

Modelling In-Plane Failure of New Zealand Unreinforced Masonry (URM) Buildings Using OpenSees

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ABSTRACT

New Zealand's unreinforced-masonry (URM) building stock is highly vulnerable to seismic loading. To predict the behaviour of this building type, researchers and practitioners often rely on the Equivalent Frame Model approach by dividing masonry walls into macro-elements (piers and spandrels). This is commonly done using either commercial software or tailored elements. Tailored elements, that explicitly define stress-strain or force-displacement relationships, can be implemented into the Open System for Earthquake Engineering Simulation (OpenSees) analysis platform. However, within the New Zealand context, there is currently no simplified numerical modelling approach to simulate the cyclic degradation due to pier rocking behaviour, which is an observed and critical failure mode in experimental tests. This study develops and implements a modelling framework in OpenSees that captures the non-linear flexural behaviour of fibered-based elements and brittle shear failure through the incorporation of uncoupled shear spring elements. The model's hysteretic law is calibrated against test results from New Zealand pier-and-spandrel systems.

The analysis results closely matched test measurements, validating the proposed approach. This framework, therefore, offers a practical and cost-effective solution for the seismic assessment of New Zealand URM buildings. While calibrated with New Zealand data, the method is generalizable to similar historic URM typologies, enabling broader use in seismic performance evaluation and risk reduction.

1 INTRODUCTION

Following the 22 February 2011 Christchurch earthquake, inspection of 471 unreinforced masonry (URM) buildings identified 112 yellow-tagged (26%) and 291 red-tagged (62%) structures, for which repair was considered infeasible (Dhakal 2011). An investigation of the national URM building stock reported approximately 3750 URM buildings in New Zealand, predominately two-to-four-storey structures (Russell 2010). These figures highlight the significant vulnerability of the URM building stock and have drawn considerable attention from both researchers and practitioners. A common deficiency in New Zealand URM

buildings is the lack of adequate connections between walls, floor diaphragms, and roof (NZSEE 2011), which has been identified as a major cause of severe damage. As a result, seismic retrofit measurements have largely focused on restraining Out-of-Plane (OOP) all failure. While such interventions have been shown to effectively reduce OOP collapse mechanisms (Moon et al. 2014), the global stability of URM buildings governed by In-Plane (IP) behaviour remains a critical concern. This is particularly true for early 19-century constructed building stock, where ageing mortar and brick materials further exacerbate vulnerability (Dizhur et al. 2010). Therefore, there is a clear need for modelling approaches capable of reliably predicting the IP behaviour of URM building stock.

Although finite element (FE) micro-modelling offers the most detailed representation by explicitly modelling masonry units, mortar, and interfaces, it is computationally intensive and impractical for large-scale seismic assessment (Scullion et al. 2021). As an alternative, the equivalent frame approach is widely adopted for modelling URM structures (Hogan et al. 2018; Knox 2012; Pasticier et al. 2008; Scullion et al. 2021). However, in the New Zealand context, two key challenges currently limit widespread application.

First, software availability presents a significant barrier. Common software packages for URM building analysis, such as AEDES (Aedes 2016), SAM (Magenes et al. 2006) and SAP2000 (CSI), are commercial products that are not accessible for every researchers. This restricts transparent and reproducible modelling workflows and limits the ability to benchmark URM performance.

Second, there is a lack of New Zealand specific implementation within open-source platforms. OpenSees (McKenna et al. 2000), an open-source finite-element framework, provides flexible user-defined material and element formulations that could address this gap. Numerous studies have used OpenSees to model the combined flexural-shear behaviour of URM walls (Ayoubi 2021; Requena-Garcia-Cruz et al. 2023; Shabani & Kioumarsi 2022; Vanin et al. 2019). However, these models are primarily calibrated and validated using European construction practices. Given the notable differences in construction techniques and material properties in New Zealand URM buildings (Russell et al. 2006), direct application of these approaches remains uncertain.

This study, therefore, aims to develop a practical OpenSees-based modelling strategy for simulating the in-plane seismic performance of New Zealand URM perforated walls. The proposed model is validated against cyclic experimental tests on New Zealand representative sub-assembly (piers and spandrels) and is intended for integration into building-level seismic assessment workflows, supporting fragility development, retrofit prioritisation, and cost-benefit evaluations.

2 OPENSEES MODEL DEVELOPMENT FOR NEW ZEALAND BRICK URM BUILDINGS

Across material testing (Lumantarna et al. 2014), sub-assembly experiments (Knox 2012), and full-scale building programmes (Bothara et al. 2010; Hogan et al. 2018), a consistent understanding of New Zealand URM has emerged:

- Masonry material properties are characterised by relatively soft mortar combined with stronger fired-clay units, for which prism-level strength correlations are well-established.
- The In-Plane response transitions between stable rocking and brittle shear failure depending on pier aspect ratio, spandrel geometry and axial stress.
- At the building scale, macro-element (equivalent-frame) modelling provides reliable predictions when diaphragm action and orthogonal-wall interactions are approximately represented.

In line with this understanding, an equivalent frame approach is adopted in this study to model URM sub-structures within the OpenSees platform. The modelling strategy is summarised as follows:

- Each perforated URM wall is idealised as a frame system comprising vertical piers and horizontal spandrels connected by rigid nodes.
- Piers and spandrels are modelled using force-based beam-column elements with fibre-discretised cross-sections to capture axial–moment interaction.

- Shear behaviour is represented through phenomenological uniaxial laws coupled to the element kinematics, implemented differently for piers and spandrels, as described in the following subsections.

2.1 Axial-flexural behaviour of piers and spandrels

The OpenSees Concrete02 material, which incorporates a Kent-Scott-Park-type compression model with linear tension softening, is widely used for modelling the fibre-section response of URM walls (Ayoubi 2021; Peruch et al. 2019; Requena-Garcia-Cruz et al. 2023). Accordingly, this material is adopted to represent the axial-flexural behaviour of both piers and spandrels. This choice allows the model to capture strength degradation associated with cyclic loading, as observed in experimental testing of URM structures (Knox 2012).

Material parameters are defined based on New Zealand specific testing experimental data, as summarised here:

- Compression: The masonry compressive strength, f'_m , is taken from sub-assembly testing reported by Knox (2012). The corresponding masonry strain, ϵ_m , is calculated using the empirical relationship proposed by Lumantarna et al. (2014). The post-peak crushing strength, f_{cm} , is assumed to be 20% of the peak compressive strength consistent with recommendations by Lumantarna et al. (2014).
- Tension: The URM components exhibit high compressive capacity but negligible resistance, particularly in ageing construction (Peruch et al. 2019). While zero tensile strength is often assumed, this can lead to numerical convergence issues in OpenSees. To mitigate this, small artificial elastic fibres are commonly introduced for numerical stability (Ayoubi 2021; Peruch et al. 2019). In this study, tensile behaviour is defined separately for piers and spandrels. For piers, the tensile strength, f_{t_pier} , is assumed to be 2% of the compressive strength, representing brittle tensile behaviour (Requena-Garcia-Cruz et al. 2023). This value also aligns within common reported masonry tensile strength range of 1% to 10% of compressive strength (Angelillo et al. 2014; Segura et al. 2021). For spandrels, experimental testing of New Zealand URM sub-assemblies has shown that spandrels predominantly fail through sliding along bed joints under tension (Knox 2012). Accordingly, the tensile, $f_{t_spandrel}$, is taken as measured bed-joint shear strength of 0.51 MPa reported by Knox (2012).

2.2 Shear model

2.2.1 Shear spring in series with the fibre-based element for piers

Figure 1a and 1b, based on experimental sub-structures tests reported by Howlader et al. (2020), clearly illustrate the contrasting responses of piers governed by flexural and shear failure mechanisms. For modelling purposes, although the distributed plasticity approach is widely adopted in the literature (Ayoubi 2021; Peruch et al. 2019; Raka et al. 2015), shear degradation under cyclic loading is not well-captured for piers that fail through rocking. This limitation arises when the predicted shear capacity exceeds the column's flexural yield strength and the walls are governed by a flexural response characterized with limited strain hardening. A similar issue has been identified in reinforced concrete columns (Elwood 2004). To address this limitation, piers in the present study are modelled as a shear spring in series with fibre-based element, illustrated in Figure 1c, so that flexural deformation is modelled by the beam-column element and shear deformations is carried by the shear spring.

In addition, the uniaxial law inside the pier shear spring is defined using SAWS hysteretic material available in OpenSees. It has been successfully used for cyclic shear springs in structural joints (Aleman et al. 2015; Lukic et al. 2018). The SAWS parameters, summarized in Table 1, are defined through a two-step procedure. First, the pier rocking shear capacity is calculated using Equation 1, which is commonly adopted for New Zealand URM walls (Knox 2012; Russell 2010). Second, the degradation parameters are calibrated to reproduce the experimentally observed pinching behaviour and energy dissipation trends under cyclic loading. This procedure enables explicit control of shear strength and degradation while remaining consistent with available experimental evidence.

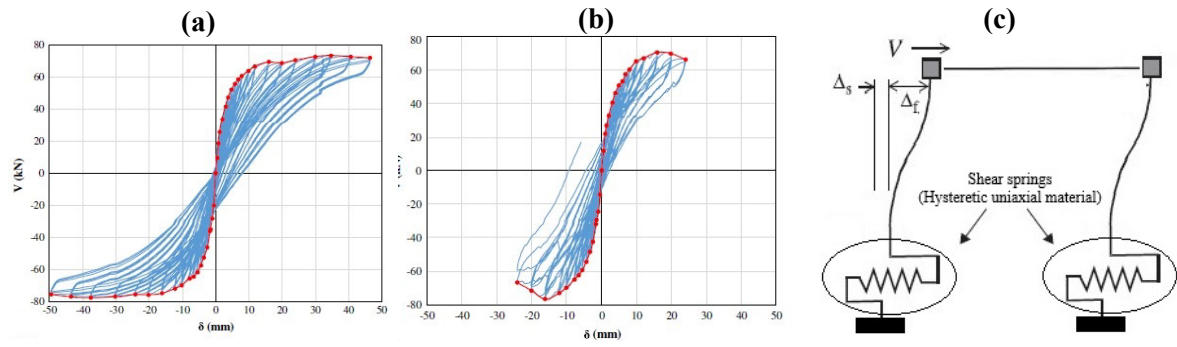


Figure 1: Illustrations for: (a), (b) URM wall governed by flexural and shear responses adopted from (Howlader et al. 2020), and (c) sub-structure with shear spring in series model

Table 1: Calibration of the material laws for shear spring.

F0	FI	DU	S0	R1	R2	R3	R4	alpha	beta
Eq.1	0.7F0	Eq.2	Eq.3	0.05	-0.058	1	0.055	1.1	1.2

$$V_{dt} = \frac{0.9b_p l_p f_{dt}}{\xi} \sqrt{1 + \frac{\sigma_{avg}}{f_{dt}}} \quad (1)$$

where V_{dt} = shear strength of a pier; b_p = width of pier; l_p = length of pier in-plane; f_{dt} = Diagonal tensile strength of masonry; and σ_{avg} = average axial compression stress on a pier.

$$DU = 0.004h_{eff} \quad (2)$$

where DU = displacement at ultimate strength; and h_{eff} = effective pier height.

$$S0 = 0.8 G_m A_{pier} / h_{eff} \quad (3)$$

where S0 = initial stiffness taken as elastic stiffness of a cantilever beam; G_m = shear modulus; and A_{pier} = gross cross-sectional area of pier.

2.2.2 Section-level shear via section Aggregator for spandrels

Since only shear failure mechanism was observed from experimental test (Knox 2012), an approach used by other researchers (Ayoubi 2021; Requena-Garcia-Cruz et al. 2023) is employed by attaching a phenomenological shear law directly to the section using OpenSees section Aggregator. As a result, fibered flexure-axial response is added with a cyclic shear law to the section. The hysteretic with a trilinear backbone allowing pinching and degradation is presented in Table 2.

Table 2: Trilinear backbone parameters.

f1	d1	f2	d2	f3	d3
$0.9V_{spandrel}$	0.001	$V_{spandrel}$	0.0015	$0.4V_{spandrel}$	0.006

3 MODEL VALIDATION

3.1 Experimental database and validation scope

Model validation is conducted using cyclic quasi-static testing on six representative URM pier–spandrel sub-assembly (PS1 to PS6) reported by Knox (2012). The experimental programme provides an appropriate benchmark for assessing the model ability to reproduce the global IP lateral response of URM wall

substructures. Since the key purpose of this modelling is capturing the transitions between stable rocking and brittle shear failure, the validation explicitly targets backbone force-displacement shape, peak lateral strength, post-peak strength degradation trend and displacement capacity. In addition, two justifications are considered to align experimental results with modelling assumptions:

- **Selection of experimental specimens:** Among the six specimens, PS4 and PS6 are excluded from the validation based on test issues reported by Knox (2012). While PS4 experienced unintended pre-damage in the push direction due to a loading system malfunction during test setup, PS6 exhibited localised damage mechanisms when the deep spandrel translated horizontally rather than rotating compatibly with the piers. These behaviours are incompatible with the modelling assumptions of undamaged initial conditions and compatible pier-spandrel kinematics. Their exclusion is therefore methodologically justified and does not bias the validation outcomes. The retained specimens still cover a meaningful range of axial stress levels, pier geometries, and spandrel depths.
- **Experimental-numerical differences:** In the experiment documented by Knox (2012), horizontal load was applied at the pier-spandrel junctions, which does not fully represent distributed seismic inertia forces. Consequently, validation emphasises relative response trends and backbone characteristics (shape, degradation and ductility), rather than exact force levels. Predicted backbone shapes were also cross-checked against an independent Australian sub-assembly testing programme (Howlader et al. 2020), providing additional confidence in the generality of the observed response trends.

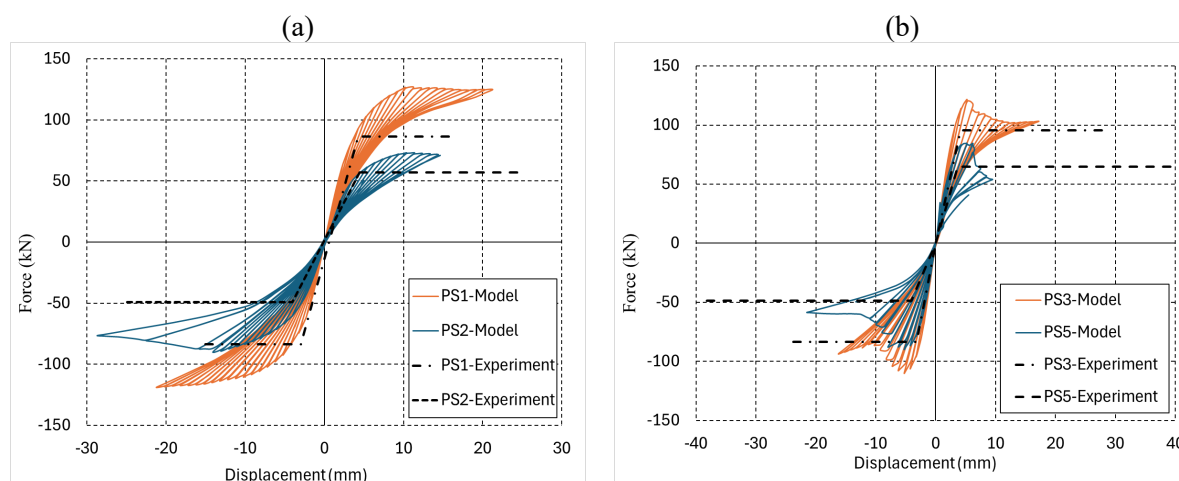
On the other hand, cyclic energy dissipation, residual strength and full-building behaviour are outside the scope of this study. These limitations are intentional and consistent with the intended application of the model.

3.2 Comparison of global force–displacement response

Predicted force-displacement backbone curves for PS1, PS2, PS3 and PS5 are shown in Figure 2, grouped according to geometric similarity. Due to the unavailability of full experimental data, the bilinear idealisation envelopes of the URM substructures reported by Knox (2012) are presented, allowing comparison between the substructures in term of strength and ductility. In addition, quantitative comparisons of peak strength and displacement capacity are summarised in Table 3. The maximum displacement reported in Table 3 corresponds to the displacement at which the lateral resistance drops to 80% of the peak strength (shear-dominated response), or the maximum displacement reached in the test where the degradation was not observed (flexural-controlled response). The same definition is consistently applied to both experimental and numerical results to ensure comparability.

Table 3: Model experiment comparison to bilinear parameters reported by Knox (2012).

Specimen	Max_Base shear (kN)			Max_Displacement (mm)		
	Model	Experiment	Ratio of model/experiment	Model	Experiment	Ratio of model/experiment
PS1	126.89	96.30	1.32	22.25	15.00	1.48
PS2	73.16	60.98	1.20	28.75	24.93	1.15
PS3	121.39	104.90	1.16	17.25	23.74	0.73
PS5	84.64	75.30	1.12	21.54	40.26	0.56



PS1: Piers (0.23x1.19, L=2.09m, axial stress = 0.52 MPa) / Spandrel (0.23x0.59, L=2.43)
 PS2: Pier A (0.23x1.19, L=2.09, axial stress = 0.36 MPa) / Pier B (0.23x0.71, L=2.09, axial stress = 0.58 MPa) / Spandrel (0.23x0.59, L=2.69)
 PS3: Piers (0.23x1.19, L=2.265, axial stress = 0.48 MPa) / Spandrel (0.23x0.94, L=2.43)
 PS5: Piers (0.23x1.19, L=2.265, axial stress = 0.22 MPa) / Spandrel (0.23x0.94, L=2.43)

Figure 2: Predicted force-displacement curves for PS1, PS2, PS3 and PS5

The model successfully distinguishes between flexure-dominated and shear-dominated behaviour. Specimens PS1 and PS2 exhibit stable, flexural (rocking-controlled) responses with limited post-peak degradation, while PS3 and PS5 display pronounced strength degradation consistent with shear-dominated behaviour. This transition is correctly captured and is attributed to the increased spandrel depth in PS3 and PS5, consistent with the assumed pier-spandrel interaction mechanism.

Peak lateral strength is systematically overestimated by approximately 10% to 30%. This bias is primarily attributed to conservative assumptions regarding masonry tensile strength and shear modulus, for which specimen-specific material data were unavailable, and is consistent with the validation limitations outlined in Section 3.1.

Predicted displacement capacities exhibit greater scatter but consistently exceed a drift ratio of 0.8% of pier height, aligning with deformation limits commonly reported for New Zealand URM walls (Knox 2012; Marino et al. 2019).

4 CONCLUSION AND FUTURE RESEARCH

This paper proposes a New Zealand-calibrated, open-source equivalent-frame modelling strategy for the IP seismic response of URM perforated walls. The primary contribution is a transparent and reproducible OpenSees workflow that captures both rocking-dominated and shear-degrading behaviour within a unified framework, explicitly aligned with observed New Zealand URM response.

Within the defined validation scope, the model adequately reproduces the global backbone response, governing deformation mechanisms, and strength degradation trends of URM pier-spandrel substructures. While peak strength is overestimated, the trends are consistent and attributable to material uncertainty rather than modelling inconsistency. By relying exclusively on standard OpenSees material models, the framework also supports scalable application across a wide range of URM wall geometries and axial load conditions relevant to the New Zealand building stock. This provides a practical pathway for researchers and engineers to evaluate URM wall performance using transparent, non-proprietary tools. Ongoing work is extending the approach to full-building models, enabling system-level seismic performance assessment.

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