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# Simplified Non-Linear Response History Analysis of Low-Rise Unreinforced Masonry Buildings

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## ABSTRACT

This paper describes a simplified Non-Linear Response History Analysis (NLRHA) process for the seismic assessment of low-rise masonry buildings. The key step to the simplified NLRHA process involves “condensing” the 3-Dimensional building down to a series of equivalent 2D nonlinear frame models (macro models). Specifically, for buildings containing flexible timber diaphragms, each wall line is represented by an individual 2D nonlinear frame model (the flexible diaphragm idealisation assumes the wall lines are decoupled from each other). To limit the number of individual analyses, a process to identify and then evaluate only the most critical wall lines using NLRHA is adopted – any wall line not assessed using NLRHA would be deemed no worse than the critical wall lines that are assessed. Each 2D macro frame model comprises of three inter-connected sub-models to capture the key vulnerabilities of masonry buildings – in particular, the sub-models include an in-plane model, an out-of-plane model, and a leaning column model. All three sub-models are constrained via a diaphragm model which can also include an allowance for floor-to-wall nonlinear connection behaviour. This paper outlines details of each sub-model, along with numerical output, and how the NLRHA assessment compares to more traditional assessment methods.

## 1 INTRODUCTION

The seismic assessment of low-rise unreinforced masonry (URM) buildings is commonly assessed using simple analysis techniques in New Zealand. Methods to assess URM buildings are covered in Chapter C8 of the MBIE Seismic Assessment of Existing Buildings Guideline (MBIE et al., 2017), with recent updates contained in the Public Comment Draft that was released in August, 2025 (MBIE et al., 2025). The Assessment Guideline suggests the use of linear techniques be supplemented with simple nonlinear

techniques (aka Simple Lateral Mechanism Analysis, SLaMA) for Basic URM buildings - this approach is generally considered appropriate for most URM buildings in New Zealand that are not considered complex. A “Basic” URM building is defined in the Assessment Guideline as being up to two storeys in height, and containing flexible (timber) diaphragms. Also inherent in a Basic URM building is that lines of lateral resistance are located along each edge of the diaphragm, in both directions. This combination of features is expected to result in limited interaction between lines of lateral resistance in each direction i.e. each line of lateral resistance is evaluated using a simple tributary-area based analogy.

In contrast to the method for “Basic” URM buildings in New Zealand, linear static, linear dynamic, nonlinear static, and nonlinear dynamic analysis procedures can be adopted in North America using ASCE41-23 (ASCE, 2023) which permits either complete 3-Dimensional analysis methods, or a series of independent 2-Dimensional “wall line models” if the diaphragm can be idealised as flexible. Linear procedures are only permitted for “URM Common Building Type” that meet several limitations relating to height, number-of and plan configuration of wall lines, and diaphragm flexibility.

In the Netherlands, masonry buildings are assessed for induced seismicity using a range of methods, including detailed FEM nonlinear dynamic analyses, through to more contemporary macro-based nonlinear pushover methods (i.e. equivalent frame models), using the Dutch Practical Guideline NPR:1998 (NPR:9998, 2021). Earthquakes in the northeastern part of the Netherlands exist because of the subsidence of the ground at relatively shallow depths beneath the earth’s surface due to the extraction of gas from the Groningen gas field – the Dutch Guideline refers to this seismic activity as ‘induced earthquakes’, which differ distinctly from ‘tectonic’ earthquakes.

Despite the various analysis methods available to assess URM buildings, one of the more complex aspects of these buildings is the ability to evaluate the out-of-plane (OOP) behaviour of vertically spanning bearing walls. Even when a nonlinear dynamic analysis is adopted to evaluate a URM building, the OOP evaluation is often performed as a separate stand-alone evaluation, due to the complexity of modelling the de-coupled nonlinear in-plane and nonlinear out-of-plane behaviour of vertically spanning URM wall elements within the same nonlinear model. That is, URM walls have very different nonlinear behaviour, and failure modes in the in-plane vs out-of-plane direction, which requires a different modelling technique be adopted in the two orthogonal directions for the same element.

The method to evaluate the OOP performance of URM walls in Chapter C8 of the NZSEE Guideline adopts principles of ‘virtual-work’, which compares the theoretical mid-height instability of a vertically spanning bearing wall against the spectral demand of the wall. A similar evaluation method exists in the Dutch Guideline, referred to as the Nonlinear Kinematic Analysis (NLKM). In North America, a simple force-based evaluation method is now used, with modifications to account for diaphragm flexibility, wall overburden demand, height/thickness ratio, and the presence of cross-walls (ASCE41, 2023). Often, it is the OOP behaviour of vertically spanning URM bearing walls (and their connections) that govern the global performance of URM buildings.

This paper presents a simplified approach to model the nonlinear behaviour of low-rise URM buildings, combining the in-plane and out-of-plane assessment into a single 2-Dimensional nonlinear dynamic analysis.

## 2 OVERVIEW OF THE MODEL

The nonlinear modelling process in this paper adopts a nonlinear macro-frame-model, comprising of three connected sub-models, which includes:

1. In-plane wall model
2. Out-of-plane wall model, and

### 3. Leaning column model

An overview of the complete nonlinear model is shown in Figure 1. The three sub-models are constrained to each other via a master diaphragm node at each level, as follows:

- In-plane wall model is connected to the master diaphragm node via a diaphragm spring.
- The out-of-plane model is connected to the master diaphragm node via a wall-to-roof/floor restraint (either stiff, or nonlinear).
- The leaning column (P-Delta column) is connected to the master diaphragm node via a stiff/rigid spring.

The process of creating the nonlinear model requires the 3-Dimensional building be reduced to a 2-Dimensional analysis. An example of a typical building in which this process has been used on is shown in Figure 2. This 3D image represents the model in Figure 1 used to evaluate the South wall (in-plane) for E-W earthquake loads, including an out-of-plane evaluation of the West and East walls (the West wall is outlined red).

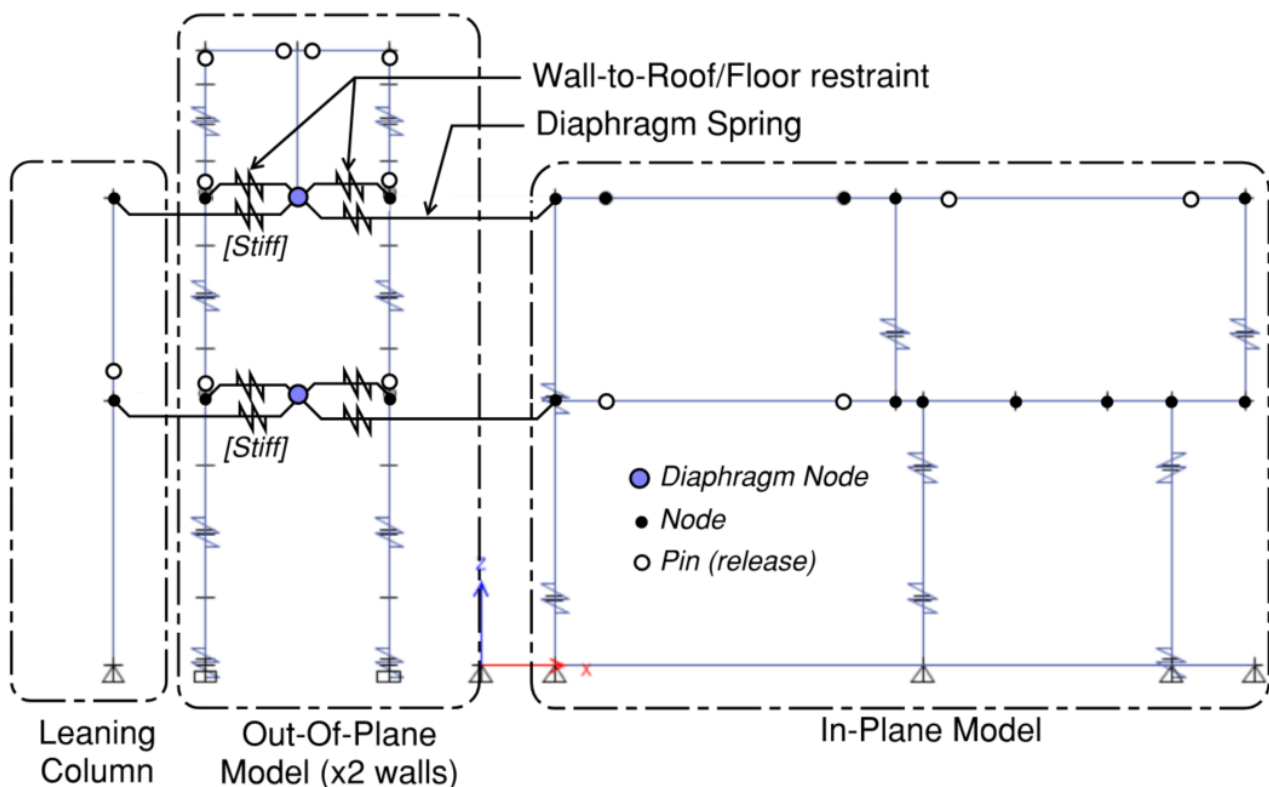
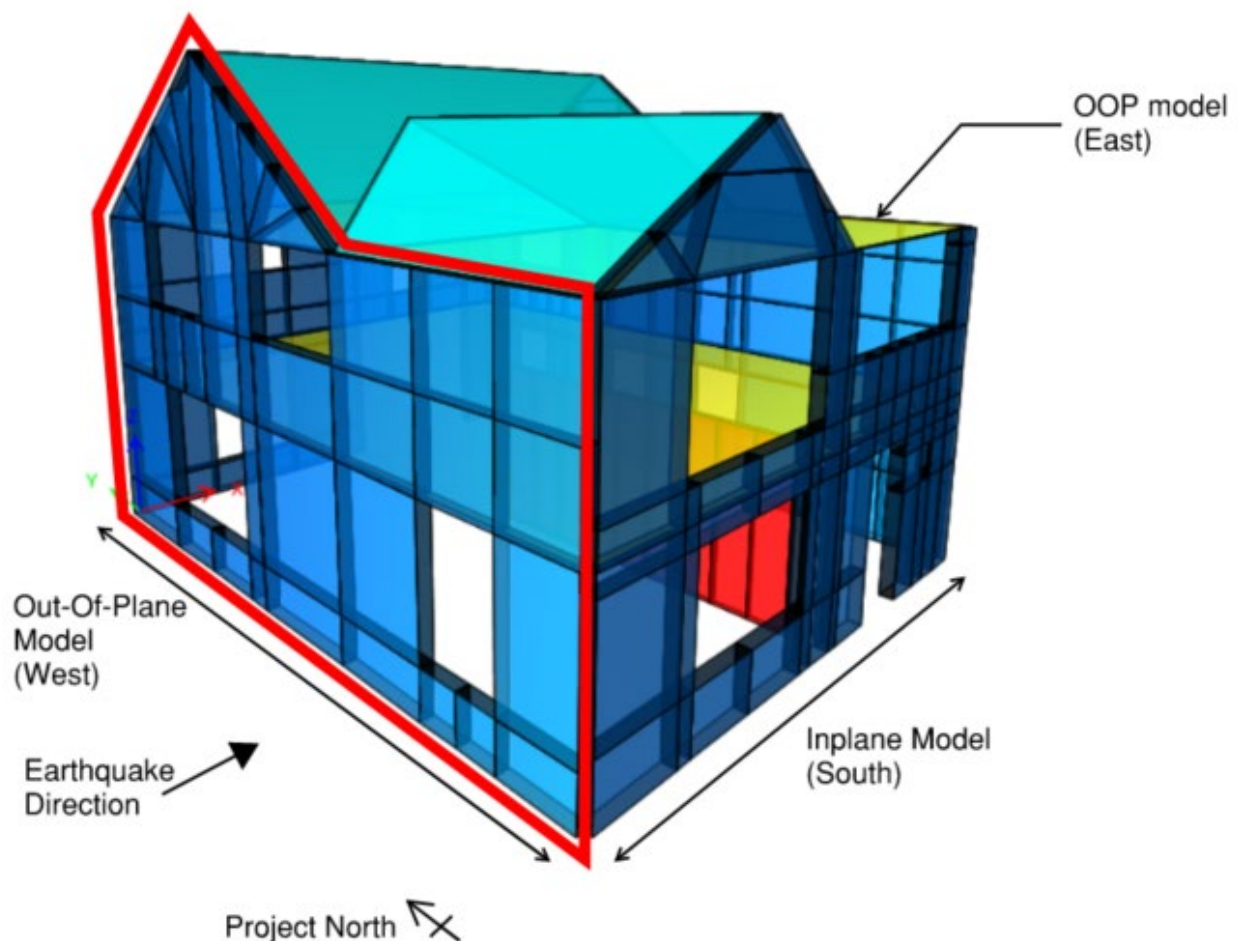


Figure 1: Non-Linear Model Overview

### 3 IN-PLANE WALL MODEL

The in-plane wall model accounts for the stiffness, strength and inertia of the masonry wall line that contributes to in-plane resistance. A 2D in-plane equivalent frame model is illustrated in Figure 3, comprising of nonlinear beam-column elements, stiff (rigid) line elements, and pin releases. Pin releases are implemented at spandrels that are not expected to provide significant strength/stiffness, and therefore not relied upon for lateral resistance. Stiff frame elements are used to model the stiff ‘panel zones’ between interesting spandrel and pier elements.



*Figure 2: Two-storey Unreinforced Masonry Building*

The elements that contribute to nonlinear response within 2D in-plane model are shown in Figure 3, comprising of nonlinear shear hinges (spandrel & pier shear), and nonlinear rotational hinges (pier rocking). Each pier element includes a nonlinear shear hinge to model bed-joint sliding shear behaviour (a common controlling shear mode), and a nonlinear rotational hinge to model pier rocking behaviour– the strength envelope for each of the shear and rotational hinges is determined based on the geometry of the pier, axial load, and brick properties (i.e. calculated outside of analysis model and input into the model).

Spandrels are modelled with a degrading stiffness/strength shear hinge, located at mid-span. The strength of the spandrel is taken as the critical mechanism (lowest strength), considering either the shear strength, or the flexural strength of the spandrel assuming the spandrel is in double curvature, and includes the degraded (residual) strength of the spandrel. Strength degradation of rocking piers is not modelled, on the basis that strength degradation of rocking is associated with toe crushing, and pier rocking is (generally) not permitted to exceed the deformation at which toe crushing occurs.

The pier and spandrel shear hinges are modelled with a nonlinear pinched hysteresis (i.e. ‘Pivot’ hysteresis in ETABS), shown in Figure 3. The rotational/rocking hinges are modelled with a nonlinear elastic hysteresis (Multi-Linear Elastic hysteresis in ETABS, also shown in Figure 3). The Multi-Linear Elastic hinge loads and unloads onto itself (no hysteretic energy dissipation) – the only form of energy dissipation from pier rocking is from the elastic damping model.

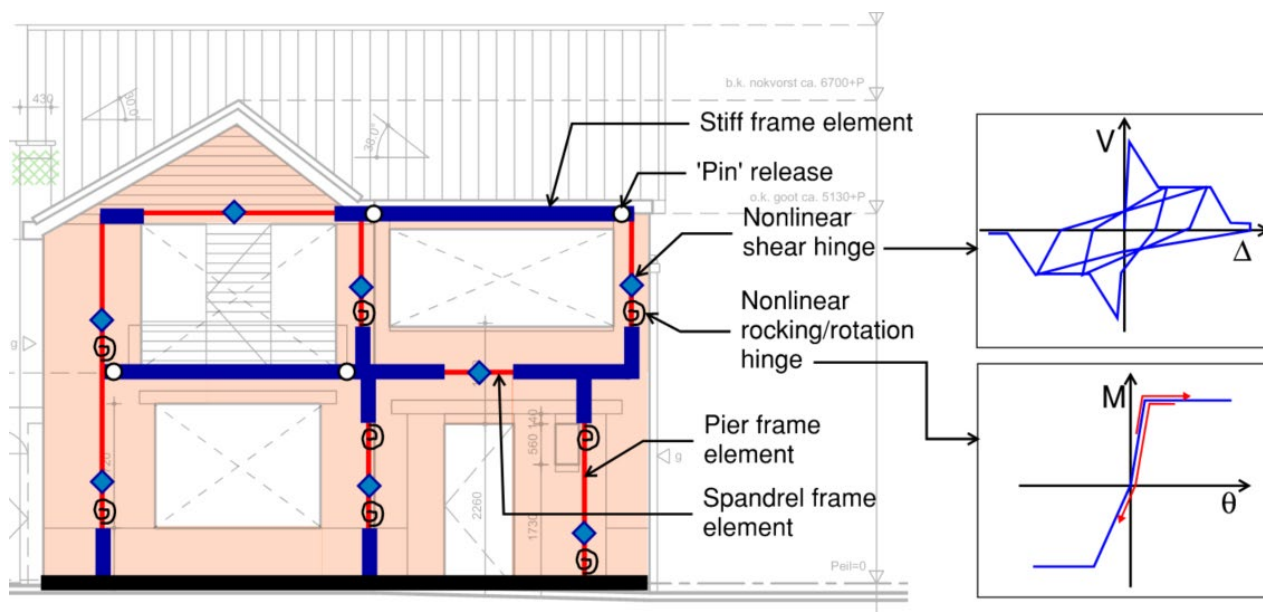


Figure 3: 2D In-plane Non-linear equivalent frame model

#### 4 OUT-OF-PLANE WALL MODEL

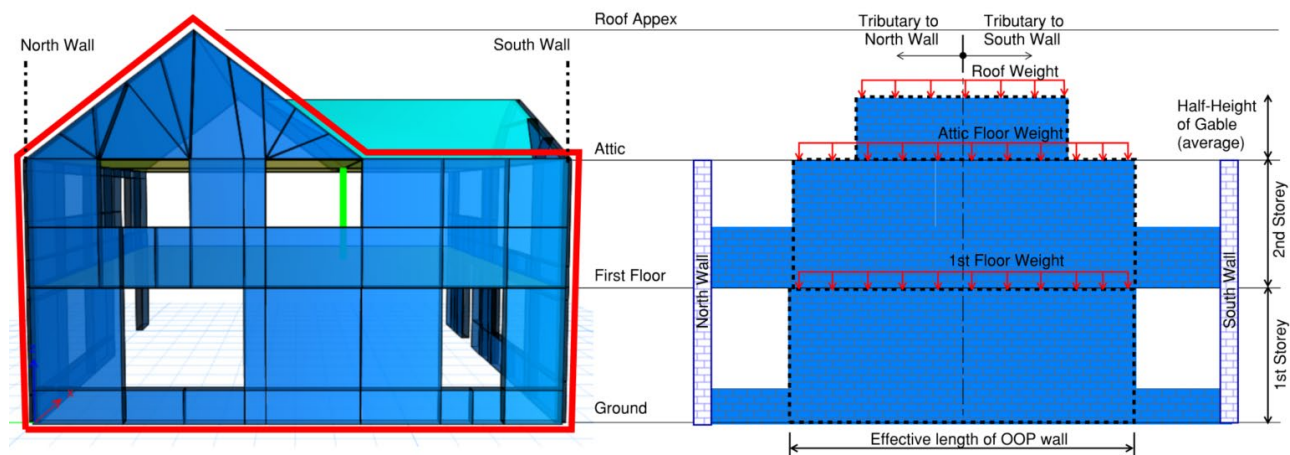
The out-of-plane (OOP) model includes a single vertically-spanning brick wall element for each face-loaded, load-bearing, perimeter wall (and internal wall, if needed). Using the building in the image of Figure 2 as an example, the building has one OOP model for the West wall and one OOP model for the East wall – at every level the OOP model is connected to the diaphragm node, and the diaphragm node is connected to the in-plane model (South wall).

The wall openings along the OOP model are consolidated to create a single (solid, no openings) OOP wall, shown in Figure 4. This assumption is only reasonable if the wall thickness and vertical span are constant, and the axial stress on the wall is relatively constant over the wall elevation i.e. the roof and floor joists apply uniform load to the OOP wall. The geometry of the resultant ‘solid’ OOP model is then halved, on the assumption that one half of the OOP resistance (and demand) is distributed to each of the in-plane perimeter walls (North and South walls) via a flexible (timber) diaphragm.

The vertical span of the gable is taken equal to one-half of the wall height from attic to roof apex – this represents the average vertical span of the gable wall. A half-height one-way spanning gable wall is considered a reasonable approximation to the more complex two-way spanning behaviour that is likely to occur.

The components of the OOP wall model are illustrated in Figure 5. A Multi-Linear (nonlinear, elastic) rotational hinge located at mid-height of the wall to capture the OOP flexural strength of the wall (mid-height rocking). A ‘pin’ is modelled at the top and bottom of each vertically spanning wall to acknowledge vertically misaligned wall openings will prevent ‘flexural continuity’ above/below a floor. A rotational hinge is located at the bottom and mid-height of the first storey wall. Both the New Zealand URM Assessment Guidelines (MBIE et al., 2025) and the Dutch Practice Guideline (NPR:9998, 2021) provide equations to determine the mid-height OOP instability displacement of a URM wall using principles of virtual work, and are dependent on the wall-floor boundary conditions (top and bottom pivot-point eccentricity). It is also important to account for the eccentric gravity load effect that exists between the floor and the wall (wall-mounted joist support, for example) – this eccentric load imposes out-of-plane flexure demands on the OOP wall which can create stabilising or destabilising effects on the wall, depending on the direction of face

loads. It is assumed that the floor reaction from timber joists pocketed into the wall, load the wall concentrically (no eccentricity).



*Figure 4: Consolidation of openings on OOP wall model*

The OOP wall model requires mass and weight to be modelled between levels. Each vertical span is divided into (4) segments within the level, resulting in (3) intermediate nodes per vertical span. P-Delta modelling is necessary to capture the negative post-yield stiffness that occurs in rocking systems. The nonlinear behaviour of the OOP model is shown in Figure 6 – the red line in the graph is the static pushover strength envelope (shear versus OOP-drift). The blue line is the response from a nonlinear (dynamic) response history analysis. This general force-deformation relationship is in line with the body of research and experimental work referenced in Chapter C8 of the Assessment Guideline. The nonlinear behaviour demonstrates the ability of the model to correctly capture the OOP response of the wall, including the degrading resistance that occurs due to geometric nonlinearity (P-Delta).

OOP restraint of the wall is modelled via a spring that connects the OOP wall to the diaphragm node (wall-floor spring in Figure 5). If OOP restraint is provided via mechanical anchors, a simple force-controlled elastic spring is used to assess the strength of the wall-floor connection. If OOP restraint of a floor (excluding roofs) is provided via joists pocketed into the supporting wall, a deformation-controlled friction/sliding inelastic spring can be used to evaluate the connection. This nonlinear friction/sliding model is illustrated in Figure 5 (right), and is achieved by modelling the OOP restraint via 3-springs 1) Static friction spring, 2) dynamic (reduced) friction spring, 3) Compression-only joist-bearing spring. Loss of support of the joists occurs when the sliding displacement exceeds the seating length of the pocketed joist. Joist sliding is not considered an appropriate modelling technique to evaluate the restraint at the top of a wall - the high variability in displacement response for walls with partial restraint at the top of the wall results in OOP wall evaluations that lack reliability.

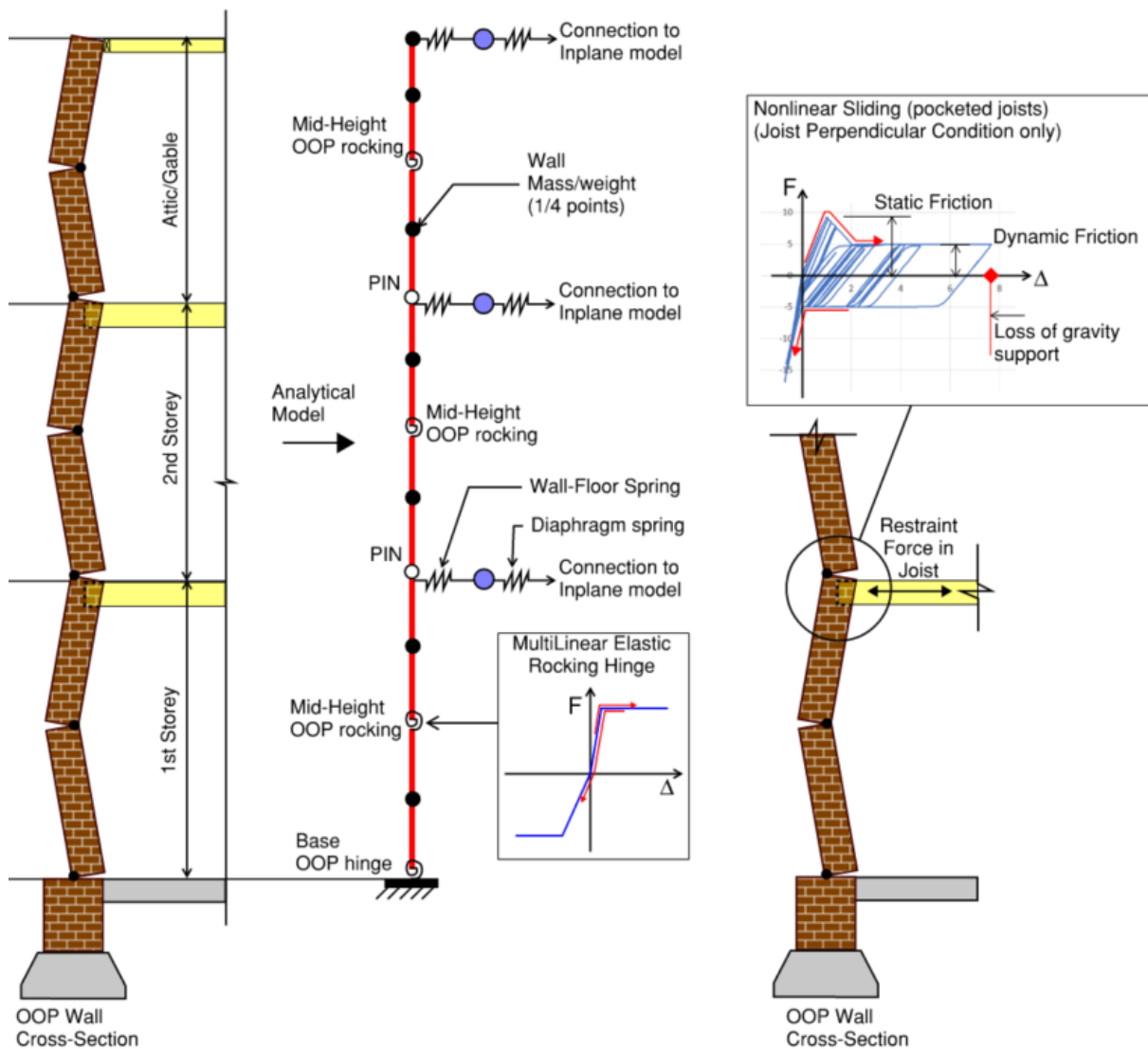


Figure 5: Out-of-Plane Wall Model

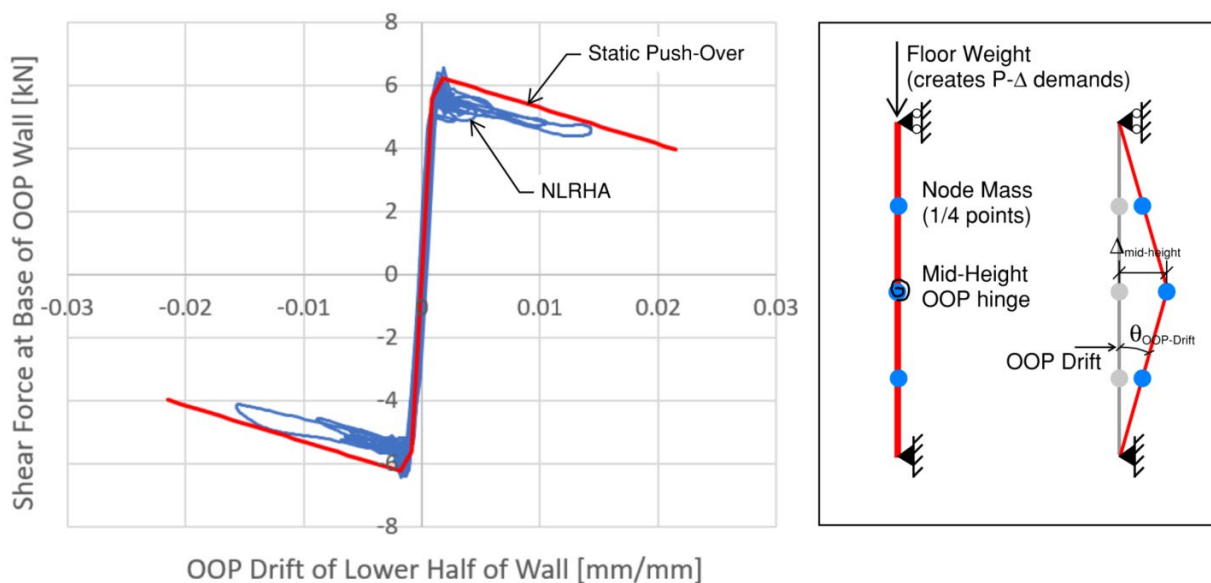


Figure 6: Behaviour of the Out-of-Plane Wall Model

## 5 DIAPHRAGM STIFFNESS AND LEANING COLUMN

The stiffness of the diaphragm spring, which couples the in-plane model to the OOP model is determined from a 3D elastic model of the building (if simple relationships are not adequate). This requires that the 3D elastic model include the in-plane stiffness of the components of the floor system that provide significant diaphragm stiffness (OSB/ply panels, floor joists, roof trusses etc). Shear transfer between the diaphragm and the in-plane wall is evaluated via a force-controlled elastic spring (diaphragm shear transfer is considered brittle when shear strength is provided via mechanical anchors, or joists pocketed into the supporting wall).

A leaning column is included in the model, being a pinned column that supports the portion of diaphragm weight which is not supported by either the in-plane wall or OOP wall model. This additional weight is necessary to ensure that the entire building weight is modelled for the purpose of capturing the global P-Delta demands imposed onto the building.

## 6 NLRHA VS. NLPO & NLKM

A nonlinear (static) pushover (NLPO) analysis is an essential tool that is used to review the behaviour of the nonlinear model. Figure 7 below shows the inelastic mechanism that results from the behaviour of the in-plane model. The resultant NLPO capacity curve is combined with the spectral demand to estimate global displacement demands of 20mm-40mm via an Acceleration-Displacement-Response-Spectrum (ADRS) plot.

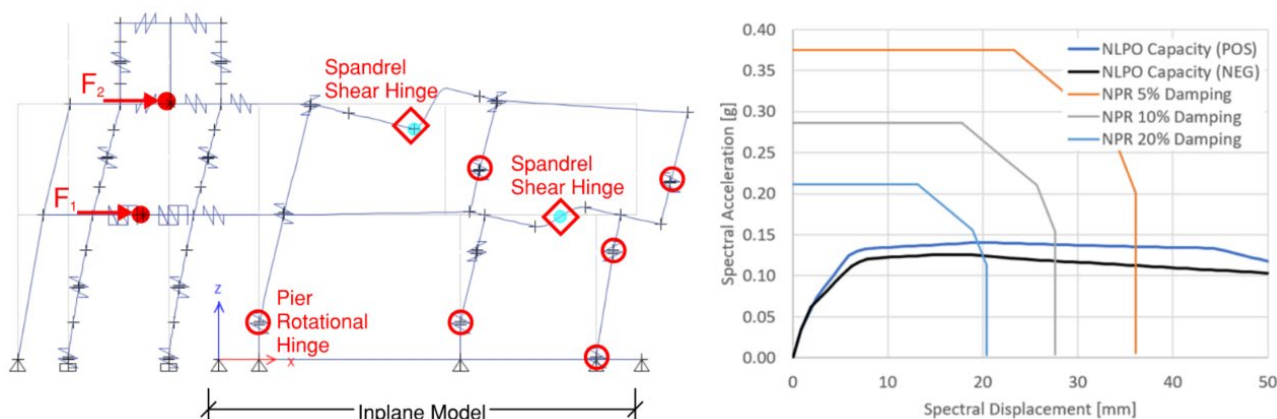


Figure 7: NLPO Behaviour of Wall Elevation

The displacement response at the First Floor and Roof diaphragm nodes for a single ground motion (scaled to the target spectrum) is shown in Figure 8, showing displacement demands of similar magnitude (circa 20mm) to that of the NLPO method.

The storey-shear (at the base of the structure) vs roof displacement is another useful response parameter to compare to the NLPO evaluation. Figure 9 compares the NLPO capacity curve against the time history response for a single ground motion (NLRHA). The NLRHA shows dominant nonlinear elastic rocking (elastic unloading) attributed to pier rocking – a minor amount of inelastic behaviour is evident, and is attributed to nonlinear spandrel shear behaviour.

One of the key advantages of the NLRHA OOP evaluation is that it removes some of the approximations inherent in the OOP virtual-work evaluation method (NLKM). Figure 10 below plots the OOP drift time-history for the east wall for a single ground motion – the low magnitude of OOP drift demands is evident (less than 1.0%), being much less than the theoretical instability displacement of the wall i.e. the wall is not vulnerable to OOP instability.

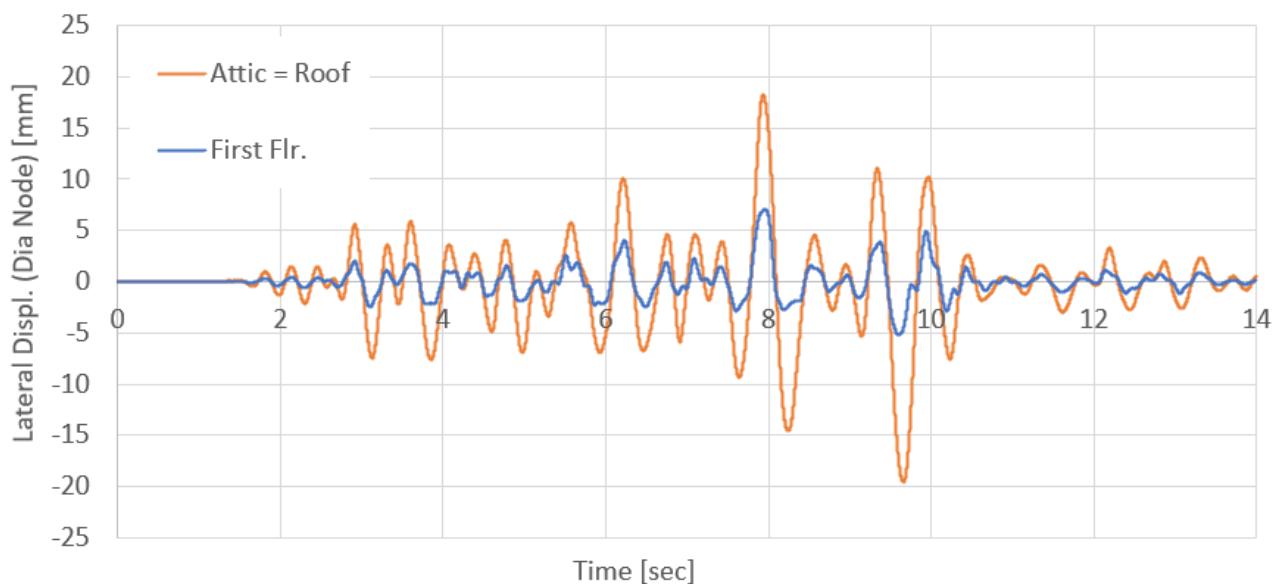


Figure 8: Displacement time-history (diaphragm nodes) for a single ground motion

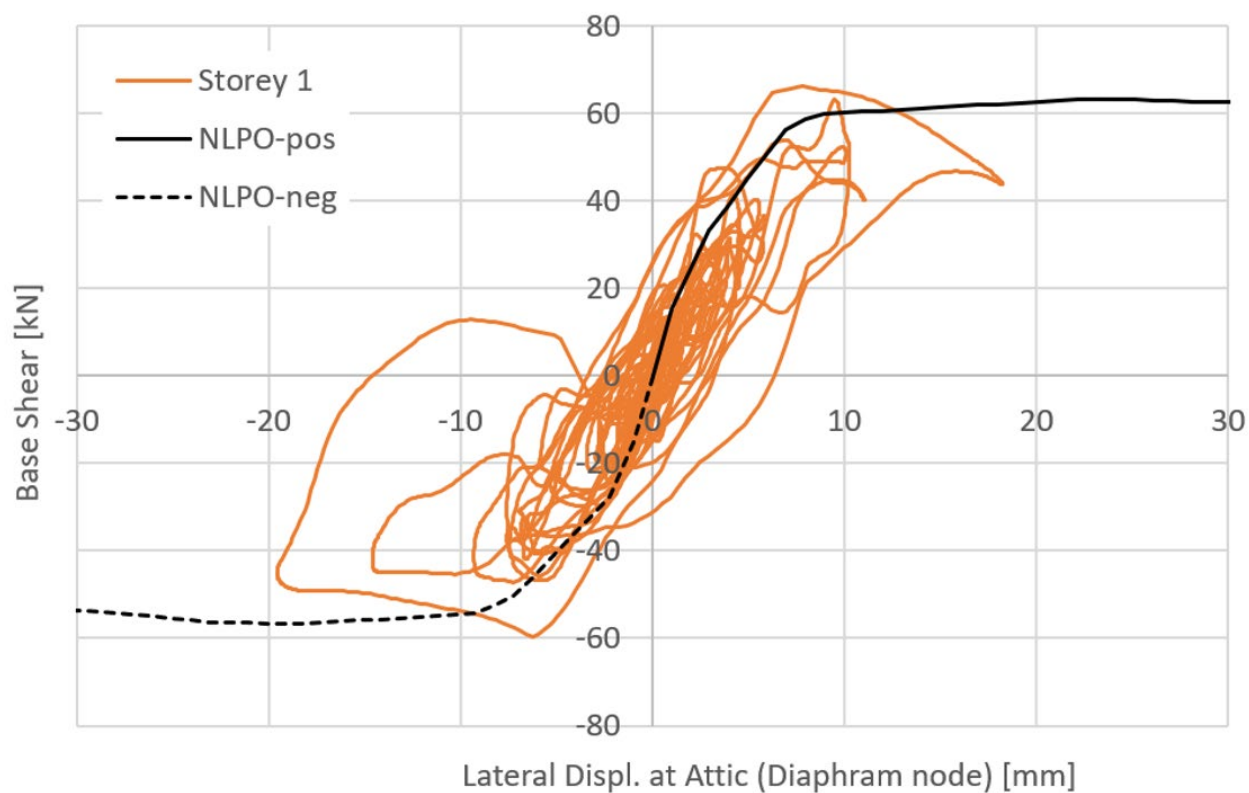
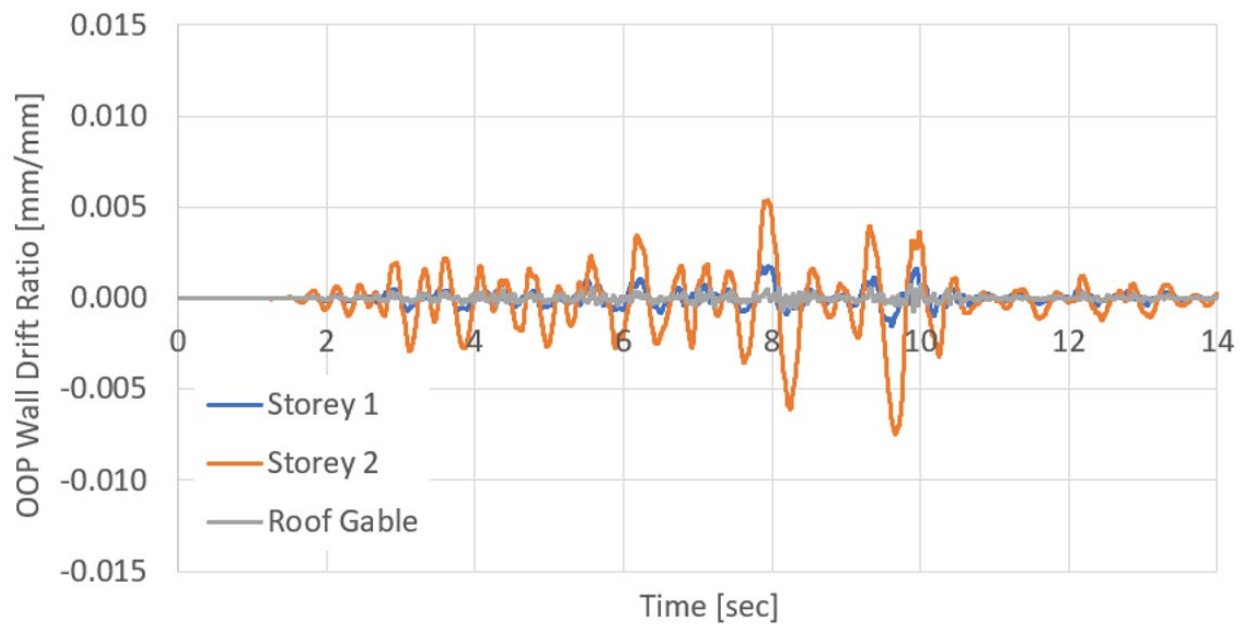


Figure 9: Storey shear vs roof displacement for a single ground motion, compared to the NLPO capacity curve



*Figure 10: OOP wall drift time history (East wall) for a single ground motion.*

Table 1 below compares the OOP wall assessment (demand-to-capacity ratios, DCR) between the NLRHA evaluation and the NLKM evaluation (values greater than 1.0 indicate instability). For both methods, the mid-height OOP displacement limit is equal to 60% of the instability displacement of the wall. The NLRHA OOP evaluation uses the mean of the maximum OOP displacement demands from the ground motion suite. The NLRHA evaluation reports a less-conservative OOP wall evaluation (lower DCR's) – NLRHA is able to take advantage of the coupling generated between the OOP and In-plane wall models to better quantify the (dynamic) demands on the OOP wall. Interestingly, both methods indicate the OOP wall between the First-Floor to Attic is the most vulnerable wall (largest DCR); however, the NLRHA method indicates the wall is much less vulnerable than the NLKM evaluation would suggest (0.20 vs 0.79).

*Table 1: OOP mid-height instability demand-to-capacity ratios – comparison between NLRHA and NLKM*

| OOP Wall Level        | East Wall<br>NLRHA | East Wall<br>NLKM | West Wall<br>NLRHA | West Wall<br>NLKM |
|-----------------------|--------------------|-------------------|--------------------|-------------------|
| Attic to Roof         | <0.05              | 0.71              | <0.05              | 0.60              |
| First-Floor to Attic  | 0.20               | 0.79              | 0.19               | 0.56              |
| Ground to First-Floor | <0.05              | 0.17              | <0.05              | 0.13              |

## 7 CONCLUSIONS

This paper has outlined the details of a 2-Dimensional NLRHA procedure for the seismic assessment of low-rise masonry buildings. It should be noted that the process described herein is not expected to predict the seismic performance of masonry buildings; however, it is intended to remove some of the conservatism inherent within more simple methods to assess the in-plane and out-of-plane seismic vulnerability of low-rise URM buildings.

The 2-Dimensional NLRHA has the following benefits:

- Intuitive to verify and review i.e. simple to detect modelling errors and interrogate component behaviour.
- Uses fundamental modelling techniques that are adopted in nationally-recognised seismic assessment guidelines (MBIE et al, 2025., NPR:9998, 2021., and ASCE, 2023).
- Low computational effort, and rapid analysis run times (compared to complex 3-Dimensional analyses).

While there are strong benefits above, the process to convert a 3-Dimensional building response to a series of 2-Dimensional analyses does require a good understanding of how low-rise masonry buildings are expected to behave in an earthquake, to ensure modelling decisions are founded on sound principles of earthquake engineering and structural dynamics.

## 8 REFERENCES

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