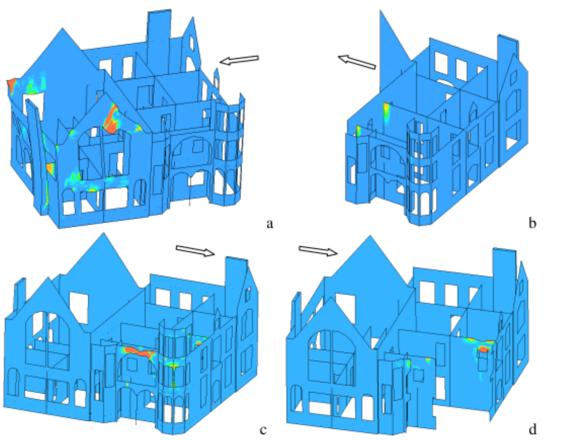

Figure 14: *Deformed shape and damage for +Y: (a) wide damage of the Council Chambers and (b) damage without the Council Chambers, and –Y: (c) collapse of the top arch in South façade and (d) damage without SE corner*

In the +Y direction, from Figure 14a there is deformation of the West façade to the outside and there is extensive damage on the gables of the North and South façades, with cracks in the interior walls near the Council Chambers (Figure 14b). In the –Y direction, the top arch on the right of the South façade collapses and the turret in the SE corner is damaged in Figure 14c, and there are cracks in the interior walls near the East façade (Figure 14d).

Figure 15 shows the capacity curves for all analysis corresponding to maximum load factors of 0.16, 0.13, 0.24 and 0.15 for +X, –X, +Y and –Y loadings respectively. Here, this is the percentage of the weight of the building applied horizontally, or the base shear force in proportion. Therefore, the building can withstand a static horizontal action of 0.13g, being more vulnerable in the –X direction.

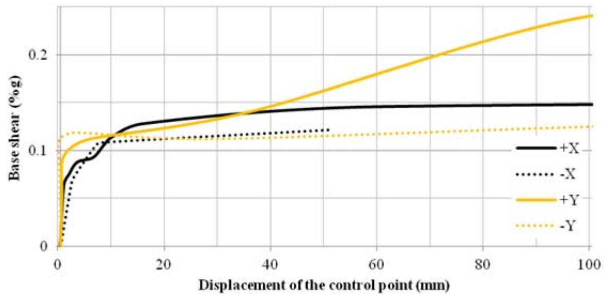

Figure 15: Capacity curves from the FEM analysis

A different behaviour is identified for the +Y direction, which, from the observation of the evolution of damage and displacement in the computed response, seems to be due to the fact that the transversal walls to the East façade are working as buttresses for the building. These walls are working in elastic range (without significant damage) up to a displacement of 100 mm, allowing to considerably increase the base shear strength, after that the damage in the walls appears with corresponding decreasing of the base shear, see Figure 14a-b. This effect is not observed in the –Y direction, where the transversal walls present early damage, as shown in Figure 14c-d.

The failure mode in the X direction is the collapse of the SW corner of the building, and the roof is not modeled in the simulation. As the roof might provide a certain level of restraint at the top of those walls, another model was prepared using tyings between the nodes of the top left of the South façade and the top right of the North façade. This tries to simulate the existence of a relatively stiff roof in the Council Chamber, with a good connection between roof purlins and walls, or the addition of steel ties between the walls. The new analyses in the X direction provide similar results for each loading sign. In Figure 16 it is shown that a much different behavior of the building is found due to ties in the Council Chamber (refer to Figure 13a-b). The new capacity curve for +X direction can be found in Figure 17, when compared with previous analysis, where it is shown that the capacity increase is moderate.

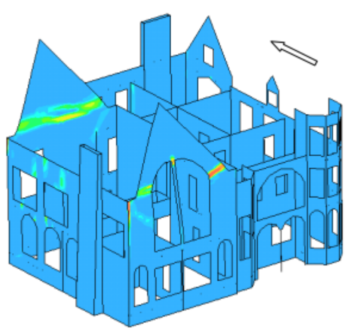

Figure 16: *Deformed shape and damage state for the +X analysis with the tyings option.*

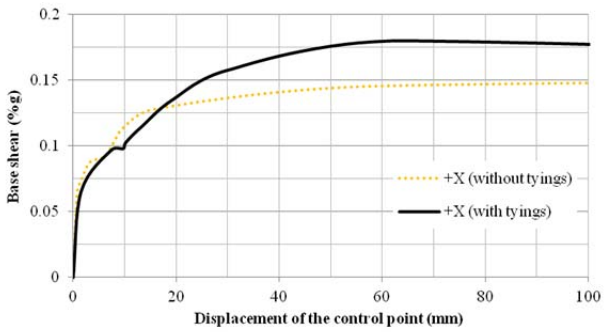

Figure 17: *Comparison of capacity curves without and with tyings*

6.2. Results Corresponding to the Altered Structure through the EFM Model

The studied building was subjected to structural changes by a renovation in 1989, which has been identified on a collection of sketches obtained from the Christchurch City Council. These drawings show that several alterations have been made inside the building with much use of reinforced concrete. Then, a new macro-element model was created by introducing the main changes in the building structure, with the aim to evaluate the influence of the alterations in the pushover response. From the computed capacity curves in Figure 18, it seems that the influence in the global building response of the intervention made is very low, only a slightly improved ductility being observed due to the reinforced concrete elements contribution.

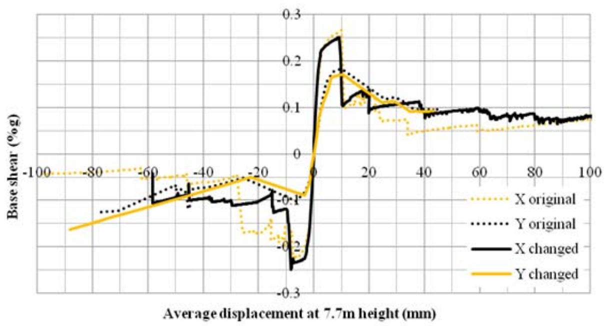

Figure 18: *Capacity curves obtained for the original and changed building structures*

A final analysis was made considering a bidirectional diaphragm for all floors of the building, which can simulate a possible strengthening intervention with adequate tying of the building. This model can also replicate the existing behavior if the previous structural alterations managed to reach this goal, which cannot be confirmed at this stage without a careful inspection of the building from the inside. The results in Figure 19, comparing the unidirectional and bidirectional floors, clearly show a significant improvement of the building response for bidirectional floors, both in terms of base shear and ductility. The minimum capacity of the building (in the +Y direction) is now about 20% of the weight, or 0.2g. The capacity increase for the –Y direction is dramatic.

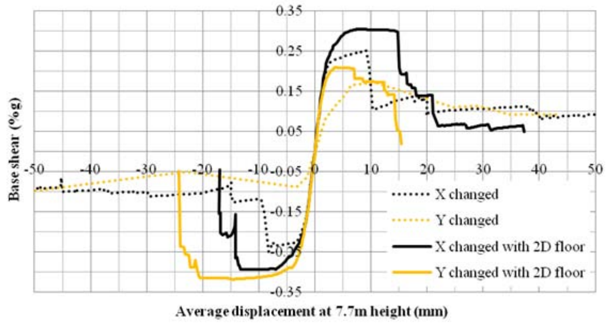

Figure 19: *Capacity curves obtained for the altered building considering 1D and 2D floors*

7 FINDINGS OF THE SEISMIC ASSESSMENT

A significant uncertainty remains about a number of features of the building, such as the floor system, due to modifications to which the building has been subjected in the 1989 renovation and to the fact that access to the building interior is impossible. For this reason, a performance-based seismic assessment including a force verification was preferred, instead of a displacement-based safety assessment that can be very sensitive to non-validated deformation characteristics. In opposition to the procedure in the N2 method [Fajfar and Fischinger, 1988], where an equivalent single-degree-of-freedom system is used, in this study the idealized bilinear response of the real building was considered. The inelastic capacity of the building is accounted through a ductility-based behavior factor q, and it was assumed that the building needs to withstand the maximum spectral acceleration (MSA) registered in the February 2011 quake.

Figure 20 compares the capacity curves obtained from the pushover analysis on the FEM and 1D floor EFM models. Note that the results from EFM and FEM are not directly comparable, given that the diaphragmatic action is considered for the floors in the EFM model while out-of-plane failure is considered in the FEM model, and given that the assumed response of masonry piers in the EFM is brittle. Effectively, a discrepancy is observed of the predicted peak strength in the X direction, which is possibly due mostly to absence of floors in the FEM model, and because the floors in the EFM model work mainly in the X direction. In the Y direction the approximation between the two models is better. The results of both predictions seem to be very conservative, in the sense that the building survived the February 2011 Christchurch quake with a MSA of 1.0g (note that the MSA in the New Zealand code [NZS, 2004] is 0.66g). Therefore, it is assumed that the EFM with the 2D floor model provides a better representation of the present condition of the building, as the new capacity curves in Figure 21 provide higher strength and ductility.

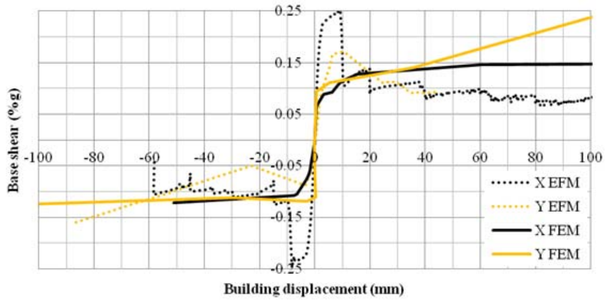

Figure 20: *Comparison of capacity curves obtained for the FEM and 1D floor EFM models*

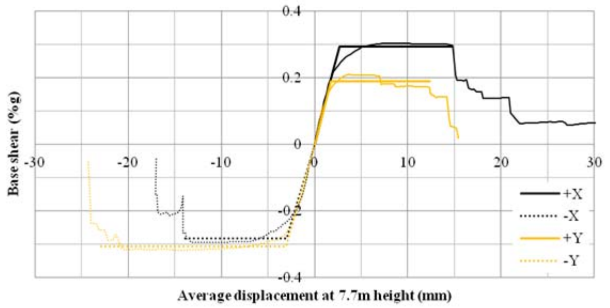

Figure 21: *Idealized bilinear responses of the building for the EFM with 2D floor model*

The subsequent computations are based on the definition of an idealized bilinear representation of the building capacity curves. The idealized bilinear response is computed by defining the initial branch with a secant stiffness k for 70% of the maximum base shear in the capacity curve, and the subsequent horizontal plateau is defined by equalizing the energy of the actual and idealized responses until a displacement (d_u) corresponding to a post-peak base shear capacity of 80%. The yield point of the idealized response is defined by the displacement d_y and by the load factor F_y/W, where W is the building weight. The idealized bilinear response of the building corresponding to all loading directions is presented in Figure 21, with the associated parameters from Table 2. A ductility-based behavior factor q_μ is also computed according to Tomaževič [2007] as:

$$q_\mu = \sqrt{2\mu - 1} \tag{1}$$

where μ is the global ductility computed as the ratio between d_u and d_y.

Table 2: *Parameters of the idealized bilinear responses*

	d_y (mm)	k (kN/m)	F_y/W	d_u (mm)	μ	q_μ
$+X$	2.67	930311	0.294	14.80	5.54	3.18
$-X$	3.01	789091	0.282	14.10	4.68	2.89
$+Y$	1.78	898841	0.190	12.40	6.97	3.60
$-Y$	3.09	837500	0.307	23.13	7.49	3.74

On the other hand, the reference spectrum ordinates S_r can be computed by reducing the MSA measured (1.0g) using the behaviour factor q_μ, leading to the results given in Table 3. A basic safety factor SF can then be computed as the ratio between the normalized base shear strength F_y/W and the reference spectrum ordinate S_r. The computed SF values provide insufficient resistance of the building to survive the 2011 Christchurch quake, with the +Y direction the weakest. The reason of this underestimation is mainly due to the assumed brittle behavior of the masonry elements, which provides a conservative response both in terms of base shear and ductility. Tomaževič [2007] suggests, based in shaking table tests, an over-strength of at least 30%, as a result of underestimation for material strength. If this over-strength is accounted in the computation of an increased safety factor ISF, a higher seismic resistance is observed for the building, already close to the unit value and justifying why the same survived the earthquake, although with considerable damage.

Table 3: *Computation of reference spectrum ordinates and safety factors*

	S_r (g)	F_y/W (g)	SF	ISF
$+X$	0.315	0.294	0.93	1.21
$-X$	0.346	0.282	0.82	1.07
$+Y$	0.278	0.190	0.68	0.88
$-Y$	0.268	0.307	1.15	1.50

Although the maximum values on the capacity curves for the FEM and EFM approaches are different, the general behavior of the building is captured and the fragile and most robust zones of the building have been well identified. This information has been used in the current strengthening design of the building. Both models consistently provide largely insufficient capacity of the structure, despite the building surviving the 2010-2011 Christchurch earthquakes, heavily damaged.

8 STRENGTHENING AND RETROFIT OPTIONS

8.1 General

Two strengthening options have been considered at a conceptual level for initial costing. The options are a conventional strengthening scheme based largely on shotcreting to existing masonry walls, and a base isolation scheme providing a higher performance building. Both options would require the similar amount of deconstructive work to the severely damaged parts of the building and share the similar details for the strengthening of the roof structure. New concrete walls would be constructed to replace the demolished brick walls mainly around the Council Chamber. It is intended that the heritage features and the architecture is maintained as far as practically possible by careful deconstruction and rebuild using the original materials.

8.2 Conventional Concrete Wall Scheme

This scheme was designed for an Importance level 2 structure to a minimum of 67% NBS (full designation) for strengthening of existing elements and 100% NBS for replacement elements. Some damage to both structural and architectural elements would be expected under an ultimate limit state earthquake.

A number of walls have been identified as shear walls to resist the lateral loads and adequate connections would need to be created from the floor and roof diaphragms. New foundations would be required to accommodate the additional loads.

8.3 Base Isolation

This option would result in a much better performance under a large seismic event. Considerable protection of heritage features will be achieved without intrusive work to secure these items. Damage at much higher levels of shaking is expected to be minimal and damage to the internal contents would be significantly reduced. Some elements of the building which are currently damaged could more simply be re-instated because of the reduced level of floor accelerations imposed by a base isolated building.

The result of the EFM modelling incorporating the strengthening work carried out in 1989, shows the minimum capacity of the building is around the level of demand for a base isolated structure. Therefore, with the construction of new concrete walls to replace the severely damaged structure, it is considered that minimum strengthening work is required to the remaining areas of the existing structure.

The foundations would need to be designed for a Maximum Considered Event (MCE) which would require either piling to a gravel level 4m below ground of ground improving such as grout injection. The ground floor would need to be replaced with a rigid concrete floor to tie the structure together at ground floor level.

This option would be a preferred solution and requires careful consideration of the additional costs benefits of BI especially given the preservation of a building of heritage significance such as the Old Municipal Chambers,

Although the cost estimates for both options have not been completed, we understand that cost premium for the base isolation concept may be approximately 10% over the conventional scheme.

9 CONCLUSION

The studied building is a very important cultural and societal heritage in New Zealand and contains fabric and features of exceptional significance. Furthermore, it is representative of many traditional masonry buildings around the world.

The Old Municipal Chambers building behaved largely as expected, fact that is in agreement with the predictions from the computational modelling. Work carried out in 1989, which included installation of a reinforced concrete lift shaft, and mainly the tying of the floors into the masonry walls greatly improved the building ability to withstand earthquake shaking.

The building has suffered significant damage during the recent earthquake events, and some sections of the building are beyond repair. Reinstatement and strengthening of the building is feasible, and both conventional and base isolation retrofit schemes have been considered to a conceptual level.

A base isolation retrofit would require less intrusive work to, and result in a much greater level of protection to, the heritage fabric of the building. If the cost premium is confirmed to be in the order of 10% for base isolation, we consider this would be a very good investment for the future enabling protection of the building for use by future generations.

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