
Updates to Section C8 (Unreinforced Masonry Buildings) of the Engineering Assessment Guidelines

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ABSTRACT

Despite recent proposed changes to the Earthquake-prone Buildings legislation, unreinforced masonry buildings (URM) appear likely to remain at the centre of mandatory seismic upgrades prescribed by legislation. A draft update to Section C8 (Unreinforced Masonry Buildings) was issued in 2025, to collate new knowledge in the field of masonry assessments and to address some of the issues and gaps which engineers have reported with the 2017 edition. This paper summarises the key changes and provides an introduction to the new and expanded methods for assessment provided in the updated edition.

Highlights include new methods for assessing masonry gables by extending the rocking analysis model for vertical walls to new shapes; additional guidance on boundary conditions for rocking walls; expanded clarification for how to select an equivalent frame when assessing the in-plane response of a penetrated unreinforced masonry wall; a more thorough treatment of cavity masonry walls; new guidance on horizontal spans in masonry including the design of strongbacks; and a new set of tables for out-of-plane wall and parapet responses. The generation of the new tables has led to the identification a small number of errors in the 2017 tables which may have resulting in unconservative assessments of a small subset of parapets. In addition to covering the updates, this paper serves to provide notice and information to engineers who may have used the tables for parapet assessments.

1 INTRODUCTION

The Building (Earthquake-prone Buildings) Amendment Bill had its first reading on 16 December 2025. The Bill outlines proposed changes to settings for mandatory minimum seismic upgrades for buildings in New Zealand. The Bill proposes that one of two building typologies “in scope” for mandatory seismic upgrades

will be unreinforced masonry buildings (URM). In addition, the Bill notes that some URM buildings will require “full retrofit” while others will only need “façade securing” (NZ Gov’t, 2025).

Building population survey work carried out by Beca as part of the background policy development for the proposed new legislation indicates the approximate number of URM buildings expected to require seismic upgrades, if the Bill is passed into law. Table 1 shows a breakdown of these buildings and the probable upgrade category (full retrofit to EPB threshold vs façade securing) into which they fall (Beca, 2025).

Table 1: Summary of earthquake-prone URM buildings in New Zealand

Earthquake-prone URM buildings	Current EPB register	Proposed new EPB regime			
		Removed (low zone)	Risk Register only	“Full Retrofit to EPB threshold”	Facade Securing
1-2 storey URM	2,672	701 Removed	824	-	1211
3+ storey URM	245	68 Removed	-	104	9

Note: The region of Otago still has until 2032 to complete the process of identifying EPBs.

From Table 1, it can be seen that approximately 1300 URM buildings will require mandatory seismic upgrades. Noting that the “full retrofit” category will include work to the façade, it is clear that continued attention to the response and performance of masonry facades is warranted.

The 2025 update to Section C8 is largely focused on elements commonly occurring within URM façades in NZ – gables, cavity walls, horizontal spans, parapets, and boundary conditions for rocking (OOP) wall elements. This new content is expected to be relevant in the process of providing seismic upgrades to the URM buildings encompassed by the Bill, as well as buildings undergoing voluntary seismic improvement.

At present the 2025 update can be accessed from

https://www.engineeringnz.org/documents/2681/Part_C8_for_Public_Comment.pdf

2 GABLES

Gables merit specific consideration due to their location, the influence of boundary conditions, and relative complexity compared to rectangular walls. Their location, at the uppermost elevations of the building, means they are subjected to the greatest amplification of seismic input. Their performance during an earthquake is highly dependent on their boundary conditions. Furthermore, due to their triangular geometry it is inappropriate to consider their response on a per unit length basis as can be done for a regular vertical spanning wall or a regular cantilever. Instead, the complete geometry must be considered to correctly determine the self-weight and overburden loads, accounting for the variety of triangular gable configurations (see Figure 1) which is essential for an accurate assessment of the gable response.

The assessment procedure presented in the guidelines is based on rocking dynamics and is a displacement-based approach. For each identified gable configuration, equations are provided to calculate the rotational mass moment of inertia, from which the rocking period and participation factor are determined and subsequently used to evaluate the displacement capacity. These formulations are derived by establishing the equations of equilibrium applying the principle of virtual work. Practical examples for each gable configuration are included in the guidelines to illustrate the application of the proposed methodology.

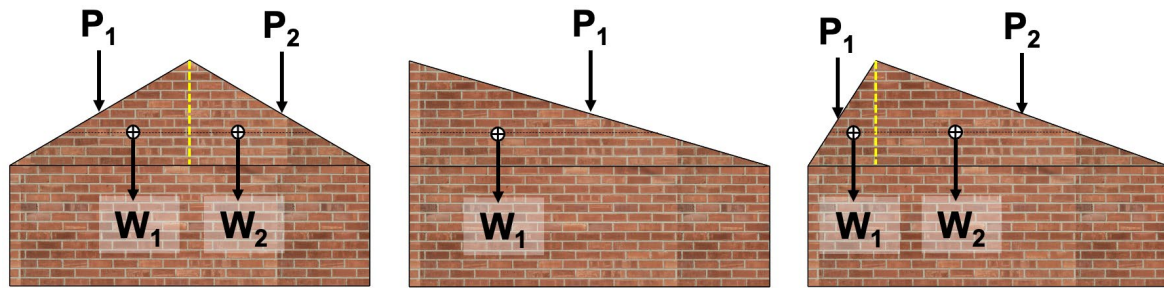


Figure 1: Gable geometric variations

Three general cases can be identified for gables based on the degree of fixity provided by the roof, by the ceiling diaphragm, and by the perpendicular return walls. A reduction in wall thickness at the height of the gable where the diaphragm is supported may also be a contributing factor, as may any openings, associated concrete lintels, or concrete ring beams present either in the gable or the wall directly below.

2.1 Case 1: Vertical cantilever gables

Gables failing as triangular vertical cantilevers are triggered by a discontinuity at the base of the gable. This failure mode may result from a change of wall thickness between the gable and the wall below the gable or by the presence of detailing such as a well-connected ceiling diaphragm, a reinforced concrete lintel, or a concrete bond beam. Additionally, this failure mode requires that there be no evidence that the gable is adequately fixed to the roof structure. This scenario may also arise when the wall below the gable has thickenings or buttresses that terminate at the base of the gable. These conditions cause a rotational weakness to occur at the base of the gable, which becomes the hinge point for the cantilevered failure. This failure mode was frequently observed in the Canterbury earthquakes, with examples shown in Figure 2.



(a) Concrete lintel discontinuity



(b) Diaphragm discontinuity



(c) Top triangle failure

Figure 2: Gables failing as vertical cantilevers

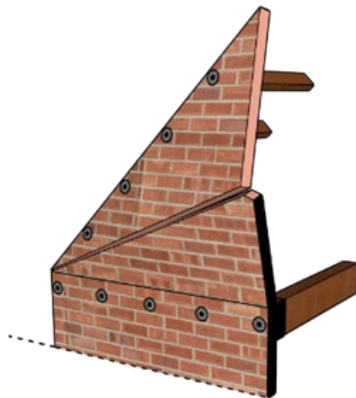
2.2 Case 2: Vertically spanning gables

Vertically spanning gable failure is associated with fully restrained boundary conditions where reliable fixity is provided around the full perimeter of the gable triangle having a symmetrical isosceles geometry (see Figure 3). The mechanism also occurs for the scenario of a sawtooth gable with a free vertical edge and is triggered by discontinuities similar to Case 1 at the base of the gable and by the top edge of the gable being well-connected to the roof.

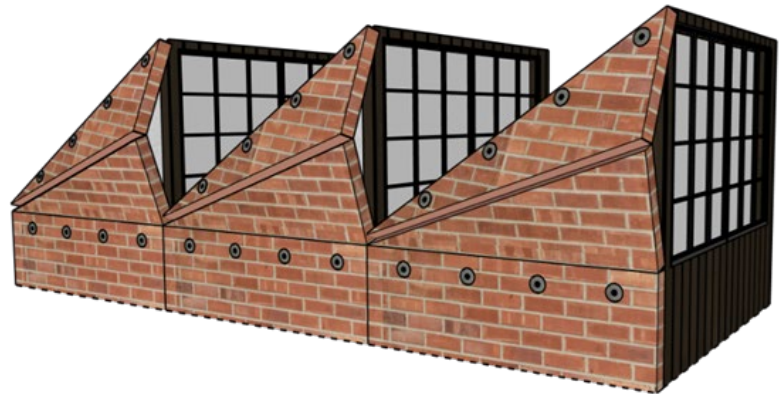
For a half gable with a free vertical edge, a diagonal crack forms to divide the gable into two equal triangular wall panels.

For a full triangular gable, three wall panels are formed with diagonal cracks meeting at the centre of the gable and a vertical crack reaching the apex to form a symmetrical inverted Y mechanism

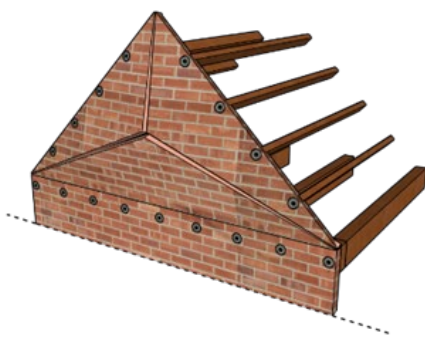
This failure mechanism (and the associated boundary conditions) occurs less often when compared to other gable failure mechanisms. It is noted that this failure mechanism can be difficult to identify from post-earthquake damage observations.



(a) Diagram of half triangular gable



(b) Sawtooth gable with free vertical edge



(c) Diagram of symmetrical isosceles full triangular gable



(d) Experimental results from Correia et al (2018)

Figure 3: Vertically spanning gable failure

2.3 Case 3: Vertical-horizontal spanning gables

Vertical-horizontal spanning gable failure occurs when neither the roof nor the ceiling diaphragm provides suitable fixity, or when the gable does not have a ceiling diaphragm that provides constraint at the base of the gable. In this case the complete gable overturns along with a portion of the wall below, and the base of the failure mechanism has a curved shape. The specific geometry of the failure mechanism is dictated by the distance between the return walls and the wall height below the gable. It is uncommon for the full height of the wall below the gable to fail except for cases where the distance between return walls is several times greater than the height of the wall below the gable or cases where the wall below the gable is highly penetrated with openings. Case 3 is the most complex failure mechanism, as shown in Figure 4, and it was observed moderately often in the Canterbury earthquakes.

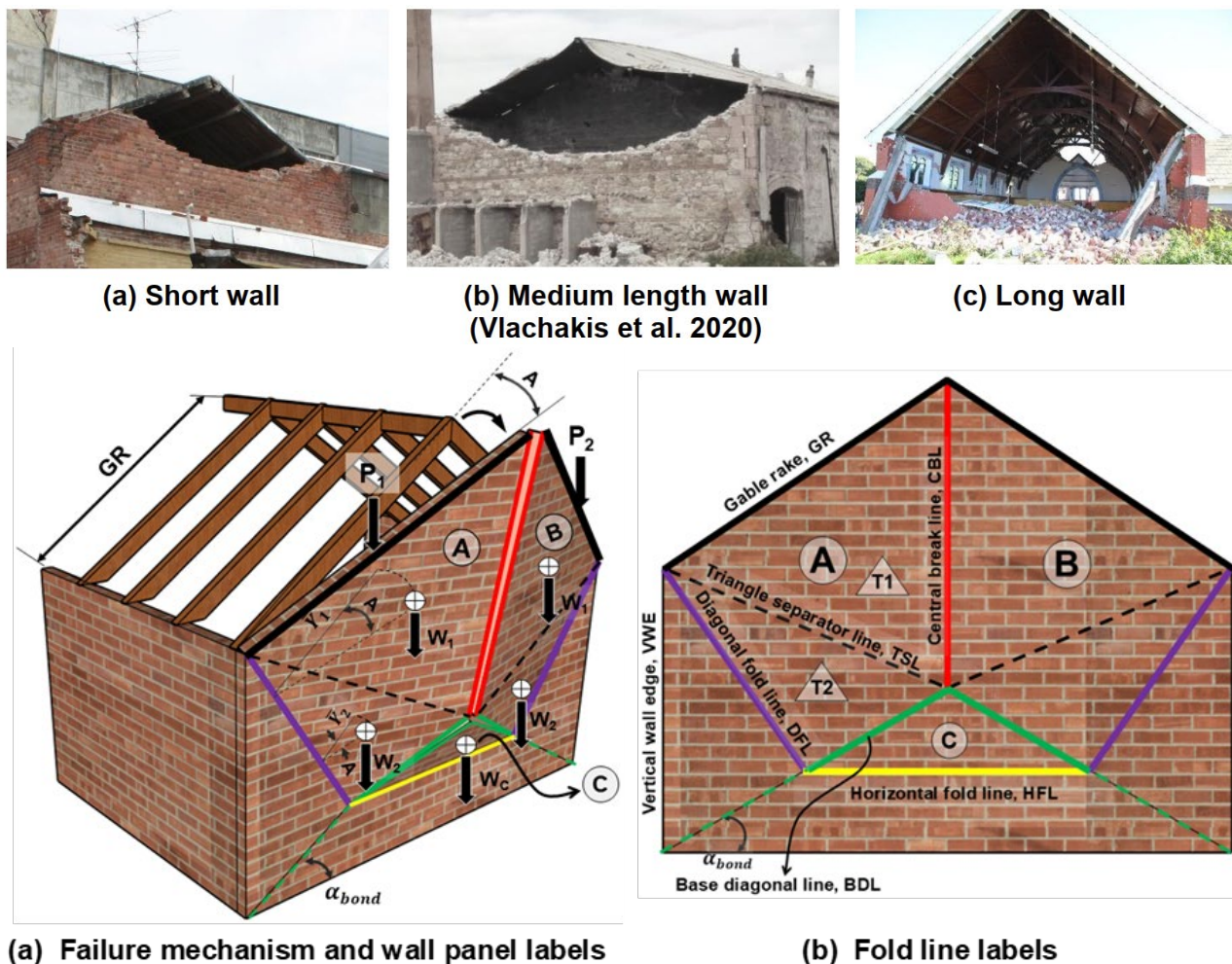


Figure 4: Vertical-horizontal spanning gables

3 SELECTION OF EQUIVALENT FRAME

Expanded advice is given regarding assessment of heavily penetrated walls, and transformation into equivalent frames for piers and spandrels. In unreinforced masonry the beam-column joints can be treated as rigid end offsets, and this has been made explicit.

Guidance has expanded to include treatment of walls which have differing openings, when the piers are relatively weak compared to associated spandrels. In this case, the effective height of piers can be increased beyond simple opening height, to reflect the formation of stepped cracking along mortar joints (therefore doesn't apply if this crack pattern is suppressed due to concrete bond beams/lintels etc.)

The update to C8 gives more guidance regarding pier height for in plane rocking walls, highlighting this pier span is not necessarily the floor to floor height. [Guidance is based on fig C11-1 from ASCE 41-23 (ASCE, 2023).] A further addition explicitly allows for toe crushing to enable a form of limited redistribution of in-plane load between piers of differing length, provided that these piers are adjacent and along the same line.

The figure outlining the generalised force-deformation relationship for rocking walls/piers has been updated to align with the Life Safety performance level from ASCE 41-23 for simplified nonlinear procedures. It is suggested that practices that have master spreadsheets and other similar tools to calculate in plane capacity of walls update these to reflected increase in allowable rocking displacement.

4 CAVITY WALLS

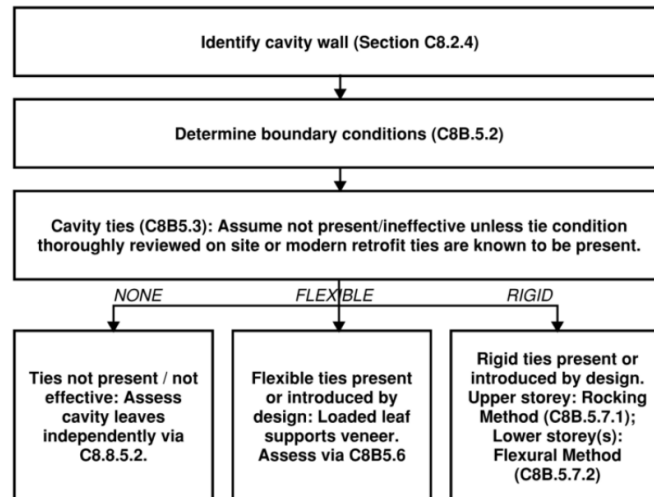


Figure 5: Cavity wall assessment flowchart

New advice regarding the assessment of masonry cavity walls is included in the revised C8. As shown in Figure 5, the revised section provides additional guidance on the boundary conditions present in various configurations of as-built cavity walls. Examples include the common scenario of differences between top load and effective span for inner and outer leaves of a cavity. The revised section proposes three general tie conditions and modes of analysis: *no ties/ineffective ties*, which is assumed to be the default state of most walls; *flexible ties*, which drag demand from a veneer leaf to an inner leaf; and *rigid ties*, with the ability to carry shear and moment.

For the *rigid tie* category, two methods of analysis are provided. For upper storeys, where as-built wall moment capacities are assumed to be minimal due to absence of top load, a “rocking” model is proposed, in which the ties are checked for their ability to lift a rocking wall section, permitting the assessment of the cavity wall as though it were solid.

A cavity wall has lower mass than a solid wall due to the presence of the cavity. The reduction in mass due to the cavity decreases the restoring force and also decreases the demand. The reduction in demand from reduced weight is linear. However, the reduction in restoring force occurs more slowly, as cavity walls have most of their weight at the edges, where the weight is most effective in producing restoring force. The “missing weight” at the cavity, in the middle of the wall, does not provide much of the restoring force. Due to this beneficial relationship between demand and capacity, and provided that the rigid behaviour of the ties has been established, it is appropriate to assess rigidly-tied cavity walls as solid walls. Differences in the wall period for solid vs cavity walls (due to changes in the rotational inertia) are not significant for common wall configurations.

The second *rigid tie* analysis method provided is the “flexural” model. In this model, the flexural capacity of the masonry is considered, along with the capacity of cavity ties. This model is considered more suitable for walls located lower in a subject building, where the presence of top load on the wall section under analysis generates additional moment capacity in the masonry. The “flexural” model is anticipated to be employed using generic structural analysis software which most design offices will already be using.

5 HORIZONTAL WALL SPANS

This section addresses horizontally spanning masonry walls in a limited context. The intent of this section is to provide assistance for engineers assessing load paths involving horizontal spans. Examples of the intended usage include:

- Assessment of wall sections at circulation voids (lifts, stairs, risers etc), where there is no support for walls at floor levels.
- Assessment of wall sections spanning horizontally between piers, pilasters, or buttresses (although in cases where windows are present between piers, engineers will need to use judgement in applying the method).
- Assessment of wall sections spanning horizontally between vertical strongbacks. These strongbacks may be part of an existing seismic improvement scheme within a building that is being re-assessed, or they may be part of a retrofit programme.

The procedure and examples presented in this section assume that wall panel flexure occurs along a vertical line at the midpoint between points of support. The line of flexure is stepped (bricks stronger than mortar). The physical area considered consists of the horizontal and vertical interfaces within the line of flexure, at each course. Wall panel flexural capacity is due to the combination of cohesion between courses of masonry and friction due to top load. These mechanisms provide resistance to rotation of the brick at the interfaces along the line of flexure. The calculated resistance to rotation is derived by analogy to the flexural tension capacity in a reinforced section. The horizontal moment capacity of the wall section derives from the lever arm between a compression surface on the head joint (on the compression side of the wall section), and the resistance to rotation and sliding in the bed joint on the “tension” side of the wall section (refer Figure 6).

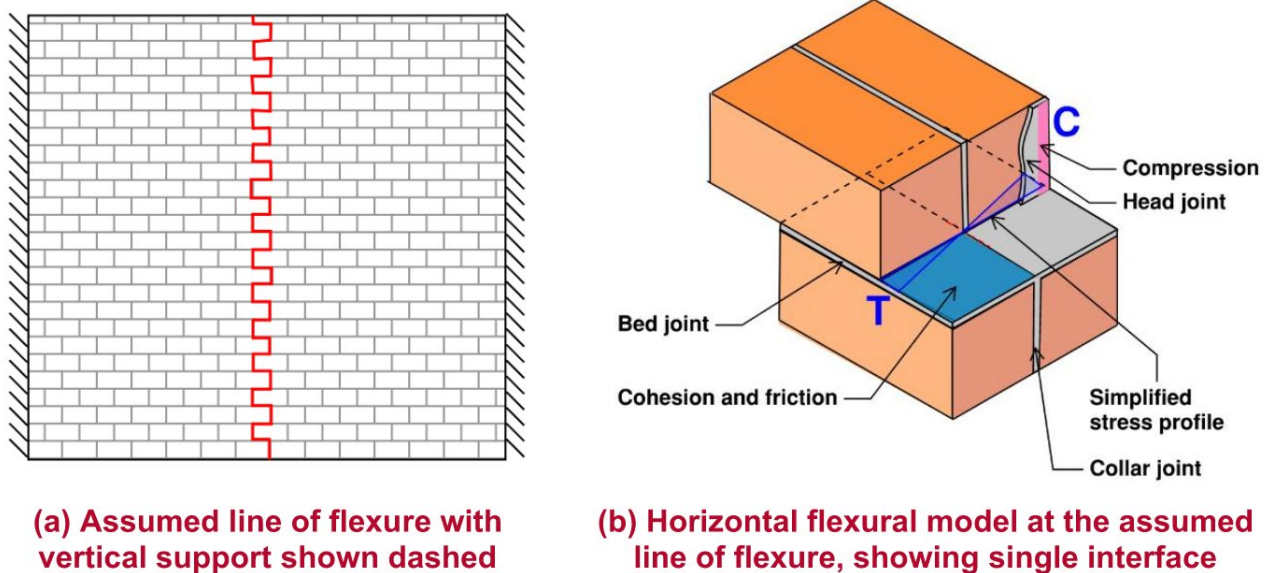


Figure 6: Flexural response assumptions for horizontally spanning walls

6 ERRORS IN PARAPET TABLES IN 2017 EDITION

Appendix C8C contains charts for the assessment of out-of-plane responding walls. It was established that some errors existed in these charts and hence the charts have been updated and redrawn. An example of the new charts is shown in Figure 7.

A statement from the Joint Committee regarding the likely scope and impact of these errors is expect to be issued by Engineering New Zealand in the coming weeks (at time of writing). It is understood that the error

affects only assessments which used the charts to assess the parapets (assessments using the calculation method are unaffected), and only certain ranges of parapet heights would have been affected.

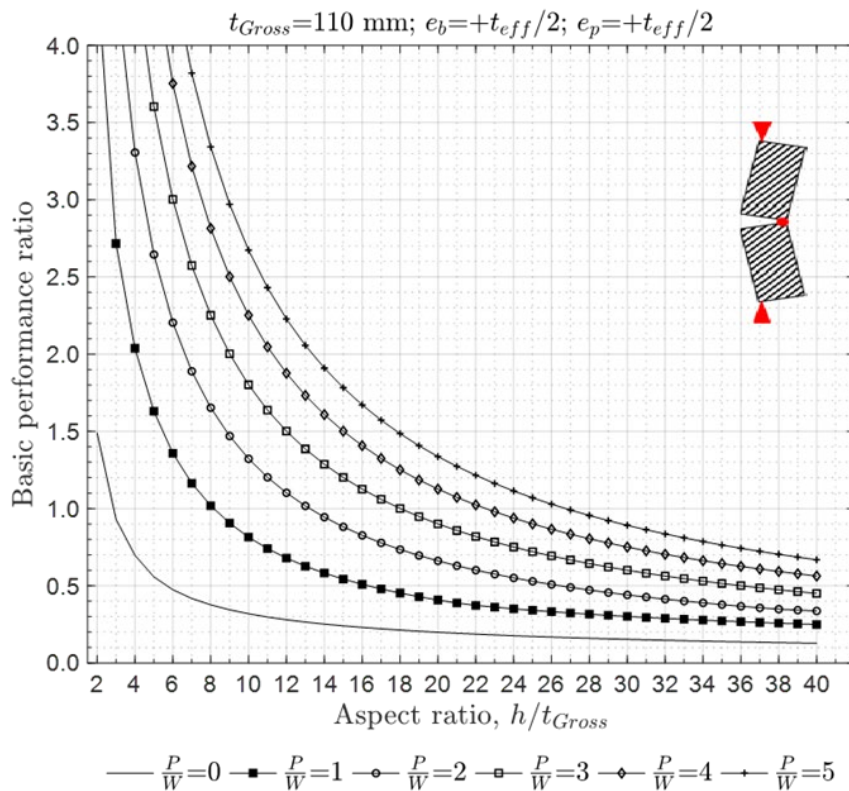


Figure 7: Example of redrawn charts

7 SUMMARY

A 2025 update to Section C8 (Unreinforced Masonry Buildings) has been issued, to collate new knowledge in the field of masonry assessments and to address some of the issues and gaps which engineers have reported with the 2017 edition. This paper summarises the key changes and provides an introduction to the new and expanded methods for assessment provided in the updated edition. The new information presented in the revised edition is expected to assist with the targeted “façade securing” schemes which may become mandatory under proposed new seismic legislation.

REFERENCES

- ASCE. 2023. Seismic evaluation and retrofit of existing buildings. ASCE Standard ASCE/SEI 41-23. Reston, VA: ASCE
- Beca Ltd, (2025). CBA Results for Revised Seismic Risk Mitigation Approach for New Zealand's Earthquake-Prone Buildings. Retrieved 2 February 2026 from: <https://www.mbie.govt.nz/building-and-energy/building/earthquake-prone-building-and-seismic-risk-management-review/proposed-changes-to-the-earthquake-prone-building-epb-system>
- Correia, A. A., Tomassetti, U., Campos Costa, A., Penna, A., Magenes, G., & Graziotti, F. (2018). Collapse shake-table test on a URM-timber roof substructure. In *Proceedings of the 16th European conference on earthquake engineering, 16ECEE* (pp. 18-21).
- NZ Gov't, (2025). *Building (Earthquake-prone Buildings) Amendment Bill*. Retrieved 27 December 2025 from: <https://www.legislation.govt.nz/bill/government/2025/0238/7.0/whole.html#LMS1558674>
- Vlachakis, G., Vlachaki, E., & Lourenço, P. B. (2020). Learning from failure: Damage and failure of masonry structures, after the 2017 Lesvos earthquake (Greece). *Engineering Failure Analysis, 117*, 104803.