

Advanced Non-linear Modelling for Heritage Resilience: Finite Element Seismic Assessment of the Dunedin Town Hall

J. Dymock

WSP, Christchurch, New Zealand.

ABSTRACT

The Dunedin Town Hall and Municipal Chambers, located in the heart of Dunedin, New Zealand, comprise an iconic heritage building complex with considerable historical and civic value. We conducted a comprehensive detailed seismic assessment of the 1880s Municipal Chambers and 1929 Town Hall, addressing challenges related to their Heritage Category 1 and 2 status and complex unreinforced stone masonry structures.

By employing state-of-the-art digital workflows and advanced finite element modelling we simulated the building's seismic response to strong ground motion inputs. Utilising the latest digital workflows, we were able to make effective use of point-cloud scans to build a robust analytical model that accurately reflected as-built conditions. Using advanced non-linear analysis techniques with constitutive models for composite masonry, reinforced concrete and unreinforced stone masonry walls, we identified the building's key potential seismic vulnerabilities. This enabling us to develop a deep understanding of the structure's seismic response including revealing complex load paths and clearly identifying the structure's hierarchy of failure.

Our thorough understanding of the building's behaviour, along with tools that displayed crack patterns, wall displacements and element failure limits, allowed us to clearly and effectively communicate the findings to a wide range of stakeholders. This helped support informed decisions on risk and strengthening priorities.

1 INTRODUCTION

Seismic assessment of heritage buildings brings two main challenges. First, many were built in an era when unreinforced masonry was common and their capacity can be difficult to define. Material behaviour is complex, and load paths are often uncertain. Second, common to any seismic assessment but particularly for heritage, outcomes must be communicated clearly and effectively to a wide range of stakeholders.

WSP was tasked with assessing The Dunedin Town Hall, a heritage structure with significant historical value. The assessment process presented considerable challenges as the building incorporated not only

unreinforced masonry but also complex, multi-layered stone/concrete/brick walls. The building's complex geometry and load paths required careful attention to ensure that the analysis represented likely behaviour.

Effectively communicating the results of the seismic assessment to the client and stakeholders was a key part of the project requirements. We presented clear visualisations and concise summaries of the complex analysis. This enabled the client to make informed decisions about strengthening priorities.

2 BUILDING DESCRIPTION

The Dunedin Town Hall complex is located in The Octagon and is comprised of three distinct sections named The Town Hall, Glenroy Auditorium, and Municipal Chambers. The three sections are shown in a 3D render in Figure 1. The Municipal was constructed first in 1880 with The Town Hall and Glenroy Auditorium being constructed in 1929. The relative positioning of them in plan can be seen below in Figure 2. There is a construction gap between the Glenroy Auditorium and Municipal chambers that is treated as a seismic separation.

The Municipal Chambers is a Category 1 listed heritage building and the Town Hall and Glenroy Auditorium are listed as a Category 2 heritage building.

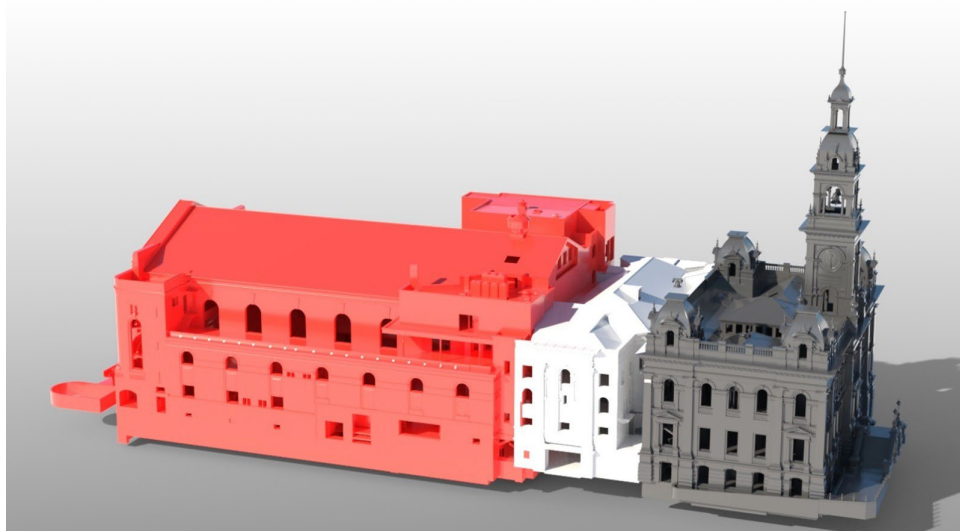


Figure 1. 3D render of the Dunedin Town Hall complex showing The Town Hall (red), Glenroy Auditorium (white), and Municipal Chambers (black)

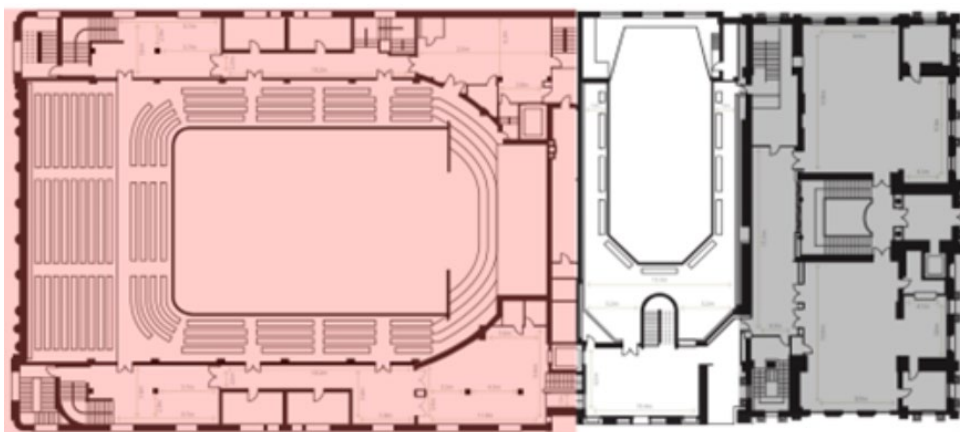


Figure 2. Floor plan of the Dunedin Centre showing the Town Hall (Red), Glenroy Auditorium (White) and Municipal Chambers (Black).

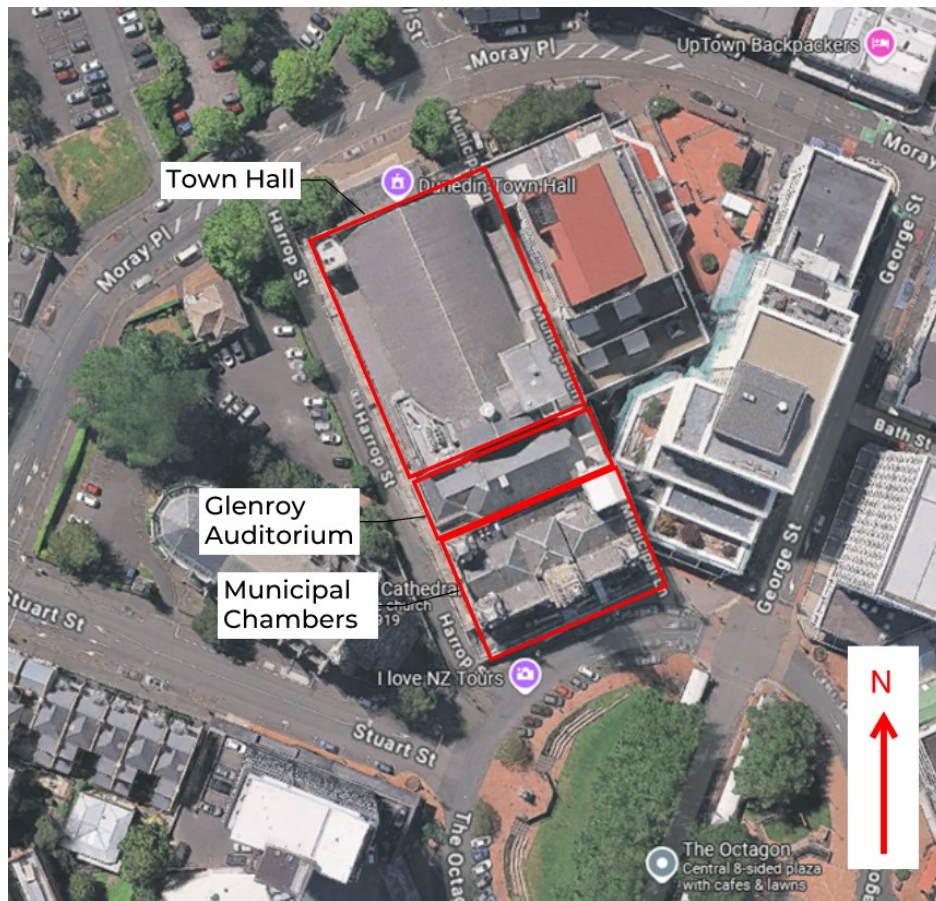


Figure 3. Building location in Dunedin and showing the Town Hall, Glenroy Auditorium and Municipal Chambers sections

2.1 Municipal Chambers

The building footprint measures approximately 40m x 20m with an average roof height of 17m excluding the clocktower which measures approximately 26m high from the top of the roof, with a total height of 45m above ground floor level.

The main roof consists of slate over a lightweight timber frame, supported by unreinforced masonry perimeter and internal walls. Perimeter walls are made of external stone with brick lining and rubble infill, while internal walls are typically brick. Reinforced concrete walls were added around the stairwell during 1988 alterations. Stone wall thickness and composition vary throughout the building. The masonry walls extend to ground level and rest on shallow concrete foundations. Suspended timber floors are supported by joists spanning the masonry walls.

Lateral loads are resisted in both the longitudinal and transverse direction by both perimeter and internal walls, constructed of unreinforced masonry. Much of the building's seismic mass is due to the weight of the walls themselves. The exterior walls are all primary walls and resist lateral loads.

2.2 Town Hall and Glenroy Auditorium

The Town Hall and Glenroy Auditorium structures were additions to the original Municipal Chambers and were originally completed in 1929 with a significant redevelopment from 2011 to 2013. The general construction is of reinforced concrete floors and steel beams with an external perimeter façade of stone, concrete and brick.

The overall plan dimensions of the Town Hall and Glenroy Auditorium structures are 68m in the longitudinal direction; 53m for the Town Hall and the remaining 15m for Glenroy. Both structures are the same width at 40m in the transverse direction. The Town hall roof apex is 24m high while the Glenroy roof apex is 18m high.

The Town Hall roof features slate tiles on timber framing, supported by steel roof trusses. These trusses are fixed to steel columns encased in concrete. Stone masonry perimeter walls with large window openings are provided between the steel columns but the main weight of the roof is supported by the steel columns.

The first and second floors of the Town Hall are constructed of reinforced concrete slabs that span onto reinforced concrete beams, which are supported by both reinforced concrete columns and the concrete-encased steel columns mentioned above. At the basement level, reinforced concrete and concrete-encased steel columns bear on reinforced concrete piers with shallow pad footings below. The perimeter stone masonry is supported by a shallow strip footing.

The Glenroy Auditorium vertical load resisting system is identical to that of the Town Hall structure outlined above. The only addition is that the conference room on the first floor is supported by steel trusses. These steel trusses are also supported by the concrete-encased steel columns.

The lateral load resisting system of the Town Hall in the longitudinal direction consists of steel roof cross bracing, bringing the seismic weight of the slate roof out to the masonry walls on the perimeter, comprised of stone and brick with concrete filling the cavity between. The masonry walls transfer lateral load in shear down to the shallow foundations, and then into the ground. The gable end wall is restrained by the reinforced concrete floors at the ground floor level, steel cross-brace support at both the first and second floors by the lower and upper galleries and by steel purlins running to roof cross-bracing at the roof level. Tension-only cross-bracing provides the main lateral load path back to the perimeter walls.

The transverse direction relies on a combination of portal frame action of the roof trusses, combined with the concrete-encased steel columns cantilevering up from the lower roof level, where shear walls transfer lateral loads down to the foundations and into the ground. Cross bracing in the roof drags some load to the stone masonry end walls. The roof cross bracing consists of steel bracing that was installed originally in 1929 and refurbished with additional Reidbrace cross bracing in 2013.

The Glenroy Auditorium lateral load resisting system consists of masonry walls in both the longitudinal and transverse directions. There is no steel bracing in the roof, and the roof's weight is transferred out to the perimeter walls via diaphragm sarking action in the roof, aided by hip rafters.

3 MODEL DEVELOPMENT

The Detailed Seismic Assessment (DSA) of the Dunedin Town Hall used two models: a linear elastic model for static loads and a nonlinear dynamic model with time history loads. The linear elastic model was developed in ETABS with 2D shell elements and 1D frame elements which is the limit of the software's capabilities for wall buildings.

To deliver a more meaningful analysis for the client, a nonlinear dynamic model was developed using LSDyna. The assessment required highly non-linear material behaviour, which is best handled using an explicit solver. ETABS does not provide an explicit solution approach. LSDyna also supports material erosion, which allows elements to be removed during the simulation when defined criteria, such as strain limits, are exceeded.

This enabled realistic wall failure mechanism and failure hierarchies to be observed and communicated clearly. Table 1 below summarises these features.

Table 1. Key features compared

	Linear Elastic Model	Nonlinear Dynamic Model
Analysis Software	ETABS	LSDyna
Solver Type	Implicit	Explicit
Material Erosion	No	Yes
Wall Elements	2D shell elements. One shell per wall	3D solid elements. Multi-layered walls with separate solid elements for each material
Floor Elements	2D shell elements	2D shell elements
Frame Elements	1D Frame elements	1D Frame elements

The LSDyna model includes each wall material layer explicitly to better represent the building's seismic response. The Dunedin Centre includes a range of wall compositions including solid concrete, unreinforced brick, and triple-layered Oamaru Stone/Concrete/Brick shown below in Figure 4.

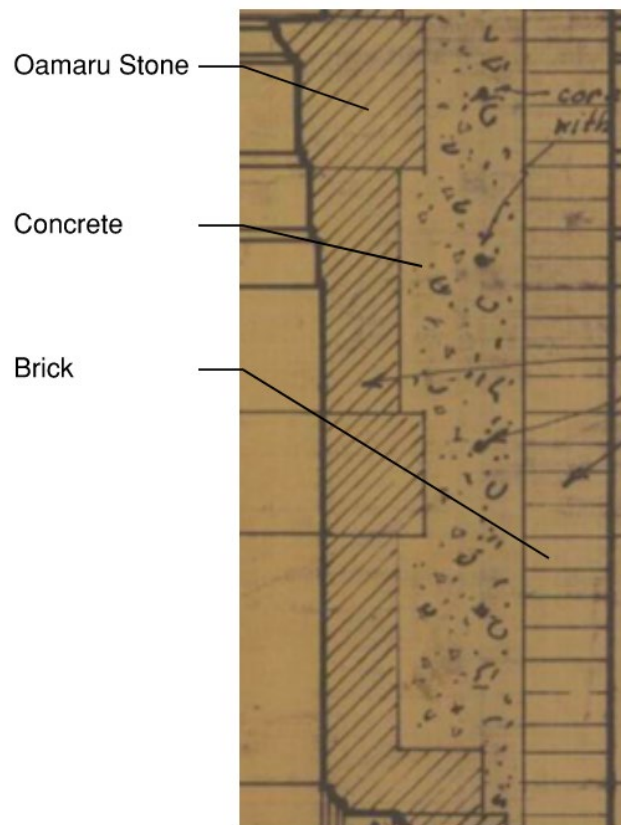


Figure 4. Typical section through Town Hall exterior walls showing outer stone facade(at left), inner brick(at right) and concrete infill layer.

The model was developed from multiple sources including the original 1927 construction drawings, photographs and information from various refurbishment over the years, a detailed BIM model based on point cloud scanning information (Figure 5 and Figure 6), site inspections by WSP engineers and intrusive material testing.

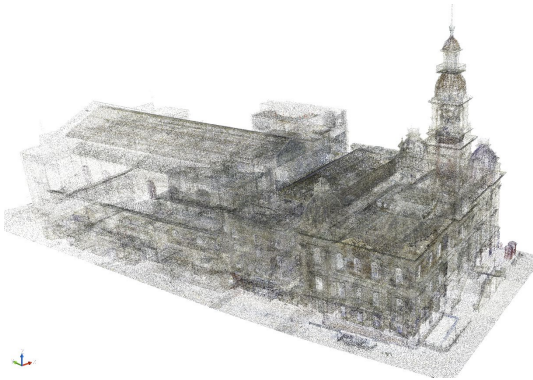


Figure 5. Point Cloud.

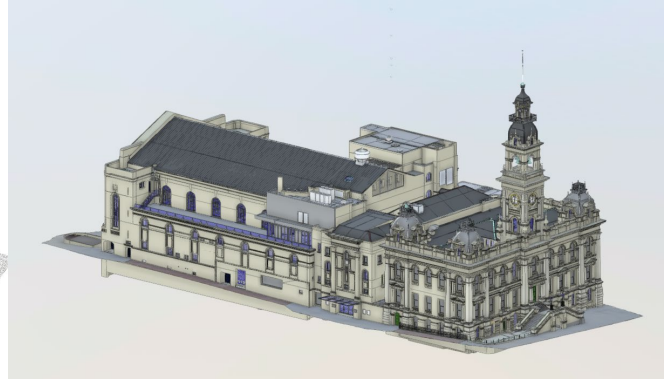


Figure 6. Revit Model.

The complete Revit model data was imported but only the geometry relevant to the building's structure is retained (Figure 7). A refined model (Figure 8) is generated that includes all geometry related to the seismic assessment of the structure. Non-structural components and architectural finishes are generally omitted from the analysis model unless they are expected to influence global seismic response or mass distribution significantly. Their mass is still included in the geometry that remains. The model balances accuracy to the as-built structure with computational efficiency, ensuring that the seismic assessment reflects the building's expected seismic performance.

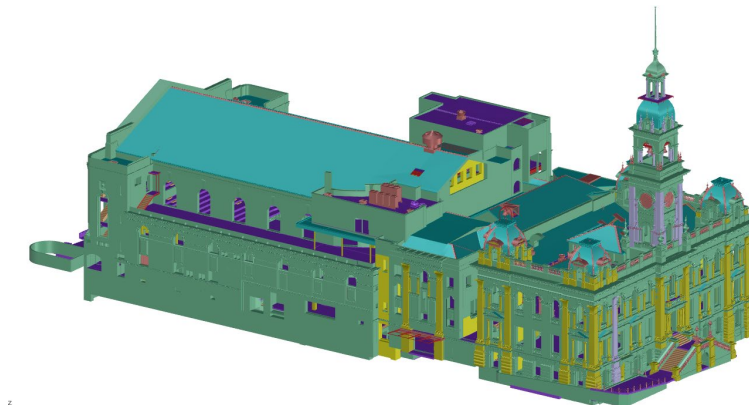


Figure 7. Relevant geometry pulled from the Revit model

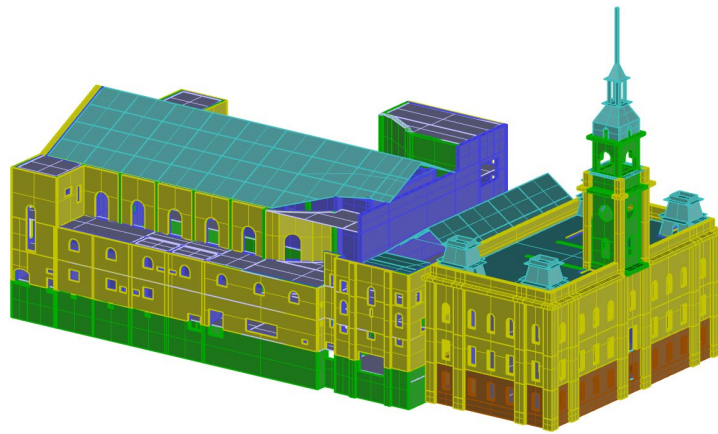


Figure 8. Analysis model geometry in preparation for arranging in parts

The geometry model of Figure 8 is imported into specialised software for discretising the model into a finite element mesh. The first process is to allocate various component parts in preparation for meshing (Figure 9). This includes components for each material layer of the walls. For example, the stone, concrete and brick multi-wythe walls have three separate component parts for each structural wall at this step. The next step is to generate and refine a 3D Finite element mesh based on the component parts (Figure 10)

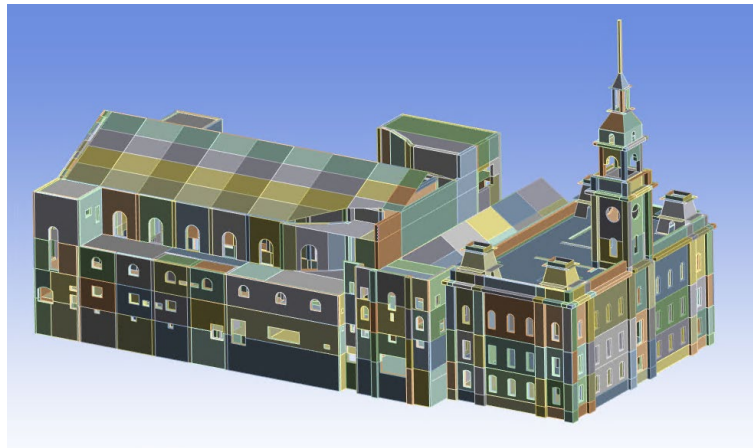


Figure 9. Analysis model refined for various structural parts in preparation for meshing

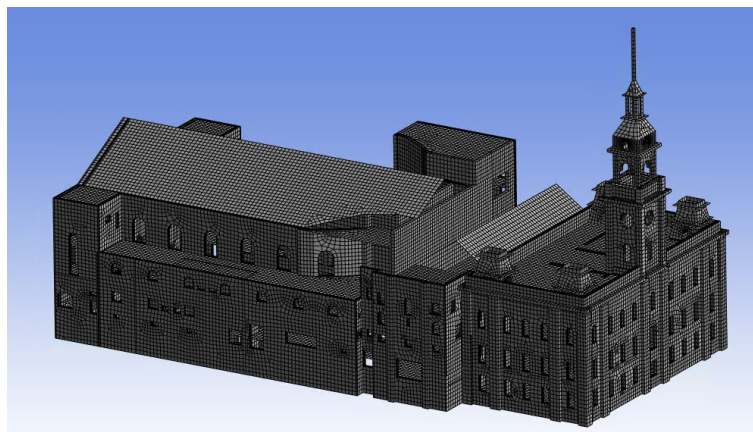


Figure 10. The analysis model after meshing into finite element objects.

The final step is to apply material properties and loads to the model which results in the final FEA model shown in Figure 11 below.

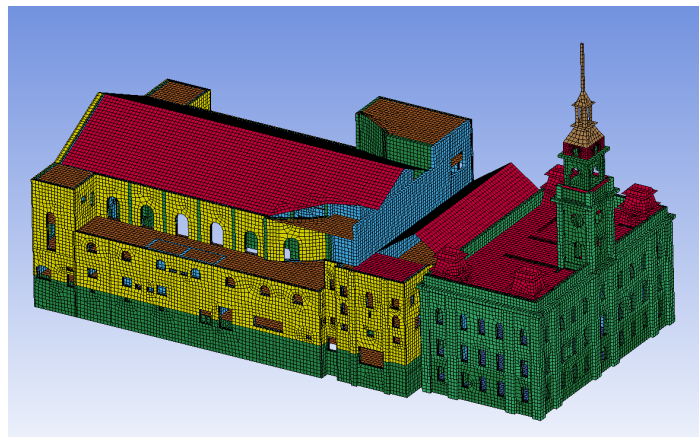


Figure 11. The final FEA analysis model with material assignments

The following Figure 12 shows a section cut through one of the exterior walls in the Town Hall showing the multi-layer wall modelling detail.

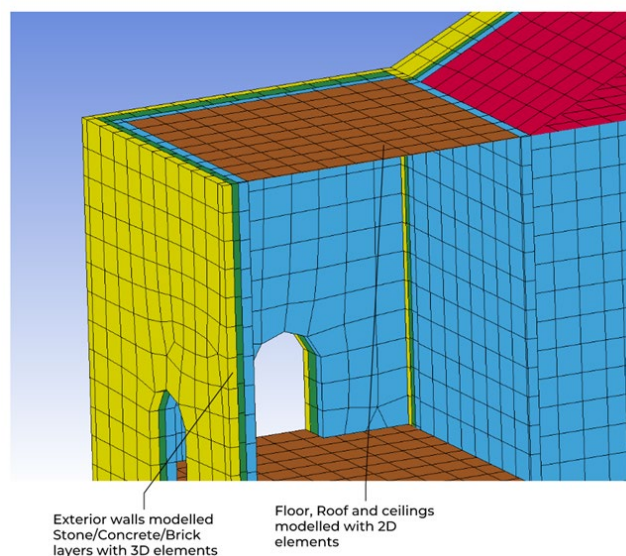


Figure 12. FEA model section cut showing the multi-layer wall modelling

4 ANALYSIS AND RESULTS

A strong ground motion record was selected and scaled to the procedure in NZS1170.5. Scaling of the single record was to the Ultimate Limit State (ULS) and the Collapse Avoidance Limit State (CALs). The non-linear time history analysis (NLTHA) used one record only, because the objective was to visualise the building's response under a single strong ground motion scenario. A full quantitative validation of the ETABS model was not required but the LSDyna results did provide useful insight for interpreting the ETABS outputs.

The two orthogonal horizontal components of the record were applied to the model with the vertical action being omitted. Base shear outputs are separated for the Town Hall/Glenroy Auditorium and the Municipal Chambers because the two structures were constructed at different times and we wanted to understand their responses individually.

Wall performance points, for both in-plane and out-of-plane limits, are shown on the base shear time history plots in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines C8. The Town Hall and Glenroy Auditorium base shear plot over time is shown in Figure 13 below. Figure 14 shows the associated base shear over time plot for the Municipal Chambers. The base shear results for both show the wall in-plane and out-of-plane limits being reached early in the time history which is consistent with the results of the ETABS model outputs and calculations.

Pushover analyses of the entire building were conducted to validate the NLTHA results. The pushover results demonstrated good agreement with the NLTHA output and provided valuable insights for confirming the assumptions within the linear elastic ETABS model. Supplemental pushover analyses were performed on isolated building components such as a wall along a specific gridline to confirm the material property inputs.

The main value of the LSDyna model outputs is the visualisations which can be reported on to the client and stakeholders. We provided a video output of the building subjected to the strong ground motion action showing cracking forming, elements eroding and displacement limits being reached. A representative summary of the outputs from the LSDyna results is shown in Figure 15, Figure 16, and Figure 17. They clearly show how different parts of the building respond to the strong ground motion input.

Overall, the LSDyna results align with the ETABS outputs and associated calculations, with particularly strong agreement for the in-plane wall response. For out-of-plane behaviour, the LSDyna assessment indicates better performance than the ETABS-based assessment. The analysis also suggests that the Municipal Chambers walls experience lateral load-resisting failure before collapse of the clocktower. As the walls soften, they reduce the lateral load transmitted from the ground into the clocktower above.

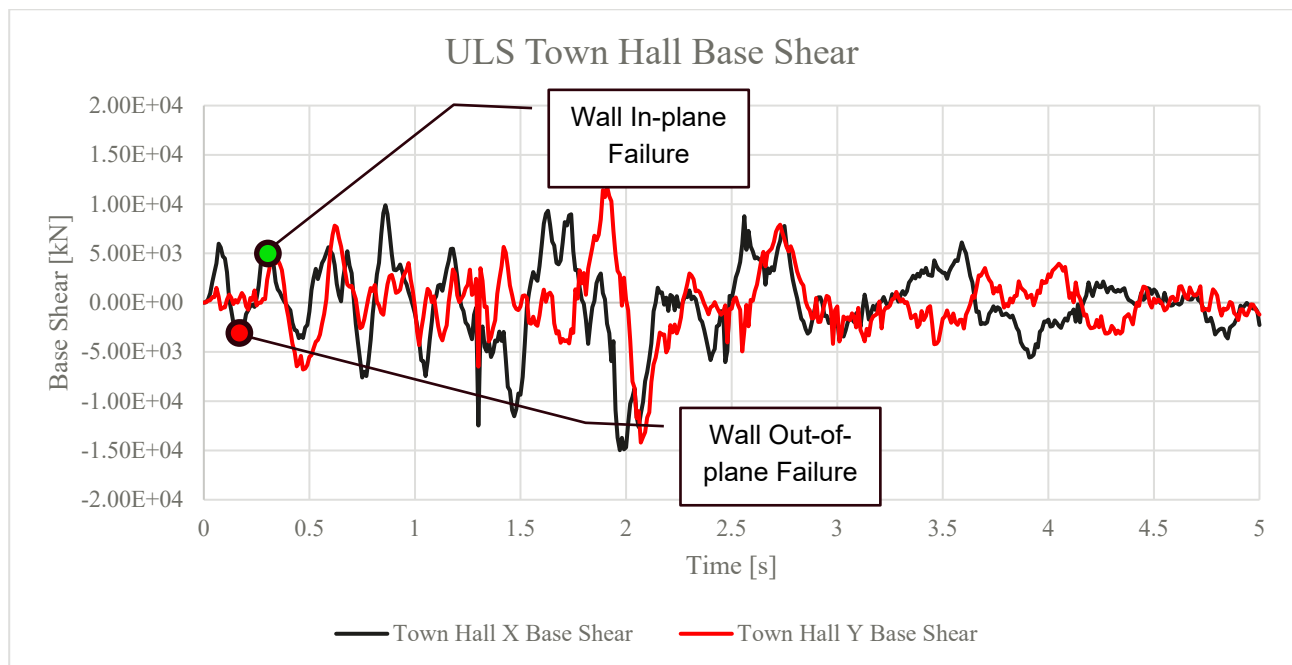


Figure 13. Town Hall base shear over time.

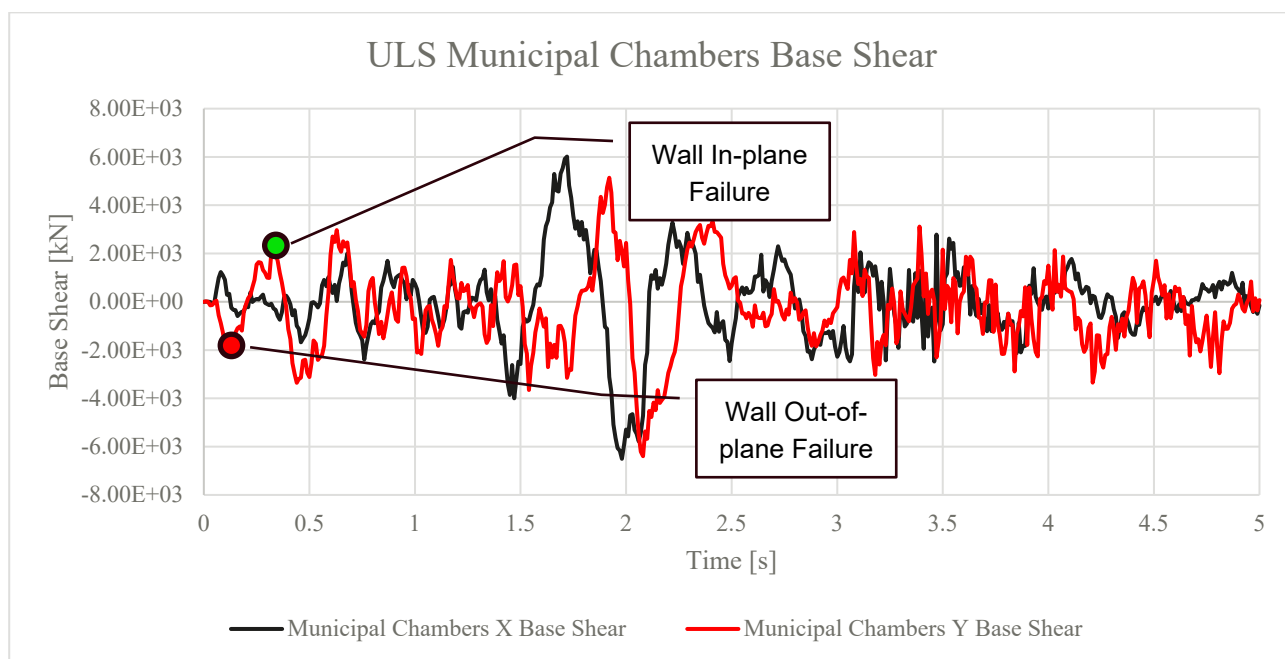


Figure 14. Municipal Chambers base shear over time.

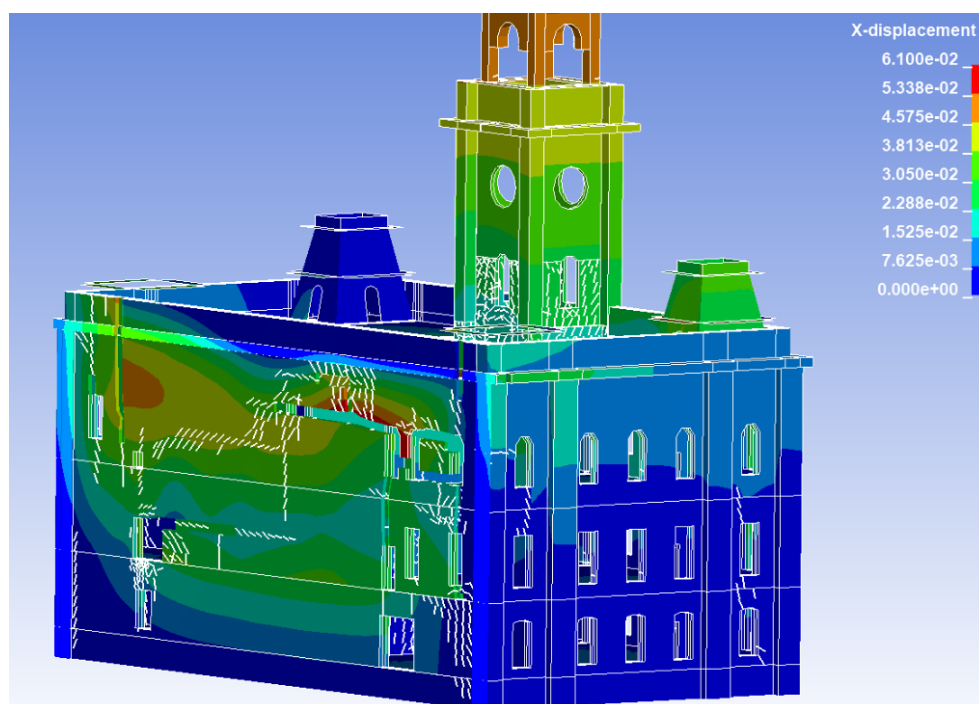


Figure 15. Municipal Chambers results output showing material cracking, element erosion, and wall displacement from peak longitudinal motion.

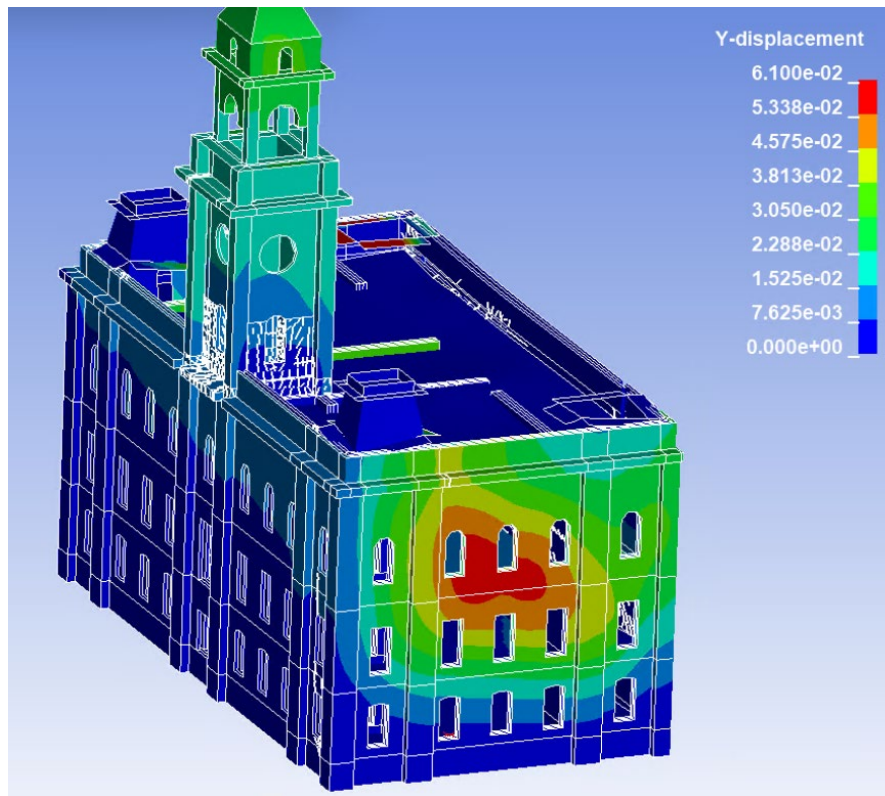


Figure 16. Municipal Chambers results output showing material cracking, element erosion, and wall displacement from transverse direction peak motion.

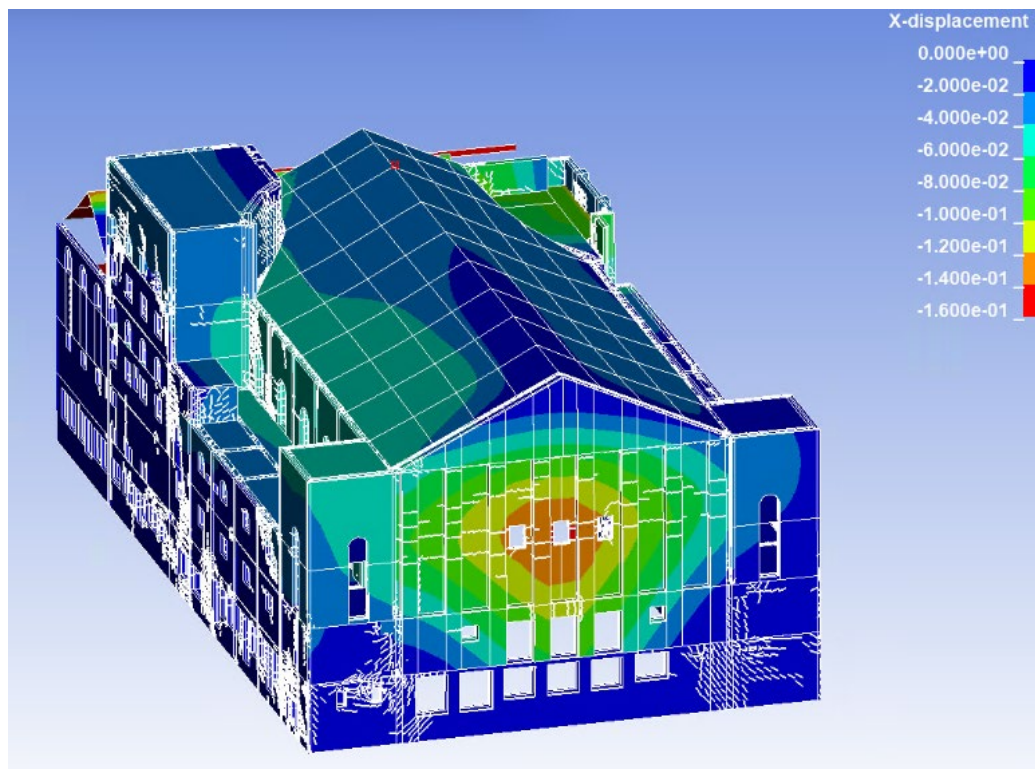


Figure 17. Town Hall results output showing material cracking, element erosion, and wall displacement from longitudinal direction peak motion.

5 CONCLUSIONS

The use of the LSDyna model in this project provided significant advantages in accurately capturing the complex nonlinear behaviour of the multi-layer wall systems under seismic loading. This advanced modelling approach allowed for a detailed assessment of failure modes and failure hierarchies including material cracking and element erosion, delivering results that more closely reflect real-world responses.

The in-plane results for the walls showed strong agreement between the LSDyna model and hand calculations to the NZSEE guidelines: C8, likely because both approaches relied on the same site-specific material tests. The LSDyna assessment suggested better performance than the hand calculations mainly because the calculations are based on displacement-based checks, while the LSDyna model incorporates detailed material properties. Additionally, it's important to highlight that guideline C8 is intended for unreinforced masonry, whereas many walls in this building were built with three layers: Oamaru Stone, Concrete, and Brick; all of which were considered in the LSDyna model.

A key insight from the LSDyna analysis was its ability to account for wall softening, period elongation, hysteretic damping and energy absorption which reduced the seismic force transmitted to the tall clocktower - something not captured by the linear elastic ETABS model.

For the client and stakeholders, the adoption of LSDyna finite element model facilitated clear and effective communication of these results through animated and static visual outputs ensuring that technical findings and implications were clear.