A methodology for the seismic assessment of infilled frames with significant vertical discontinuities

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ABSTRACT

Seismic assessment of infilled frame buildings in New Zealand follows Section C7 of the NZ Assessment Guidelines or ASCE41-17 for Tier 3 Nonlinear Dynamic Procedures. The latter approach was applied in the recent assessment of the Wellington Railway Station, one of New Zealand's most complex heritage structures.

The building features numerous configurations of concrete-encased riveted steel frames with masonry infill. Many of these are beyond the direct applicability of prescriptive methods contained in available guidelines, due to significant vertical stiffness discontinuities caused by large openings and deep concrete spandrel beams.

The available guidelines typically point to the adoption of an explicit compression-only strut modelling approach for these non-standard configurations. This approach was found to present several challenges in application.

To address these challenges, a literature review was conducted, and a novel modelling approach was developed. This "Four-Strut" method remains aligned with the intent of ASCE 41-17, while enabling the influence of significant vertical discontinuities to be captured directly in the analysis. Further enhancements were developed to account for the potential for infill to amplify the demands on steel frame connections, including column splices and partially restrained beam connections.

Whilst specifically developed for assessing the varied configurations at the Wellington Railway Station, the outlined method can serve as a reference for assessing other buildings outside existing guideline boundaries.

1 INTRODUCTION

A Detailed Seismic Assessment of the Wellington Railway Station has been undertaken, adopting the 2017 version "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, version 2017" (referred to as the NZ Assessment Guidelines). Considering the complexity of the structural system of the Wellington Railway Station building a Nonlinear Time-History Analysis (NLTHA) had been undertaken using the ASCE 41-17 NDP Tier 3 compliance pathway stipulated in the NZ Assessment Guidelines.

Figure 1: Aerial photograph of the Wellington Railway Station, from the Heritage NZ website

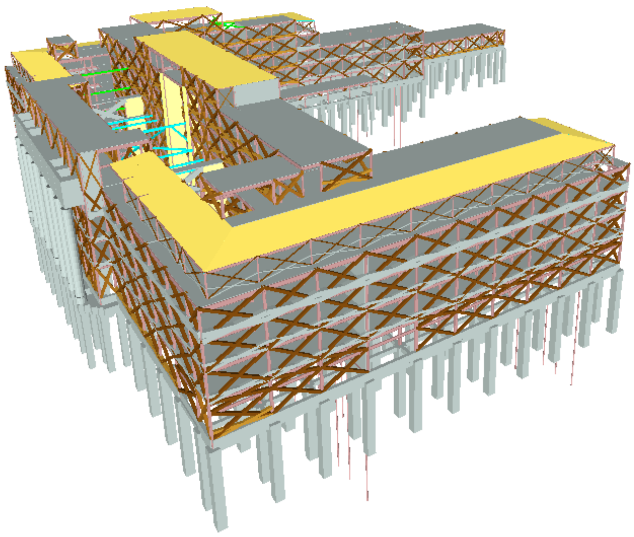


Figure 2: 3D analysis model developed for the DSA

The Wellington Railway Station (Figure 1C) is a heritage listed building, constructed in the 1930's. It comprises a structural steel frame as its primary structural system. The frame has partially restrained (PR) riveted connections in the beam-column joints and is concrete encased, typically with minimal supplementary reinforcement. In between the steel frame are various configurations of masonry infill with a cavity and a masonry veneer that form the facades of the building.

For the development of the 3D analysis model of the building a compression-only strut analogy has been adopted. This analogy is set out to capture the various configurations of the frame system of the building with the various masonry infill configurations.

At the outset of the Wellington Railway Station DSA project, the intention was to adopt the provisions of ASCE41-17 Section 11.4 (Masonry Infills) as the primary compliance pathway, supplemented (as required) with Section C7 of the NZ Assessment Guidelines (NZSEE, 2017). As the analysis progressed it became clear that there are several gaps in applicability of both provisions to the full range of infill frame scenarios present in the building. This prompted a literature review and a series of analytical investigations. Ultimately, a novel modelling approach was developed, capable of representing infill bays with significant vertical stiffness and / or strength discontinuities, referred to herein as the "Four-Strut" implementation.

1.1 Existing infill bay configurations

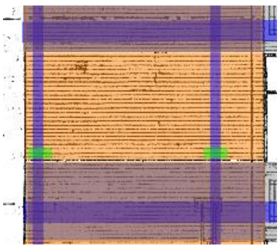
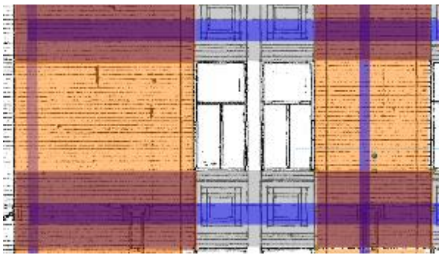
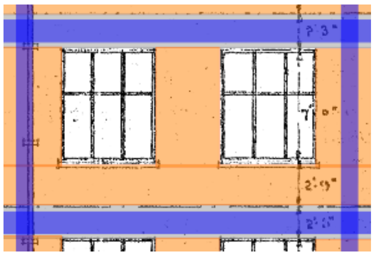
The Wellington Railway Station building encompasses a wide range of masonry infill bay configurations due to variability in:

- Bay geometries, influenced by storey heights, aspect ratios and relative location and proportion of openings.

- Masonry construction, including location of cavity and number of wythes of masonry confined within/floating outside of the bounding steel frame.
- Upstand and downstand spandrel geometry and construction (i.e. masonry and concrete).
- Steel column section geometries, plating, orientation, splice types and locations.
- Partially restrained beam end connection types (approx. 500 different details defined in the drawings, encompassing a variety of configurations).
- Overall horizontal and vertical building irregularities.

Error! Reference source not found. illustrates a selection of the typical masonry infill arrangements present in the building. Each contains a significant vertical discontinuity.

Table 1: Selection of masonry infill bay arrangement present at the Wellington Railway Station building

Solid Infill atop concrete upstand spandrel, column splices (green) above spandrel	**Moderate opening with upstand concrete spandrel**	**Large openings with upstand masonry spandrel**
		

2 EXISTING GUIDELINES – APPLICABILITY AND LIMITATIONS

Several existing guidelines were reviewed to assess their applicability and limitations with respect to the various infill frame scenarios present at the Wellington Railway Station:

- ASCE41-17 Section 11.4 – Masonry Infill
- NZ Assessment Guidelines, Section C7 (2017) – Moment Resisting Frames with Infill Panels
- Loyola (2020) - Seismic Assessment of Infilled Steel Frames
- Stavridis et al. (2017) – Updating the ASCE 41 Provisions for Infilled RC Frames
- Martin & Stavridis (2018) - Evaluation of A Simplified Method for the Estimation of the Lateral Resistance of Infilled RC Frames.
- Schneider et al. (1998): Lateral Strength of Steel Frames with Masonry Infills Having Large Openings

Key findings of this review are described below:

2.1 ASCE41-17 – Section 11.4 – Masonry infills

The ASCE41-17 Section 11.4 provisions encompass several key principles that warrant consideration when assessing the seismic performance of infilled frames:

- Overall infill frame behaviour is a function of the complex interaction of the stiffness and strength characteristics of the infill and surrounding frames.
- Previous research has demonstrated that failure mechanisms of infill bays can be very complex and difficult to predict reliably. Because of this, a modelling approach that represents the envelope behaviour of multiple potential mechanisms / limit states is generally considered acceptable
- For solid infill, or where only small openings exist, a single pair of concentric compression-only struts is an effective analytical approach.
- Where large openings exist, it may be appropriate to use an "explicit" approach, whereby a pair of compression-only struts is used to represent each pier and spandrel element.

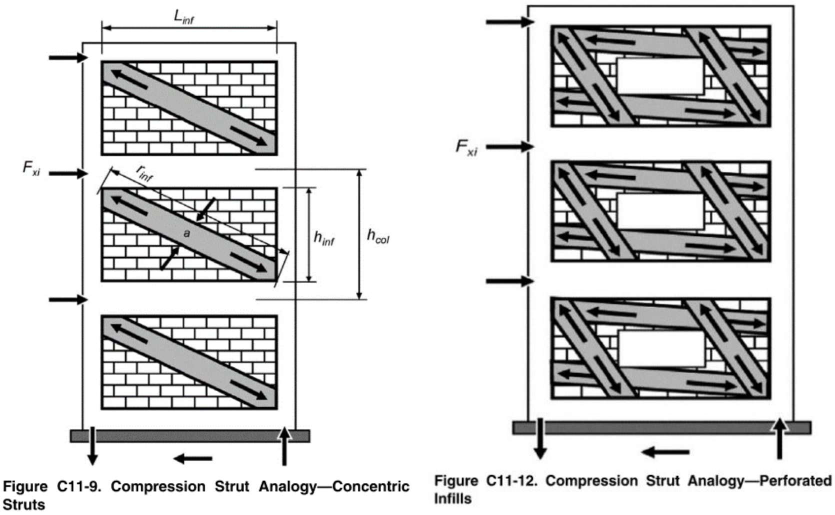


Figure 2: Use of compression struts per ASCE41-17 – for solid panels (left) and panels with large openings (right)

However, the ASCE41-17 approach has several shortcomings when applied to the Wellington Railway Station:

1. It is focused on reinforced concrete infill frames. While there is some commentary around how it can be extended to steel frames, this is more anecdotal and less well calibrated by testing. For example, the classification between strong/weak infill and ductile/non-ductile frames, is somewhat empirical and not directly applicable to steel frames.
2. It is focused on solid infill. Modification to account for small openings is permitted. Stiffness relationships are intended to be applied only to frames with <40% openings and strength relationships where there are <20% openings. Some commentary is provided around perforated infills, but there is little in the way of explicit guidance for infills bays with large openings.
3. It is a prescriptive methodology. Many of its relationships have an empirical basis and combine multiple limit states. There is little guidance on how the underlying principles might be isolated such that they can be leveraged to suit a broader range of scenarios.
4. It does not consider the influence of partially restrained beam connections, nor column splices, both of which are present in the structure and have potential brittle failure mechanisms.
5. It contains limited guidance around calibration of strut properties for use in nonlinear time-history analysis.

2.2 Loyola thesis (2020)

This master's research project was carried out by Loyola under the supervision of Stavridis, one of the leading researchers behind the infill provisions contained in ASCE41-17, also refer the work by Stavridis et al. (2017) below. The project is very much aligned with the ASCE41-17 approach. However, it also identifies shortcomings and makes recommendations for how the ASCE41-17 approach could be better adapted to infilled steel frames.

Some of the key findings and recommendations in Loyola (2020) include:

- The methodology that underpins ASCE41-17 Section 11.4 is inaccurate when applied directly to infilled steel frames.
- It is possible to achieve a better result by recalibrating the methodology against results of infilled steel frames testing programmes.
- Numerous modifications are required, including to the relationships for determining drift at peak strength, drift at residual strength, initial stiffness and yield point and location of plastic hinges in columns.
- Four distinct limit states are considered appropriate for infilled steel frames:
  - Loss of cohesion (at low drifts)
  - Failure of leeward columns (at large drifts)
  - Tension yield of windward column (P_{lim})
  - Corner crushing of struts

These points make Loyola (2020) an attractive basis for the infill implementation for the Wellington Railway Station building. However, it still has several of the same shortcomings as the ASCE41-17 provisions (items 2,3,4, and 5 above).

2.3 Engineering Assessment Guidelines – Section C7 – Moment-resisting frames with masonry infill panels

As with ASCE41-17 and Loyola (2020), Section C7 of the Engineering Assessment Guidelines (2017) is primarily focussed on solid infill panels and advice on modelling large openings is limited. It is also primarily intended to be used with reinforced concrete bounding frames. However, the underlying principles for determining strength and stiffness of individual infill panels are relatively simple and more easily adapted for use with an explicit modelling approach.

2.4 Stavridis et al. (2017)

Stavridis et al. (2017) provides additional support to the notion of using an explicit modelling approach in the case of large openings, (with pairs of struts used to model each individual masonry element (pier and spandrel)). Refer Figure 3 below. Limited commentary is also provided around how strut elements may be noded out eccentrically with either the columns or the beams to more accurately capture local demands in these elements. However no guidance is offered around how such methods may be calibrated in practice, and it appears that detailed finite element modelling may be required for this step, which is impractical for a project the scale and complexity of Wellington Railway Station.

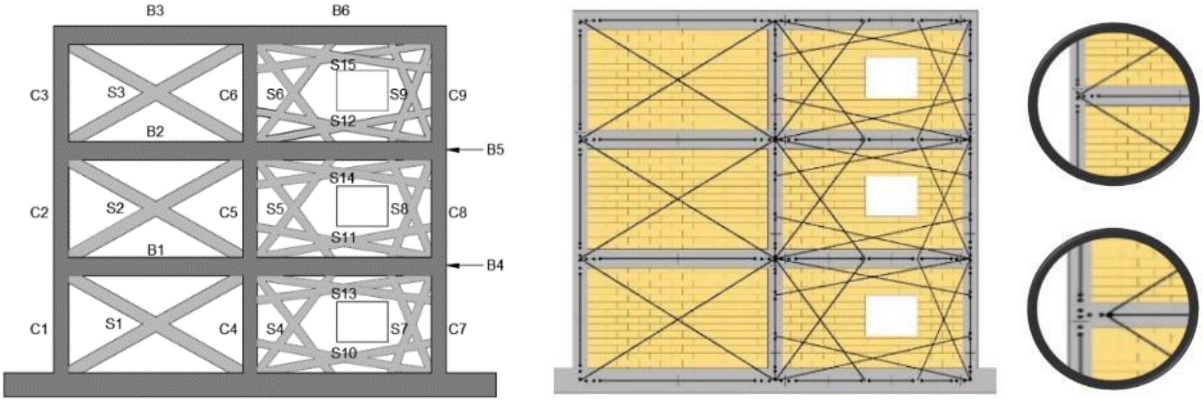


Figure 3: Use of explicit struts, reproduced from Stavridis et al. (2017)

2.5 Schneider et al. 1998

Schneider et al. (1998) provides useful insights for modelling bays with large openings. This paper summarises the results of cyclic testing on several specimens which have characteristics consistent with the Wellington Railway Station building, including steel bounding frames and large openings with relatively slender piers. The paper is particularly relevant where there are slender pier elements adjoining the concrete-encased steel frame at the Wellington Railway Station building (refer Figure 12).

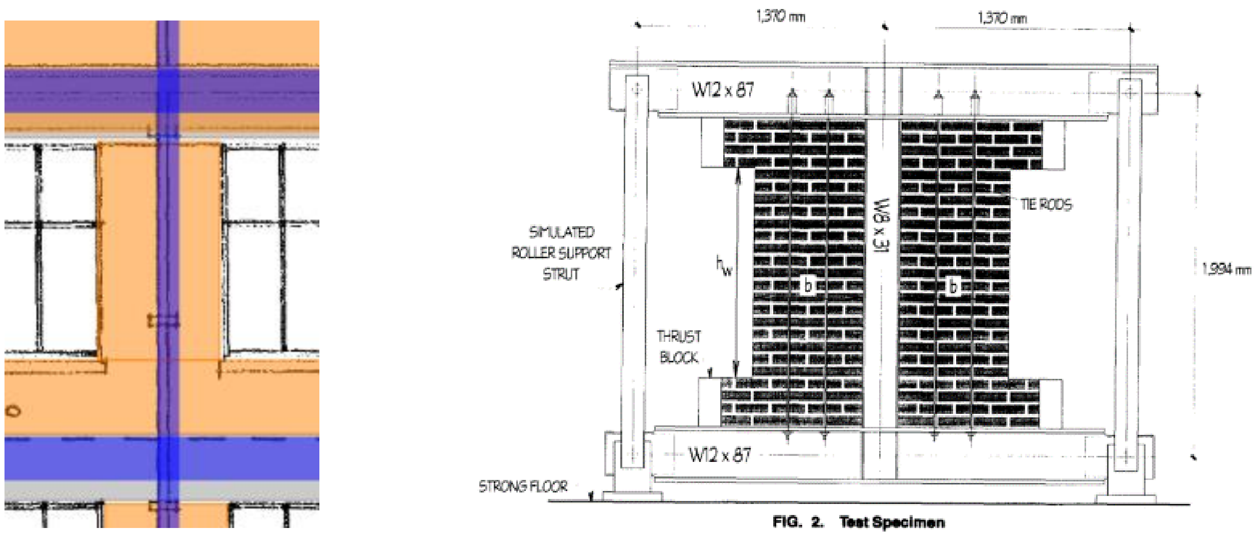


Figure 4: Similarity between Schneider 1998 specimen and slender piers at the Wellington Railway Station

3 EXPLORATIONS INTO EXPLICIT MODELLING

Nonlinear cyclic displacement pushovers were used to assess the potential for an explicit modelling approach to represent configurations beyond the applicability of ASCE41-17 / Loyola (2020) (refer Figure 5).

3.1 Reliability of strut calibration

The explicit modelling approach was found to have some potential to capture anticipated failure mechanisms. However, strut property calibration allowing for multiple limit states was found to be challenging in practice. For a simple concentric strut model, story drift and strut extension can be related using a simple geometric relationship. This makes it possible to achieve an idealised target storey shear force vs. drift backbone for an overall assemblage. For an explicit model however, any flexibility in the bounding frame will tend to relieve the deformation of the strut as drift increases. The direct relationship between drift and strut extension breaks down and may lead to struts not triggering intended failure criteria, even at high levels of drift. Limit states associated with low drift, such as loss of masonry cohesion, were found to be difficult to adequately account for with this approach.

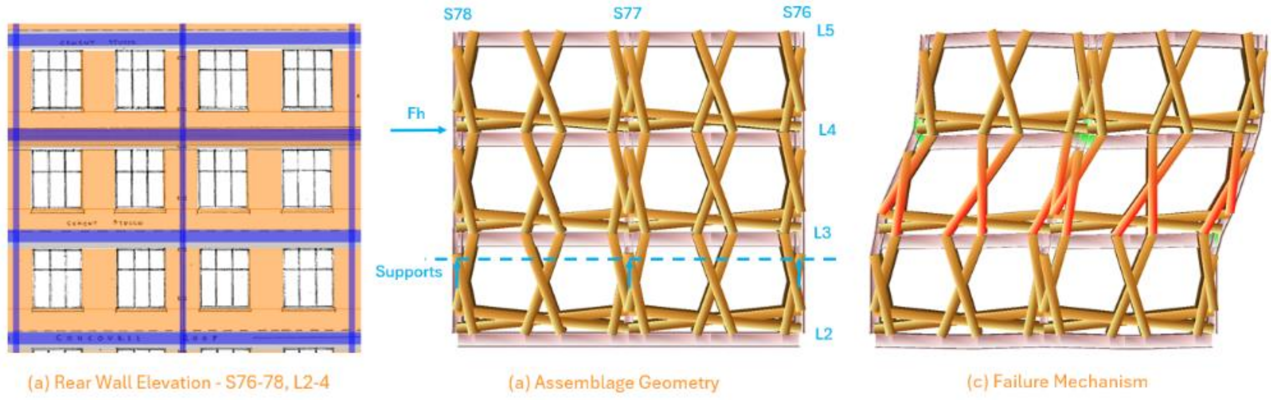


Figure 5: Illustration of explicit modelling approach explored

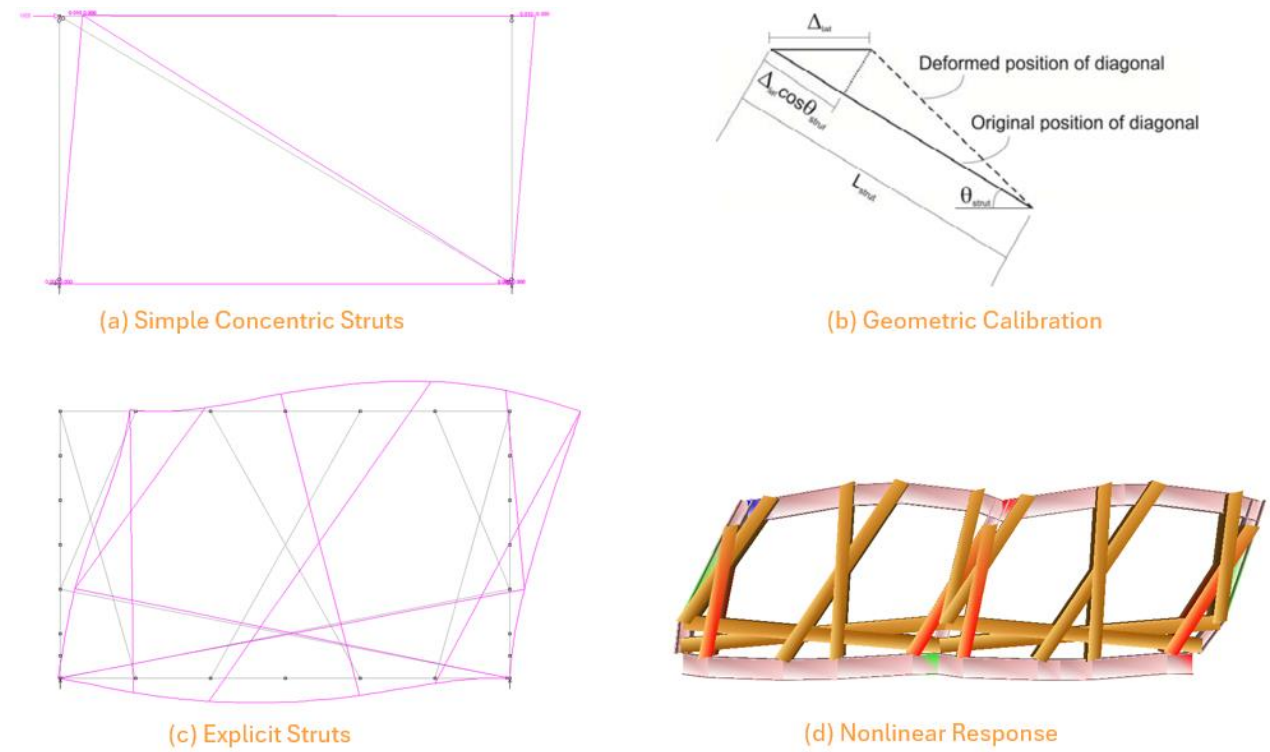


Figure 6: Illustration of challenge of calibrating explicit struts

4 ADOPTED INFILL IMPLEMENTATION

Following the above research, two distinct implementations were adopted to cover the range of infill bays that comprise the Wellington Railway Station building:

1. Conforming Loyola (2020) implementation.
2. Modified Loyola (2020) "Four-Strut" implementation

These are described in more detail in the following sections.

4.1 Conforming Loyola (2020) Implementation

This approach was adopted for bays with a full bounding frame, openings <20%, and no concrete upstand spandrel:

- A single pair of compression-only infill struts for each bay (Refer Figure 7)
- Properties determined using Loyola (2020), augmented by ASCE41-17.
- Struts are noded out concentrically with the steel beam-column joints.

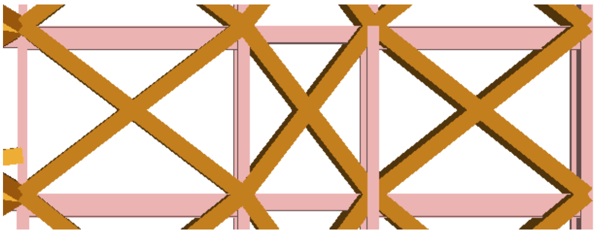


Figure 7: Typical arrangements of Conforming (2020) Infill Bays (<20% openings)

4.2 Modified Loyola (2020) "Four-Strut" Implementation

This approach was adopted for bays with a full bounding frame and significant vertical discontinuity:

- A hybrid approach developed to account for significant vertical discontinuity in the stiffness and/or strength of infill.
- Applied to bays characterised by large window openings (>20%), as well as all bays with a concrete upstand spandrel (with or without openings).
- Up to four compression-only struts. One pair representing the spandrel (if present). The other pair representing the infill above the spandrel. Where openings are present, this second pair of struts represents the masonry piers in aggregate.
- Properties determined using a procedure based on Loyola (2020), but with some modification using first principles as well as calibration with Schneider et al. (1998).
- Pier struts and base of spandrel struts noded out concentrically with columns, to ensure reliable calibration. Pier and spandrel struts act in parallel. Top of spandrel struts join columns at top of physical spandrel, to allow explicitly for potential short column effect and direct assessment of column splice performance (splices are typically located near the top of spandrels).

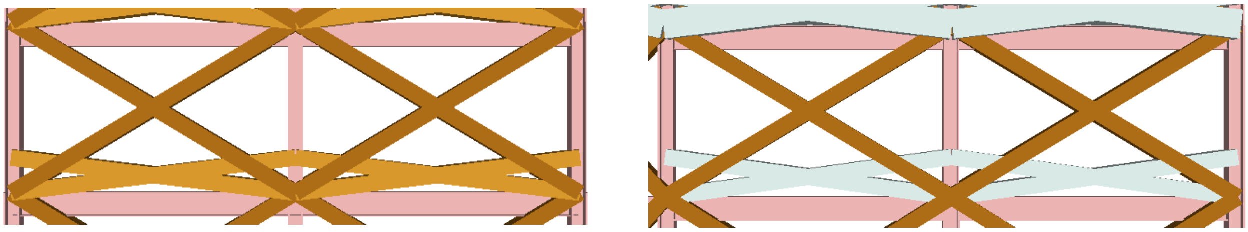


Figure 8: "Four Strut" Implementation, applicable to bays with both masonry and concrete upstand spandrels (hybrid modelling approach)

5 DERIVATION OF STRUT PROPERTIES

5.1 Conforming Loyola (2020) method

Both the above implementations are developed to approximate the quadrilinear Shear-Force – Drift backbone curve adopted by Loyola (2020), as illustrated in Figure 9. This curve is characterised by three key points – Yield (B), Peak Strength (C) and Residual Strength (D).

The general form of the curve used by Loyola (2020) includes the contribution of both the steel frame and the infill. To implement in the analysis model, the key points are used to derive properties and ASCE41-17 backbone parameters for compression-only brace elements representing the infill only.

An elastic-perfectly-plastic relationship is adopted to represent the loading branch (A-B-C). This requires calculation of an extra point (e_{bi}, P_{max}) as indicated in Figure 10. This point is determined using the principle of equal areas. The effective width and cross-sectional area of the struts are adjusted so that P_{max} is achieved at an extension of e_{bi}.

Loyola (2020) uniformly sets the residual drift ($e_{res,bay}$) and residual strength ratio ($P_{res,bay}$) for the overall bay at 1.1% and 0.8, respectively. This appears to be informed by the force-based principal that 'failure' is characterised by a 20% reduction in shear capacity. When using NLTHA, it is appropriate to consider behaviour beyond this limit when assigning modelling parameters and incorporating strength degradation in the model. Therefore a residual drift ($e_{res,strut}$) and residual strength ratio ($P_{res,strut}$) of 2.0% and 0.1 has been adopted as baseline properties for the infill struts. These provide a good match to Loyola (2020)'s overall bay response at drifts <1.1% while also capturing anticipated behaviour at higher drifts, where the infill contribution is likely to be more significantly degraded.

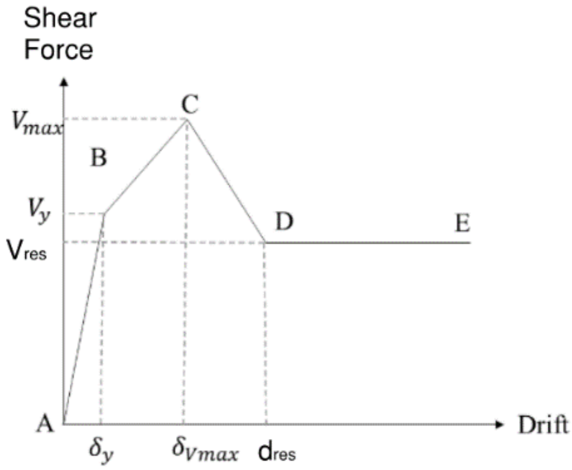


Figure 9: General quadrilinear backbone curve for overall bay of infilled steel frame, per Loyola (2020)

These relationships are compared in Figure 10. Acceptance Criteria and working definitions for the ASCE41-17 'a' and 'b' modelling parameters are also illustrated. Acceptance Criteria are set based on the basic relationships defined in Table 11-13 of ASCE41-17. These limits are converted to equivalent strut extensions and tracked for each brace element, rather than for the overall bay.

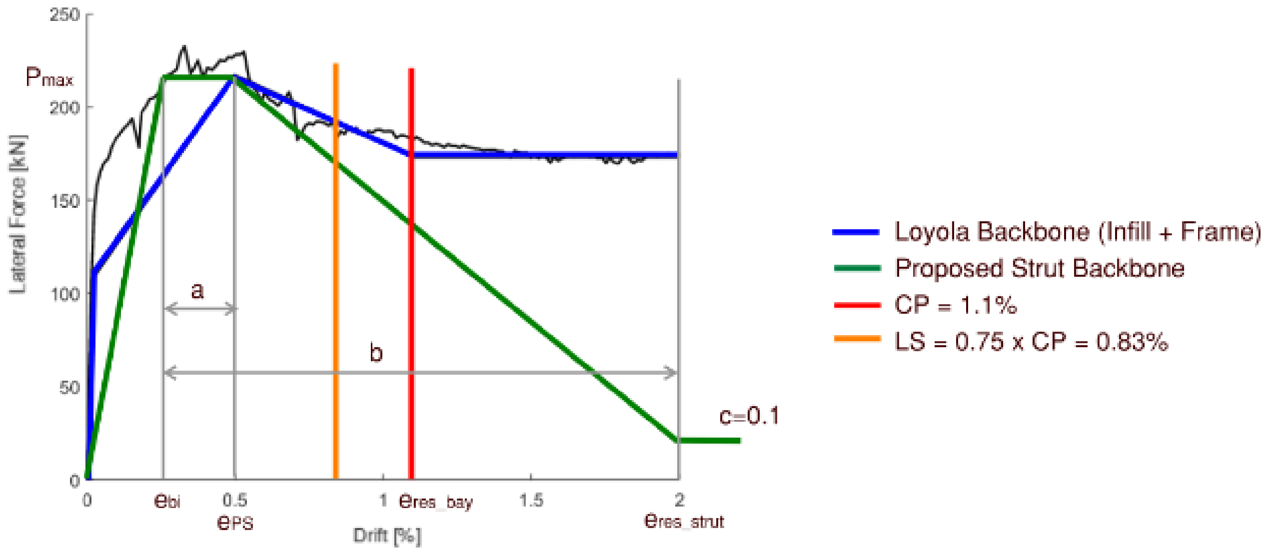
5.2 Modified Loyola (2020) "Four-Strut" method

The Four-Strut method was developed to overcome challenges described in Section 3 related to a full Explicit Implementation. It is a hybrid approach which aligns well with the general principles of Loyola (2020) and ASCE41-17, while also being capable of more directly capturing potentially critical behaviours associated with significant vertical discontinuity in infill strength and stiffness, including column splice failure and short-column effect due to the presence of deep upstand concrete spandrels, and/or large openings.

Properties for the struts are determined using a modified Loyola (2020) approach. First principles are used to modify the relationships used by Loyola (2020) to determine the overall stiffness and account for vertical load effects for the overall bay. Loyola (2020)'s relationships for shear strength are modified and applied independently to the struts representing each of these aspects.

A compression-only strut approach was considered appropriate for the concrete spandrels because the compressive strength of the masonry and concrete are in a similar range, and, at the Wellington Railway Station building, any horizontal reinforcement in the spandrels is understood to terminate at the face of the steel columns. This means the concrete spandrels will tend to behave in a similar way to masonry spandrels – forming a diagonal strut in compression and forming cracks adjacent to the columns in tension. The masonry infill and concrete spandrel behave as a single unit until an initial crack starts to open up at base of the masonry pier or top of the spandrel.

The Four-Strut method was further augmented using insights gleaned from Schneider et. al. (1998). With reference to Figure 11, we have considered the correlation between pier aspect ratio (AR = height / length) and drift at peak strength to account for the tendency of slender piers to reach their peak strength at higher drift levels (approximately 0.5% higher for AR=3.0 as compared with AR=1.5). Similarly, a 0.5% spread was allowed for in CP limits for the same range of aspect ratios (1.25% - 1.75%). This remains consistent with the CP limit previously proposed for the Explicit Implementation (1.5%).



AR-0.5

Figure 10: Generalised strut force vs. extension relationship adopted for Conforming Implementation compression-only struts, overlaid upon Loyola (2020)'s Model 6 results (Specimen 1, Aspect Ratio = 0.5)

Table 2: Summary of strut modelling parameters adopted for Conforming Loyola (2020) and Four-Strut method (interpolated as required)

<table>
<tr><th rowspan="2">Implementation method</th><th colspan="4">Modelling parameters</th><th colspan="2">Acceptance criteria</th></tr>
<tr><th>AR</th><th>Δ_{peak}</th><th>Δ_{res}</th><th>V_{res}</th><th>LS (0.75·CP)</th><th>CP</th></tr>
<tr><td>Conforming Loyola (2020)</td><td rowspan="3">All</td><td rowspan="3">For AR ≥ 0.80 $\Delta_{peak} = 0.80\%$
For AR < 0.80 $\Delta_{peak} = AR$</td><td rowspan="5">2.0%</td><td rowspan="5">0.1</td><td rowspan="3">0.83%</td><td rowspan="3">1.10%</td></tr>
<tr><td>Four-Strut: Spandrel</td></tr>
<tr><td>Four-Strut: Solid masonry above concrete upstand spandrel</td></tr>
<tr><td rowspan="2">Four-Strut: Masonry piers</td><td><1.5</td><td>0.8%</td><td>0.94%</td><td>1.25%</td></tr>
<tr><td>>3.0</td><td>1.25%</td><td>1.31%</td><td>1.75%</td></tr>
</table>

Table 2 summarises the adopted relationships for both the Four-Strut and Conforming Loyola (2020) implementations. For convenience, properties are assigned to piers struts based on the average aspect ratio of all the piers in a given bay. Linear interpolation is used for aspect ratios between 1.5 and 3.0. Struts representing spandrels and squat piers (AR < 1.5%) are assigned parameters consistent with the Conforming Loyola (202) approach.

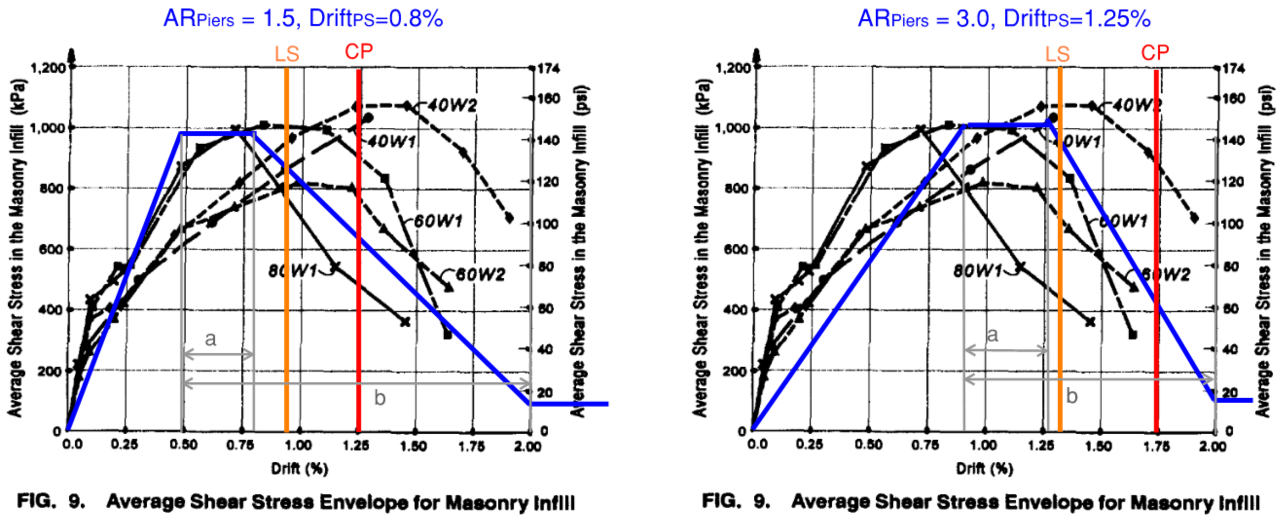


Figure 11: Adoption of parameters for masonry piers, from Schnieder et al (1998)

6 VERIFICATION OF MODELLING PARAMETERS

To develop confidence in the proposed infill implementations, they were tested using a series of representative assemblage models. The models were subjected to cyclic displacement pushover analysis and comparisons drawn between the hysteretic response of the overall model and individual elements. The cyclic displacement pushover trace is applied at the top of the level of interest, and vertical pin supports are provided at the bottom of this level to isolate the behaviour of a single storey. However, the models also include the structure for the levels above and below to ensure a realistic degree of constraint is provided.

6.1 Four-Strut method example - Bunny Street corner façade

One of the evaluated assemblages comprises three adjacent bays of the Wellington Railway Station building, between the second-floor level and the third-floor level (refer Figure 12). Two of the bays have large openings. Because concrete upstand spandrels are present in all bays, the Four-Strut implementation is adopted in all cases.

The response of two versions of the model is compared – one with infill struts between the second-floor level and the third-floor level, and another with those struts removed. This provides a comparison of the contribution of the whole bay with that of the bare steel frame. The assemblages are subjected to a single pushover trace with a peak drift of 4.0%.

Figure 12: Assemblage model of the Wellington Railway Station

Nonlinear backbones were also derived for the concrete struts. However, for this specific assembly, the spandrel struts were found to remain elastic in all cases as the load they experience is limited by yield of the steel columns and pier struts.

6.2 Assemblage overall response

Figure 13 illustrates the applied cyclic displacement pushover traces as well as the deformed shape and element performance at three key points. The first is just after first yield of all the pier struts in the positive drift direction (T=1.52s, 0.8% drift). The second is just after all the pier struts have exceeded their CP limits (T=3.9s, 1.33% drift) and the third is after the steel frame first begins to yield, with the columns hinging above the concrete spandrels (T=5.32s, 2.45% drift).

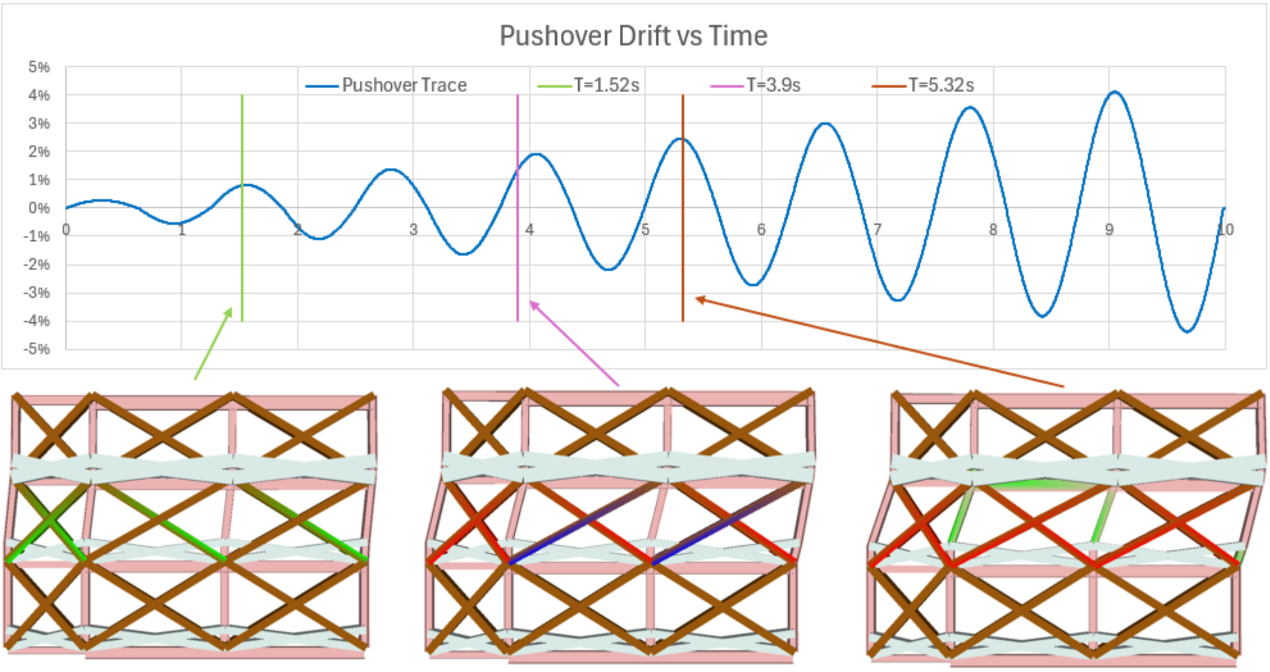


Figure 13: Assemblage model input - Applied cyclic displacement pushover trace, deformed shapes and element performance at key points

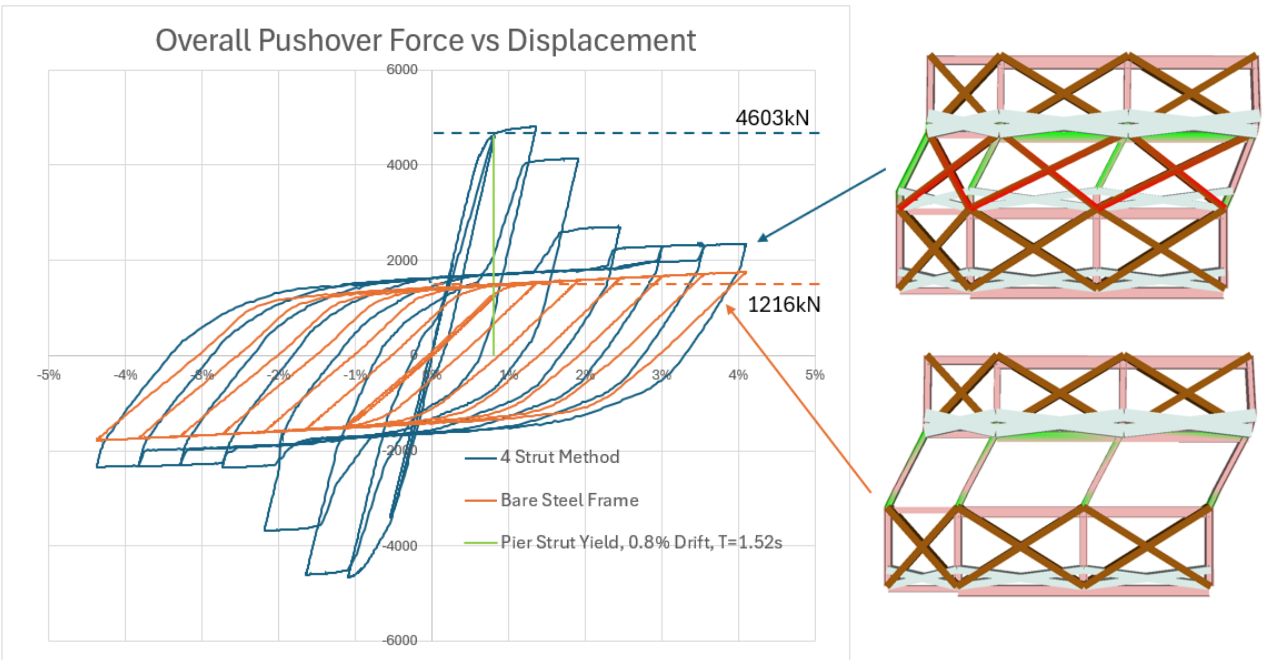


Figure 14: Assemblage model results - Overall pushover force vs. displacement

Figure 14 illustrates the overall pushover force vs. drift response of both the infilled and bare steel assemblage versions. The deformed shapes and element performance at the peak applied drifts is also provided. In these images, green indicates element yield, blue indicates exceedance of LS and red indicates exceedance of CP performance criteria.

The pushover trace confirms that the peak shear strength of the overall bay is developed at approximately 0.8% drift and the contribution of the masonry to shear resistance has degraded to its residual capacity (10% of peak) just beyond 2.0% drift.

6.3 Detailed response of infill struts

Figure 15 illustrates the element force vs extension hysteresis for the pier infill struts active under positive applied drift. Also plotted are the target quadrilinear backbones, the LS and CP performance criteria and the stress state just after first yield of the pier struts (T=1.52s, 0.8% drift). These show good agreement with the target behaviours.

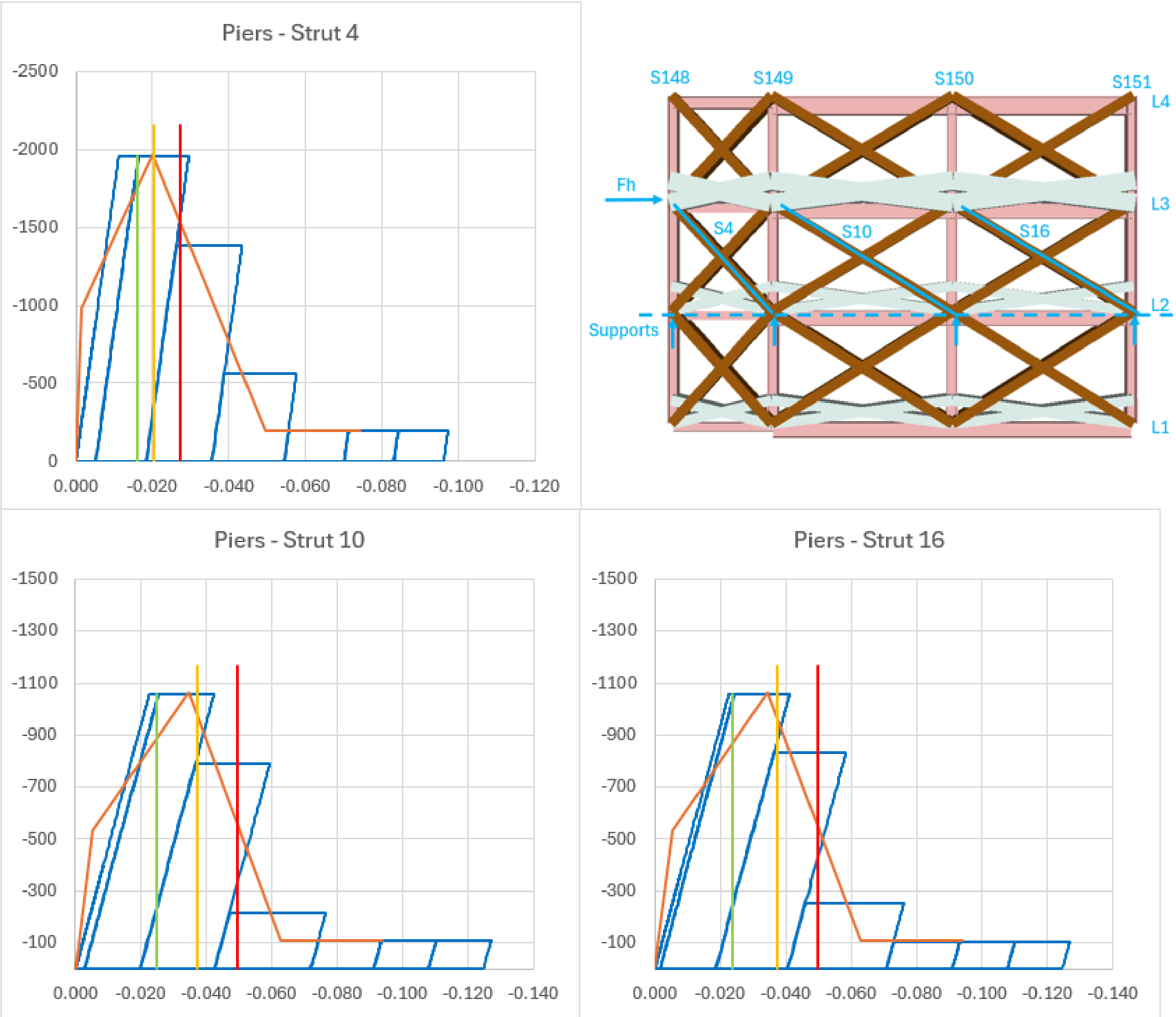


Figure 15: Assemblage 2 - Force vs. Extension hysteresis in active pier strut elements under 0.8% positive drift, T=1.52s (green line in Figure 13).

7 SECONDARY EFFECTS ON PERIMETER FRAME

While developing the Infill Strut implementation, it was identified that the Loyola (2020) procedures may not adequately account for the potential for strut migration to amplify shear and rotation demands on beam partially restrained connections. Because of this, a method for assessing this was developed, based on observation of results presented in Loyola (2020) and derived from explorations into explicit strut modelling. The rationale is as follows (refer also Figure 16):

- At low drifts (<0.5%), there is a tendency for columns to bear against the infill.
- At high drifts (1.5-2.0%) there is potential for the beam to be forced into a single curvature hogging shape, rather than the usual double curvature expected in a moment frame. This can be considered to be caused by the rotation of a triangular rigid body of infill at the leeward column (shaded in Figure 16**Error! Reference source not found.**). The result is zero rotation / moment at the leeward end of the beam, and the formation of a mechanism at the windward end, with hinges at the windward partially restrained (riveted) connection and at the midspan of the beam. As a result, beam plastic rotations may be underreported in the analysis by a factor of approximately 2.0.
- In the transition between the above two states, there is potential for the vertical component of a migrating strut to exceed the shear capacity of the beam / end connection. **To account for this, the ability of beam end connections needs to be reviewed to either resist this component elastically or respond to shear yield in a ductile manner.** Note that this is expected to be a transitory state which will tend to be self-regulating, in that the occurrence of yield will tend to relieve the build-up of stress in the masonry and cause the strut to migrate further.

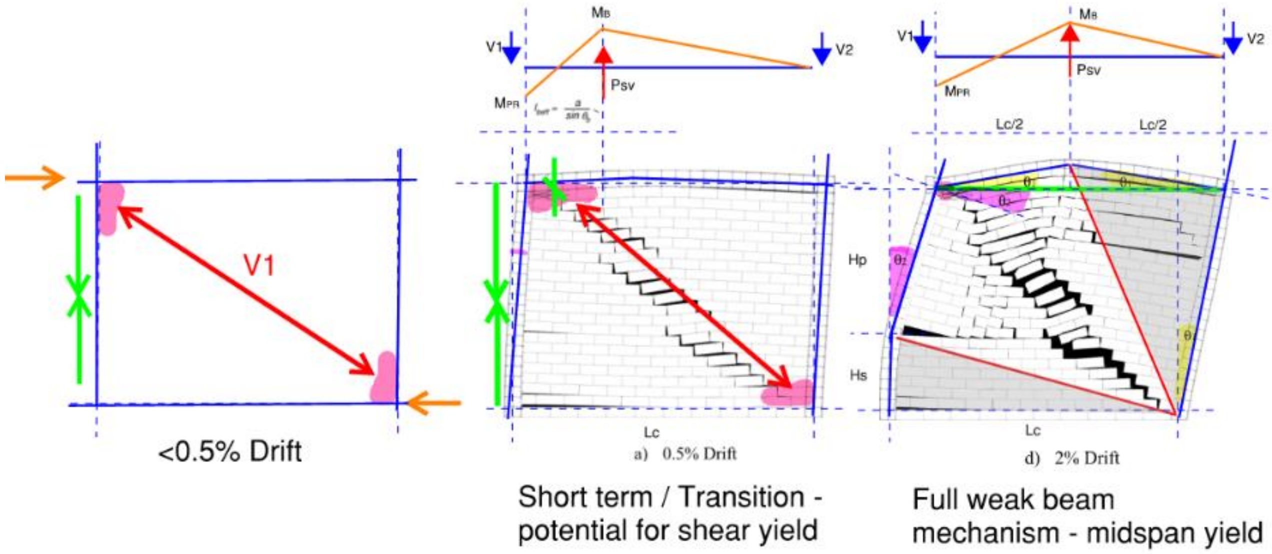


Figure 16: Potential for infill strut migration to amplify beam moments, damage pattern from Loyola (2020) Specimen 1 results.

A number of other complexities were accounted for in the modelling but are beyond the scope of the present paper.

8 CONCLUSIONS

The NLTHA method was used to carry out a DSA of the Wellington Railway Station. The building features numerous configurations of concrete-encased riveted steel frames with masonry infill. Many of these are beyond the direct applicability of prescriptive methods contained in available guidelines and standards, due to significant vertical stiffness / strength discontinuities caused by large openings and deep concrete spandrel beams.

A hybrid “Four-Strut” modelling approach was developed to address these limitations. Further enhancements were developed to account for the potential for infill to amplify the demands on steel frame connections, including column splices and partially restrained beam connections.

This “Four-Strut” method remains aligned with the intent of ASCE 41-17 Section 11.4 Masonry Infill guidelines while addressing several gaps in applicability. We therefore recommend it be considered in future development of ASCE41-17 and the NZ Assessment Guidelines. The reframing of existing standards and guidelines to be less prescriptive and provide a more isolated consideration of component Limit States may also improve their adaptability to a wider range of real-world scenarios. The proposed methodology would also benefit from calibration with an experimental testing programme targeting infilled steel frames with large openings and partially restrained beam connections.

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