
Challenges and complexities in seismic assessment and strengthening: a 26-storey building case study in Wellington, New Zealand

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ABSTRACT

This paper presents a case-study seismic assessment and strengthening of a 26-storey reinforced concrete mixed-use building in Wellington, addressing challenges faced as well as identifying areas where clearer guidance in New Zealand Seismic Assessment Guidelines would be valuable. Between 2017 and 2025, multiple Detailed Seismic Assessments (DSAs) and retrofit strategies were undertaken. Ultimately Nonlinear Response History Analysis (NLRHA) was employed to provide better understanding of floor drifts and accelerations. Floor drifts obtained from NLRHA were validated using alternative Modal Response Spectrum Analysis (MRSA), in which effective stiffness properties at maximum displacement demand were adopted instead of commonly used elastic cracked properties. Conventional methods for estimating Peak Floor Accelerations (PFA) were also evaluated against NLRHA results and were observed to underestimate accelerations in the direction affected by vertical irregularity. This observation acknowledges potential limitations in the application of these methods and suggests that clearer guidance on their applicability may be beneficial for structures of similar type. The paper further discusses practical challenges and ambiguities in precast floor assessment, particularly regarding unit classification and governing drift direction. Multiple diaphragm strengthening solutions as well as different retrofit measures for precast floors are presented providing practical examples for the wider industry. Overall, the paper attempts to share key technical insights, lessons learned and challenges faced to improve industry understanding and assist the ongoing development of seismic assessment and retrofit practices in New Zealand, beyond the case study itself.

1 INTRODUCTION

1.1 Building Description

As shown in Figure 1a, the building comprises 26 stories generally divided into two zones: Levels 1–11 for carpark floors, and Levels 12–25 for offices. The plan includes a main tower and an adjacent service core incorporating lifts and stairs. Lateral resistance in the north–south direction is provided by reinforced concrete moment-resisting frames (MRFs) at both ends. In the east–west direction, shear walls resist lateral loads below

Level 12, transitioning to MRFs above. The interior gravity system consists of four columns with connecting beams.

Floors are 300mm Hollowcore (HC) with 65mm in-situ concrete topping and non-ductile mesh, acting as diaphragms transferring lateral loads. The lift and stair core is laterally cantilevered around the main tower. Foundations comprise belled concrete piles (10–15 m) founded in rock, tied by a 300 mm slab-on-grade diaphragm.

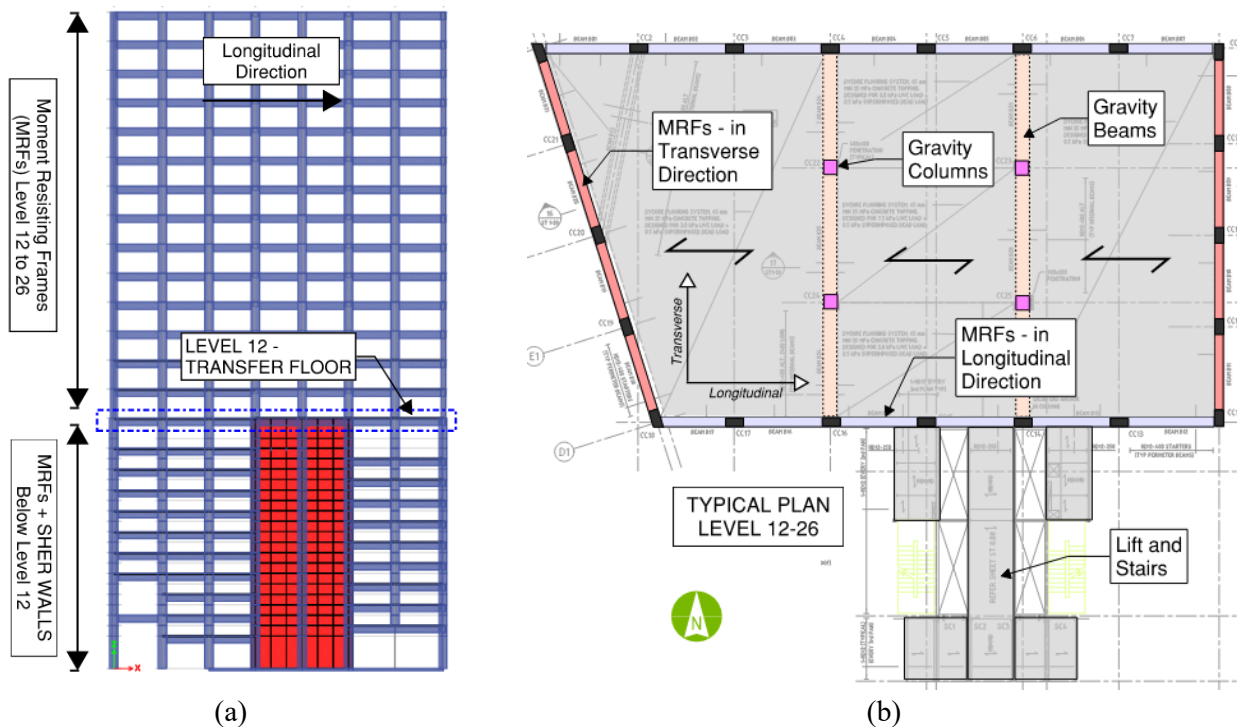


Figure 1: Building Description: (a) Elevation view (North and South frames) and (b) Plan view

1.2 Key technical challenges

1.2.1 Diaphragm assessment and strengthening

It was found that conventional code-prescribed methods for prediction of floor acceleration, such as pESA (NZS1170.5, 2004) or alternative methods referenced in ASCE 7-22 (American Society of Civil Engineers, 2022) may fail to capture this abrupt change. In case of the building, application of these methods underestimated floor accelerations near Level 12 leading to unconservative diaphragm force estimates while overestimating the accelerations for top floors. In this manner, Section 3 provides more detailed discussions.

Diaphragm behaviour is influenced by both vertical and horizontal irregularities and by openings such as stairs, which disrupt direct load paths and introduce discontinuities. At Level 12, specifically, the system transitions abruptly from shear walls to MRFs above, creating a pronounced vertical irregularity. Storey shear at this level must be reliably transferred to the walls below, requiring careful evaluation of the load path. Further discussion of these issues, along with the associated strengthening measures for both typical and transfer diaphragms, is provided in Section 4.1 and Section 4.2.

According to recent updates in (NZ Guidelines Part C2, 2025), Clause C2E.4, for ductile structures and for diaphragm action, earthquake forces should be assumed to act along any axis, and all lateral load-resisting systems are expected to develop their full overstrength capacity. Section 4.3 provide detailed discussions around different methodology for implementing this requirement and associated implications.

1.2.2 Assessment and strengthening of precast floors

The building contains almost 1,000 HC units, with each typical floor comprising 40–50 units. The assessment and strengthening of these units presented multiple challenges. Multiple failure modes—including Negative Moment Failure (NMF), Loss of Seating (LOS), and Positive Moment Failure (PMF)—were identified across the building. In addition, due to the spatial configuration of the HC units, all unit classifications (Alpha, Beta, and ordinary) were present.

The classification of HC units and the correlation between drift in principal directions and its influence on different failure modes are not explicitly defined in existing guidance. While it is acknowledged that substantial progress has been made in recent years through updates to precast floor assessment methodologies (Brooke et al., 2019, 2022; Buker et al., 2022), some aspects would benefit from further clarification. In the case-study building and given significant number of units, it was observed that relatively minor variations in assessment approach could lead to notable differences in strengthening scope, with corresponding implications for cost and constructability. Ambiguities in the assessment approach are discussed further in Section 3.3, while the strengthening measures adopted for the various failure modes are presented in Section 4.4 and Section 4.5.

2 SEISMIC ASSESSMENT METHOD

2.1 Nonlinear Response History Analysis

Nonlinear Response History Analysis (NLRHA) was performed following ASCE 7-22 (American Society of Civil Engineers, 2022) and the latest SESOC guideline (Structural Engineering Society New Zealand, 2024). The target response spectrum was developed using NZS 1170.5 (NZS1170.5, 2004) Site Class B and RotD50 adjustment. A suite of 11 ground motions representing crustal and subduction events was selected and scaled so that the mean RotD50 spectra matched the target spectrum over the building's period range of interest. Given proximity of the site to adjacent faults, almost half of the earthquake records were selected to have forward directivity pulse. The results of NLRHA will be discussed in the subsequent sections.

3 SEISMIC ASSESSMENT OUTCOME

3.1 Building drift profile

Figure 2 presents the mean storey drifts for all earthquake records in both principal directions. NLRHA results were compared with conventional Modal Response Spectrum Analysis (MRSA) and the Effective MRSA method proposed by Priestley (Priestley MJN et al., 2007). Conventional MRSA generally agrees with NLRHA, but noticeable discrepancies occur at mid-height levels in the longitudinal direction (Figure 2a), due to vertical irregularity: upper stories experience significant inelastic behaviour, while lower levels remain largely elastic owing to the reinforced concrete shear walls.

In contrast, Effective MRSA incorporates reduced stiffness corresponding to the maximum displacement demand, accounting for inelastic behaviour and yielding mechanisms along the building height. This method aligns more closely with NLRHA, providing a more accurate representation of vertical irregularity effects on seismic response. Notably, applying the drift modification factor K_{dm} (NZS1170.5, 2004) produces conservative estimates, slightly overpredicting drifts.

3.2 Floor accelerations

Figure 3 presents calibrated PFAs from different analysis approaches for both principal directions: NLRHA, pESA (NZS1170.5, 2004) and ASCE 7-22 alternative method (American Society of Civil Engineers, 2022).

In the transverse direction (Figure 3b), traditional methods reasonably captured floor accelerations. The ASCE 7-22 method shows decreasing accelerations with height, reflecting greater flexibility of upper floors while slightly amplifying the top 20% of stories. The pESA method generally provides an upper-bound estimate, equating accelerations to the PGA for most floors; however, it underestimated the top two stories. This observation is attributed to the building's long fundamental period in the transverse direction ($T \approx 4.3$ s), where upper-floor accelerations are governed by the PGA rather than ESA (Equivalent Static Analysis), contrary to the standard pESA formulation. We note that this aspect of pESA might need further evaluation as the long-period structures are more prone to experience whip effect at top floors, amplifying the floor acceleration.

In the longitudinal direction (Figure 3a), both pESA and ASCE 7-22 significantly misestimate PFAs, underestimating mid-height accelerations by up to 40% while overestimating upper-floor responses by a similar margin. These discrepancies are primarily due to vertical irregularity and abrupt stiffness transitions, which simplified methods fail to capture.

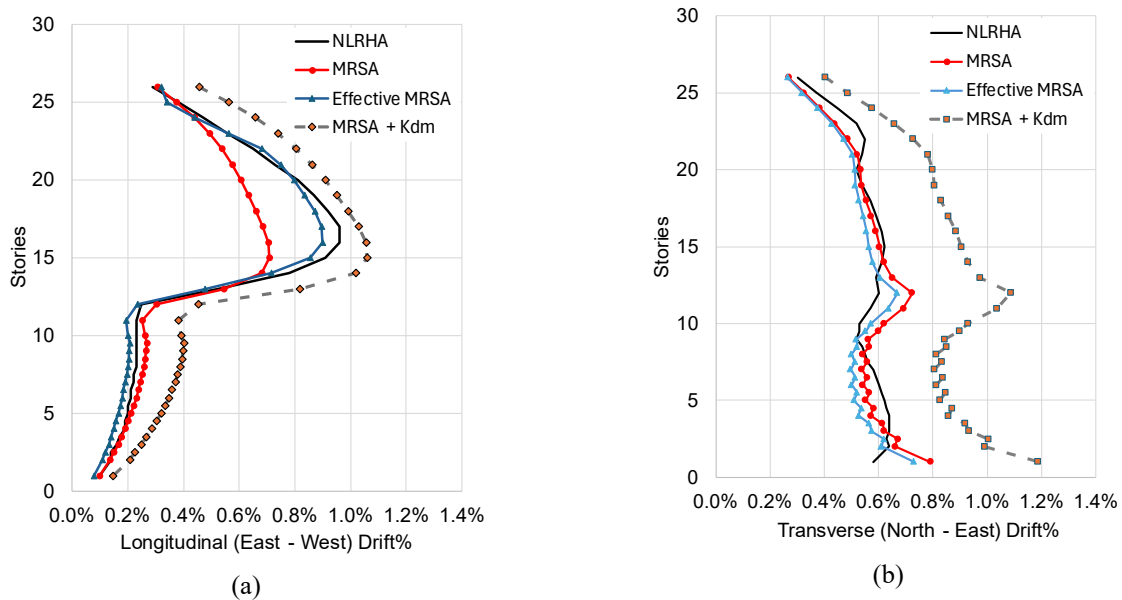


Figure 2: Drift plot from NLRHA and MRSA analysis: (a) longitudinal (East – West) direction and (b) transverse (North – South) direction

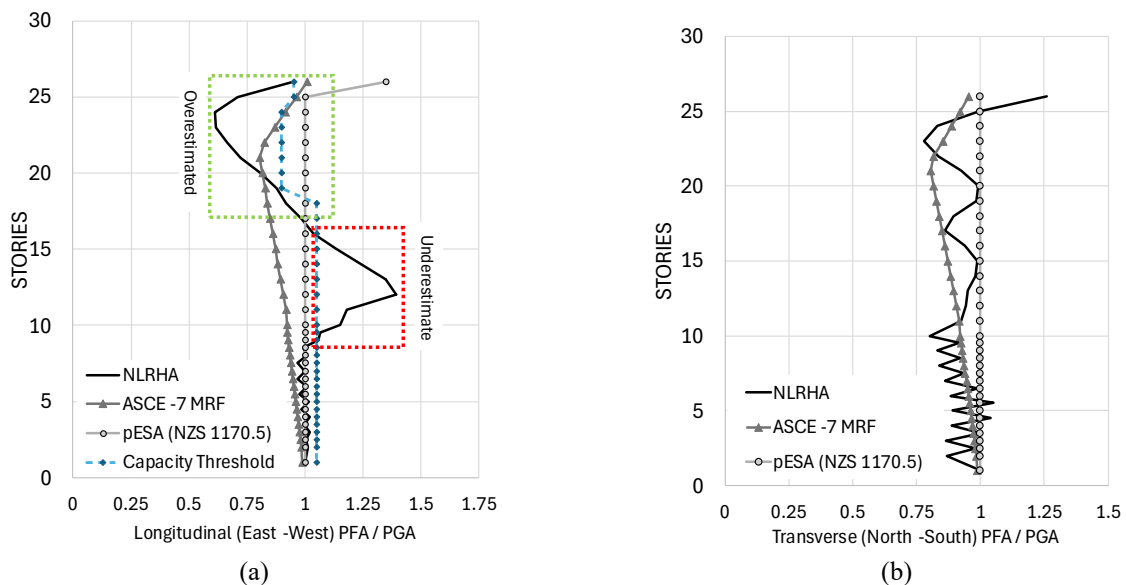


Figure 3: Overstrength peak floor acceleration plot from NLRHA, ASCE 7-22 and pESA methods: (a) longitudinal (East – West) direction and (b) transverse (North – South) direction

These discrepancies can have significant implications for both structural resilience and project cost, as they may result in strengthening the wrong floors and applying strengthening measures based on inaccurate demand estimates. As shown in Figure 3a, Both pESA and ASCE methods would incorrectly exempt mid-height floors (Floors 9–16) from strengthening based on the calibrated capacity curve while over penalizing the upper floors (Floors 17–26) with sufficient capacity.

3.3 Hollowcore (HC) floor units

Ambiguities were identified in the assessment of HC floor units, primarily relating to unit classification and governing drift direction. For example, the HC unit shown in Figure 4a could be classified as a Beta (Category 2) at end (a), ordinary unit (Category 3) at end (b) or, in short-span cases, even as Category 1 due to torsional susceptibility from incompatible end deformations (NZ Guidelines Part C5, 2025). Current guidelines do not clearly address whether different classifications may apply to each end of a unit. For this case-study building, a conservative approach was adopted by classifying each unit based on its most critical end condition.

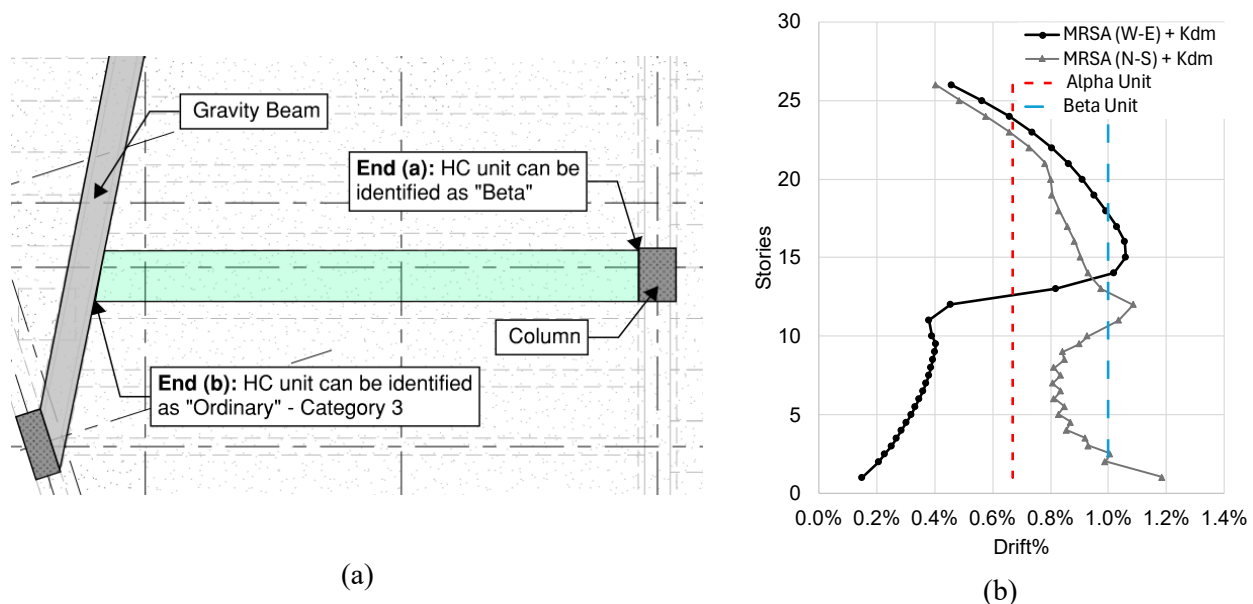


Figure 4: Discussions on ambiguities in HC unit assessment (a) HC unit with different end situation and (b) MRSA including K_{dm} drifts vs Alpha & Beta failure drift

Further ambiguity exists regarding the direction of inter-storey drift governing damage, particularly for positive moment failure (PMF). Existing guidance (refer to appendix C5E (NZ Guidelines Part C5, 2025)) does not clearly indicate whether Alpha and Beta units are more sensitive to drift parallel or perpendicular to their span. These uncertainties can result in significant cost implications by unnecessary strengthening or reduced resilience if it is misinterpreted.

For the sake of argument, consider Figure 4b, which compares MRSA (conservative case with K_{dm}) drifts result with the drift limits for Alpha and Beta units (NZ Guidelines Part C5, 2025). If the maximum drift from both principal directions is used, nearly all floors would require Alpha unit strengthening, whereas considering only the longitudinal direction (parallel to the HC span) would reduce the scope by roughly more than half.

Given that experimental studies were conducted under bidirectional loading (Brooke et al., 2019, 2022; Buker et al., 2022) and NLRHA results were available, the maximum drift from both principal directions was taken as governing for Alpha and Beta units, whereas for ordinary units, drift parallel to the unit span was adopted.

It is worth noting that the use of NLRHA results, rather than conservative MRSA-based estimates, led to a substantial reduction in the strengthening scope. For the case-study building, the number of precast units

requiring retrofit decreased from nearly 300 to approximately 175 out of a total of 1,000 units, illustrating the practical impact of adopting advanced analysis methods.

4 SEISMIC STRENGTHENING

4.1 Level 12 transfer floor strengthening

A transfer level is defined as a floor where lateral forces are redistributed due to changes in stiffness, geometry, or load paths. Level 12 functions as the primary transfer floor, directly above the shear walls, where lateral forces from the MRFs above level 12 are to be transferred to the shear wall below. The transfer forces at this level are on the order of 21,000 kN, in addition to the inertia forces generated by the floor diaphragm. This combination necessitates a substantial strengthening solution, as illustrated in Figure 5.

Figure 5a shows elevation view of the collector beams strengthening where lateral forces generated by the MRFs are transferred to the shear walls through axial behavior of the collector elements. To improve the effectiveness of this load transfer mechanism, the perimeter beams were strengthened using unbonded steel plates installed on their top surface, bearing against columns and anchored into the shear walls.

Importantly, the strengthening was designed to preserve the original flexural capacity of the reinforced concrete beams, which are key components of the ductile frame system. By avoiding a significant increase in beam strength, the intended flexural behavior of the beam was unchanged.

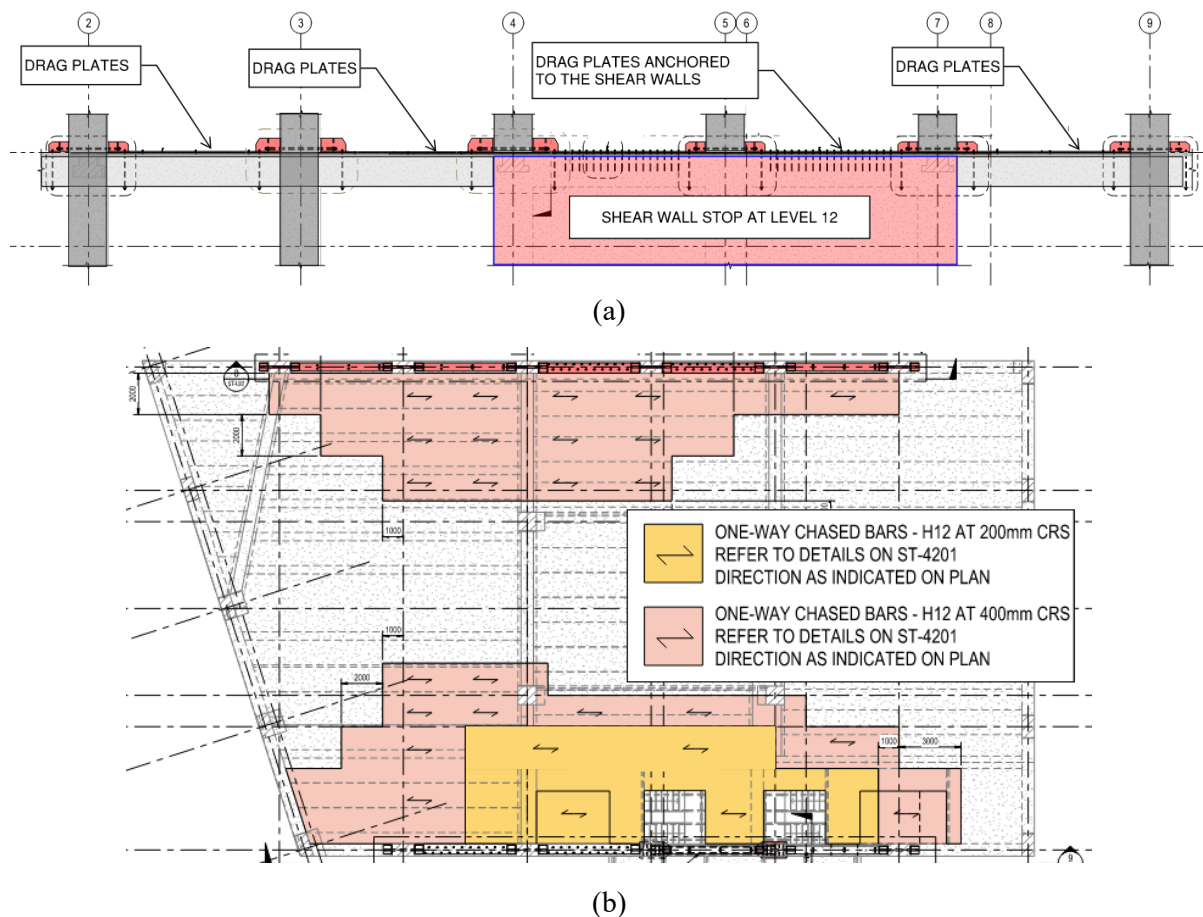


Figure 5: Level 12 strengthening: (a) plan view of chaser-in bar strengthening and (b) elevation view showing collector plates and fixings into shear walls.

Due to the deformation compatibility requirements of the diaphragm and collector beams, a portion of the transfer load wants to travel into stiff diaphragm slab rather than through the collector beams, before transferring into the shear walls. This behaviour necessitated an enhancement of the diaphragm tensile capacity as illustrated in Figure 5b. Accordingly, deformed reinforcement will be chased into the topping slab as per (ACI PRC-440.2-23, 2023). The chases are 22 × 22 mm and infilled with high-strength, low-shrinkage grout. New reinforcement is lapped with existing starter bars and mesh where required and detailed to avoid cutting existing reinforcement.

4.2 Typical diaphragm strengthening

Similar to Level 12, the chased-bar solution was adopted to enhance the diaphragm tensile capacity at ten floors. Alternative retrofit options were also investigated, including steel plates installed on top of the topping slab and steel tension only bracing systems placed beneath the floor. Among these options, the chased-in reinforcement approach was found to provide the most cost-effective solution when considering implications to building services and architecture. It is worth noting that the use of NLRHA results, led to a substantial reduction in the strengthening scope. For the case-study building, the total number of diaphragms needing remediation reduced from 26 (all) to 10 floors, illustrating the practical impact of adopting advanced analysis methods.

4.3 Concurrency actions on Diaphragms

Part C2 of recent Guidelines (NZ Guidelines Part C2, 2025) requires diaphragms to be assessed for worst-case skewed earthquake actions, with all lateral load-resisting systems assumed to develop their full overstrength capacity. While this reflects the conventional approaches to ensure an elastic diaphragm behaviour, no explicit direction is provided on how this should be implemented specifically around how the skewed angle inertia can be derived.

To address this gap, this section first outlines key observations from time-history floor acceleration responses. It then discusses practical approaches for assessing and designing diaphragms under skewed seismic demands.

4.3.1 Different mathematical approaches

Several mathematical approaches can be used to represent skewed earthquake effects. The first approach, SRSS (the Square Root of the Sum of Squares), combines maximum orthogonal seismic responses assuming they are statistically independent. This results in a constant combined demand regardless of direction, as expressed in Equation (1). From a physical perspective, the SRSS approach effectively represents the simultaneous development of full overstrength actions in both orthogonal directions, aligning with the intent of current guidelines.

$$SRSS = \sqrt{(Max F_x(t))^2 + (Max F_y(t))^2} \quad (1)$$

While simple to apply, the SRSS method has a key conceptual limitation: it implicitly assumes that peak responses in the two orthogonal directions occur simultaneously, which is not physically representative of actual earthquake behaviour.

The second approach is vector summation, which combines orthogonal accelerations at each instant in time during an earthquake. This method provides the most realistic representation of the seismic demand but requires detailed floor acceleration time-history data. As a result, it typically necessitates nonlinear response history analysis (NLRHA), which may not be practical for all projects.

$$F_{\theta}(t) = \sqrt{F_x(t)^2 + F_y(t)^2} \quad (2)$$

Finally, the 100% + 30% rule (often referred to as the orthogonal combination procedure), as specified in various building codes e.g., (American Society of Civil Engineers, 2022; NZS1170.5, 2004), is another widely used method for combining orthogonal seismic effects in structural design.

4.3.2 Discussion

Figure 6 (a, c, e and g) presents the acceleration time-history responses for four representative floors (Levels 25, 19, 15, and 13). The lateral load-resisting systems in both directions consist of ductile moment frames, which were observed to perform within their expected inelastic range.

A key observation from these records is that peak accelerations in the two orthogonal directions do not occur simultaneously. However, they occur sufficiently close in time that their combined effect remains significant.

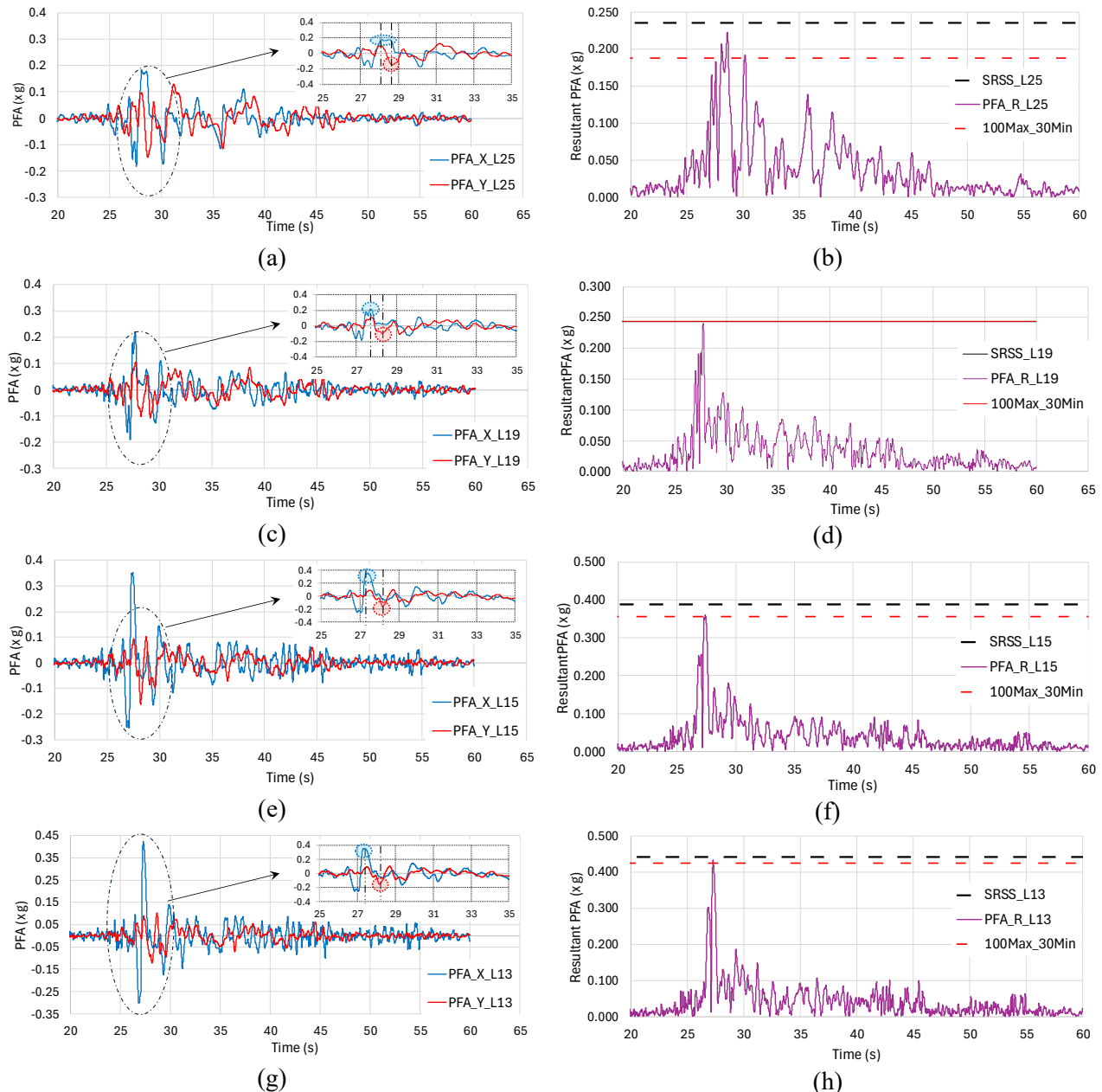


Figure 6: Floor acceleration time-history responses and comparison for levels 26, 19, 15, and 13: (a, c, e and g) acceleration time-history responses, (b, d, f and h) comparison between resultant acceleration, SRSS and 100%-30% Methods

This is further illustrated in Figure 6 (b, d, f and h), where the resultant acceleration obtained through vector summation of the orthogonal components is compared with the SRSS values. The results show that the vector sum is consistently close to the SRSS estimate, with SRSS values being marginally higher in all cases.

The latter figures also present the results of the 100%–30% method. While this approach generally produces values close to the vector sum, there are instances where it fails to capture the worst-case acceleration. For example, Figure 6(b), corresponding to Level 25, shows that the maximum observed acceleration is approximately 20% higher than that predicted by the 100%–30% method, indicating a degree of unconservatism.

Although peak responses are not coincident, their proximity in time results in combined demands comparable to SRSS values. While the 100%–30% method often approximates the vector sum, it can underestimate peak accelerations and be unconservative. The SRSS method, being consistently slightly higher, provides a more reliable upper-bound estimate of bidirectional demand, supporting its use in diaphragm design.

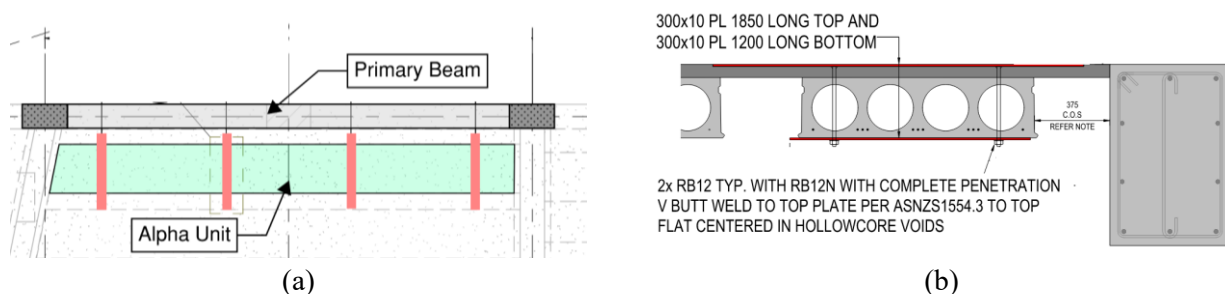
For this case study building, considering the size and height of the structure and the lack of explicit guidance on the implementation methodology, the design team adopted the SRSS method.

4.3.3 Minimum action on the diaphragm

Consideration of concurrent actions in conjunction with the pESA methodology remains ambiguous in current guidelines. When PGA governs floor accelerations in both principal directions, any of the above approaches would result in an artificially elevated PGA of up to 40% above the code-prescribed value. Accordingly, it is suggested that concurrent action provisions be not applied to floor levels where PGA governs floor accelerations.

4.4 Alpha and Beta unit retrofit

Figure 7 shows the typical strengthening of Alpha and Beta HC units, designed to provide alternative gravity load paths (catch system) in case of web cracking or web splitting. As shown on plan views (Figure 7a, c), four pairs of steel plates are evenly spaced along each unit. At each location, the steel plates will clamp the HC units, enabling gravity loads to be transferred to adjacent beams or neighbouring HC units, thereby maintaining structural continuity. The approach follows the ReCast Floors research project principles (Brooke et al., 2019, 2022) and has been applied in other buildings in Wellington. Alternative retrofit options, such as strongbacks, were also considered. However, in consultation with contractors and architects, these were found to be overly intrusive to existing building services and costly to implement.



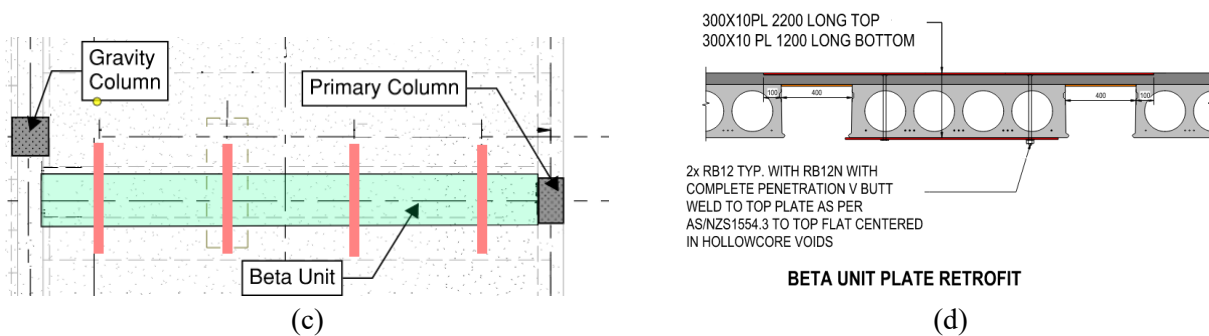


Figure 7: Alpha unit retrofit via clamping steel plates (a) alpha unit plan view, (b) alpha unit retrofit detail view, (c) beta unit plan view and (d) beta unit retrofit detail view

4.5 Strengthening for negative moment failure (NMF)

Negative moment failure strengthening is undertaken to enhance the negative flexural capacity of the HC units such that it exceeds the design negative moment demand, thereby preventing cracking and potential loss of flexural integrity in the units. This strengthening is achieved by chasing new deformed reinforcement bars into the concrete topping slab as per detailed proposed in (ACI PRC-440.2-23, 2023). These bars are detailed to lap with the existing starter bars and extend over a sufficient length so that the upgraded section is capable of resisting the required negative bending moments. Typical strengthening details are shown in Figure 8.

In accordance with the recommendations in the (Brooke et al., 2019, 2022), the new chased-in reinforcement bars must not extend into primary beams, as this would increase the negative demand for the HC units and may render the strengthening ineffective. For this project, a minimum clear distance of 40 mm has been adopted between the face of the supporting beam and the termination of the chased-in bars.

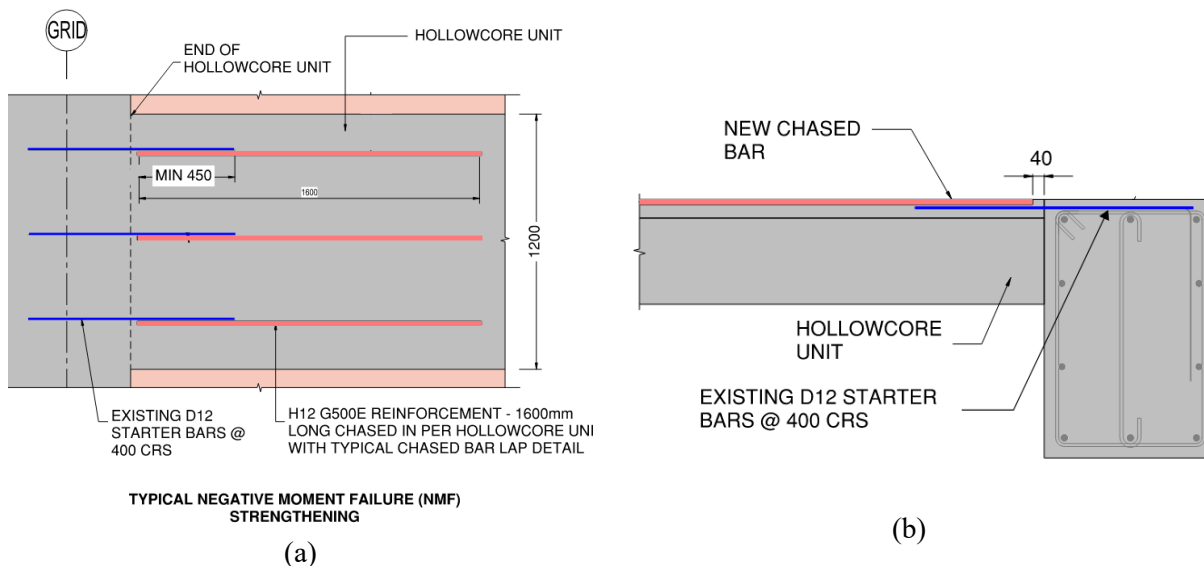


Figure 8: Strengthening for negative moment failure: (a) plan view and (b) elevation view

5 SUMMARY AND CONCLUSIONS

This paper presents the seismic assessment and strengthening of a 26-storey reinforced concrete building in Wellington, emphasizing the importance of continuity and uniformity in the vertical lateral load-resisting system.

Nonlinear response history analysis (NLRHA) was employed to provide more accurate estimates of floor drifts and accelerations. A key outcome of NLRHA was a reduction in the initially estimated seismic demands, which in turn substantially reduced the scope of required strengthening, and most importantly, identified the correct floors to strengthen. This effect was particularly evident for precast floor units, where several floors were found to be exempt from strengthening based on NLRHA-derived drift demands. Similarly, many upper storeys exhibited lower acceleration demands than predicted by simplified methods, further narrowing the strengthening scope. To give a better understanding of the scope reduction for the readers, the total number of diaphragms needing remediation reduced from 26 (all) to 10 floors, and the total number of precast units needing retrofit was reduced from nearly 300 to 175 out of 1,000.

To validate the NLRHA drifts, two alternative approaches—conventional modal response spectrum analysis (MRSA) and effective MRSA—were employed. Effective MRSA, which incorporates stiffness properties corresponding to the maximum displacement demand, closely matched NLRHA results, whereas conventional MRSA with K_{dm} provided the most conservative estimates.

The case study demonstrated that discontinuities in the vertical system can impose significant diaphragm remediation requirements on the affected and adjacent floors. Nonlinear response history analysis (NLRHA) revealed that conventional methods, such as pESA and ASCE 7-22, can underestimate peak floor accelerations by up to 40% in directions affected by vertical irregularity. In the same manner, they all overestimated the demand for the top five floors. One explanation could be that these methods implicitly assume uniform floor stiffness and regular geometry along the height of the structure which therefore seems to limit their application for such structures. This finding highlights the importance of advanced nonlinear response history analysis (NLRHA) for irregular structures and highlights the need for clearer guidance in future seismic assessment standards. Although current building codes address vertical irregularities by imposing certain limitations and applying design penalties, the practical consequences of such irregularities—particularly their impact on floor acceleration demands—remain insufficiently clarified or investigated. Greater emphasis and prescriptive guidance in seismic assessment standards would help engineers better understand these effects and apply more consistent strategies.

NZ Guidelines Part C2 (2025) require diaphragms to be assessed under worst-case skewed earthquake actions, assuming full overstrength of the lateral load-resisting systems. Time-history results indicate that, although peak acceleration responses in orthogonal directions are not strictly coincident, they occur sufficiently close in time to produce significant combined demands. While the 100%–30% method generally approximates the vector sum of orthogonal responses, it may not capture these peak resultant accelerations and can therefore be unconservative. In contrast, the SRSS method consistently produces slightly higher values, providing a more reliable upper-bound estimate of bidirectional demand. This approach also aligns with the physical interpretation of near-concurrent development of overstrength actions in both orthogonal directions. Accordingly, for the case-study building, the SRSS method was adopted as a practical and conservative basis for diaphragm design.

The assessment of HC floor units also expressed some ambiguities in current guidelines, particularly regarding unit classification and the critical direction of inter-storey drift. For the case-study building, a conservative approach was adopted: Alpha and Beta units were assessed using maximum drift from both principal directions, while ordinary units were assessed based on drift parallel to the HC span. These measures ensured structural resilience while managing potential cost implications.

The paper also outlines a suite of strengthening measures—including diaphragm tensile reinforcement, collector beam upgrades, and HC unit retrofitting—that may serve as practical examples for the wider industry. Overall, the study emphasizes the importance of considering vertical irregularities, clarifying HC unit

guidelines, and integrating advanced analysis and targeted retrofits to achieve safe, cost-effective, and constructible solutions for high-rise buildings in high-seismicity zones.

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