Whole-of-Building Approach to Improve Seismic Retrofits of Reinforced Concrete Buildings in New Zealand

"White Paper" on Challenges and Opportunities Related to the Retrofit of Multi-Storey RC Building Structures in NZ

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EQC Toka Tū Ake, BIP

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Glossary and Notations

ACI	:	American Concrete Institute
BIP	:	Building Innovation Partnership
CBD	:	Central Business District
CDF	:	Cumulative Distribution Functions
EQC	:	Earthquake Commission
ESA	:	Equivalent static analysis
FEMA	:	Federal Emergency Management Agency
FRP	:	Fibre Reinforced Polymer
HRC	:	Headed Reinforcement Corporation
IL	:	Importance Level
JCSAEB	:	Join Committee on Seismic Assessment of Existing Buildings
LOS	:	Loss of Support
MBIE	:	Ministry of Business, Innovation and Employment
MCE	:	Maximum Considered Earthquake
NMF	:	Negative Moment Failure
NSHM	:	National Seismic Hazard Model
NTC	:	Norma Técnica Complementaria (in Spanish)
NZ	:	New Zealand
NZGS	:	New Zealand Geotechnical Society
NZS	:	New Zealand Standard
NZSEE	:	New Zealand Society for Earthquake Engineering
PEER	:	Pacific Earthquake Engineering Research Center
PGA	:	Peak Ground Acceleration
PMF	:	Positive Moment Failure
RC	:	Reinforced Concrete
SESOC	:	Structural Engineering Society of New Zealand
SLaMA	:	Simplified Lateral Mechanism Analysis
TS	:	Technical Specification
ULS	:	Ultimate Limit State
USA	:	United State of America
WSF	:	Web Splitting Failure
%NBS	:	Percentage of the New Building Standard as calculated by application of C5 guidelines
b(y)	:	Width of the cross-section of the element
h(x)	:	Length of the cross-section of the element

L_c	:	Shear span, distance of the critical section from the point of contra-flexure
l_s	:	Length of the lap splice of longitudinal reinforcement
l_w	:	Wall length
M_n	:	Nominal moment capacity
M_y	:	Yielding moment
$M/(Vl_w)$	:	Shear span to depth ratio
P_{fail}	:	Failure probability index
$P/A_g f'_c$	:	Axial force ratio
$\frac{N^*}{A_g f_c'}$	:	Axial force ratio used to calculate plastic rotation capacity
$\mathbf{Q_{mu}}$	:	Shear capacity (Japanese guidelines for seismic assessment)
$\mathbf{Q_{su}}$	:	Shear demand (Japanese guidelines for seismic assessment)
S_p	:	Structural Performance Factor
T_1	:	Period of the first mode of the structure
t_w	:	Wall thickness
$V_{c\text{-}n}$	:	Column shear strength provided by concrete and axial force
$V_{p,col0}$	:	Undegraded column shear strength
$V_{s,col}$	:	Column shear strength provided by the transverse reinforcement
V_u	:	Maximum expected base shear from pushover analysis
V_y	:	Base shear at first yield
$V_{y,x}$	:	Base shear at first yield in the x direction
$V_{y,y}$	:	Base shear at first yield in the y direction
W	:	Building seismic weight
Δ_{cap}	:	Onset of lateral strength degradation at exceedance of the probable deformation capacity
Δ_f	:	Displacement for onset of loss of gravity load carrying capacity
Δ_p	:	Plastic displacement of the element
Δ_y	:	Effective yield displacement of the element
θ_a	:	Plastic hinge rotation at onset of axial failure
θ_p	:	Inelastic rotation capacity
μ	:	Structural ductility factor in accordance with NZS 1170.5:2004
ρ_l	:	Longitudinal reinforcement ratio
ρ_t	:	Transverse reinforcement ratio, calculated as the area of transverse shear reinforcement divided by the product of spacing and column dimension perpendicular to the direction of shear force
$\rho_{t,x}$	:	Transverse reinforcement ratio in the x direction
$\rho_{t,y}$	:	Transverse reinforcement ratio in the y direction

Executive Summary

In response to the pressing need for retrofit guidelines in New Zealand identified by the Building Innovation Partnership (BIP), Toka Tū Ake, and the Joint Committee on Seismic Assessment of Existing Buildings (JCSAEB), an effort has been initiated to develop a retrofit guide for existing reinforced concrete (RC) multi-storey buildings. New Zealand's unique construction practices, such as the combination of precast and cast-in-place RC and the use of structural systems with limited stiffness, create specific challenges for seismic retrofitting that cannot necessarily be addressed using existing international practices. Furthermore, current overemphasis on the vulnerability index known as 'percent new building standard' (%NBS) has led to issues such as retrofit schemes overlooking essential components, inconsistent vulnerability estimates, disputes, unnecessary expenses, and solutions that may not address critical deficiencies. The objective of the proposed guide is to address the critical need to assist engineers in a) prioritising vulnerabilities in existing buildings, b) ensuring compliance with standards, and c) adopting new knowledge and improvements in retrofit practices.

Workshops held in 2023 and a literature review conducted in 2024 identified challenges and opportunities associated with retrofitting buildings in New Zealand. In response, this white paper proposes solutions to some of the identified challenges, which may be explored in the development of the proposed retrofit guide. The most salient points are as follows:

- New Zealand guidelines currently have limited guidance regarding the design of retrofits, which leads to inconsistent effectiveness of retrofit solutions.
- Explicit assumptions and corresponding checks for retrofit design (presented in Table 1) may produce more consistent results.
- Current methods to estimate the deformation capacity of RC components lead to comparisons of measurements and calculation results with large scatter (Section 2.2.2).
- C5 methods to estimate deformation capacity have not been calibrated to be used for retrofitted elements. In their current form, their use for retrofitted elements requires adaptations and/or assumptions based on engineering judgement (Section 2.3)
- Precast floors are susceptible to brittle failure at drifts as low as 0.5-1% (see section 2.2.3), which often governs the performance index of a building. Imminent updates to C5 address some of the challenges related to the assessment of floors and diaphragms (Section 3.2).
- Japanese methods to design and evaluate retrofitted elements are straightforward. Retrofitted buildings designed according to Japanese retrofit guidelines have been observed to perform well in major earthquakes (Section 3.3 and Appendix A4)

Case studies were conducted as part of this white paper, and several observations were made regarding their outcomes:

- Adding members to stiffen the structure, adding supplemental damping, or implementing base isolation all can reduce drift demands that may be sufficient to prevent brittle failures of vulnerable elements without individual element retrofit. (Section 4.1)
- Buildings with sufficiently stiff but brittle elements benefit from individual element intervention and may not require foundation or geotechnical intervention (Section 4.2).

- Any amount of effective intervention is better than not retrofitting a vulnerable building. Buildings should be retrofitted to reduce vulnerability rather than to pursue higher %NBS targets (Section 4.1).
- At least in the initial phases of design, the engineer is better off considering retrofit alternatives on the basis of crude indices representing each alternative (e.g. building period, as done in Section 4.1) rather than on results from elaborate analyses.

The goal of this paper is to seek input to inform the drafting of the guide for RC retrofit that it describes. The readers are asked to pay particular attention to Section 3, which provides proposed initial directions for the guide. Please provide your views through this form[1]. The writers do not plan to revise this paper on the basis of the sought input. Instead, the input will be used exclusively to inform the production of the guide.

1 https://docs.google.com/forms/d/e/1FAIpQLSdFWbXOJCPtHiJjvCm2EMV_FS0xlRzSZNsrmjw40yoI55iwww/viewform?usp=sf_link

Introduction

The intersection of the imminent seismic risk in several regions of New Zealand (e.g., Wellington) and the existence of a large inventory of multi-storey reinforced concrete (RC) buildings with known vulnerabilities (ranging from lack of stiffness to deficient detailing) requires a coordinated effort to facilitate building retrofitting. Other countries with similar challenges have produced guides to help aid engineers in that process. Examples include the documents FEMA-547 (2006), ASCE 41-23 (2023), and ACI 369.1-22 (2023) in the USA, the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings and the Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings published by the Japan Building Disaster Prevention Association in Japan (2001), and the Complimentary Technical Standard for the Assessment and Retrofit of Existing Buildings NTC-2023 in Mexico (2023). To address the need for a similar guiding document addressing challenges specific to multi-storey RC buildings in the New Zealand context, Toka Tū Ake EQC and Building Innovation Partnership BIP have joined efforts to support a collaboration between the University of Canterbury and the University of Auckland to draft the needed guidelines. The proposed guideline is intended to provide simple and economic retrofit solutions for reinforced concrete buildings in line with New Zealand specific contexts.

The proposed guide shall fit within the overall regulatory framework in New Zealand through a larger project conceived by the Ministry of Business, Innovation, and Employment (MBIE). A need for general guidance for retrofitting buildings in New Zealand was identified by MBIE's Joint Committee on Seismic Assessment of Existing Buildings (JCSAEB). This committee represents New Zealand Society for Earthquake Engineering (NZSEE), the Structural Engineering Society of New Zealand (SESOC), and the New Zealand Geotechnical Society (NZGS). The general guidance from MBIE shall help engineers focus on the most critical vulnerabilities in a building, serve the needs of the market and regulatory bodies, and allow for methodical adoption of new knowledge. Within the scope of that general effort that addressed retrofits of all types of construction, a need for specific guidance on retrofitting RC buildings was identified by BIP and EQC. That is the need addressed by the guide discussed here.

To produce the needed guide, the project team has collected information through workshops and questionnaires, and it has also produced an extensive literature review (available upon request through Toka Tū Ake EQC and BIP). The literature review offers references to research on conventional retrofit techniques, and information on novel techniques that may not be common in New Zealand yet. The literature review may be of help to engineers searching for alternatives that may accommodate better specific project constraints.

This white paper is being produced with two main objectives in mind:

1. Identify challenges that can be addressed through the proposed retrofit-design guideline.
2. Draft feasible solutions to the most critical of the mentioned challenges **to seek input** from industry and appropriate stakeholders before the first draft of the guide is produced.

The first draft of the guide shall be vetted through public commentary. The whole effort is overseen by JCSAEB, mentioned above, and by a steering committee composed of practicing engineers. The white paper begins with a summary of findings collected from a series of workshops held in key areas of the country. Based on these findings, the key challenges related to retrofit of RC buildings are summarized. Next, possible solutions to the posed challenges are proposed. Finally, two case studies are presented that follow the proposed solutions.

1. Workshops Held in 2023

To define the scope of the new retrofit guide, workshops were held in Wellington (July 18, 2023), Napier (August 29, 2023), and Christchurch (September 14, 2023). Opinions from industry, academia, and council representatives about the contents and format of the proposed retrofit guide were compiled.

Online surveys were also sent to invitees before the workshops. Responses for three questions are shown in Figure 1. The responses suggested that the main challenges affecting seismic retrofits of RC buildings in New Zealand are 1) lack of guidance, 2) costs, and 3) the uncertainty of existing retrofit methods to provide reliable solutions (Figure 1 (a)). Participants were also asked whether they were familiar with FEMA 547 and whether a similar guiding document would be useful to the New Zealand practice. The answers suggested that adapting FEMA 547 and similar documents to the New Zealand context would be of help (Figure 1 (c)).

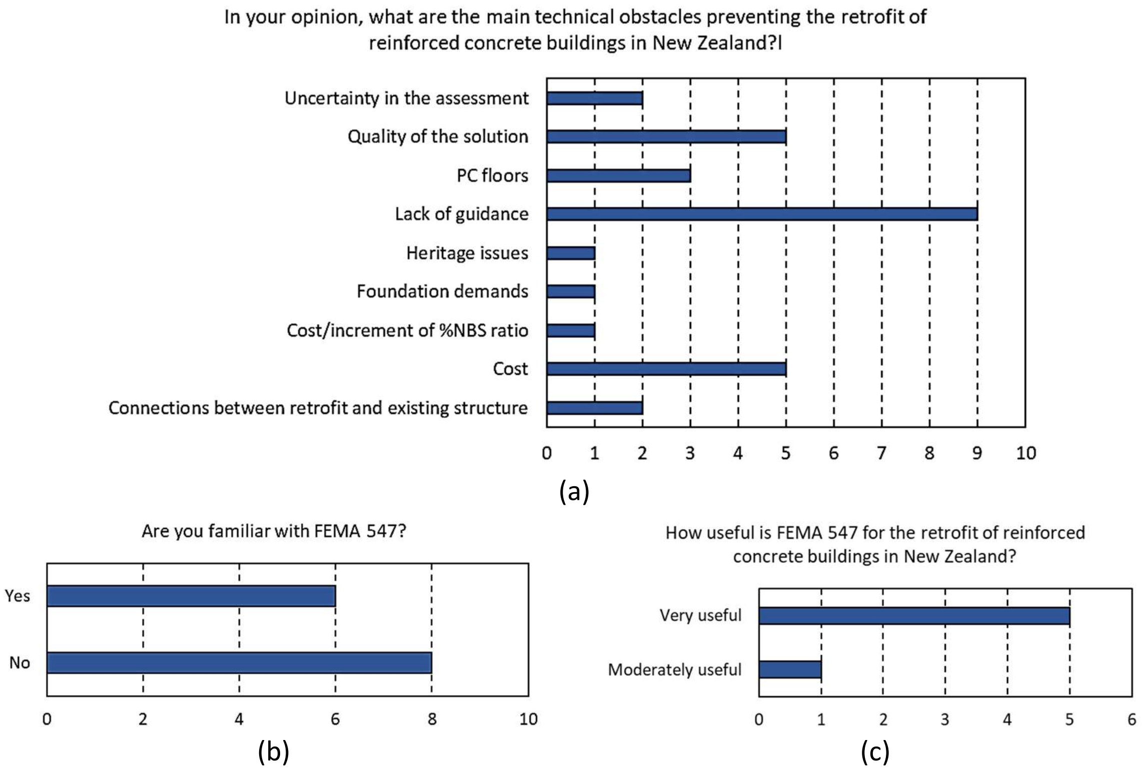


Figure 1: Responses to questions asked to define the scope of the guidelines.

The topics covered in the workshops were: 1) general guideline contents, 2) FEMA 547, 3) overall structural retrofit techniques, 4) methods to retrofit vertical elements comprising the lateral force resisting system, 5) retrofit of floors, and 6) considerations of soils and foundations. During the workshop, attendees were asked to complete a questionnaire about the need to include specific subjects and retrofit solutions the guideline. Results are summarized in Appendix A1. Key conclusions from the workshops include:

- The new guide should not introduce new variations to the procedures used to assess existing buildings.

- The guide should fit within wider regulations (to be produced by MBIE) defining. performance objectives and design motions for retrofits (for all types of construction)
- To keep the guide concise, supporting information, data, measures of uncertainty, and references should be included in a separate commentary.
- Case studies would be helpful but should also be kept separate from the guide.
- The guide should concentrate on specifics for the most common retrofit methods in NZ.
- The main goal of the guide should be to provide uniformity in practice.
- The guide should emphasize the need to consider deformations and their effects:
    - Deformations of the original structure relative to deformations in the added structural elements, frames, or walls.
    - Deformation limits affecting the lateral and vertical strengths of original elements.
    - Deformation limits affecting non-structural elements.
    - Alternative retrofit methods not yet common in New Zealand should be a) tested as needed before introducing them into the guide, and/or b) provided to users through the mentioned commentary and/or the literature review that preceded this white paper.

2. General Challenges

The described workshops and the preceding literature review identified four groups of key challenges:

- Objectives and approaches to design retrofits,
- Challenges related to determining the drift capacity of the existing structure and its influence on retrofit decisions,
- Challenges related to the determining drift capacity of the retrofitted structure, and
- Lack of retrofit techniques that are both cost-efficient and reliable.

The following sections address these groups of challenges in the light of opinions gathered in workshops and other communications with practicing engineers, international retrofit guidelines, and the current assessment guidelines in use in New Zealand.

Current assessment guidelines in use in New Zealand today were released by the Ministry of Building Innovation and Employment (MBIE) in 2017. These guidelines prescribe methods to assess the vulnerability of existing buildings to seismic hazards, with the following organization:

Part A – Assessment objectives and principles

Part B – Initial seismic assessment

Part C – Detailed seismic assessment

In Part C, document C5 (2017) covers assessment related to reinforced concrete buildings, and covers typical building practice, behaviour, expected material properties, probable element capacities, and global capacities for different building typologies. There are updates to C5 that have been proposed for 2024 (Brooke, 2024) which will seek to reduce conservatism in some assessment processes. The work presented here uses the current methodologies in C5.

The assessment guidelines address retrofits in Sections A10 and C5.8 through these general recommendations:

- New elements should be designed to current building code(s), with demands factored for the target %NBS
- New systems may be designed to resist the greater proportion of overall seismic demand
- Displacement compatibility should be carefully considered
- Demand distribution to new elements may be limited by displacement capacity of existing elements
- The building should be re-assessed with new elements and assuming probable strength properties, but this is not generally required when simply adding elements and using linear analysis.

The listed recommendations appear to provide much room for interpretation. As a result, the project team received reports of a myriad of issues affecting assessments and retrofits including:

- Lack of attention to diaphragms
- Inadequate or no consideration of displacement compatibility
- Lack of attention to all relevant "severe structural weaknesses"
- Insufficient consideration of retrofit constructability

- Economic constraints compound the problem. Owners without financing resources are having difficulty implementing retrofits, which they question as being too conservative and expensive. One of the goals of the proposed guide is to provide ways to use simpler and more economical retrofit solutions.

2.1 Clarifying Retrofit Design

The most common goal in the retrofit effort in New Zealand is centred on the index called %NBS. This index refers to the ratio of the strength of the structure to the strength required for a new building of similar characteristics and at the same location as the building being retrofitted. As an alternative to a ratio of strengths, the designer is also allowed to produce a ratio of drift capacity to drift demand to calculate %NBS. Drift demands can be determined using a variety of analysis techniques ranging from a simple equivalent static analysis through to more advanced nonlinear time history analysis.

One difficulty with %NBS is that it has developed over time, and was primarily envisioned as a metric to identify the poorest performing structures seismically. It was not designed to compare different buildings to each other, but has been pressed into other uses (and beyond engineering e.g. real estate/insurance/banking) as a proxy for building rating.

Regardless of the type of ratio chosen, the project team has identified differences among engineering offices related to whether the existing structure is considered in the estimation of the %NBS index, and what limits should be imposed on the analysis. Limits are needed to reflect the fact that the existing structure may lose a large fraction of its lateral resistance (or even gravity-load resistance) before the structural components added through retrofit become effective.

To respond to the recent update of estimates of seismic hazard (TS 1170.5:2024), the MBIE has clarified that the %NBS index refers to the previous hazard (NZS 1170:2004) instead of the latest shaking intensity estimates. This decision demonstrates the nature of the index. It is not an absolute measure of the robustness of a structure. It is only a relative index to identify the most vulnerable structures. In that respect, engineers should convey to clients the idea that satisfying the current regulations does not eliminate risks. The case studies included in this document illustrate this idea in quantitative terms.

Visual inspection of retrofits in NZ has also revealed that not all design offices consider the differences in the deformed shapes of the existing structure and the components added through retrofit. Figure 2 shows a steel frame with eccentric braces that has been fastened to an older RC structure. Connection has been achieved using bolts going through the web of the beam element (excluding the link of the eccentrically braced frame). There is no visible room for the connection to accommodate the differences in the deformed shapes of the concrete and steel frames. The problem is often addressed by making the holes in the web slotted. But that practice does not seem to be followed consistently. That is the type of problem that the new guide will help solve.

Figure 2. Connection between EBF and Concrete Frame not allowing for differences between deformed shapes.

2.2 Quantifying Drift Capacity of Existing Elements

As mentioned above, drift or deformation limits are needed to reflect the fact that the existing structure may lose a large fraction of its lateral or vertical resistance before added structural elements reach their full resistance or before peak deformations estimated for the design earthquake are reached. Deformation limits for existing RC elements are defined in the assessment guidelines C5. A technical proposal to revise C5 was published in 2018 and serves as the current guideline to assess non-earthquake prone buildings. A new revision to C5 is expected to be released in 2024 (Brooke, 2024). The efforts to refine the methods in C5 have resulted in improvements in estimates of the strengths of structural elements, but methods for estimating deformation capacities still have demonstrable scatter. That scatter is not illustrated in detail in the assessment guidelines and, therefore, it is not always communicated to stakeholders well. A perspective on the "reliability" of estimates produced using C5 is provided here, in reference to a database of results from RC column tests published by ACI 369 (Ghannoum & Sivaramakrishnan, 2015).

The reliability of deformation capacities obtained though C5 is relevant to retrofit in at least two ways: 1) conservatism in those estimates, which results from the mentioned scatter, increases the number of elements that can be flagged as needing retrofit, and 2), that scatter and the fact that the procedures in C5 were calibrated for conventional elements (without retrofit) should prompt engineers to question the applicability of the procedures to retrofitted elements for which there is less test data available to check and, if necessary, adapt the mentioned procedures.

2.2.1. Procedures in C5

As part of the detailed seismic assessment of a building, the guidelines provide methods to estimate drift capacities for structural elements. These estimates are based on probable material properties, geometry, and combinations of mechanics and observation The current version of C5 includes two methods to estimate drift capacities: the moment-curvature method (an adaptation of the plastic hinge analogy first developed by (McCollister et al., 1954), for beams resisting monotonically increasing demands), and the "direct rotation method," which is a statistical regression adapted from other guidelines (ASCE 41-17, 2017).

2.2.1.1. Probable Flexural Capacity (C5.5.2.2) and Probable Shear Capacity (C5.5.5)

Methods to estimate the drift capacity of RC elements often require estimates of strength. Following C5.5.2.2, probable flexural strength is calculated using probable material strengths (C5.4) and conventional formulations for RC sections resisting bending. Column flexural strength is to be assessed considering axial forces expected from gravity and seismic actions. On the other hand, probable shear capacity is given by the sum of contributions from concrete and transverse reinforcement:

$$V_{p,col0} = (V_{c-n} + V_{s,col})$$

Figure C5.23 and equation 5.71 of C5 relate the probable shear capacity to rotation ductility, or the column rotation normalized with respect to the probable rotation at yield. Shear strength is assumed to decay linearly from $V_{p,col0}$to $0.7V_{p,col0}$ for rotation ductility values ranging from 2 to 6, with larger rotations not being assumed to cause further decay.

2.2.1.2. Moment-Curvature Method (C5.5.3.4)

This method relies on information about material properties, geometry, sectional analysis (C5.5.2.2), an assumed plastic hinge length (C5.47), and curvature limits related to: a) maximum steel or concrete strains (Table C5.10), b) bar buckling (Eqn. C5.57), c) lap splices (Eqn. C5.6), and e) the intersection of curves representing expected variations of flexural resistance and shear capacity with rotational ductility (Figure C5.23). The limiting curvature is used to calculate the plastic rotation, θ_p, of the column (Eqn. C5.46), which is in turn used to calculate inelastic deformation, Δ_p/L_c (Eqn. C5.11). The inelastic deformation is added to the elastic deformation, (Δ_y/L_c, Eqn. C5.10), to get an estimate of drift capacity, Δ_{cap}/L_c.

2.2.1.3. Direct-Rotation Method (C5.5.3.3)

Probable material properties and member geometry are entered into a parametric equation directly producing an estimated maximum admissible plastic rotation. There are several equations available for different structural elements that have been calibrated using statistical regression and existing test data. Below is the equation for columns with deformed bars not controlled by inadequate splices:

$$\theta_p = 0.031 - 0.032\frac{N^*}{A_g f_c'} + 0.47\rho_t - 0.017\frac{V_y}{V_{p,col0}} \geq 0.0$$

The equation is supposed to be applicable to both elements with brittle response and ductile elements. Nevertheless, the equation is not recommended for columns with splices not long enough to develop the yield stress of the longitudinal reinforcement.

Once plastic rotation capacity is determined, the corresponding plastic deformation and the yield deformation are added as in the moment-curvature method.

2.2.1.4. Limit Related to Axial Failure

For columns, the drift capacity from either method is compared (through Eqn. C5.8) against two-thirds of the probable drift at loss of axial load-carrying capacity (Δ_f / L_c , Eqn. C5.12), and the smaller of the two values is chosen as the probable drift capacity. At the same time, rotation at axial failure θ_a has a lower bound equal to the plastic rotation calculated using either the moment-curvature or direct rotation methods, which makes the process not straightforward. For columns, for instance, the limitation related to axial failure is only mentioned in a note and not directly in connection with any numbered equation.

Engineers have different readings of the cited provisions and limits. If nothing else, the matter demonstrates the need for simpler assessment and design provisions. In this paper we make the case that the direct-rotation method suffices, creating an opportunity to simplify assessment by reducing the number of options to consider.

2.2.2. Discussion on Scatter

To evaluate the reliability of the described methods to estimate column drift capacity, test results compiled by Committee ACI 369 were compared with calculation results. In total, 326 results from tests of columns with rectangular cross sections and varying geometries, testing configurations, and material properties were considered. All of these columns were subjected to displacement reversals applied along a single axis with protocols with different intensities (increments of displacement and repetitions). To try to produce better results, measured concrete strengths and steel yield stresses were used instead of probable material properties.

Moment-curvature analyses were performed for each specimen to obtain shear-rotation curves. Estimated peak shear and limiting curvature were obtained from the intersection of the shear-rotation curve and a curve representing shear capacity as a function of rotation (Eqn. C.71). Figure 3 shows an example of such an intersection normalized with respect to the yield rotation.

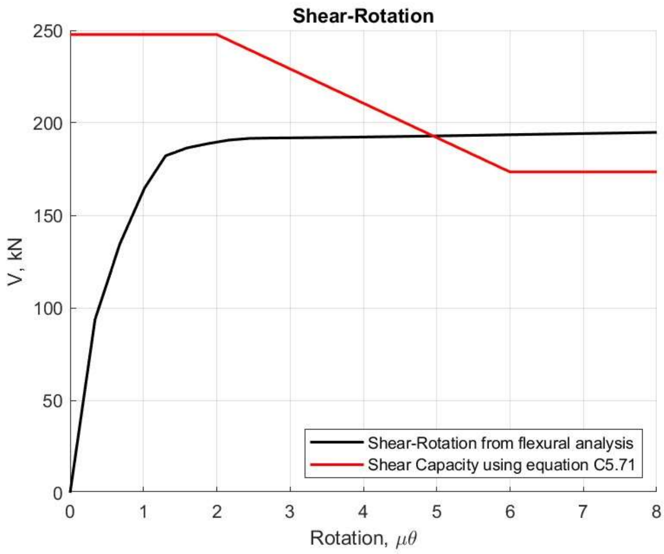


Figure 3: Example of intersection of Shear-Rotation response with C5 Shear capacity estimates.

If flexural demands do not exceed shear capacity $V_{p,col}$, the estimated peak shear and limiting curvature calculated correspond to the smallest of the curvatures related to 1) maximum concrete or steel strains (Table C5.10), 2), splice strength (Eqn. C5.6), and 3) bar buckling (Eqn. C5.57).

Figure 4 shows a comparison between maximum reported shear and the maximum calculated shear C5. The mean ratio of reported to estimated peak shear is 1.09, with a standard deviation of 0.26. With a few conservative exceptions, C5 assessment guidelines provide reliable estimates of shear strength for rectangular columns in the ACI369 database. This observation is not surprising given that most of the studied columns reached their flexural capacities, and flexural capacity is fairly insensitive to assumptions about limiting deformations and, therefore, relatively simple to estimate.

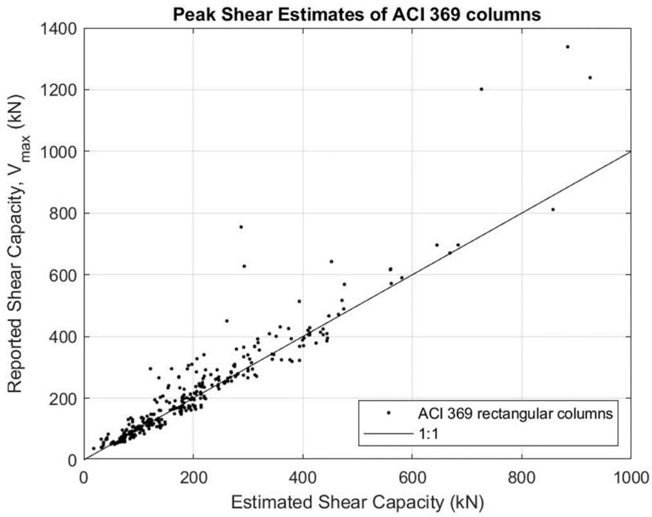


Figure 4: Peak Shear estimates vs reported values for ACI 369 Columns.

Figure 5 shows estimates of probable drift capacity, Δ_{cap}/L_c, obtained from the direct-rotation and moment-curvature methods. Figure 6 compares estimated and reported drifts at axial failure. Of the 326 tests considered, only 40 reported drifts at axial failure. Figure 5 and Figure 6 illustrate large deviations between calculated and measured column drift capacities. The issue is not exclusive to the methods in C5 (Pujol et al., 2022). An evaluation of the method included in the Japanese assessment standard, which states that drift capacity is a function of the ratio of shear strength to shear demand, produced results of similar quality.

Figure 5 and Figure 6 are not presented here to critique C5. The problem of estimating drift capacity is not simple. The figures are presented because they illustrate that uncertainty in our estimates a) should be quantified and used to inform decisions about retrofit, and b) often leads to large conservatism which adds to the costs and extent of retrofits.[2]

A few key ideas related to Figure 5 and Figure 6 follow:

- Mean and median ratios of reported/calculated drift capacities are 2.0 and 1.6 for the moment-curvature method and 2.9 and 2.2 for the direct rotation method. The corresponding standard deviations are 1.5 and 3.4. The direct rotation method does not appear to produce drift capacity estimates larger than approximately 2.5%.

[2] Machine-learning algorithms can be trained to produce better estimates of drift capacities (Luo & Paal, 2019; Aladsani et al., 2021; Deger et al., 2023; Lee et al., 2024). An open-source algorithm is available on https://colab.research.google.com/drive/1nmvUfavaIdTOy2FnuRQEzs_MpmsfovUZ?usp=sharing. But the vast majority of the engineers polled through our workshops expressed reluctance to use machine-learning algorithms.

- The mean ratio of reported to calculated drift at axial failure is 2.8 (using the moment-curvature plastic rotation as a lower limit) and 2.9 (using direct rotation plastic rotation as a lower limit). The corresponding standard deviations are 1.7 and 1.6. The conservatism may be warranted as drift at axial failure has been observed to be sensitive to the number and direction of displacement reversals, which are both difficult to estimate.

Approximately 3/4 of column drift capacity estimates are limited by two-thirds of the probable drift at axial failure, Δ_f/L_c. Concerning the last bullet point, Figure 7 shows drift capacity estimates obtained ignoring the limitation of two-thirds of the drift at axial failure. Means and standard deviations become 1.6 and 1.4 for moment-curvature, and 2.5 and 3.4 for direct rotation methods, respectively. A salient observation from the presented plots is that the direct-rotation method tends to produce safe and conservative estimates.

It is understood that modifications are underway to make the procedures used to produce Figures 5-6 less conservative through an update to C5 to be released in 2024. The essence of the procedures, however, is not changing, which means that the observed scatter is not likely to decrease.

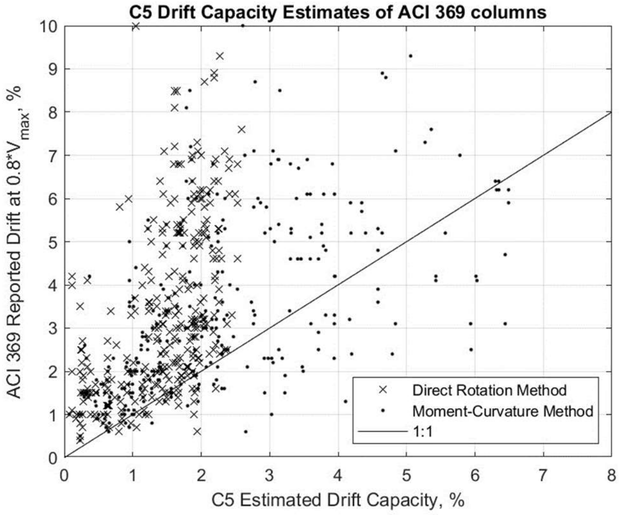


Figure 5: Reported and calculated drift capacities.

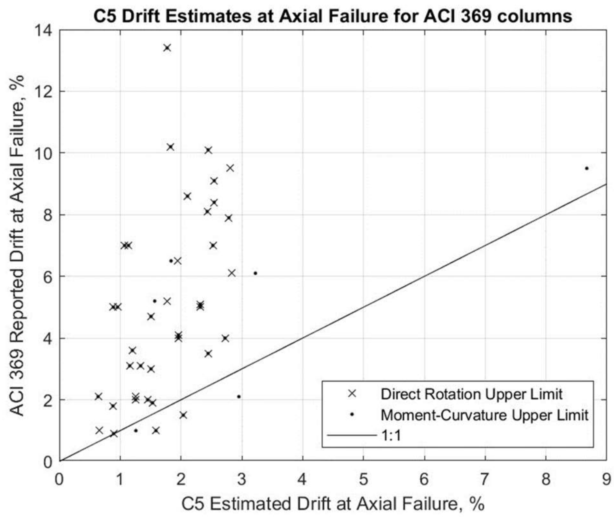


Figure 6: Reported and calculated drift at axial failure.

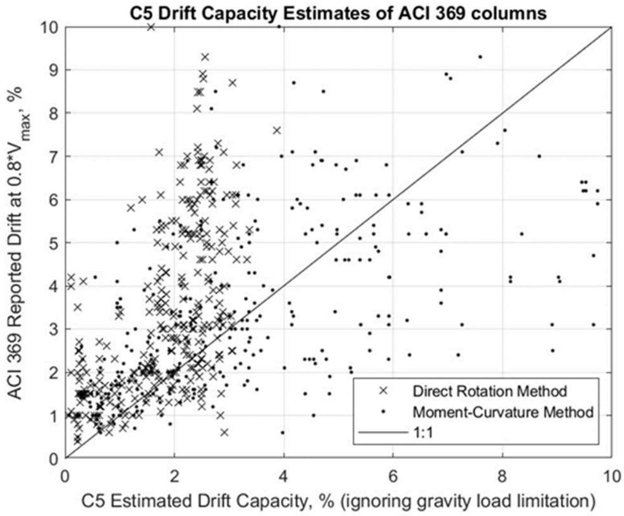


Figure 7: Drift capacities estimated ignoring limit related to axial failure.

Difficulties are inevitable when attempting to estimate the drift capacity of RC elements. In Figure 3 as an example, the difference between a rotation ductility of 1.3 and a rotation ductility of 8 is only 17kN, meaning a 10% increase in shear strength leads to an expected increase in rotation of over 500%. This is the reason a segment of the literature on drift capacity has moved from approaches based on highly idealized mechanical constructs to regressions using parameters known to be critical, and, more recently, machine-learning algorithms. Nevertheless, it is understood that retrofit practices in New Zealand need stability and there is not much room for new changes to assessment practices. The critical issue here is, however, that the data and experience available on drift capacity of retrofitted elements are more limited relative to what is available for un-retrofitted elements. The issues illustrated through Figure 5 to Figure 7, which relate to conventional (un-retrofitted) columns, are therefore likely to be compounded by retrofits. Plausible ways to address that problem are discussed in Section 3.

2.2.3. Floor Drift Capacity

C5 (2018) prescribes assessment procedures for hollow-core floor units, as show in Figure 8. The potential failure modes of loss of support (LOS), negative moment failure (NMF), positive moment failure (PMF) and web splitting failure (WSF) are assessed in reference to expected drift demands to achieve a low likelihood of collapse of the floor unit. A similar procedure is followed for other precast elements but, given the prevalence of hollow-core flooring in New Zealand, only the assessment procedure for hollow-core floors is addressed here for brevity.

LOS is assessed based on expected beam elongation and rotation demands that produce horizontal movement of the floor unit as well as spalling of the support or the end of the floor unit. The drift capacity of the floor unit is defined as the drift at which the total movement exceeds the seating length (as affected by spalling). NMF is checked by comparing the negative moment capacity of the floor unit at the section where starter bars (or dowels) end, and an estimate of negative moment demand. The negative moment demand is obtained from gravity-loads and a superimposed moment equal to the product of the force causing yielding of starter bars times their vertical distance to the centroidal axis of the floor unit. The starter bars can yield because of elongation in beams and relative rotations between beams and floor units. The drift limit assigned to this mode of failure currently is 1%. That limit is reduced to 0.5% to provide a margin of safety.

If the negative moment demand exceeds the negative moment capacity, the drift capacity is estimated to be 1%. PMF needs to be assessed where cracking is expected to result from positive moments. Expected displacement demands from beam elongation and rotation are checked in reference to critical crack width. PMF is likely to occur if the critical crack width is greater than the diameter of the strand of the floor unit. In addition, WSF caused by deformation incompatibility and torsion in the floor unit needs to be checked too. Finally, the drift capacity of the floor unit is estimated as the minimum of the drift capacities associated with the described failure modes.

Appendix E of the C5 (2018) requires that the drift capacity of the floor should be reduced by a factor of 2 because of the significant life safety risk. This factor is likely to govern the assessment of the building.

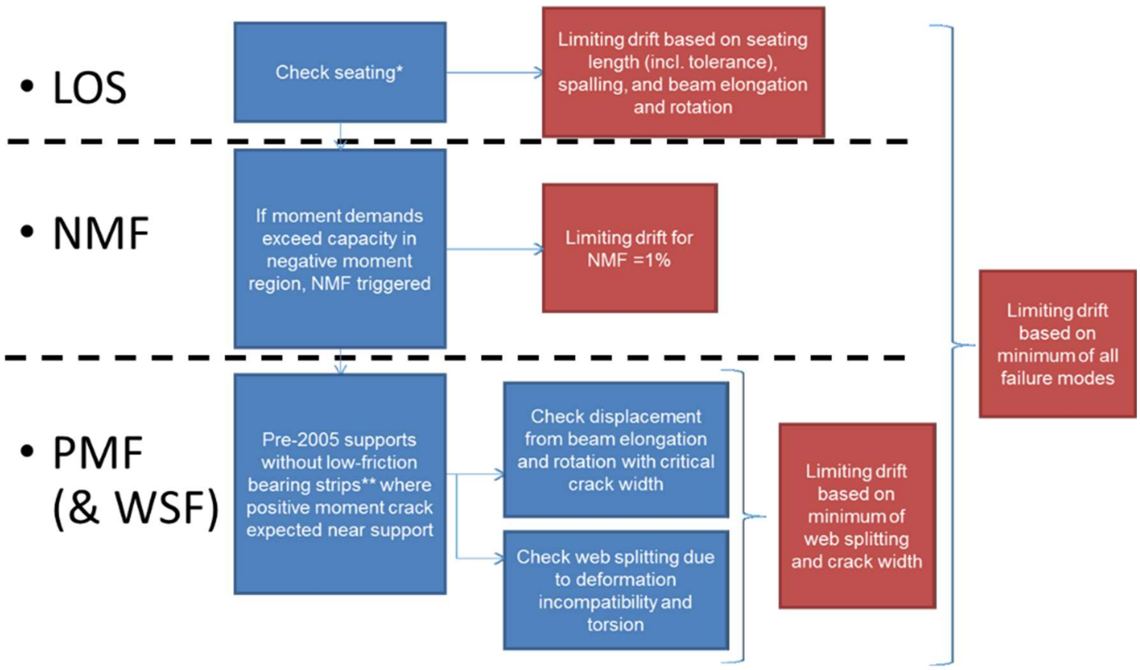


Figure 8: Assessment procedures for determining the drift capacity of hollow-core floor units [5].

The in-plane capacity of the diaphragm is typically limited by the topping reinforcement which often consists of cold-drawn, non-ductile mesh. When assessing the diaphragm capacity, C5 requires the engineer to:

- Ignore the contribution of the mesh if the strut-and-tie, or an equivalent analysis technique, is used (because of the assumption inherent in such approaches that plastic strains develop in the diaphragm reinforcement), or
- Analyse the diaphragm assuming it is linear (elastic), while ensuring that strains don't exceed 0.3%.

Both options typically result in low estimates of diaphragm capacity that trigger retrofitting. Nevertheless, C5 provides a "deemed-to-comply" pathway for diaphragms of regular buildings classified as having an 'importance level' (IL) of 2 (which includes most apartment and office buildings). While this pathway does not provide an estimate of capacity for the diaphragm, it does result in a %NBS score exceeding 34%.

2.3 Quantifying Drift Capacity of Retrofitted Systems

2.3.1 Drift Capacity of Retrofitted RC Elements

There is currently limited guidance in New Zealand regulations or standards on how to calculate 1) the strength and 2) the deformation capacities of retrofitted RC elements (columns, beams, joints, walls). Alternatives being considered for the new guidelines on RC retrofit include:

a. Referring to international consensus documents (such as publications by ACI Committee 440, 2023) for specific types of retrofits. This option has two drawbacks: consensus documents do not always illustrate how the recommendations in them were calibrated, making it difficult to judge the reliability of the methods, and such documents do not exist for all of the retrofit methods at our disposal.
b. Referring the engineer to the available literature on general methods to estimate the drift capacity of retrofitted RC elements (e.g. Özcan and Binici, 2020). The main drawback in this case is simply that the scatter in comparisons between measurement and calculation is quite large.
c. Adopting a standardized method, such as the method recommended in the Japanese guidelines for RC building retrofit or an adaptation of the methods prescribed in assessment standard C5 to assess the deformability of existing RC elements. Again, there is no clear information on scatter, and initial spot-checking exercises done in the preparation of this white paper suggests the scatter may be large again. In the case of the methods in C5, their use for most retrofit techniques would clearly be outside the ranges of applicability considered in their calibration. For instance, even in the case of RC jackets, questions about the effectiveness of the original transverse reinforcement (with tie 'legs' shorter than the legs of new ties) and whether shear strength should be calculated for the new or the old value of concrete strength arise and need to be addressed. C5 does not address them. In contrast, Japanese retrofit-design methods have been explicitly formulated for retrofitted elements and their success seems hard to deny based on reconnaissance evidence.

Considering the advantages and disadvantages of these alternatives, three ways to address the issue of the deformability of retrofitted elements are put forward in Section 3.4. In addition, Table 1 in Section 3.1 describes an alternative to reduce the number of elements requiring retrofit by reducing drift demand as well as an additional option.

2.3.2 Drift Capacity of Retrofitted Diaphragms

Despite the fact that hollow-core floor units typically have low drift capacity (e.g., NMF at 1% Büker, 2023), experimental investigations in ReCast project (BRANZ, 2024) demonstrated that a storey drift of 5% can be achieved before losing gravity load path if appropriate retrofit is implemented (e.g., strongback retrofit). This observation indicates that a storey drift limit of 2.5% consistent with NZS 1170.5:2004 can be achieved for buildings with hollow-core floor system for a life safety performance objective. Nevertheless, the floors may sustain significant damage at this drift which may lead to building demolition.

Supplementary shear reinforcement can potentially be a cost-effective retrofit solution for PMF (Büker, 2023, p.314) in conjunction with supplementary positive moment reinforcement and supplementary seating (Brooke et al., 2022). Ongoing experimental research projects on the potential of supplementary shear reinforcement to increase the rotational capacity of hollow-core floors is underway.

For the retrofit of the in-plane diaphragm capacity of precast floors, work is also underway investigating the use of FRP. Initial results are promising with rotation capacities of up to 6% having been observed for units not subjected to elongation of beams. These results do require significant attention to FRP anchorage detailing to achieve these rotation capacities. Testing of units retrofitted with FRP and subjected to both rotation and elongation shall follow the tests excluding elongation.

2.4 Lack of "Efficient" Options for Retrofit

One of the challenges to retrofit in New Zealand and elsewhere is to minimise the impact on building occupancy caused by the retrofit works. Feedback from the workshops carried out in the start of the project indicated the need for more efficient and less disruptive retrofit options. Often retrofit needs to be done while the building is occupied, hence strengthening elements from the exterior of the building is preferred.

For global retrofit, the options include the use of concrete shear walls, pier-spandrel frames, and steel-braced frames placed adjacent or within the plane of the exterior walls of the building. The main challenges affecting the use of these systems relate to space between buildings (to allow safe construction) and connections that can accommodate differences between the deformed shapes of existing and new structural elements. The PITA-column system that is commonly used in Japan (Pita-Column Association[3]) deserves consideration in New Zealand. It has been successful in that it seems to require minimal space and equipment, and it does not seem to be prone to problems related to incompatibility of displacements as evidenced from its performance in the 2024 Noto Earthquake (Appendix A4).

For local retrofit of RC members (e.g. columns), conventional retrofit options often require access to the entire perimeter of the member, causing costs associated with the modification or replacement of windows, partitions, and façades. An alternative is discussed in Appendix A2.1.

And last, with respect to 'horizontal' structural components, a myriad of challenges affects buildings with precast floors. Appendix A2.2 describes in more detail some of these challenges which have to do mostly with costs associated with accessing and intervening every floor unit in an entire building as required by currently available precast-floor retrofit options.

2.5 Challenges with Base Isolated Retrofits

Base isolation is recognized as one of the most effective seismic protection systems for buildings, significantly reducing seismic demands by decoupling the superstructure from the ground motion. Despite its benefits, several challenges hinder its widespread implementation in New Zealand, particularly in retrofitting existing structures.

1. Limited expertise and resources: There is a limited pool of engineers and contractors in New Zealand with expertise in base isolation technology, particularly in retrofit applications. This

[3] https://www.pita-kyoukai.jp/index.html

limitation can lead to delays in project timelines and increased costs. Often the pool of engineers who are experienced in Base Isolation are not experienced in retrofit or vice versa.

2. Structural and architectural limitations: Retrofitting existing buildings with base isolation requires significant foundation modifications (excavation, strengthening), including provision of a moat/rattle space around the building. This can be particularly challenging in densely built-up areas, and can be further complicated by site boundary/easement issues if the rattle space moat needs to extend over the boundary.
3. Financial implications: The cost of implementing base isolation may be higher than the cost of conventional seismic strengthening methods. This cost includes not only the installation of isolation units but also the associated expenses in analysis, materials, and prototype testing. The lack of local manufacturing facilities for isolation units in New Zealand further increases the costs due to the need for importation and the need for custom designs to meet specific local requirements.
4. Regulatory and standardisation challenges: base isolation is an 'alternative pathway' to comply with the New Zealand Building Code, which leads to inconsistencies in design practices. Engineers must rely on the 2019 Draft NZSEE Seismic Isolation Guideline (NZSEE, 2019), international standards such as those from Japan, Europe, or the United States, or some combination of the available options. This may lead to potential discrepancies in safety and performance expectations (Pietra et al., 2015).
5. Technical and practical constraints: base isolation for retrofitting poses unique technical challenges, including the need for detailed assessment of the existing building's condition. The weight, stiffness, and dynamic characteristics of the building must be compatible with the proposed isolation system. For example, challenges can be encountered with achieving necessary superstructure rigidity if the building is of flexible typology, such as RC moment frames.
6. Performance uncertainty in moderate and severe events: a careful balance must be achieved by designing base isolators that are responsive enough to engage and move under lower seismic intensities (maintaining operational functionality), while also ensuring low risk in the Collapse Avoidance Limit State scenarios. In general, there is uncertainty about the consequences of the exceedance of the ability of the base isolators to accommodate displacements.

Some of the above challenges are not unique to New Zealand. They are broader technical challenges related to base isolation. While it is recognized that better guidance is desired for base isolated building retrofit (and also new design), specific solutions to the challenges above go beyond matters pertaining exclusively to multi-storey RC buildings.

2.6 Challenges with Foundation/Soil Retrofits

In general, retrofit at the soil/foundation level is due to one of the following reasons:

1. A new lateral load resisting structural element (e.g., shear wall, braced frame) is added to the building and the overturning loads need to be resisted.

2. Expected deformation behaviour of existing foundation elements is unreliable or undesirable.

The selected retrofit and details of the intervention are implicitly tied to the reason for and cause of the issues. The first reason has a direct cause (due to the retrofit intervention at the structural level), and focus is typically directed towards having minimal deformation at the soil-foundation level such that the new structural elements can limit the deformations of the building. The second reason around deformations can manifest from a variety of causes. There are two important considerations to make. **First** is the location of the expected deformations:

- Deformation within foundation elements, typically due to a yielding or failure of foundation elements. The primary causes of this are:
  - Retrofit of existing structural elements such that the element forces would overload existing foundation elements
  - A poorly designed foundation element is identified
- Deformation of soil and displacements of foundation elements. The primary causes of this are:
  - Retrofit of existing structural elements such that the element forces would overload the soil or soil-foundation interface
  - The existing design did not account for liquefaction-induced effects (e.g., settlement, differential settlement, tilt, lateral spreading, uplift forces, kinematic soil loads, foundation sliding)
  - Site stability

The location of deformations is important, in that some classes of techniques are only relevant for within-element deformation (e.g., strengthening foundation elements) and others to soil deformation and interface displacements (e.g., ground improvement, site stabilisation). In fact, some techniques to address soil deformation can exacerbate element deformations and vice versa.
The **second** consideration is the type of foundation (e.g., isolated/strip footings, raft/mat foundation, deep/piled foundations), because both failure mechanisms and retrofit options are often unique to a foundation type.

Refer to Section 3.6 for proposed solutions addressing some of the issues above.

3. Solutions Being Considered for Inclusion in New Guidelines for RC Building Retrofit

The following subsections are proposals to address some of the challenges identified above. They are produced here **in an attempt to seek input**. The writers would welcome different proposals that may lead to more, affordable, and effective retrofits.

3.1 Deformation Checks

From correspondence with practicing engineers, the checks shown in Table 1 have been preliminarily selected to be included in the guideline.

Table 1: Proposed deformation checks for the retrofitted structure.

Contribution of the lateral load resisting capacity of an existing structural element be considered in estimation of %NBS index?	Check
Yes	Deformations estimated for the design event shall not exceed deformation limits related to loss of lateral resistance in any un-retrofitted and considered component of the existing structure
No	Deformations estimated for the design event shall not exceed deformation limits related to loss of vertical resistance in any un-retrofitted component of the existing structure. Alternatively, it shall be demonstrated that loss of axial-load carrying capacity can be accommodated through redistribution of forces.

While only two pathways are given in the table above, there is value in adding an additional check for drift capacity associated with the lateral resistance of un-retrofitted elements even when they are not considered in the %NBS estimate. The aim of this check would be to minimize failure of components in the building even when they are not relied on for strength. Such a 'stepped' approach is better aligned with the NZSEE Seismic Grading classifications. Similarly, it would be prudent to calculate deformation demands for the full intensity of the design earthquake, as opposed to a fraction corresponding to the value of %NBS being targeted in the retrofit design. To enable this approach, better methods to estimate deformation capacities would be helpful. In that regard, the discussions in 2.2.1 and 2.2.2 and appendix A3 suggest that the direct rotation method is simpler, more conservative, and its results do not lead to more scatter in comparisons with measurements. The direct-rotation method may also be easier to adapt to retrofitted elements.

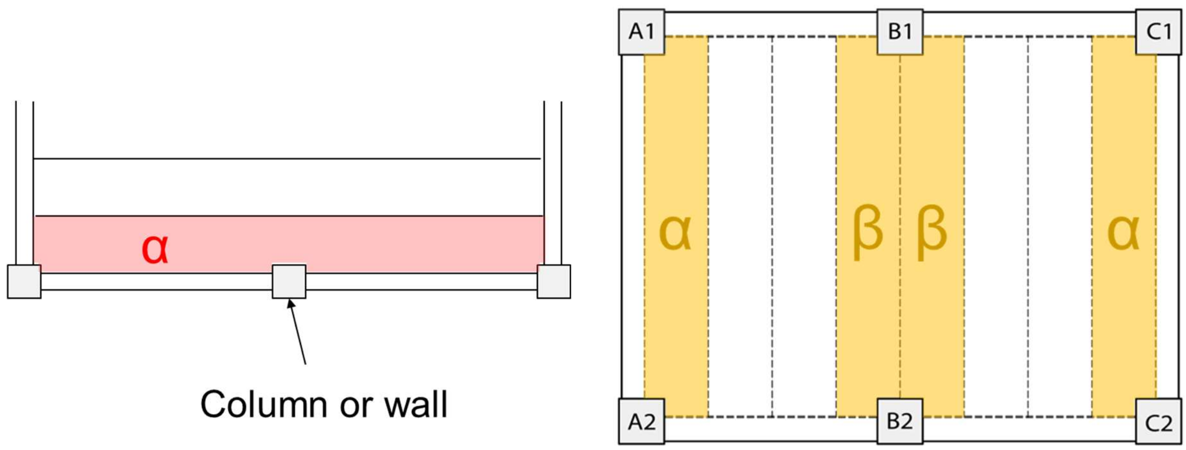
In addition to the deformation checks described in Table 1, the issues illustrated in Figure 2 need to be addressed to ensure the effectiveness of the retrofit. Connections between the existing structure and added elements should therefore be detailed to account for differences in the deformed shape of both systems. In general, engineers should focus on the deformation capacity rather than force capacity (except in the context of capacity design, of course).

3.2 Floor and Diaphragm Capacities

As described above, one of the key challenges in retrofitting New Zealand buildings is that precast concrete floors, in particular hollow-core floors, sustain damage at drifts that can be less than 1% depending on the failure mode. The 2018 Appendix E of C5 provided drift capacity based on the best available data at the time of publication, however the completion of the ReCast Floors project has provided new data (Brooke et al., 2022; SESOC, 2021; Büker et al., 2022) that is being incorporated in the proposed update to C5 being released in 2024. One of the most challenging criteria for assessment in the 2017 version is that units that are expected to fail in a brittle failure mode (i.e. web-splitting failure, loss of support, negative moment failure, etc.) must have their drift capacity divided by a factor of two and be compared to unfactored Ultimate Limit State (ULS) drift demands. This factor of two effectively limits allowable drift demands for NMF or web-splitting of alpha units (see Figure 9) to 0.5% drift at ULS. As per, additional data from the ReCast Floors project, there is now a proposal to change this reduction factor from 2 to 1.5. Additionally, for further understanding of the vulnerabilities of beta units (see Figure 9), the update to C5 is proposing the following limits in Table 2. As these limits will be divided by 1.5, drift capacities will range between 0.67% - 1.33%. It is recommended that targeted retrofits be considered in which all category 1 units be retrofitted as buildings typically have few of these units and the proposed retrofits (SESOC, 2022) are minimally intrusive. For category 2 and 3 retrofits, it is recommended to minimise the number of units requiring retrofit through either the reduction of storey drifts through a global retrofit (e.g. addition of walls, braces, or base isolation), or for lower seismic hazard regions where the expected risk is lower (e.g. Auckland) determine if the floors are likely to exceed these drifts.

Table 2: Classification of hollow-core units to experience web cracking failure.

Category	Drift Limit	Description
1	1.0%	**Highest potential for web cracking** - Alpha units (defined in Figure 9) that span past a vertical element (e.g. column or wall) - Units controlled by NMF
2	1.5%	**Moderate potential for web-cracking** - Other alpha units not in Category 1 and without link slab - Beta units - Units subjected to significant torsional demands
3	2.0%	**Lower potential for web-cracking** - Other hollow-core units



(a) (b)

Figure 9: Definition of critical hollow-core units susceptible to web cracking failure. Note: Alpha unit = hollow-core unit spanning between corner columns, Beta unit = hollow-core unit spanning between intermediate columns.

3.3 Japanese Guideline Drift Capacity Method

The Japanese retrofit guidelines appear to have been effective in multiple earthquakes (Appendix A4). The guidelines being planned can benefit much from the methods used in Japan. For instance, Japanese methods to estimate drift capacity are relatively simple, and have ranges of applicability that have been extended to retrofitted elements. Specifically, the Japanese method for estimating drift capacity of columns is a comparison of the minimum calculated shear capacity and the calculated flexural capacity. Ultimate displacement ductility is expressed as follows:

$$1 \leq 10\left(\frac{Q_{su}}{Q_{mu}} - 1\right) \leq 5 \qquad \text{(Eqn. 1)}$$

Where Q_{su} is the shear strength of the column, and Q_{mu} is the shear associated with the flexural strength of the column.

In Japanese codes, yield drift ranges from 0.4% to 0.67% based on the aspect ratio of the element. The ACI 369 column database used in Section 2.2.2 was used to evaluate the described methodology of the Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings. The results are shown in Figure 10. The methodology was modified to use New Zealand formulation for shear strength based on C5 and the restriction on the maximum tensile reinforcement was removed. This preliminary inspection of the reliability of this expression suggests that the scatter in plots of measured vs calculated drift capacity is as large as the scatter observed for the methods in C5. In that respect, the advantages offered by this expression that are relevant to this document are:

1. Eqn. 1 is relatively simple.
2. Eqn. 1 has been used for retrofitted elements, and field evidence suggests that such use has been successful in Japan.

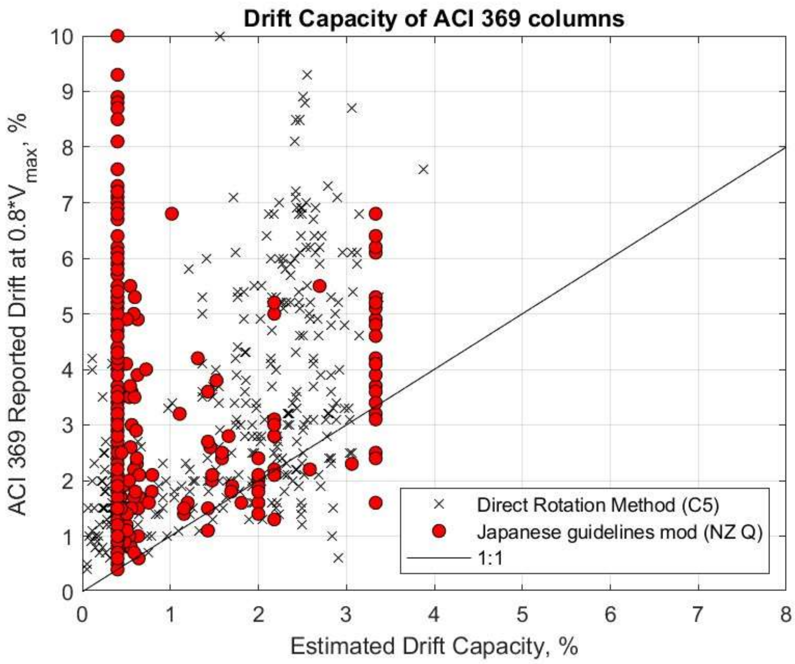


Figure 10: Comparison of scatter in the estimated drift capacity of RC columns using the direct-rotation method and the Japanese guidelines.

3.4 Proposed Alternatives to Estimate the Deformation Capacity of Retrofitted RC Elements

Considering the advantages and disadvantages of the alternatives described in Sections 2.2.1 and 3.3, three ways to address the issue of the deformability of retrofitted elements are put forward here ***to seek input from readers***:

1. Use capacity design to preclude brittle modes of failure (e.g. shear failure, bond failure, and axial failure).
   - Use a ratio of capacity to demand of at least 1.35.
   - Retrofitted components should not be expected to resist axial loads exceeding 1/3 of the axial capacity calculated considering the retrofit (e.g., a steel jacket). This

condition can be bypassed if the engineer can demonstrate that the structure can redistribute loads in case of axial failure.

   - The formulation described in Section 3.3 would indicate that the suggested capacity-demand ratio would lead to drift capacities exceeding 2%. Given that a) drift is accommodated by both vertical and horizontal elements in most situations, and b) compliant new RC elements have drift capacities comparable with the mentioned projection of 2% (Pujol et al., 2022), this approach would seem both simple and reliable.

2. Use capacity design, as in 1). A ratio of capacity to demand smaller than 1.35 can be used as long as the drift capacity of all relevant elements is checked.
   - For retrofitted elements, detailed checks on drift capacity could be prescribed with adaptation(s) of the expressions in C5. If the readers of this paper so choose, effort shall be invested to try to develop those adaptations, but that process can take time. Methodical research would be required, and implementation of this alternative may need to be done through updates to an initial –and more basic- version of the new guidelines for RC retrofit.
3. Use capacity design as in 1) and a ratio of capacity to demand smaller than 1.35, and let the engineer justify their choice on the basis of experimental evidence or available literature. This option would require peer review.

The authors suggest that options 1, 2, and 3 should be deemed adequate substitutes for requirements of detailing specified in standards for new RC construction (e.g. NZS3101). That is, to simplify retrofit design and construction, no 'strain-or curvature-limits,' for instance, and other similar limits imposed on the design of new structures would be imposed on retrofitted RC elements.

Option 1 is attractive to the writers because it can be justified on the basis of Japanese practices that have been observed to be successful in the field in multiple instances. Nevertheless, a few notes regarding the potential implementation of this option are in order:

- The listed options <u>refer exclusively to retrofitted elements</u>. Estimating deformation capacity for un-retrofitted elements shall keep pertaining to the assessment methods in C5.
- The designer may choose to stiffen the structure to lower deformation demands and reduce the number of elements needing retrofit. The designer may also ignore the lateral stiffness and strength of the original structure, as described in Table 1, for the same purpose. For squat walls (with aspect ratios smaller than 1 to 2), Option 1 may not work as it is difficult to get such walls to yield in flexure before reaching high shear stresses. A different criterion is needed for such walls if they are to be considered to be part of the lateral force resisting system.
- In general, in retrofitted elements required to contribute to the lateral resistance of the building, modes of failure expected to reduce drift capacity below the limit implied by Eqn. 1 and Option 1 would need to be precluded through retrofit. For instance, a wall with low amount of longitudinal reinforcement would need to be strengthened (e.g. with addition of sprayed concrete overlay with additional vertical bars) to ensure its flexural strength is larger than its cracking moment (associated with the tensile strength of concrete). Similar considerations apply to retrofitted elements with short lap splices, and retrofitted elements prone to out of plane buckling (e.g. thin and singly-reinforced walls which may need to be thickened or strengthened in their lateral directions or be retrofitted with boundary elements).

- The retrofit design method should allow the retrofit of elements to prevent loss of vertical-load carrying capacity without enforcing requirements related to lateral deformation capacity. In such case, the retrofitted components should not be considered as part of the lateral-force resisting system.
- Other issues that the guidelines need to address are requirements for the strengthening of beam-column and slab-column joints, as well as specifications for retrofits to delay buckling of longitudinal bars and retrofits to confine lap splices.

For the purpose of calculating a value of %NBS for the retrofitted structure, an option such as Option 1 could automatically lead to a value of 100% for each element meeting its requirements. Deviations from those requirements would be associated with reduced values of %NBS. As an alternative, Eqn. 1 could be used as a means to obtain estimates of drift capacity for retrofitted elements that can be used to produce values of %NBS.

3.5 Alternative Retrofit Solutions

To lower costs and speed up the retrofit process, the profession needs to explore the use of retrofit alternatives beyond what is being used today.

Appendix A2 describes options being explored to address brittle columns with limited accessibility (A2.1) and precast floors (e.g., hollow-core floors) for which fixing every single precast unit is prohibitively expensive (A2.2). Other alternatives, some of which have been used successfully overseas, are described in the preceding literature review.

3.6 Geotechnical Considerations

Soil and foundation retrofits are generally both expensive and disruptive and therefore it is pragmatic to consider the building and foundation system holistically and determine whether deformation at the soil and foundation level requires intervention, or alternative strengthening of the structure can achieve the desired result (FEMA 547 2006) (Roeder et al. 1996). The allowable foundation deformation is a function of the deformation capacity of the supported structure, and the overall stability of the building. For instance, a ductile structure can tolerate larger foundation deformations than a brittle structure, but overturning of the building must be resisted in both types of structures. In the case where soil or foundation retrofits are considered necessary, the selection of available options should balance the following site-specific issues: access and height restrictions for equipment, noise and vibration limits, restrictions from existing utilities, restrictions from ongoing operations, dealing with contaminated soil (FEMA 547 2006).

Geotechnical solutions are dependent on-site characteristics, foundation type, and retrofit objective. Prescriptive recommendations for foundations may be out of scope for retrofit guidelines pertaining to RC structures, so existing studies, standards, and guidelines like FEMA-547 (2006) should be used at the discretion of a geotechnical engineer. Nevertheless, the following are general considerations and techniques that are applicable to all foundation types:

- Site investigations, including determining foundation type, geometry, and strength (through existing building documentation or test pits), and reducing uncertainty in soil properties and structural capacity (through in-situ testing of soil),
- Soil-foundation flexibility considerations, where rocking foundations, sliding foundations, and load redistribution may lead to reduced structural demands,
- Ground improvement techniques, and

- Techniques to increase substructure rigidity and strength

Where new members are intended to resist lateral and vertical loads, supporting new structural elements can be achieved by:

- Connecting to existing the foundation (assuming the existing foundation is sufficiently robust to resist the additional loadings of the new elements)
- Adding pad foundations to resist overturning
- Adding driven or cast-in-place piles (where space and access is not a concern)
- Installing micro-piles or screw piles (where space is limited)

The following are considerations and techniques applicable to specific foundation types:

- For shallow pad and strip foundations:
  - Bearing area can be increased by casting additional perimeter pads around the existing footings, connecting isolated footings, or completely replacing footings.
  - Footing strength can be increased by adding supplementary piles (typically with new pile caps tied to the existing footing)
  - Alternatively, a footing may be relocated to a deeper or shallower depth to increase capacities, load testing can be conducted to confirm capacities of existing foundations, or foundation elements can be strengthened as covered in the above discussion on supporting new structural elements.
- For shallow mat and raft foundations:
  - The foundation can be converted to a piled raft by drilling through the existing foundation and installing piles (typically micro-piles), or
  - Alternatively, cutoff walls can be installed to reduce uplift forces and deformations (Liu and Song 2006) or ground improvement (like compaction grouting) can be completed through a slab if there is sufficient headroom in the basement of the structure.
- For deep foundations:
  - Load testing of existing piles can be conducted to reduce uncertainty in load and displacement capacities as well as pile stiffness, or
  - Additional piles can be added adjacent to existing piles and tied to the existing foundation by expanding the pile cap.

Given the complexity and potential cost of foundation retrofits, close coordination between the geotechnical and structural engineers is essential in determining if, and to what extent foundation strengthening is required for the target performance of the retrofitted structure.

4. Case Studies

To investigate the outcomes of the two approaches specified in Section 3.1, two case studies were produced. These case studies examined two buildings, and retrofits were selected ignoring or considering the contribution of the existing structure. The cases were selected to illustrate the effects of stiffening a building, in one case, and the effects of increasing member capacities to meet deformation targets, in the other.

In each case study, retrofit options were formulated, and each option was subjected to 1) static lateral forces according to NZS 1170.5 (for design purposes) and 2) nonlinear dynamic analyses for ground motions scaled according to NZS 1170.5 Section 5.5 (2004) and the New Seismic Hazard Model (NSHM) (GNS & MBIE, 2022) as represented in Draft Technical Specification TS1170.5 (2024) as available at the time of writing. Scaled ground acceleration records from:

- 1940 El Centro (RSN6 Imperial Valley-02, 5/19/1940, El Centro Array #9),
- 1999 Kocaeli (RSN1148 Kocaeli Turkey, 8/17/1999, Arcelik),
- 1999 Chi-Chi (RSN1504 Chi-Chi Taiwan, 9/20/1999 TCU067), and
- 1999 Duzce (RSN 1605 Duzce Turkey, 11/12/1999 Duzce)

were selected from the PEER NGA Strong Motion database (Mazzoni & Way, 2013) to represent plausible ground motions.

4.1 Red Book Building (Retrofit Design Ignoring Existing Structure)

The first case study is based on a modified version of the RC moment-frame building described in the 1998 “Red Book” *Examples of Concrete Structural Design to New Zealand Standard 3101* (Bull & Brook, 2008), which was a training tool commonly used in NZ during the moment frame era.

The “Red Book” frame building is notable for having floors made with hollow-core precast units, which are known to be susceptible to damage at low drifts. For illustration purposes, it is assumed that the building is located in Wellington instead of Christchurch, as assumed in its original design (Bull & Brooke, 2008). In addition, to improve relevance to this paper, columns are redesigned with deficient detailing. External retrofit frames are sized to meet different values of %NBS without considering contributions from the existing structure (as per the second option described in Section 3.1).

4.1.1 Building Description

The building has ten storeys, is regular, nearly symmetrical and -as designed- has detailing and proportions conforming to New Zealand concrete standard NZS 3101 except for the mentioned modifications introduced for columns. Details are provided in Appendix A5. The period of the building is approximately 2 seconds for "Effective" moments of inertia (I_e) calculated for cracked concrete cross sections (and approximately 1.4 seconds for gross cross sections).

4.1.2 Building Assessment

The building is assessed here using nonlinear dynamic analysis given the following alterations to the original design assumptions: the building is assumed to be located in Wellington (on a site subsoil class C) and the amount of column transverse reinforcement is less than what is prescribed in the Red Book. Figure 11 shows the original and modified column transverse reinforcement for Ground-1st Floor columns. Modified columns have fewer ties of smaller bar diameter with wider spacing.

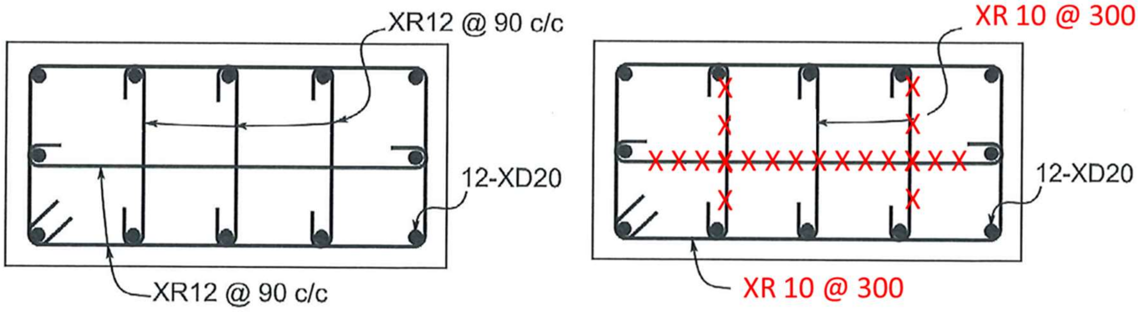


Figure 11: Original (left) and Modified (right) Red Book Columns. Crosses indicate omitted ties.

For simplicity, design forces were estimated using the equivalent static analysis (ESA) specified in NZS 1170.5 section 5.2, and a reference spectrum (for a damping ratio of 5%) shown in Figure 12.

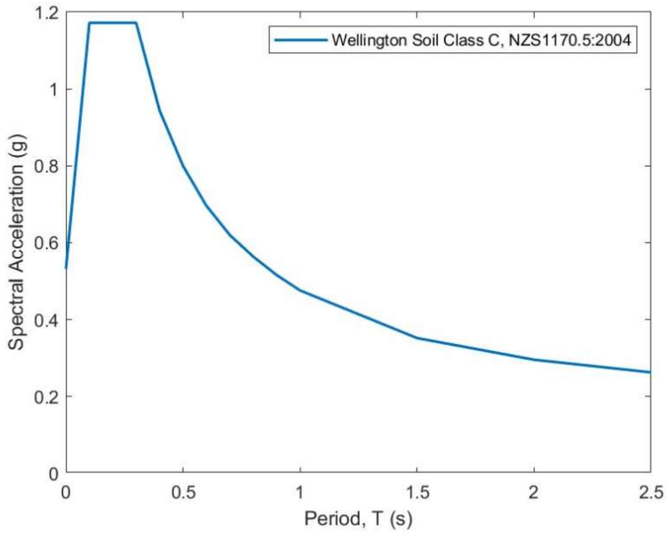


Figure 12: ULS Design Spectra for Wellington Soil Class C using NZS 1170.5 (2004) (for a damping ratio of 5%)

This spectrum refers to what the New Zealand standards call the 'ultimate limit state.' Retrofit variations comprised Eccentrically Braced Frames (EBF) with members sized to resist equivalent lateral forces estimated for the described spectrum at 1.6 s (obtained in the Red Book documentation as $T=0.11h_n^{0.75}=1.6$ s to provide a common reference to the different retrofit alternatives studied here)[4], a 'Ductility Factor', μ of 4, a 'Structural Performance Factor', S_p of 0.7, and three differing levels of demand. These demands are obtained by taking 34%, 68% and 100% of the equivalent lateral forces (reduced using a factor of $S_p/k_\mu = 0.7/4$).

To resist the resulting forces, three structural alternatives are considered (Figure 13):

[4] It may seem reasonable to estimate %NBS on the basis of an estimate of period and damping consistent with the properties of the retrofitted structure considering all its elements (existing and added). Nevertheless, it is not obvious whether current regulations require that explicitly. It could be argued that doing so would penalize decisions to stiffen a structure as well as decisions to consider the lateral strength and stiffness of the existing elements if %NBS is calculated in terms of force. Calculating %NBS in terms of displacement would avoid this issue and would therefore be preferrable. Section A.10 of the assessment guidelines seems to address the question of how to define period for the retrofitted structure suggesting different approaches depending on the type of analysis chosen. The writers would like to suggest that more specific guidance is needed. The guidance should consider the relative nature of the %NBS index and the need to promote retrofits to control drift.

- Alternative B: one EBF on each exterior face,
- Alternative C: two sets of EBFs on each exterior face,
- Alternative D: three sets of EBFs on each exterior face

Although EBFs were sized without considering contributions from the existing RC frame, nonlinear static and dynamic analyses were conducted using numerical models in which the original structure was considered to contribute to building stiffness and strength. This was done to get a sense for how effective different design targets may be in the context defined by Option 1 in Table 1.

Periods and base-shear coefficients obtained from nonlinear static analyses performed considering the contribution from the original structure are shown in Table 3 T1 is first-mode period, V_y is base shear at yield of the first element, V_u is the maximum expected base shear from nonlinear static analysis, and W is total building weight. The variations in period would suggest reductions in drift demands (for linear structures) of nearly 30%, 40%, and 50% (for Alternatives B, C and D and using one significant figure to acknowledge uncertainties). The rate of decrease in drift decreases –if drift is assumed proportional to period– while strength increases nearly linearly, indicating that the installation of the first retrofit frame has the biggest impact relative to the installations of the additional second and third frames. Four scaled ground motions were used in nonlinear dynamic analyses. The motions and the procedures used to scale them are described in Appendix A5. Geotechnical considerations are provided in Appendix A6.

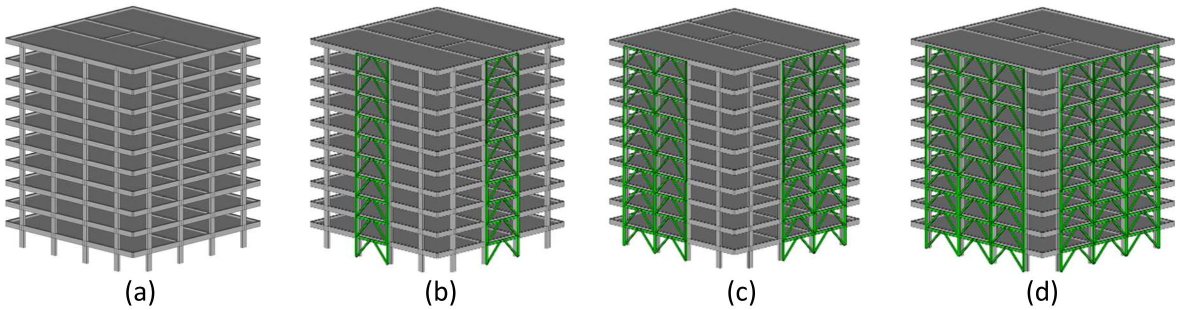


Figure 13: Building models (a) Unretrofitted case, and retrofitted buildings with EBF system for (b) 34%, (c) 68%, and (d) 100% NBS.

Table 3: Dynamic properties and results from the seismic design of the unretrofitted and retrofitted models.

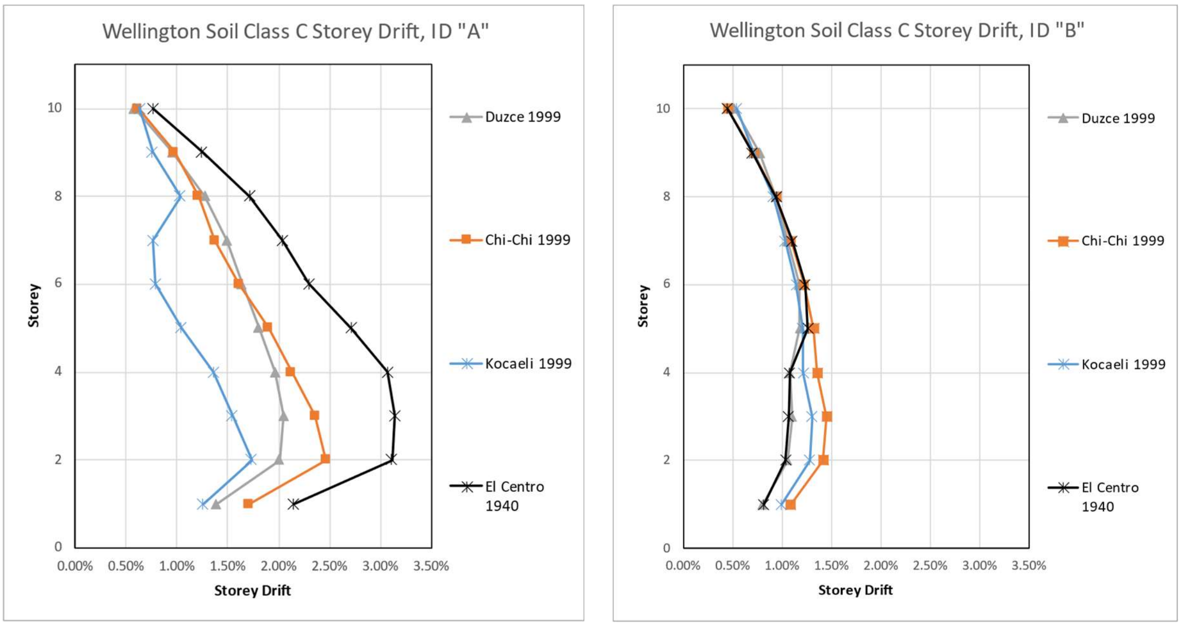
Case	ID	T_1 (s)	V_y (kN)	V_y/W	V_u (kN)	V_u/W
RC Frame	**A**	2.0	5000	0.08	5500	0.09
RC + 34%NBS EBF	**B**	1.5	6500	0.10	9000	0.15
RC + 68%NBS EBF	**C**	1.2	8000	0.13	11000	0.18
RC + 100%NBS EBF	**D**	1.1	10000	0.16	12500	0.20

Figure 14 illustrates peak values of storey drift ratio estimated using nonlinear dynamic analyses for the scaled ground motions used (see Appendix A5). The means of the maxima in each plot indicate reductions in deformation demands of ~45%, for alternatives B and C, and ~60% for alternative D. In general, the estimated drift maxima are approximately proportional to initial period (see Figure 16). This is despite the nonlinearity of the system, and variations among records. This observation is convenient in design (Sozen, 2003), as it allows the engineer to rapidly judge the effect of retrofit interventions without elaborate analysis.

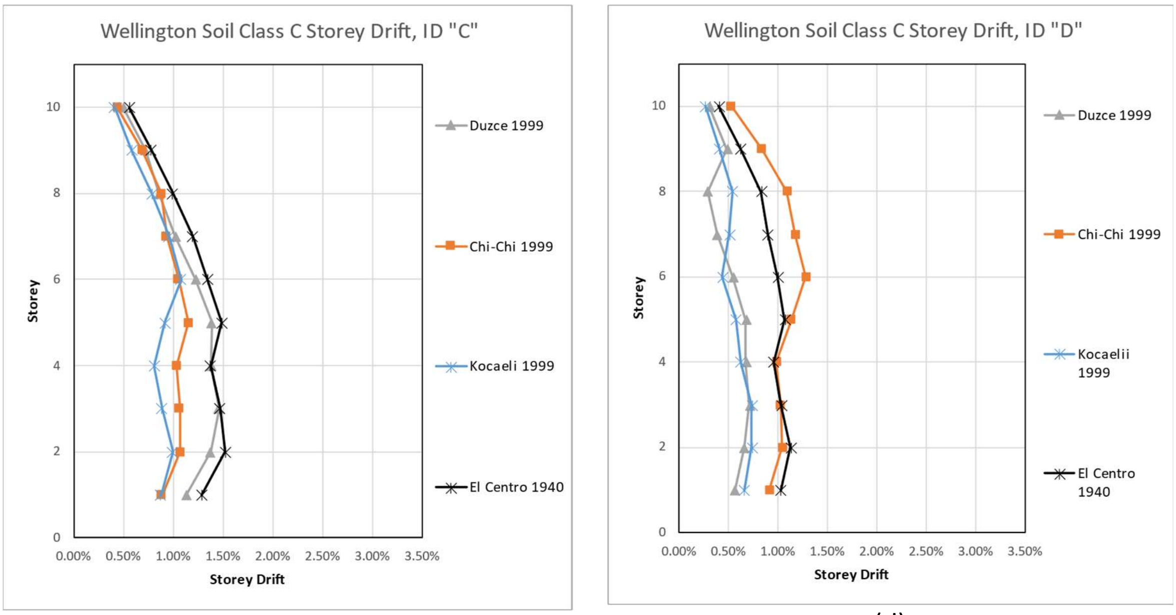
The variations in drift suggest that investing in Alternative B (one frame per elevation) produces the best return in terms of reduction in drift and potential for damage. Considering only the use of braced frames over the lower two thirds of the building height (rather than full height) may be the most economical alternative.

Figure 15 focuses on a single scaled record, instead of four records as in Figure 14, but considers two scaling factors: one to match the seismic hazard estimates currently considered (as prescribed in NZS1170.5, 2004, soil class C), and one to match estimates being released as draft TS1170.5 (assuming translation to soil class III). The stark differences in Figure 15 reflect that the bulk of the uncertainty in earthquake engineering is attributable to the estimates of ground motion intensity. To provide the relevant markets with stability, and to reflect current legislation in regard to earthquake prone building (which is tagged to 2017) no existing buildings are to be assessed against the draft TS1170.5. Similarly, the draft TS1170.5 is not being used for design yet. The new hazard estimates (in draft TS1170.5) are being introduced by MBIE for comment as to the impact on the design of **new** buildings **exclusively**. As per advice published by Engineering New Zealand (2021) "there are no plans for TS 1170.5 to be introduced into seismic assessment in the foreseeable future".

The comparison in Figure 16 is offered here only to illustrate that the methods and limits used to assess and retrofit a building produce not absolute but **relative** measures of the robustness of the building before and after retrofit. In that sense, at least in the initial phases of design, the engineer is better off considering retrofit alternatives on the basis of crude indices representing each alternative (e.g. building period, as done above) rather than on the basis of results from elaborate analyses. Rather than running exhaustive sweeps of analyses of complex numerical idealizations, the retrofit designer's time is better spent determining connection details and working with contractors to ensure constructability.

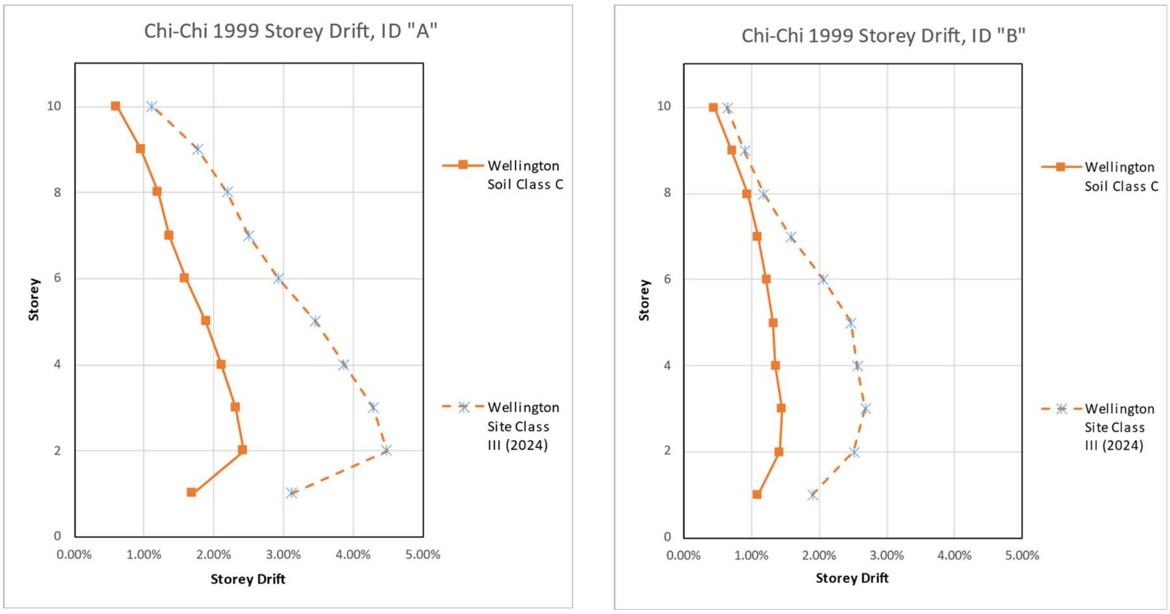


(a) (b)

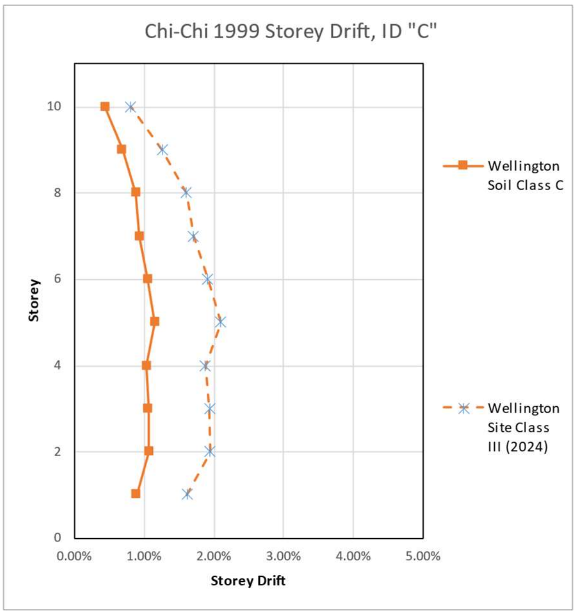


(c) (d)

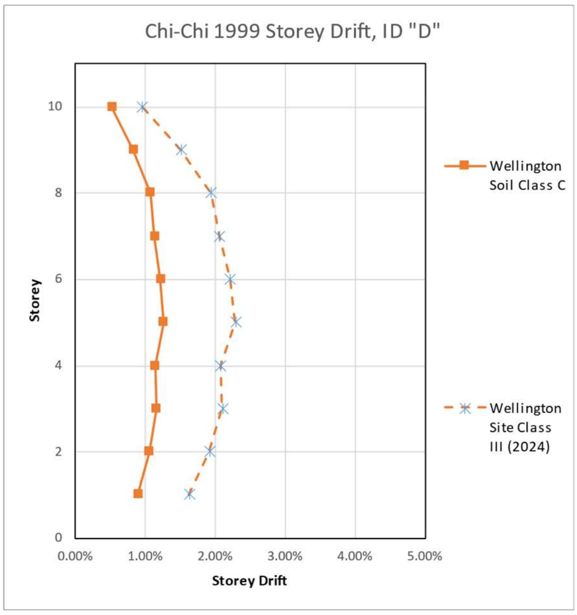
Figure 14: Storey drifts for Alternatives (a), (b), (c), and (d), subjected to the ground motion suite in Table 12.



(a) (b)



(c)



(d)

Figure 15: Storey drifts for the four retrofit alternatives (a), (b), (c), and (d) subjected to different intensities of shaking as represented by one scaled ground motion record.

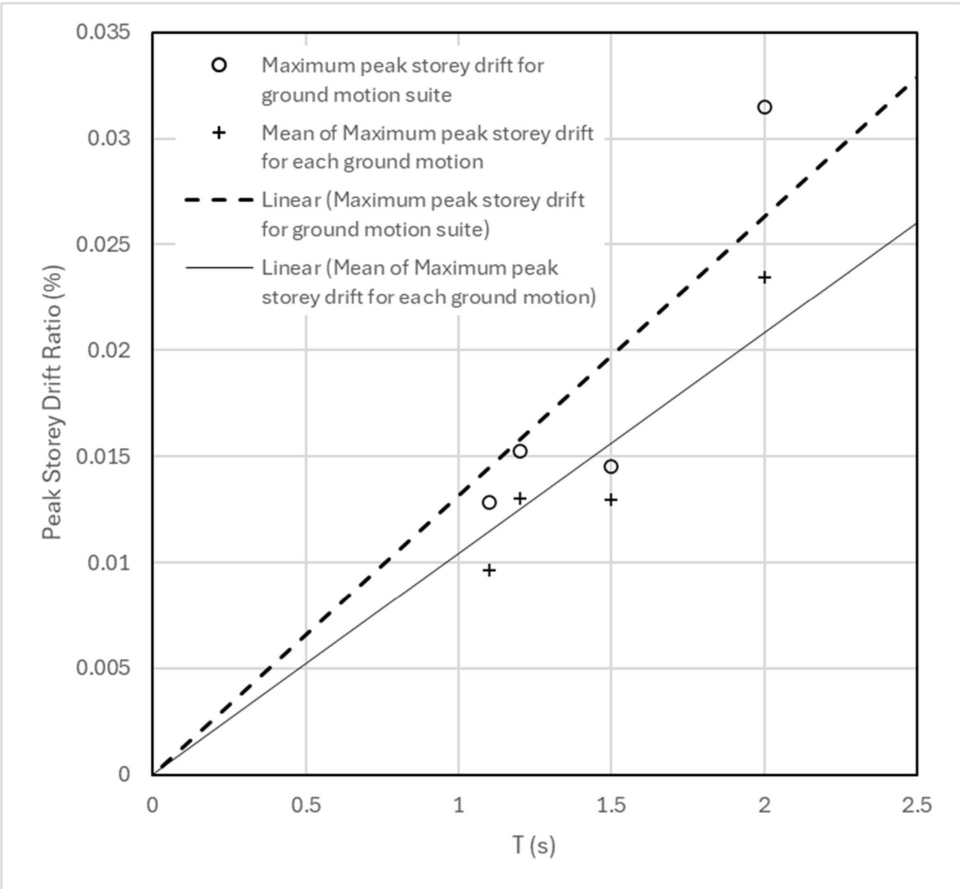


Figure 16: Peak storey-drift ratios vs. period.

To consider the potential for structural damage associated with the estimates of drift discussed above, consider Table 4, which lists estimates of the drift the capacities (Δ_{cap}) at lateral failure of the unmodified and modified (with reduced transverse reinforcement) Ground-1st floor columns. The table also lists estimates of drift at yield (Δ_y) and drift at axial failure (Δ_f). The modified columns (with reduced transverse reinforcement) have calculated capacities that achieve yield rotations but with relatively narrow margins against lateral and axial failures post yield.

Table 4: Capacities of Red Book Ground floor column

ID	h(x)	b(y)	$P/A_g f'_c$	ρ_l	$\rho_{t,x}$	$\rho_{t,y}$	$V_{y,x}$	$V_{y,y}$	$\Delta_{y,x}/L_c$	$\Delta_{y,y}/L_c$	$\Delta_{cap,x}/L_c$	$\Delta_{cap,y}/L_c$	$\Delta_{f,x}/L_c$	$\Delta_{f,y}/L_c$
	mm	mm	%	%	%	%	kN	kN	%	%	%	%	%	%
Original	900	460	31	0.9	0.8	0.7	1100	600	1.1	0.8	1.9	3	2.8	4.5
Modified	900	460	31	0.9	0.1	0.1	1100	600	1.1	0.8	1.3	1.2	1.9	1.9

Note: Symbols are defined in the Glossary Section.

The estimates in Table 4 and Figure 15 can be combined with the procedures described in Appendix A3, to provide a sense of the implications of uncertainties related to a) the ground motion and b) the limitations of our design and assessment methods. Table 5 lists estimates of peak first-storey drift ratios extracted from Figure 15, and ratios of those values to the drift capacity estimates for Reduced Capacity columns in Table 4. This is done, for illustration purposes, assuming drift concentrates in columns. Drift, of course, can be accommodated by columns and beams, both. In addition, drift ratio is seldom highest in the first storey, and one could look at averages instead of maxima. In that sense, the values in Table 5 are provided here only a) to suggest a format to consider deformations and risk, and b) to illustrate the relative effects of design choices and decisions related to policy instead of engineering. With those critical caveats in mind, Table 5 also includes estimates of probabilities of lateral and axial failures in ground-level columns obtained following Appendix A3.

Table 5: Probabilities of failure for Red Book ground floor columns (with modified transverse reinforcement)

Building ID	Peak 1st storey Drift demand	Lateral Load		Axial Load	
		$\frac{demand}{capacity}$	P_{fail}	$\frac{demand}{capacity}$	P_{fail}
	%		%		%
hazard defined by NZS1170.5 (2004)					
A (NZS)	2.2	1.7	56	1.2	9
B (NZS)	1.1	0.9	14	0.6	0
C (NZS)	1.3	1.0	23	0.70	1
D (NZS)	1.0	0.8	12	0.6	0
hazard defined by TS1170.5 (2024)					
A (TS)	3.9	3.1	89	2.1	38
B (TS)	2.0	1.6	50	1.1	6
C (TS)	2.3	1.9	62	1.3	11
D (TS)	1.9	1.5	46	1.0	5

All retrofit alternatives produce ground-storey drifts below the calculated drift capacity at loss of axial load when considering ground motions scaled to NZS 1170.5 (limit state 'ULS'). For ground motions scaled to reflect the new hazard estimates prescribed by TS 1170.5 (2024), only retrofit alternative “D” has demand to capacity ratios smaller than or equal to 1. Despite this, the probabilities that ground floor columns might lose axial load-carrying capacity are lower than 10-15% in all cases where retrofits were applied. These probability estimates, given the crude assumptions behind them, should be interpreted as indices rather than rigorous estimates. The numbers, however, can be used to make the case that our efforts to retrofit the existing building inventory are unlikely to eliminate risk. This picture is compounded by observations that drift ratios exceeding 1% are likely to require repairs to partitions with costs exceeding the cost of complete partition replacement (Algan, 1982). The reader is also invited to consider the drift estimates in Table 5 in relation to the limits in Section 3.2 that would indicate high risks associated with the performance of precast floor units. Despite all of that, relative to one another, the values in Table 5 suggest that any intervention is much better than no intervention in a vulnerable structure, and that idea supports the current approach being promoted by MBIE to favour measures that help control the costs of retrofit.

A final note regarding this example is due. The period of the structure in Alternative B (with a single braced frame on each elevation), calculated ignoring completely the stiffness of the original structure, is approximately 2.3 seconds. For that period, the maximum storey drift demand, on average, would approach 3.0% (Figure 16). The ratio of that value to the estimated drift at axial failure (Table 5) would therefore be close to 1.6, which would be associated with probabilities of axial failure of up to ~20% according to Appendix A3. That is again, if drift is assumed to be accommodated by columns only, which would be an exaggeration. It would benefit the engineer to sharpen their approach by a) considering the contribution of the original structure to lateral resistance, and b) using end rotations to make decisions. Those options would be better than simply checking deformations for a reduced level of demand consistent with the targeted strength of 34% of NBS. When it comes to issues as critical as column axial failure, comparisons made with respect to design actions and ground motions scaled to 100% NZS 1170.5:2004 would better communicate risk to clients.

4.2 Wellington “Indicator building” (Retrofit Design Considering Existing Structure)

Ghasemi and Stephens (2022) conducted analyses of the Wellington Building Inventory compiled by Puranam et al. (2019) and selected “indicator buildings” with details typical to specific clusters of buildings in Wellington. In total, 5 dominant building clusters were identified, and 9 indicator buildings were created to be representative of the identified clusters (Ghasemi & Stephens, 2022).

The purpose of the second case study is to select a retrofit for one of the archetypal buildings identified in these previous research reports. The first building cluster identified by Ghasemi and Stephens includes RC buildings constructed in the 1960s with heights ranging from 25 to 30m and a combined core wall and moment frame lateral force resisting system. Rather than assessing the created indicator buildings, a building that fits the criteria of the first building cluster was selected.

In this case, the Munro building (104 The Terrace, Wellington CBD) was selected for assessment because 1) it falls within the ranges of variables identified for building cluster 1, 2) it has been identified by the WCC as earthquake prone, and 3) its location opens discussion on the differences between NZS 1170.5 and TS 1170.5 (which was open for public comment in February 2024) when determining design actions for building retrofits.

4.2.1 Building Description

The Munro building is a 9-storey RC office building designed in 1962, with a combined RC frame and wall system providing lateral load resistance. Details are provided in Appendix A7. The ground floor is taller and has fewer columns than the next 7 storeys, which may promote a soft-storey mechanism. Effective building periods are 1.3s in the short plan (NS) direction and 1.1s in the long plan (EW) direction. The building has an inherent stiffness imbalance in its long direction, with the main structural wall butted against the South edge of the floorplan, which will produce pronounced torsion.

4.2.2 Building Assessment

The building is assessed here using nonlinear dynamic analysis based on assumptions and building dimensions specified in Appendix A7. Table 6 lists estimated initial periods for cracked sections and estimated base shear coefficients. Refer to the Glossary for definitions.

Table 6: Fundamental periods and base shear estimates for Munro building

							Retrofitted	
Case	ID	T_1 (s)	V_y (kN)	V_y/W	V_u (kN)	V_u/W	V_u (kN)	V_u/W
Long Direction	x	1.1	7000	0.2	6500	0.2	15000	0.43
Short Direction	y	1.3	5000	0.14	5000	0.14	10000	0.29

The location for the Munro Building in Wellington is identified as Soil Class B (Shallow soil sites) according to the Wellington City Council map. The soil classification system in NZ is changing as this document is being produced. In standard 1170.5, the site at hand, would be classified as Site Type B. In the newly introduced technical specification TS 1170.5, the site would be classified as Class II s. Figure 17 shows a comparison of the acceleration spectra specified for the site by a) NZS 1170.5, b) TS 1170.5, and c) the 2022 National Seismic Hazard Model (NSHM).

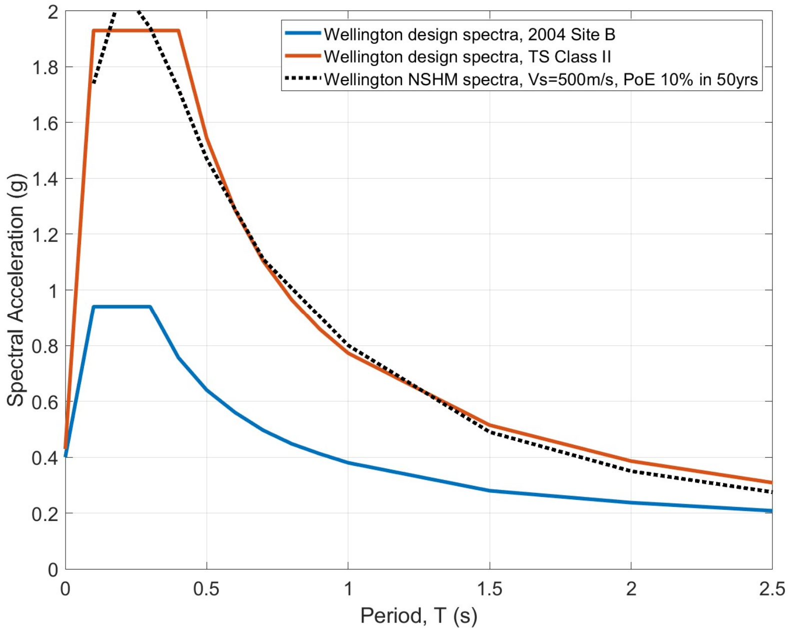
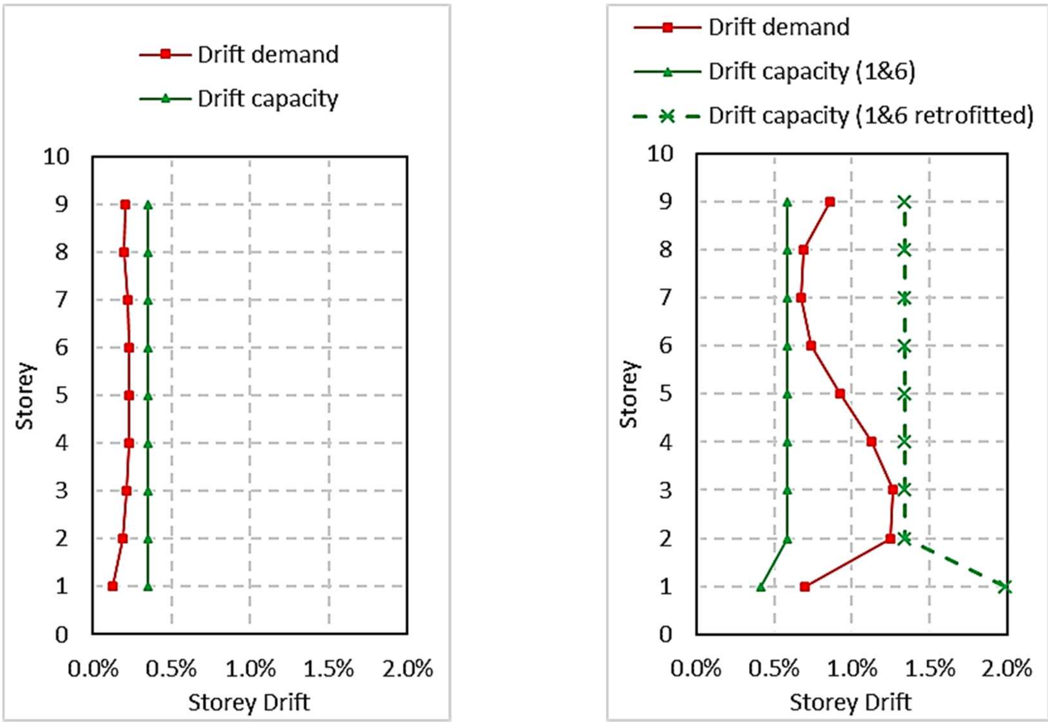


Figure 17: Acceleration spectra for the Munro Building (for a damping ratio of 5% and a nominal return period of 500 yr.).

From Figure 17, it can be seen that 1) the TS 1170.5 design spectrum is significantly higher than the NZS 1170.5 spectrum, and 2) the design spectra from TS 1170.5 and the NSHM are nearly identical. Four ground motion records were scaled as described in Appendix A7 to match these spectra. The scaled records so produced were used as input for the nonlinear dynamic analyses mentioned before. Figure 18 illustrates examples of storey-drift demands (in the long floorplan direction) obtained from these analyses for a typical exterior column along the North building facade Figure 18. Drift capacities obtained using standard C5 for the same column and in the same direction are plotted along the described drift demands. These estimates suggest that an initial goal of a retrofit plan should focus on the exterior columns before focusing on the structural wall(s). In this exercise, it is assumed that RC jackets can be used to increase the drift capacity of the existing columns. Appendix A7 shows a plausible retrofit scheme (Figure 33), which is associated with estimates of drift capacity varying with the details of the column section as shown in Figure 18b. For the sake of argument, these capacities were estimated using the moment-curvature procedures in C5 despite the uncertainty about their applicability discussed in Section 2.3. In contrast, long-direction wall drifts are less than the drift capacity estimated again using C5, indicating that wall retrofits may not be as critical. Nevertheless, a detailed study about the ductility of the reinforcement in the wall and the need for confinement, especially in wall boundary elements, would be prudent.



(a) (b)

Figure 18: Expected drift demands and estimated drift capacities, (a) long floorplan direction, wall (Gx), and (b) long floorplan direction, Columns 1 (ground floor) and 6 (upper storeys). Chi-Chi 1999 record, scaled to match 1170.5 (2004) for the ultimate limit state (ULS).

A comparison of storey drift demands obtained for the short floorplan direction and for ground motions scaled to NZS 1170.5:2004 and TS 1170.5 (2024) are shown in Figure 19. The plotted values correspond to the centroids of consecutive floorplans. The comparisons illustrate again that ground motion uncertainty represents the bulk of the uncertainty in the retrofit problem. And that serves as a reminder that the goal of the retrofit process is to improve buildings by addressing their most critical vulnerabilities instead of producing systems free of risk. Notice, nevertheless, that the spectra in Figure 17 are different by a factor close to 2, and so are the drift demands in Figure 19. This observation supports idea that moderate drift (not exceeding 4%), in the absence of structural failures, is nearly proportional to intensity and period, which is an observation that can help shorten the design process.

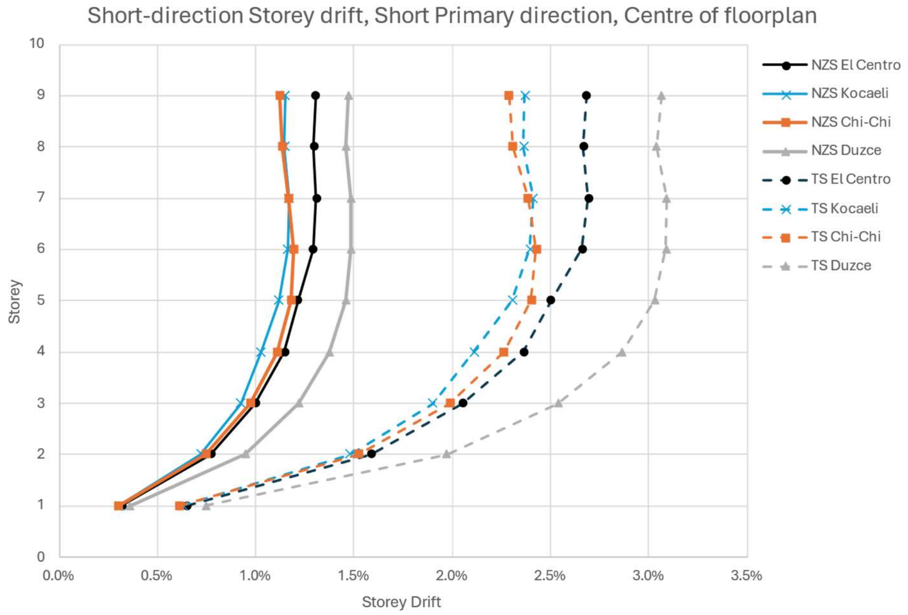


Figure 19: Differences in the drift profiles based on the use of NZS1170.5 or TS1170.5 scaling.

Table 7 and Table 8 include peak storey drifts (maxima a: along the building height for columns 5-8, and b: within the first storey for columns 1-4). The tables also list failure probability indices for each column type and for ground motions scaled to 2004 (Table 7) and 2024 (Table 8) standards. These probability indices are estimated using the procedure outlined in Appendix A3. The caveats mentioned in the previous case study in regard to these estimates apply here too. In this case, however, deep perimeter beams may help accommodate a smaller fraction of the drift than in the previous case. As can be seen in the tables, retrofitting significantly reduces the probability of failure for columns with shear or bond deficiencies (refer to Appendix A3 for details). Notice, nevertheless, that some columns remain rather brittle after retrofit. Those columns have small ratios of shear span to depth (<2), and the available formulations indicate their drift capacities are limited even when generous transverse reinforcement is provided. This difficulty illustrates two key points:

1. We need better methods to estimate the drift capacities of both conventional and retrofitted elements (including options that may go beyond traditional uses of mechanics and statistical regressions), and
2. Retrofit alternatives may be limited by the original configuration of the structure, which is not always something that can be altered easily. In this particular case, further risk reduction may require a large shear wall on the North facade of the structure.

Table 7: Drift demands and probabilities of column 'lateral' failure for NZS 1170.5 2004 scaling

Column ID	Peak Drift demand, x	$\frac{demand_x}{capacity_x}$	$P_{fail,x}$	Peak Drift demand, y	$\frac{demand_y}{capacity_y}$	$P_{fail,y}$
	%		%	%		%
Original Columns						
1	0.70	1.64	52	0.56	0.92	16
2	0.54	0.80	11	0.62	0.69	7
3	0.54	0.48	2	0.67	0.89	15
4	0.28	0.59	4	0.66	1.39	40

5	1.27	1.58	49	2.00	1.91	62
6	1.27	2.21	72	1.80	3.89	94
7	1.27	0.80	11	2.00	2.13	69
8	0.49	0.94	17	1.80	3.46	92
Retrofitted Columns						
1R	0.70	0.34	0	0.56	0.27	0
2R	0.54	0.32	0	0.62	0.35	0
3R	0.54	0.29	0	0.67	0.40	1
4R	0.28	0.17	0	0.66	0.41	1
5R	1.27	0.65	5	2.00	1.38	40
6R	1.27	0.95	17	1.80	1.34	38
7R	1.27	0.80	11	2.00	1.22	32
8R	0.49	0.28	0	1.80	1.04	22

Table 8:Drift demands and probabilities of column 'lateral' failure for TS 1170.5 2024 scaling

Column ID	Peak Drift demand, x	$\frac{demand_x}{capacity_x}$	$P_{fail,x}$	Peak Drift demand, y	$\frac{demand_y}{capacity_y}$	$P_{fail,y}$
	%		%	%		%
Original Columns						
1	1.42	3.33	90	1.15	0.92	61
2		1.65	52		0.69	41
3	1.11	0.98	19	1.37	0.89	59
4		1.20	30		1.39	79
5	2.58	3.20	89	4.17	1.91	95
6		4.48	97		3.89	100
7	2.58	1.62	51	4.17	2.13	97
8		1.94	63		3.46	99
Retrofitted Columns						
1R	1.42	0.68	6	1.15	0.27	3
2R		0.65	5		0.35	8
3R	1.11	0.60	4	1.37	0.40	12
4R		0.36	0		0.41	8
5R	2.58	1.32	37	4.17	1.38	85
6R		1.92	63		1.34	81
7R	2.58	1.62	51	4.17	1.22	80
8R		0.58	4		1.04	66

4.3 Cost and Constructability Challenges Affecting Retrofitting

Costs and constructability can dictate the outcomes of a retrofit project regardless of its structural engineering aspects. The associated constraints are site-dependent and addressing them requires input from contractors, estimators, and building owners to determine what is feasible and affordable. Constraints regarding access, disruptions to building function, and aesthetic requirements must be considered early in the project. The following is a non-exhaustive list of considerations affecting selection of retrofits:

- Material costs
- Fabrication costs
- Installation costs
- Costs and time associated with removal and reinstallation of cladding, windows, partitions, ceilings, and equipment
- Losses due to disruptions
- Fire code compliance
- Resealing building envelopes

All these considerations can add unexpected costs and can make retrofit alternatives prohibitively expensive. Early planning and communication are crucial to reduce increases in budget. With all this in mind, however, the engineering community must also make a concerted effort to quantify and communicate potential costs related to not doing anything to vulnerable buildings. Those costs are, of course, harder to quantify. But to the extent possible it would be helpful to alert clients of long-term (or life-cycle) projections of costs related to evacuation, prolonged building downtime, repair, and loss of property value, while also mentioning risk of collapse if appropriate. These matters can be presented in at least two ways: a) considering nominal probabilities of occurrence or exceedance of reference earthquake demands, and b) assuming that said reference motions will occur in the lifetimes of those concerned. The latter approach is seldom used, but it may yield figures that are easier to grasp than the often-small values produced by probabilistic methods accounting for uncertainties in ground-motion estimates. Depending on time and budget constraints, the commentary to the guide may provide guidance in this regard.

Other costs that the community must consider go well beyond each building as an isolated unit. We must admit that we face steep costs related to the potentially prolonged cordoning of urban areas. How much will it cost NZ to shift the operations occurring in Wellington until its buildings are repaired and or declared safe for reuse after a large and shallow earthquake close to its CBD? In that sense, the problem of financing of retrofits should not be left to owners exclusively. Engineers must work with owners, banks, insurers, and the government to find financial strategies to help the process. That task is as urgent as the proposed guidelines are.

5. Conclusions

The purpose of this white paper is to identify challenges related to the retrofit of RC multi-storey buildings in New Zealand. Preliminary alternatives to address the identified challenges are described, in order to seek feedback from engineers who would use a guide for RC retrofit design.

Through the literature review that preceded this white paper, several international standards addressing the retrofit of multi-story RC buildings have been identified. The guidelines being crafted for use in New Zealand shall be a compilation of adaptations of the best solutions available in those standards, as well as solutions specific to New Zealand as we have problems which are unique to NZ building stock.

The %NBS index used to rank buildings in New Zealand is not an absolute quantity but a relative metric of the 'robustness' of a building structure designed to identify the buildings that are most likely to perform the worst. Seeking to increase the lateral strength of a building, as an alternative to increase %NBS, must be accompanied by checks on the deformability of both the existing and the retrofitted structure. Minimum checks proposed for inclusion in the guidelines are listed in Section 3.1.

The evidence presented here shows that the tools available to estimate deformation capacity of existing elements produce results with much scatter and conservatism (Section 2.2.2). Engineers may want to consider the uncertainties illustrated here to select a method to estimate rotational capacities of existing elements. In the case of retrofitted elements, there are fewer procedures and data to produce reliable estimates of drift capacity. On the basis of successful Japanese experience, this white paper proposes several alternative approaches (Section 3.4), the simplest of which is to design element retrofits to prevent brittle failures through capacity design (Section 3.4. The implicit assumption in this approach is that preventing brittle failure modes (e.g. related to shear, bond, and axial load) leads to drift capacities exceeding expected demands. This approach removes the need for an explicit check on drifts, and thus navigates around the uncertainty in the tools we currently have available to estimate deformation capacity. Nevertheless, an alternative to estimate drift capacities for evaluation purposes has been proposed on the basis of Japanese practice.

Connections between added systems and the existing structure need to be designed to accommodate the differences between the deformed shapes of the existing and the new structural elements with ample margins.

For precast floors, if loss of support is mitigated (through supplemental seating or adequate existing seating) and expected storey drifts are smaller than 1% (for the full intensity of the design earthquake), evidence shows that life-safety risks are likely to be tolerable. It is difficult to retrofit precast flooring for more than life-safety without incurring major cost.

Research on retrofits that are cheaper, less invasive, and easier to implement is ongoing, and is likely to provide alternatives for retrofitting both columns with limited accessibility, and precast floors, where less intrusive construction is required.

The consensus among engineers who attended workshops informing this missive was that the retrofit systems in use in NZ today are the preferred systems that the new guidelines should support. Nevertheless, potential guideline users are invited to review the literature review accompanying this white paper because it offers alternatives that have proven affordable and effective in other countries. The "PITA-Column" system used in Japan and "Wing Walls" used in Taiwan and described in the literature review accompanying this communication are worth highlighting here.

Reconnaissance has shown that some retrofit is better than no retrofit, even when experience would suggest that the intervention may not result in acceptable performance. For example, a building in the Noto Peninsula (Japan) in which supplemental steel frames were added along only one building façade was observed to have acceptable performance despite the torsion introduced by the added frames. Of course, many buildings with deficiencies survive earthquakes. But the mentioned case is not isolated. Another example refers to the District Office Buildings in Taiwan which were retrofitted only in a single direction of their floorplan. In the perpendicular direction of these buildings, RC frames with continuous but unreinforced brick infill were relied upon for lateral stiffness and strength.

The case studies in Section 4 also suggest that initial interventions, even if they have limited scope, can have a large impact on reduction of vulnerability. The examples also show that, given the uncertainties and constraints affecting the retrofit problem, simple design approaches can guide the retrofit process as effectively as detailed analyses can.

Engineers must work with owners, banks, insurers, and the government to find financial strategies to help the retrofit process. That task is as urgent as the proposed guidelines are.

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APPENDICES

A1 Questionnaire Results

Questionnaires were handed out during each workshop, and results from each workshop are provided below. Responses reflect the opinions of the attendees of the three workshops, but also should be considered representative of the views of a diverse set of engineering firms, consultants, and academics. These results are meant to inform the format, contents, and decisions to include specific retrofit techniques. The following tables summarize the responses to questionnaires from the three workshops held in 2023.

Item on questionnaire - **General Guide Contents**	Number of responses	Average score (out of 5)
Studies/references to be included in the retrofit literature review	28	16 (Y) 12 (N)
1. Closed-Form expressions for deformation capacity of retrofitted elements	27	4.4
2. Machine-learning algorithms for estimating deformation capacity of retrofitted elements	26	2.5
3. Scatter plots of measured deformation capacity of "un-retrofitted" elements vs estimates produced by C5	28	2.8
4. Information on the sensitivity of Non-Structural components to building performance parameters	27	3.3
FEMA 547		
1. Retrofit techniques	25	4.0
2. Retrofit schemes	25	4.3
Case Studies		
1. Normal examples	24	3.9
2. Cases when things went wrong	24	3.9

Item on questionnaire – Entire Structure	Number of responses	Average score (out of 5)
1. Infills	5	3.2
a. RC	24	4.3
b. CLT	24	3.0
c. Reinforced masonry	23	2.8
d. Other	11	2.8
e. Connections to existing elements	21	4.4
2. Additional interior elements	2	
a. Bracing	24	4.5
b. Retractable bracing	20	2.7
c. Friction dampers	23	3.4
d. Kagome damping	19	2.1
e. Wing walls	23	3.2
f. Connections to existing elements	22	4.6
3. Additional exterior elements	3	
a. RC walls	23	4.3
b. Frames / bracing	23	4.3
c. Connections to existing elements	22	4.7

4. Energy dissipation devices / Base isolation	21	3.5

Item on questionnaire – Vertical LLRS elements	Number of responses	Average score (out of 5)
1. Jacketing		
a. Concrete	24	4.5
b. Steel	24	4.2
c. FRP	25	4.3
d. Hybrid	20	2.8
e. Polyester	22	2.3
f. UHPC	22	2.3
g. Steel wire mesh + mortar	19	2.5
2. Transverse post-tensioning		
a. Straps	23	3.2
b. Strands	23	3.2
c. Clamps	22	3.5
d. Shape Memory Alloy (SMA)	23	2.0
3. Steel plates and joint enlargement	24	3.3
4. Beam-column metallic haunch	23	2.8
5. Longitudinal post-tensioning	23	3.5
6. Externally bonded steel strips	24	3.5
7. FRP for walls	24	3.9
8. Cutting longitudinal bars	23	3.0

Items on questionnaire - Floors	Number of responses	Average score (out of 5)
1. Supplementary seating	18	4.3
2. Catch beams	19	3.7
3. Cable catch system	19	2.0
4. Double Tee Bracket	17	4.4
5. Double Tee articulating hanger	17	4.3
6. Strongback supports	21	4.1
7. Supplementary negative moment reinforcement	19	3.9
8. Supplementary shear reinforcement	20	3.9
9. Column ties	19	3.9
10. Steel plates for augmenting diaphragm	19	4.2
11. FRP for augmenting diaphragm	19	4.0

Item on questionnaire – Soil and Foundation	Number of responses	Average score (out of 5)
1. In-situ testing	4	4.0
a. Conventional tests (e.g., CPT, boreholes)	16	4.0
b. Test pits	16	3.8
c. Plate load testing (ASTM D1194)	16	3.1
d. Load testing existing piles (AS1250)	16	3.3

2. Ground improvement	2	
a. Jet grouting	17	2.8
b. Resin injection	15	2.5
c. Cement mixing	15	2.5
3. SFSI considerations	2	
a. Foundation rocking	18	4.3
b. Foundation sliding	18	4.2
c. Load redistribution	18	4.2
4. Shallow footing improvement	2	
a. Increase area	17	3.7
b. Increase depth	16	3.8
c. Tie element strengthening	16	3.5
d. Convert to mat foundation	17	3.7
e. Increase reinforcement	15	3.3
5. Adding pile elements	2	
a. Micro piles	17	4.1
b. Push-in piles	16	3.2
c. Screw piles	17	4.0
d. Bottom-driven steel tubes	16	2.9
e. Bored piles	17	3.1
f. Stub/shallow piles	16	3.5
g. Floating piles	15	3.3
6. Adding grade beams	13	3.4
7. Connection between new and existing foundation elements	17	4.4

Item on questionnaire – Hypothetical case study	Number of responses	Average score (out of 5)
1. Case study on incompatible retrofit	8	4.3
2. Modification of "Red Book" building example (or equivalent)	10	3.7
3. Analysis of existing, retrofitted building (pre-2017)	10	3.5
4. Discussion or examples of compatibility checks	9	4.3
5. Review of C5 rotation examples		
a. Presentation of scatter plots	6	3.7
b. Presentation of fragility curves	3	3.0

A2 Efficient Retrofit Alternatives

The EQC-BIP team is collaborating with researchers at UC and UoA who are planning two series of tests. In one series, columns with 'single-face strengthening' are going to be tested with uniaxial displacement reversals. In the other series of tests, the restraint systems will be tested under impact from falling precast units.

A2.1 Single-Face Column Steel 'Jacketing'

A number of techniques exist to retrofit RC columns. They include confining 'jackets,' 'wraps,' 'bands,' and clamps. In all cases, access to all column sides is required. An exception is the use of 'wing walls' that require access to one or two column faces which are perpendicular to the facade in the case of exterior columns. But even in this case, retrofit requires expensive modification of architectural building components (curtain walls, windows, partitions, etc). Figure 20 shows columns from the Shinkansen ("bullet-train") viaduct, in Japan, which were retrofitted by accessing exterior column faces only. This was done using a steel plate secured by post-installed anchor bolts. This retrofit technique was implemented in columns from the exterior of the area under the viaduct a) to keep the disruption to train stations and businesses to a minimum, and b) because there was limited access for cranes machinery and workers under the viaduct. The plate is expected to resist shear forces parallel to its own plane. Anchor bolts are sized to resist shear forces perpendicular to the plate. Little attention is given to confinement.

Figure 20: Example of single-face column steel jacket in the Japanese Shinkansen viaduct

Results obtained by Ishibashi et al. (2004) suggested columns with single-face retrofit plates installed as mentioned above had lateral strengths and drift capacities of at least 1.5 times and 3 times those of reference (un-retrofitted) specimens. These promising experimental results should be confirmed through additional testing for conditions relevant to New Zealand. Additional experimental evidence is being produced at UC. The setup to be used has already been used to test another column retrofit system (Pujol et al. 2024). In this way, the main investment needed is related to specimen construction. That funding has been secured through QuakeCoRE. Fabrication of seven full-scale RC columns has started and testing is expected to commence in the second semester of 2024.

A2.2 Precast-Unit Collapse Restraint System

In New Zealand, a surge in construction during the 1980s led to the construction of numerous multi-storey buildings with precast floors. Precast floors were perceived to offer advantages related to construction speed, quality, and simplicity (CCANZ, 2004a). The 2016 Kaikōura Earthquake showed the vulnerability of this type of construction, as several buildings with precast floors evidenced damage that was deemed too costly to repair. The partial collapse of floors in Statistics House was a prominent example of the vulnerability of precast floors (MBIE, 2017). A large research project named ReCast Floors (Retrofit of Precast Floors) was carried out in response to the need for methods to improve existing precast floors, particularly those constructed using hollow-core or double tee units.

One of the main conclusions of the ReCast project was that achieving life-safety performance of precast floors during strong earthquakes is typically not possible without significant retrofit. A number of retrofit techniques were studied such as supplementary seating and 'strongback supports' (Büker et al., 2022). Nonetheless, in the workshops described above, Wellington City Council personnel reported that buildings owners are finding those retrofit methods to be expensive and highly disruptive.

A more affordable alternative to retrofit precast floors has been identified. It consists of a 'catch system' composed of cables acting as catenaries perpendicular to the precast floor units. The main challenges in the implementation of the system are related to the detailing of cable anchorages and architectural constraints limiting access.

This testing shall make use of a 3D RC frame that is being built at UC to test a new precast-support system being proposed for new construction (by. R. Dhakal, and G. Lozano, with support from Quake CoRE). The mentioned frame is similar to that used in the ReCast project and will be subjected to biaxial cyclic displacement reversals through an initial testing phase. Before any testing begins. the frame will be furnished with catenaries installed between beams parallel to precast floor units. After initial damage occurs in the test frame, individual or groups of precast units will be dropped abruptly on the catenaries (using a quick-release system) to test the ability of the catenaries and their anchorages to arrest the falling floor units. Successful arrest of the fall of precast units may lead to a more affordable and easier-to-install system to protect life in older RC buildings with precast floors. Test variables shall include type of anchorage, number of cables, and initial cable sag (which shall be needed to allow cables to accommodate suspended equipment).

A3 Uncertainty in Deformation Limits

Given the unavoidable scatter in drift capacity estimates, a means to communicate the associated uncertainty is warranted. Uncertainty can be expressed in terms of an expected probability of failure given a ratio of drift demand (obtained, for instance, from dynamic analyses) to estimated drift capacity (obtained from C5 in the case of existing un-retrofitted elements). For instance, estimates of probability of failure for columns can be obtained by fitting a line through the origin of Figure 5 or Figure 6 with a slope equal to the demand to capacity ratio. The failure probability index for a column with the given demand to capacity ratio can be estimated as the number of points below the line divided by the total number of points. For example, the 1:1 line in Figure 6 depicts a case where the member demand is equal to the estimated axial drift capacity. Out of 40 columns with reported drifts, there are 2 below the 1:1 line, so the failure probability index for a column with a demand to capacity ratio of 1 is 2/40 or 5% for the corresponding calculation procedure. The points plotted in Figure 21 depict failure probability indices where demands correspond to the drift capacities reported in the ACI 369 database and capacities correspond to estimates obtained from C5 for each test column in the database.

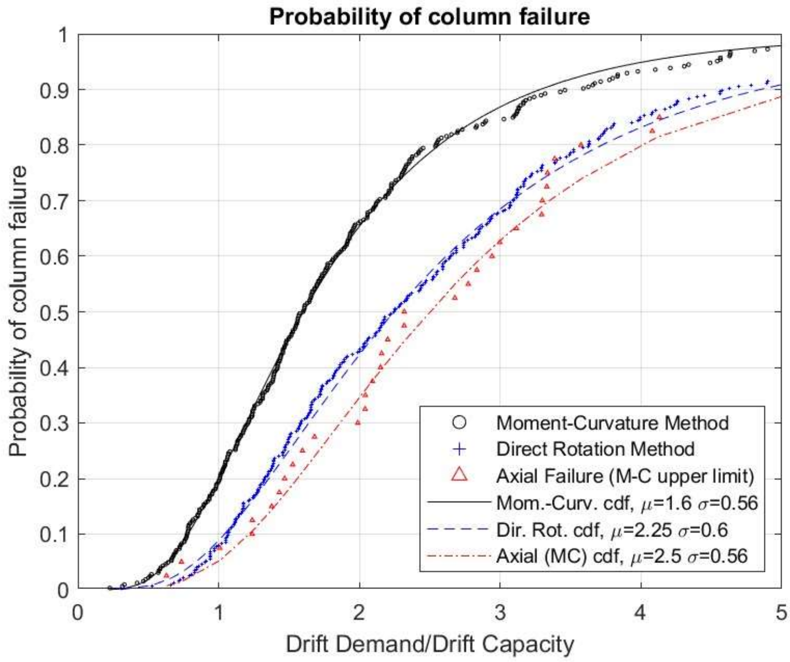


Figure 21: Probabilities of failure given demand to capacity ratios for columns using C5 assessment methods

The smooth curves shown in Figure 21 are cumulative distribution functions (cdf) for log normal distributions that are fitted to approximate the points in the figure using the indicated method and corresponding parameters listed in the legend of the figure:

$$P(Fail) = \int_{-\infty}^{\ln\left(\frac{Demand}{Capacity}\right)} \frac{e^{\frac{-(x-\ln(\mu))^2}{2\sigma^2}}}{\sqrt{2\pi}\sigma}\, dx$$

Where:

Demand	Drift demand (obtained from analysis)
Capacity	Drift capacity (obtained using C5 methods)

μ Mean of approximate lognormal distribution (taken as the median of points plotted for respective methods in Figure 21)

σ Standard deviation of approximate lognormal distribution (iterated to minimize root mean square error between points and cdf approximation)

Notice that in all the cases illustrated in Figure 21, a calculated ratio of demand to capacity equal to one is associated with estimated failure probability indices ranging between 5 and 20%. Engineers should be aware of this observation and consider it in the context of their projects, especially if the chose to use the "moment-curvature" method which is based on ideas that were formulated originally for elements subjected to monotonic instead of cyclic loads and which produces the worst results.

A4 Reconnaissance Examples of Retrofitted Japanese Buildings.

A post-earthquake reconnaissance was conducted on 15-18th of March 2024 after the Noto earthquake in Japan to study the performance of retrofitted reinforced concrete buildings. The Noto earthquake struck the Noto Peninsula in Ishikawa Prefecture in Japan at 4:50pm on 1 January with a moment magnitude of 7.6. Ground acceleration demands of over 1.0g were recorded at multiple seismograph stations (K-NET/KiK-net[5]), resulting in 241 casualties, severe structural damage in buildings, particularly in 1-3 storey timber houses, and tsunami damage. A seismometer in Togi recorded a peak ground acceleration of 2.8g during the earthquake. At least for stations recorded peak ground velocities close to 1m/s.

Distribution of seismograph stations and measured acceleration demands are shown in Figure 22and Figure 23. Acceleration demands in Shoin, Wajima, Anamizu, and Togi are comparable or above Japanese (Building Standard Law of Japan 2020) and New Zealand (NZS 1170:2004) design spectra. Spectral acceleration demands in Togi and Anamizu exceeded approximately 2.5g and 1.8g at periods of 0-1 seconds.

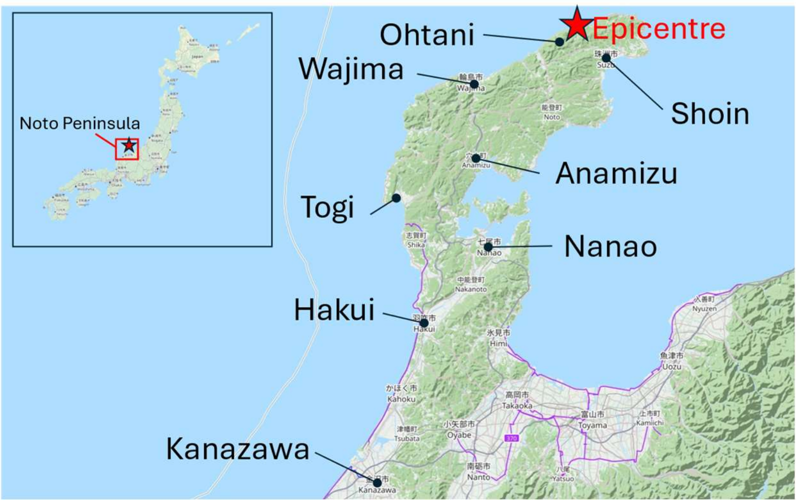
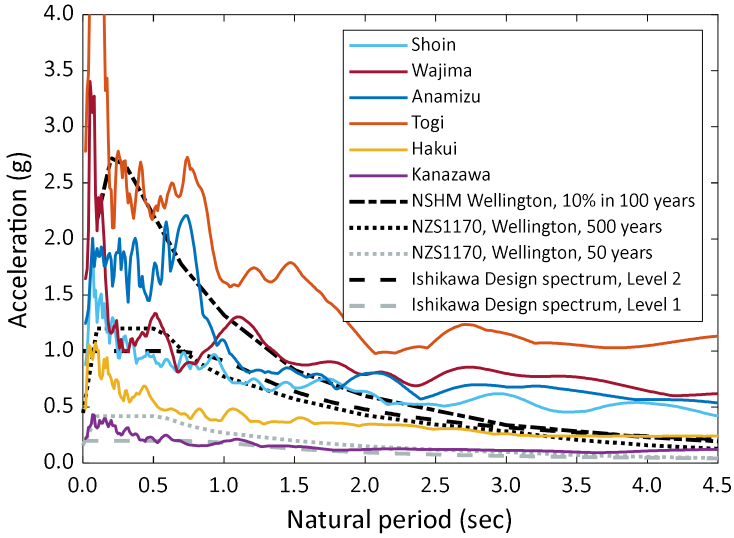


Figure 22: Distribution of seismograph stations (K-NET/KiK-net) (OpenStreetMap: https://www.openstreetmap.org)



[5] https://www.kyoshin.bosai.go.jp/

Figure 23: Measured acceleration demands in Noto Peninsula with design spectrums according to National Seismic Hazard Model (NSHM) for Wellington with a probability of exceedance of 10% within 100 years, NZS 1170:2004, and Building Standard Law of Japan 2020

Nine school buildings and five city hall buildings were inspected during the reconnaissance. Table 9 shows basic information about the inspected buildings. Japanese school buildings are typically 2-5 storey reinforced concrete buildings with rectangular floorplans. The structural systems in the longitudinal and transverse directions consist of moment-resisting frames and dual wall-frame systems, respectively. In the direction of the moment-resisting frame, RC 'standing,' 'hanging' and partition walls are typically constructed. Although these secondary walls are often ignored in structural calculations, they reduce the shear-span of adjacent columns and the columns become shear-critical.

Because the buildings have shear walls the transverse direction, no retrofit is required. However, the longitudinal direction is typically retrofitted using steel or RC braces. Other retrofit solutions, such as steel jacketing of columns, are used selectively. Where columns are susceptible to shear failure due to the presence of secondary walls, saw cut (typically referred to as "seismic slit") is made at the interface between a column and a secondary wall to prevent shear failure of the column by increasing the shear span.

Table 9: Inspected retrofitted reinforced concrete buildings in Noto Peninsula

ID	Building use	No. of storeys	Primary structural system	Retrofit solutions	City
W1	City Hall	4	RC wall-frame	RC frame	Wajima
W2	School	5	RC frame and wall-frame	Steel brace, seismic slit	Wajima
W3	School	3	RC frame and wall-frame	Steel brace	Wajima
A1	City Hall	4	RC frame	RC frame	Anamizu
A2	City Hall	3	RC frame and wall-frame	RC brace	Anamizu
A3	School	3	RC frame and wall-frame	Steel brace, seismic slit	Anamizu
A4	School	3	RC frame and wall-frame	Steel brace, seismic slit	Anamizu
H1	City Hall	6	RC frame	No retrofit	Hakui
H2	School	4	RC frame and wall-frame	Steel brace, steel jacket, seismic slit	Hakui
H3	School	3~4	RC frame and wall-frame	Steel brace, seismic slit	Hakui
S1	City Hall	5	RC frame	Steel brace	Suzu
S2	School	3	RC frame and wall-frame	Steel brace, seismic slit	Suzu
S3	School	3~4	RC frame and wall-frame	Steel brace, seismic slit	Suzu
S4	School	3	RC frame and wall-frame	RC brace, seismic slit	Suzu

A4.1 Observed Structural Damage and Performance

In general, retrofitted reinforced concrete buildings achieved life-safety objectives, preventing brittle failures, and collapse. Damage at the interface of the retrofit components (e.g., steel braces) and existing structure was insignificant, which indicates that connection details as per Japanese Retrofit Guideline effectively minimize problems related to deformation incompatibility Key observations, including potential challenges, are described for specific buildings in the following sections.

A4.1.1 School W2

School W2 consisted of two three-storey reinforced concrete buildings (East and South Building), forming a L-shape plan (Figure 24). Both buildings had been retrofitted using steel braces in their longitudinal directions.

Limited structural damage was observed in both buildings, such as minor cracking and spalling. Minor diagonal cracking was observed in shear walls in transverse direction of East Building (Figure 25 (a)). Hairline cracking and minor delamination of paint were observed in retrofitted frames, which implied that deformation incompatibility between steel braces and existing structure was minimal. This observation also supports the effectiveness of connection details specified in Japanese Retrofit Guideline (2001). On the other hand, ground settlement and large fissures were evident around the buildings, resulting in tilting of South Building towards South (Figure 25 (b)). The tilting consequently caused separation of the two building at the seismic joint. In addition, severe separation with a gap of 220mm was found at a seismic joint between South Building and a stairwell (Figure 25 (c)). Although the separation of the building can be attributed to ground settlement, it might also be a result of differences in permanent deformations exacerbated by the stiffening the buildings using steel braces in one direction but not the other.

(a) (b) (c)

Figure 24: Building W2: (a) Overview of School W2, (b) Overview of South Building, (c) Overview of North Building

(a) (b) (c)

Figure 25: Building W2: (a) Minor diagonal cracking in a transverse shear wall in West Building, (b) Tilting and separation of South Building, (c) Large separation between West Building and a stairwell at the seismic joint

A4.1.2 School S4

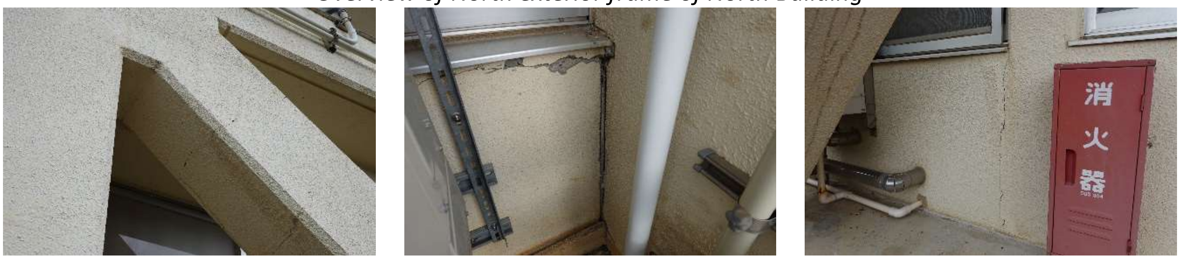
School S4 had two three-storey buildings (North Building and South Building) with parallel but offset floorplans (Figure 26). The two buildings were connected by a seismic joint over multiple bays. Four bays in South exterior frame of South Building had been retrofitted using RC braces, while no other retrofit was found anywhere else (Figure 26 (b)).

In the retrofitted frame, components sustained minor cracking and cover spalling. RC braces showed minor tension cracking (Figure 27 (a)). At the seismic slit next to the column, minor cracking and spalling was found (Figure 27 (b)(c)). Concrete spalling at the top corner of the standing wall suggests that the width of the seismic slit was not sufficient to accommodate column deformation.

In contrast, in the un-retrofitted frame (i.e., North exterior frame of South Building), substantial damage was evident. Shear walls in the first storey and short-captive columns in the second storey failed shear (Figure 28 (a)(b)). Columns in the second storey exhibited concrete spalling at the end region and reinforcement was exposed, which indicate flexural behaviour (Figure 28 (c)). These observations indicate that drift demands in retrofitted frame were reduced; however, drift demands in un-retrofitted frame were not reduced as much as retrofitted frame or amplified due to torsional effects.

(a) (b) (c)

Figure 26: Building S4: (a) Overview of School W2, (b) Overview of South exterior frame of South Building, (c) Overview of North exterior frame of North Building



(a) (b) (c)

Figure 27: Observed damages in retrofitted frame: (a) Minor tension cracking in RC braces, (b)(c) cracking and spalling at the seismic slit next to the column

(a) (b) (c)

Figure 28: Observed structural damage in un-retrofitted frame (North exterior frame) of South Building in School S4: (a) Shear failure of walls in the first storey, (b) Shear failure of short-captive columns in the second storey, (c) Concrete spalling at the end region in columns in the second storey

A4.2 Summary and Key Observations

A post-earthquake reconnaissance was conducted to observe the performance of retrofitted reinforced concrete buildings in major cities and towns in Noto Peninsula, where ground acceleration demands of over 1.0g were measured at multiple seismograph stations. Through investigating thirteen retrofitted multi-storey school and city hall buildings, the following key observations and implications were found.

- Retrofitted reinforced concrete building using braced steel frames and new exterior RC frames did not exhibit any collapse of the building, indicating life-safety objectives were achieved.
- Only hairline cracks and delamination of painting were observed at the interface between braced steel frames and the existing structure. It suggests that connection details according to the Japanese Retrofit Guideline effectively unite retrofit and existing frames, preventing connection failure.
- Shear failure in short-captive columns due to the presence of standing and hanging walls was often observed. On the other hand, columns with seismic slits did not show evidence of shear-critical behaviour. It can be inferred that the critical failure mode was changed to flexure from shear by increasing the shear span with seismic slits. In addition, if the width of the seismic slit is too small, it results in unexpected compression force demands onto an adjacent standing or handing wall.
- Severe damages, such as concrete crushing and shear failure were observed in un-retrofitted frames even if other frames in the same direction were retrofitted. This observation may imply that drift demands were not reduced in un-retrofitted frames as much as in retrofitted frames or amplified due to torsional effects.
- In Japan, retrofit steel braces are typically placed inside of existing frames to ensure deformation compatibility, whereas they have been found to be installed on existing perimeter frames in New Zealand. On the other hand, retrofit concrete braces (e.g., Pita-Column retrofit[6]) were installed on existing perimeter frames. Since the damage level on retrofitted frames using concrete braces was limited to minor cracking, connection details for concrete braces may be applicable for external steel brace retrofit.

[6] https://www.pita-kyoukai.jp/index.html

A5 Details Pertaining to Case Study from Section 4.1

The Red Book frame example used in the Section 4.1 Case study is 10 stories tall with a ground storey height of 4m and a typical storey height of 3.6m, for an overall height of 36.4m. The floorplan for the Red Book frame building is shown in Figure 29, typical cross sections of beams, slabs, and columns are shown in Figure 30, and Table 10 describes typical member dimensions.

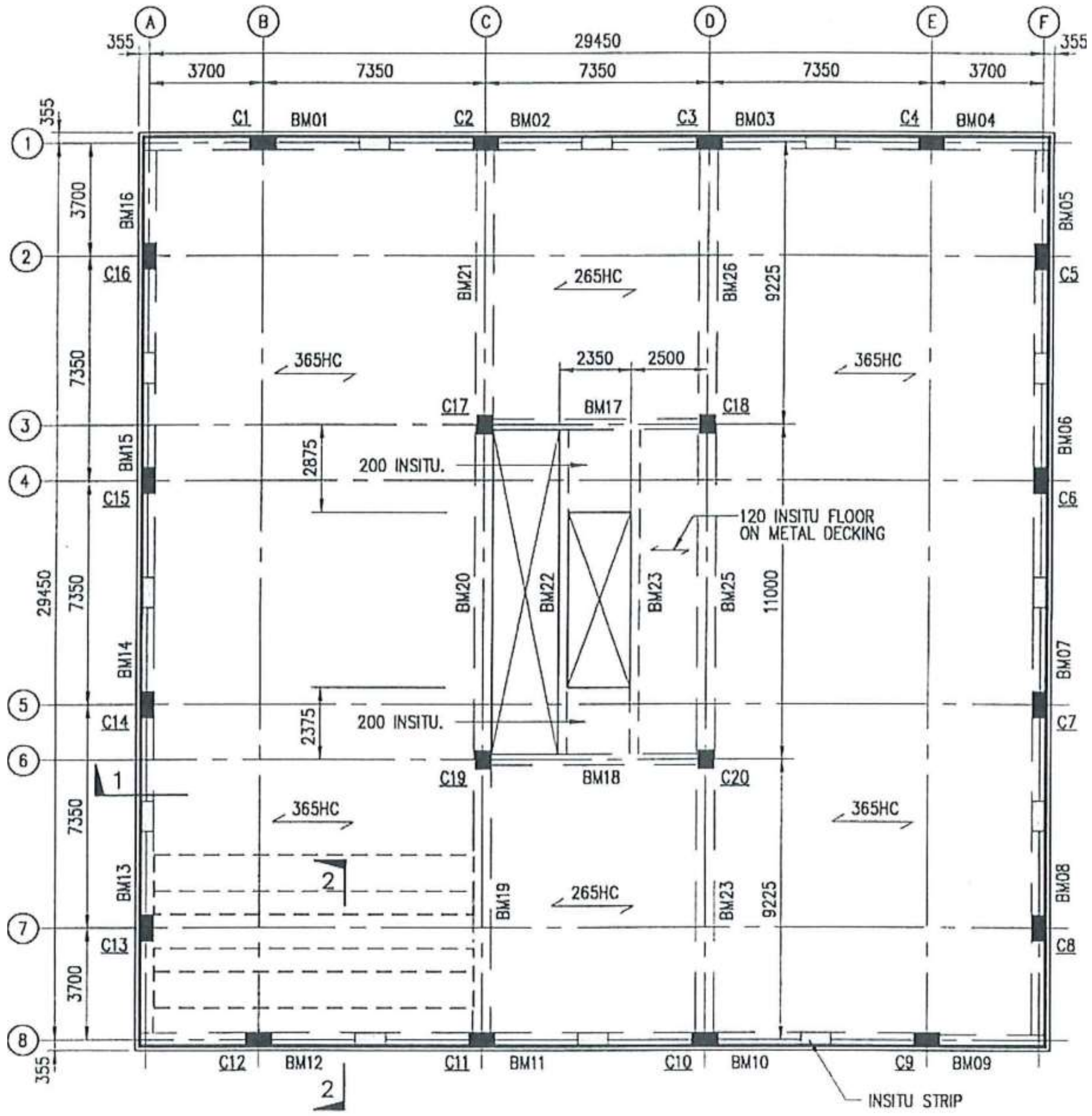


Figure 29: Red Book frame building floorplan

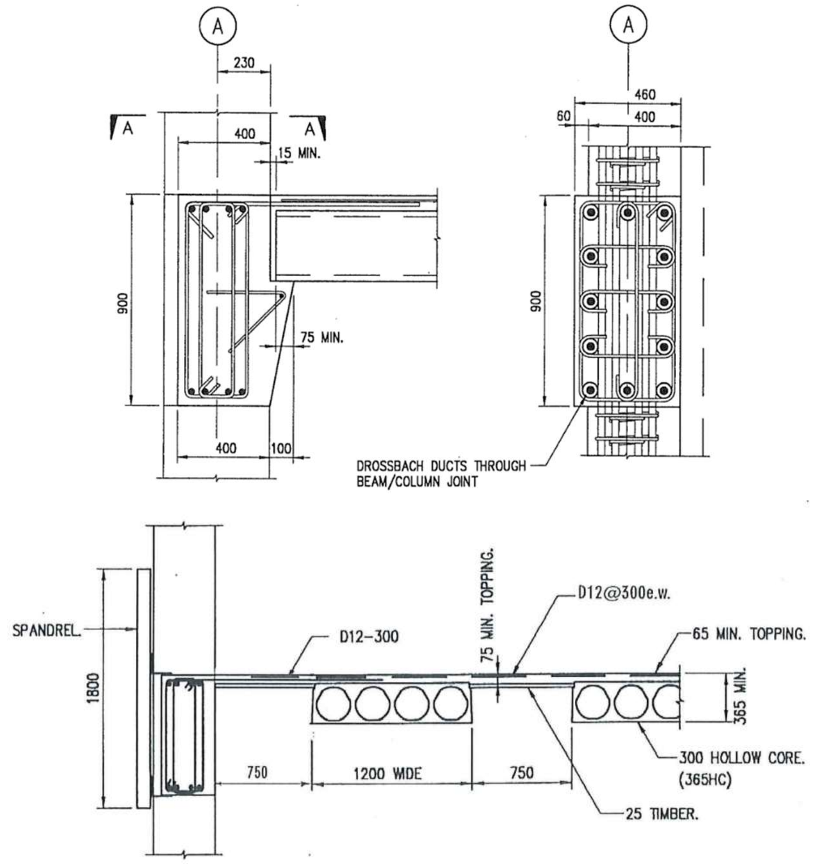


Figure 30: Red Book frame building typical beam, column, and floor sections

Table 10: Typical member dimensions for Red Book frame building

SUMMARY OF MEMBER SIZES	
COLUMNS ALL LEVELS	
C1-C16	900 x 460
C17-C20	650 x 600
BEAMS – GROUND	1,200 x 600
BEAMS – LEVELS 1-10	
BM01-BM04, BM09-BM12	900 x 400
BM05-BM08, BM13-BM16	900 x 400
BM17, BM18	550 x 350
BM19-BM21, BM24-BM26	750 x 530
BM22, BM23	750 x 250
PILES	1,000 DIA.
FLOORING	
GROUND	150 INSITU ON GRADE
LEVELS 1-10	AS NOTED

Table 11 shows members selected for external EBFs from the case study in Section 4.1.

Table 11: Steel profiles used in the external eccentrically braced frame system.

Floors	Link	Column	Brace
1-4	UB 360x171x44.7	UC 310x310x137	UC 250x250x72.9
5-7	UB 310x165x40.4	UC 250x250x72.9	UC 200x200x46.2
8-10	UB 250x146x31.4	UC 200x200x46.2	UC 200x200x46.2

Table 12 lists parameters used to scale ground motion records used in nonlinear dynamic analyses. NZS 1170.5.5.2 defines target spectral acceleration, SA_{target}, which is equal to the elastic site hazard spectrum, *C(T)*, when the structural performance factor (S_p) is equal to 1. Record scale factors, *k1*, are determined by calculating a period range of interest, T_{range}, bounded by 0.4 times and 1.3 times the largest translational period of the structure, T_1, in the direction being considered. *k1* values are then determined by minimizing the mean square of $log(k_1SA_{component}/SA_{target})$ over the period range of interest. *D1* represents the root mean square difference between the logs of the scaled primary component and the target spectra over the period range of interest. Reasonable fits have *D1* values below 1.5. To facilitate comparisons, Table 12 lists parameters used to scale ground motions to match expected ULS acceleration spectra for T=1.6s Wellington Soil Class C according to NZS 1170.5 (which defined the seismic hazard estimates used in design until 2024) and Wellington Site Class III according to the new TS 1170.5 (which defines the seismic hazard estimates to be used in NZ to design **new** buildings starting in 2024). It is understood, nevertheless, that retrofit design shall continue to be based on NZS 1170.5 for the near future.

Table 12: Parameters used in the scaling of the selected ground motions.

T_{target} = 1.6s	Unscaled		Wellington, Site Class C			Wellington, Class III (2024 TS)		
Ground motion (RSN, Primary)	PGA (g)	PGV (cm/s)	k1	PGA	D1	k1	PGA	D1
El Centro 1940 (RSN6, 180)	0.28	31	1.45	.41	1.28	2.61	0.73	1.22
Kocaeli 1999 (RSN1148, 090)	0.13	40	3.45	0.46	1.11	6.22	0.83	1.16
Chi-Chi 1999 (RSN1504, E)	0.5	92	0.60	0.30	1.15	1.09	0.54	1.20
Duzce 1999 (RSN1605, 270)	0.51	84	0.64	0.33	1.20	1.16	0.6	1.14

Primary and secondary scaled ground motions were applied to each model simultaneously, and nonlinear modal analysis having a constant damping of 1% for all modes was chosen. This value is on the lower side of conventional assumptions, but it was chosen to reflect the idea that a steel structure (in this case the EBF frames used to retrofit) tends to have lower damping than RC structures. For the latter, Lepage (1997) and Shah (2021) have shown that 2% damping produced estimates of drift consistent with measurements in a wide range of scenarios. SAP 2000 (v. 25.0.0) was used to produce displacements at each node.

A6 Geotechnical Considerations Pertaining to Section 4.1

1 Introduction

This appendix is intended to inform the case study in Section 4.1 and Appendix A5. The information provided relates to:

- The assumed soil profile at the site
- The assumed existing foundation system
- The proposed new foundation system to support the proposed new shear walls.

2 Geotechnical assumptions

The geotechnical contribution is based on the following assumptions:

The subject building is based on the “Red book” building (Bull and Brooke, 2008), with the following proposed amendments relating to geotechnical aspects:

- Location shifted from Christchurch to Wellington.
- Soil profile and assumed foundations amended to be consistent with the revised location.

A retrofit involving the addition of external braced frames supported by micropiles is to be assumed.

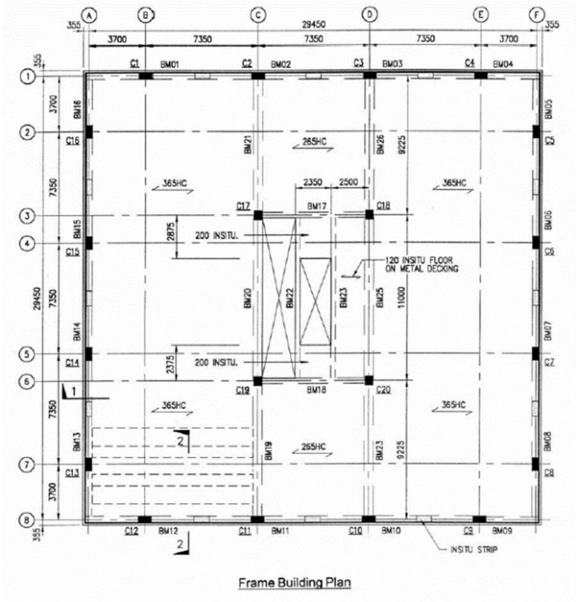


Figure A6.1 floorplan of red book building

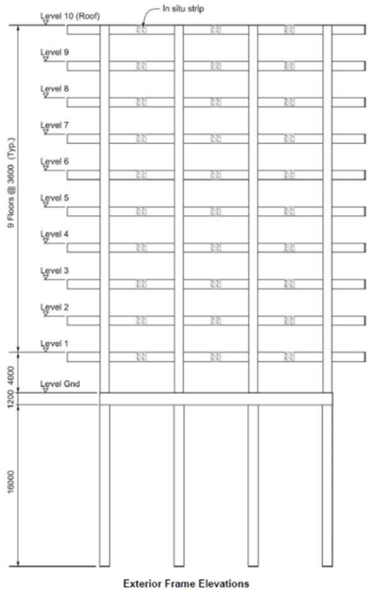


Figure A6.2 Elevation of red book building

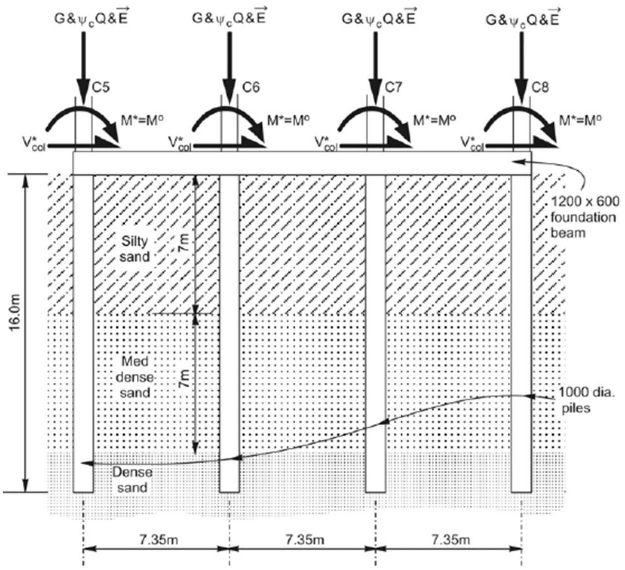


Figure A6.3 Original piles and ground profile from red book

2.1 Assumed soil profile at the site

- Geology
  - o Alluvium overlying greywacke rock
- Soil profile
  - 0 to 5 m depth: Medium dense silty sandy gravel interbedded with firm to stiff silt.
  - 5 to 40 m depth: Dense becoming very dense with depth, silty sandy gravel with occasional lenses of stiff silt.
  - 40 m depth: Greywacke rock
- Seismic subsoil class in terms of NZS1170.5:2004
  - Subsoil class C

- Groundwater level
  - 3 m depth
- Liquefaction potential
  - Low

2.2 Assumed existing foundation system

- Bored belled cast in-situ reinforced concrete piles
  - 900 mm diameter shaft. 1.5 m diameter bell
  - Founded at 12 m depth
- Substructure
  - 1200 mm x 600 mm foundation beams in both directions
- Foundation assessment conclusions (to be assumed at this preliminary stage. A contrived basis of these assumptions could be provided at a later stage if required).
  - Geotechnical and structural tension and compression capacity of the piles not critical to the assessment of the overall structure.
  - Lateral capacity of the piles in combination with that of the ground beams is sufficient to resist base shear.

3 Proposed new foundation system

The proposed new foundation system to support the proposed new braced frames should be selected collaboratively by the structural and geotechnical engineers in consultation with contractors and the client. Micropiles were selected. Bored piles constructed by a specialised compact piling rig and screw piles installed by a torque head mounted on an excavator were considered. These alternatives offered the benefits of considerably higher capacity per pile than micropiles but the constrained space around the building may not allow access for construction of these alternatives.

- Drilled and grouted micropiles are preferrable for the following reasons:
  - Can be constructed in relatively confined space
  - Can provide required tension and compression capacity by use of multiple piles
  - Can be constructed to provide a line of resistance concentric with the line of proposed load
  - Construction materials, equipment, and experience available in Wellington
- Micropile construction details
  - 175 mm diameter. 5 m free length plus 10 m bond length
  - An Ischebeck Titan bar grouted central
  - Full length grouted with corrosion protection details
  - Minimum centre to centre spacing between piles 800 mm
  - Maximum distance from existing wall face or other vertical obstruction to centre of micropile: 400 mm
- Micropile design details
  - Reduced (strength reduction factors applied) tension and compression capacity per micropile: 600 kN
  - Lateral capacity of micropiles: negligible
  - Vertical stiffness, tension; 20 to 60 kN/mm or 10 to 30 mm displacement at 600 kN tension load
  - Vertical stiffness, compression; 40 to 120 kN/mm or 5 to 15 mm displacement at 600 kN compression load.

4 Applicability

This appendix has been produced for illustrative purposes only. It pertains to the particular assumptions and opinions described and it may not be used in other contexts or for any other purpose.

A7 Details Pertaining to Case Study from Section 4.2

An overall view of the building, the 3-D model developed in SAP2000, and the first storey plan view are shown in Figure 31.

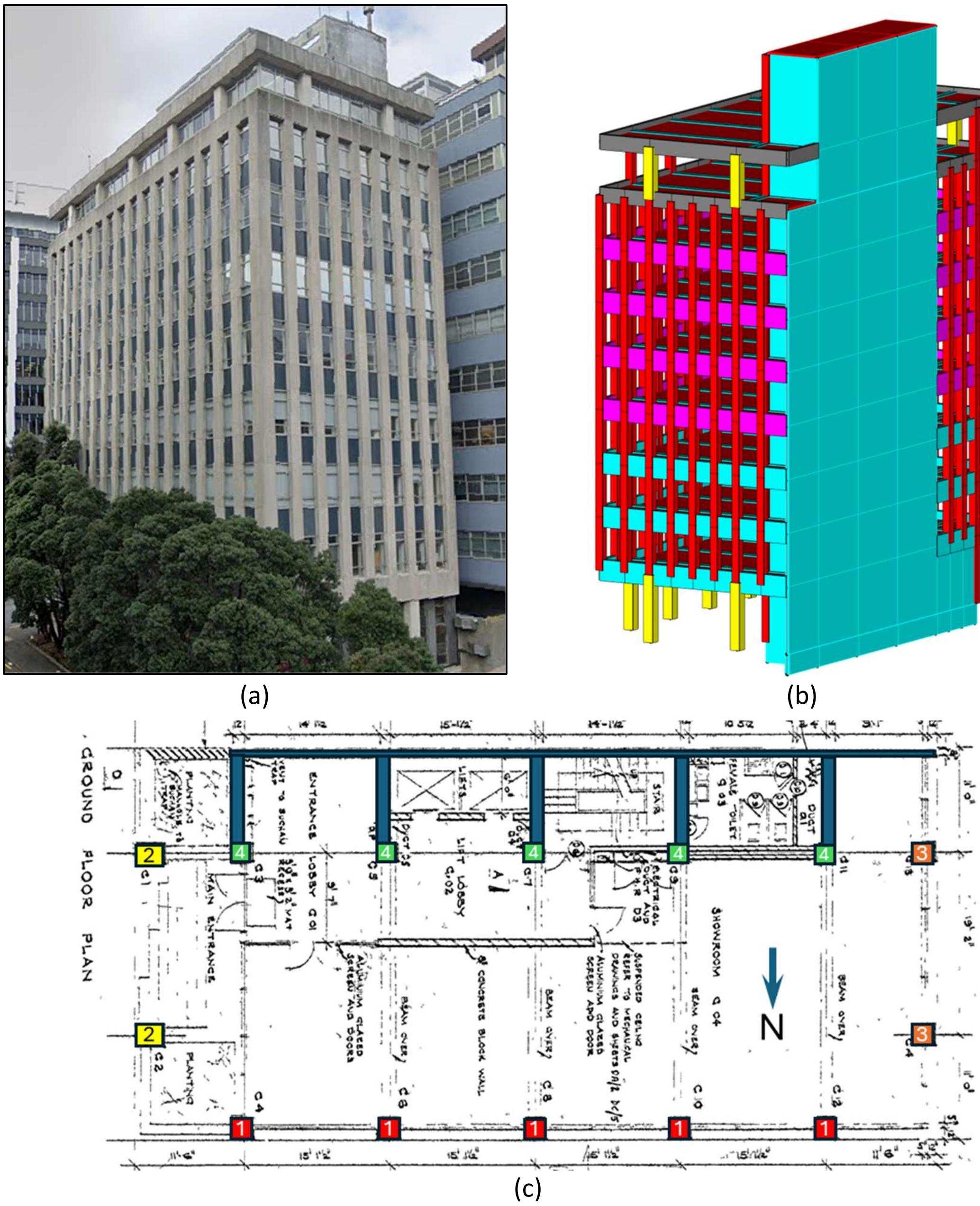
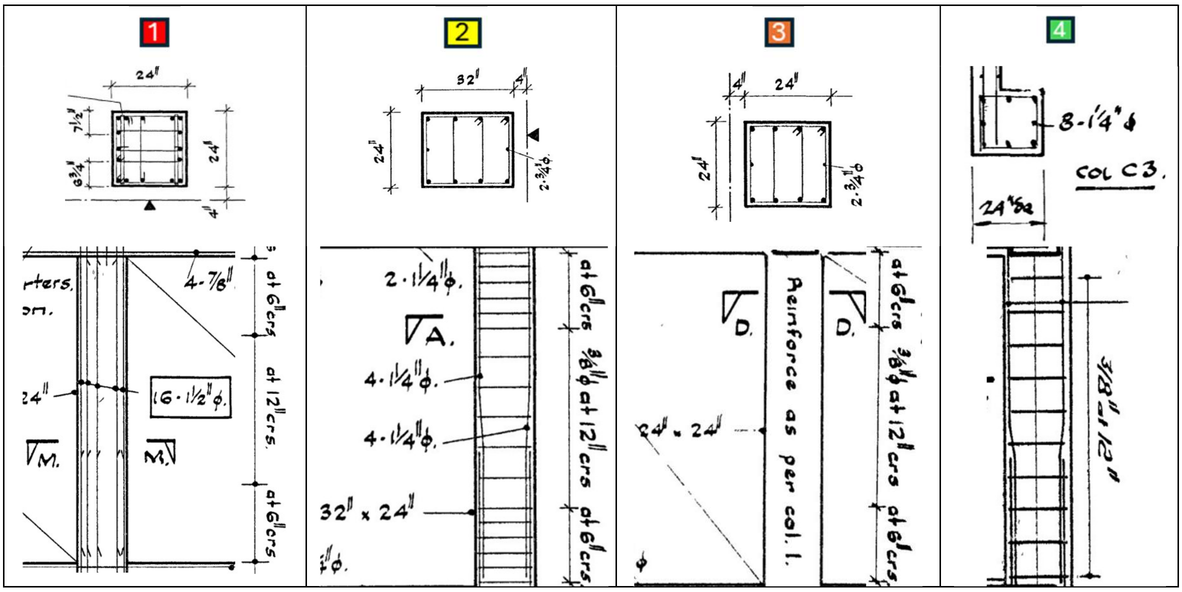


Figure 31: (a) Overall view of the building, (b) Rear of the building as modelled in SAP 2000, and (c) First storey plan view.

The locations of the walls indicate a stiffness imbalance in the long direction of the floorplans. Ground floor wall thicknesses are 8” (203 mm) in the long plan direction and 12” (305 mm) in the short plan direction. Walls in the short plan direction are reduced to 8” above the ground floor. 8” walls are reinforced with two layers of 663 H.R.C fabric (6.3 mm bars spaced at 150mm), and 12” walls are reinforced with two layers of ½” (12.7mm) diameter bars spaced at 12” (305 mm) centres in both transverse and longitudinal directions. Typical column details are shown in Table 13 for the four types of columns present in the first storey and are colour-coded to match the columns indicated in

Figure 31 (c). The columns may be susceptible to shear failure because of the wide spacing of their transverse reinforcement (3/8” bars at 6 or 12 inches). Additionally, the plans have indicated lap splices located at the bases of all columns, which will further limit column strength and ductility.

Table 13: Reinforcing details of the columns in the first storey.



As shown in Figure 32, the first[7] through seventh stories feature a denser spacing of columns around the perimeter of the structure (the original structural calculations referred to these columns as “punched walls”), one fewer wall in the short plan direction, and a wall along the long floorplan direction that is shorter by one bay compared with the ground floorplan. Colour-coded columns are shown in Table 14 and have similarly vulnerable details compared to those of the ground floor columns

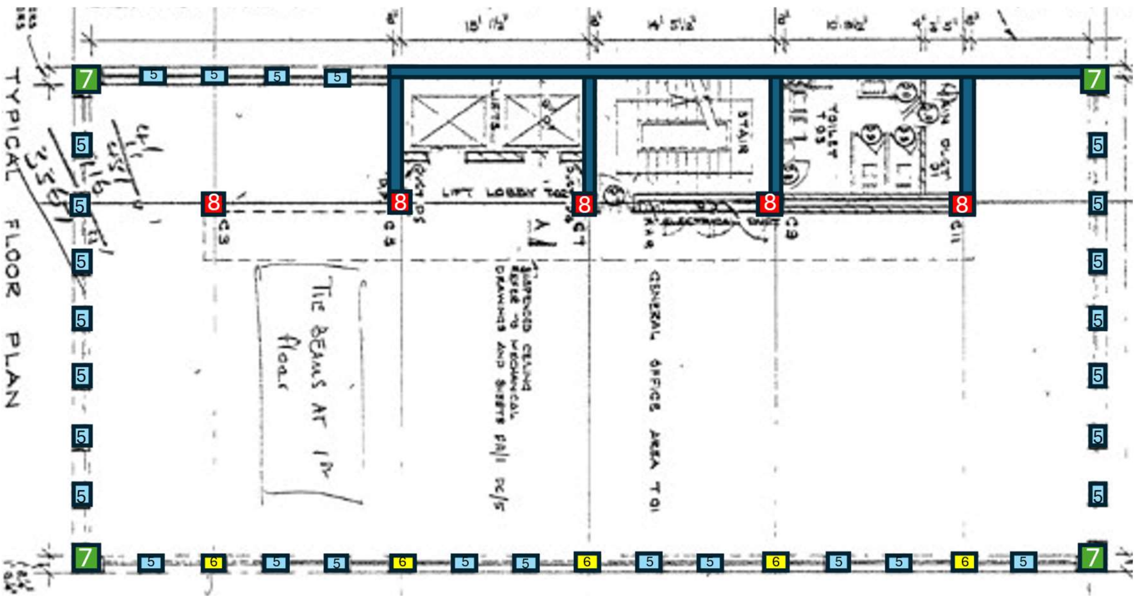
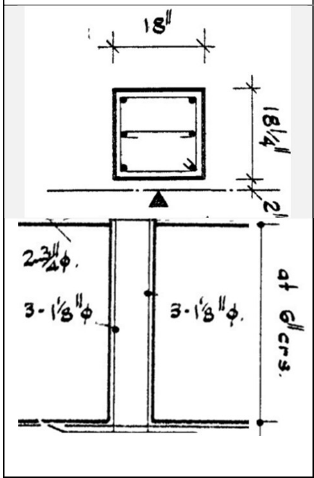
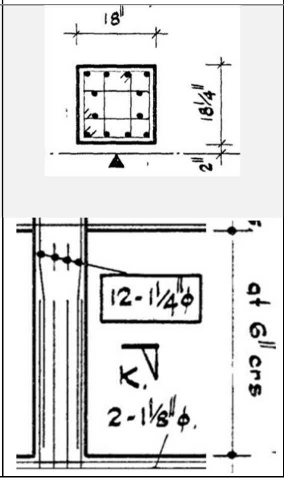
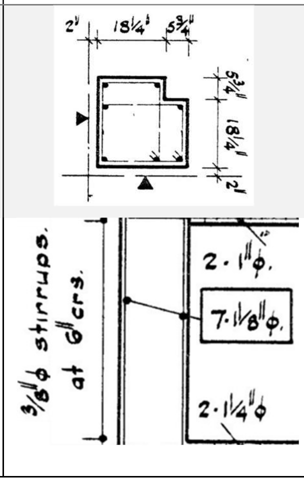
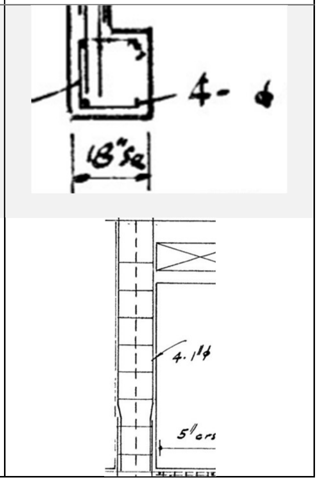


Figure 32: Plan view of the upper floors.

[7] above ground

Table 14: Reinforcing details of the columns in the upper storeys.

5	6	7	8
			

A simplified assessment was conducted for this case study, in which columns and walls were the only elements considered to estimate building lateral resistance. Perimeter beams are deep and are expected to force yielding in columns rather than yielding in beams at every floor. *Table 15* summarizes the elements being considered. The last four entries in each row of *Table 15* list the lateral drift capacity estimates for unretrofitted and retrofitted elements in the long (x) and short (y) directions of the structure. Drift is measured from centre of the storey below to centre of the storey above (rather than clear distance). As shown in Figure 33, assumed retrofits include 100 mm RC jackets with 10-mm diameter transverse reinforcing spaced at 150 mm. No dowels are shown to follow more affordable Japanese practices. Despite the uncertainty of its applicability to retrofitted elements (Section 2.1), drift capacities for retrofitted columns were calculated using the moment-curvature method, assuming the exterior dimension of the retrofitted elements as the gross cross section and the contribution of both existing and new transverse reinforcing bars to estimate shear capacities and bond development.

Table 15: Simplified seismic assessment of the columns.

														With RC Jackets	
ID	h(x)	b(y)	$P/A_g f'_c$	l_s	ρ_l	$\rho_{t,x}$	$\rho_{t,y}$	$V_{y,x}$	$V_{y,y}$	$\Delta_{y,x}/L_c$	$\Delta_{y,y}/L_c$	$\Delta_{cap,x}/L_c$	$\Delta_{cap,y}/L_c$	$\Delta_{cap,x}/L_c$	$\Delta_{cap,y}/L_c$
	mm	mm	%	mm	%	%	%	kN	kN	%	%	%	%	%	%
1	610	610	18	1219	4.9	0.4	0.2	1060	930	0.65	0.65	0.41	0.58	1.98*	1.97*
2	813	610	13	1219	1.4	0.2	0.2	810	630	0.77	0.66	0.64	0.86	1.63	1.67
3	610	610	16	1219	1.9	0.2	0.2	550	590	0.66	0.66	1.08	0.72	1.75	1.58
4	610	610	13	762	1.7	0.1	0.1	480	480	0.66	0.66	0.45	0.45	1.53⁺	1.53⁺
5	457	464	3	N/A	1.8	0.3	0.2	340	300	0.74	0.74	0.81	1.05	1.95	1.45
6	457	464	21	1270	4.5	0.4	0.4	570	680	0.74	0.74	0.58	0.46	1.34	1.34
7	610	610	0.3	N/A	1.0	0.2	0.2	310	390	0.7	0.7	1.59	0.94	1.59	1.64
8	457	457	14	762	1.0	0.1	0.1	220	220	0.62	0.62	0.52	0.52	1.72⁺	1.72⁺

* Column 1 requires jacket thickness of 150mm and 12d bars spaced at 100mm

⁺ Columns 4 and 8 require intervention (welding of splices) to mitigate bond failure

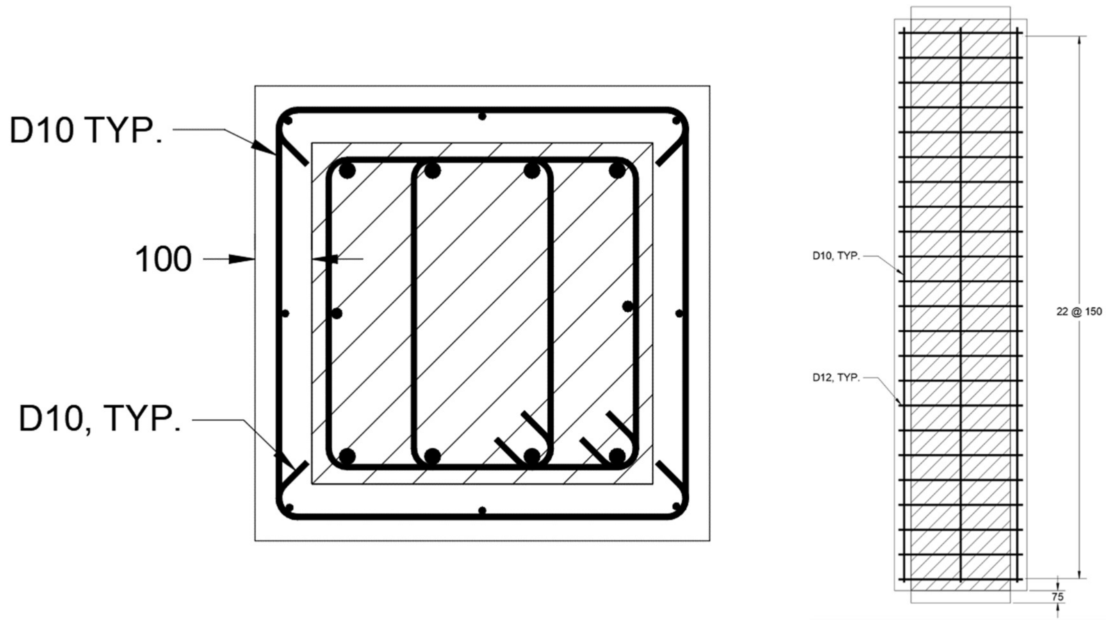


Figure 33: Typical cross section and elevation view of RC Jacket retrofit for Munro building

The presence of lap splices in most columns limits the expected drift capacity, with the interior columns (Column IDs 4 and 8) having the shortest splices and the smallest drift capacities as a result. Walls were assessed using C5.5. To simplify calculations, the wall length in the long direction was considered to be the wall length above the first story, as shown in Figure 32. Relevant parameters and estimated drift capacities of the long(x)- and short(y)-direction walls are listed in Table 16. Wall IDs with the prefix “G” represent ground floor and “F” represent first floor (above ground). Element shear span ratio, M/Vl_w, is approximated using the horizontal loads assigned using NZS1170.5 equivalent static analysis. Because of the relatively light reinforcing ratios of the walls, the plastic hinge length in the long and short directions are assumed to be 1/5 the length calculated using equation C5.21, as recommended by C5 (in the note following the plastic hinge length calculation for walls). The reduction in plastic hinge length is further supported by the lack of confinement or anti-buckling reinforcement throughout the walls. The moment capacities of the walls in the short direction were calculated considering contributions from reinforcement in the interior columns and the H.R.C fabric in the effective area of the long-direction wall. It should be noted that the probable plastic hinge is expected to occur at bottom of the first floor (above ground) for the wall(s) in the short floorplan direction because the change in section occurring at the top of the ground floor. Incidentally, the ductility of the mentioned H.R.C. fabric may merit investigation through destructive testing (i.e. extraction of samples).

Table 16: Seismic assessment of the walls.

ID	**l_w**	**t_w**	**M/Vl_w**	**l_w/t_w**	**H/t_w**	**ρ_l**	**ρ_t**	**M_y**	**M_n**	**Δ_y/L_c**	**Δ_{cap}/L_c**
	mm	mm				%	%	tn*m	tn*m	%	%
Gx	17336	203	0.99	108	147	0.21	0.21	4800	5900	0.15	0.27
Gy	3353	305	6.49	11	98	0.27	0.27	510	630		
Fy	3353	203	5.41	17	127	0.21	0.21	360	480	0.8	1.34

While the centre of stiffness in the short floorplan direction aligns with the centre of mass, the response in the long floorplan direction is governed by the eccentricity between the centre of mass and the centre of stiffness. As illustrated in Figure 34, torsion is expected to cause additional displacements in both the x- and y- directions at each floor. In extreme cases, the additional displacements estimated in the short direction are nearly equal to the displacements estimated in the long direction.

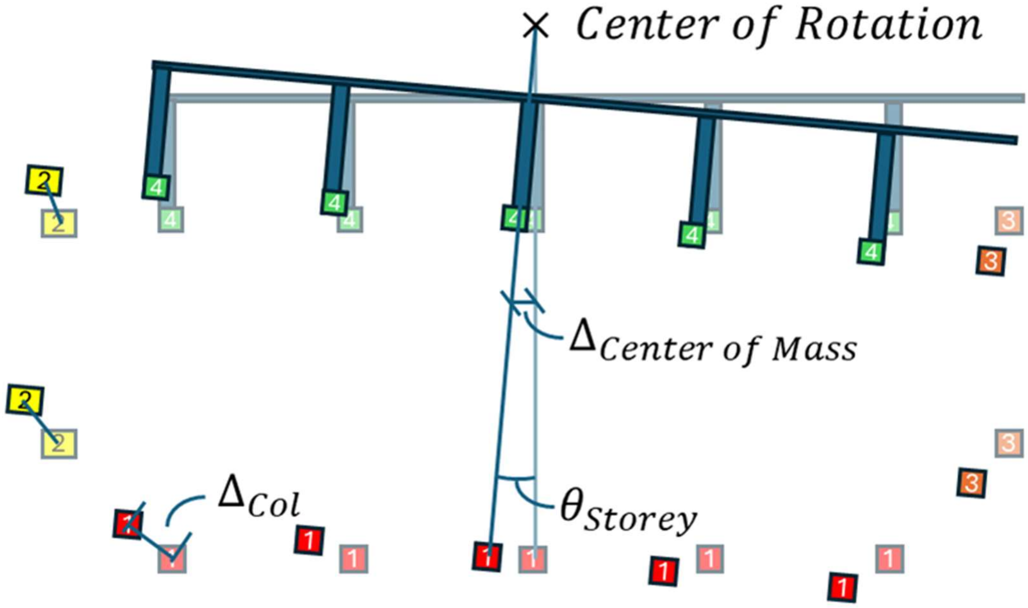


Figure 34: Story rotation due to eccentricity between centres of mass and stiffness.

From mode shapes retrieved from the numerical model built with SAP2000, the centre of rotation for each storey is between 1.9m and 2.3m outside the wall in the long floorplan direction (Figure 34). The centre of mass is 5.7m inside the same wall. As a result, the column along the north edge of the building is expected to displace as much as 7 times as much as the wall in the long floorplan direction.

A load-deflection curve was constructed using building mode shapes and the probable capacities calculated using C5.5, as shown in Figure 35 for roof drift measured at the floorplan centre. Table 6 includes building periods and other estimates relevant to the structure.

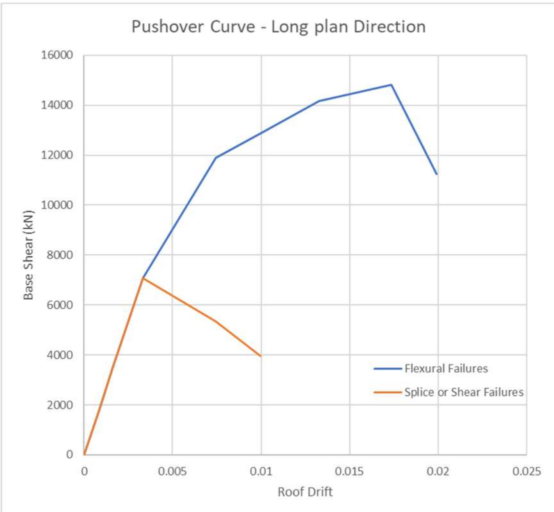


Figure 35: Existing (orange) and retrofitted (blue) long (EW) direction pushover curves

As mentioned throughout this document, the performance of retrofits is intended to be measured against existing buildings designed to NZS 1170.5, but ground motions were scaled according to both spectra prescribed in both NZS 1170.5 (2004) and the new technical specification TZ1170.5 (2024) that is mean to reflect new seismic hazard estimates. As was also done for the first case study, procedures provided in 1170.5.5 were followed to scale records used in dynamic analyses. The scale factors used are shown in Table 17. A target period of 1.2 seconds was selected as the mean of the first two fundamental periods, 1.1s and 1.3s, as opposed to producing scaled records for each direction.

Dynamic analysis was conducted in SAP 2000 (v25.0.0) using nonlinear modal analysis and constant damping ratio of 1%. The damping is again on the lower side of values commonly assumed for RC, but it was chosen for consistency with the previous examples.

Table 17: Parameters used in the scaling of the selected ground motions for NZS1170.5 and TS1170.5.

T_{target} = 1.2s			**Wellington, Site Class B**			**Wellington, Class II (2024 TS)**		
Ground motion (RSN, Primary)	PGA (g)	PGV (cm/s)	k1	PGA	D1	k1	PGA	D1
El Centro 1940 (RSN6, 180)	0.28	31	1.04	0.29	1.29	2.14	0.60	1.23
Kocaeli 1999 (RSN1148, 090)	0.13	40	2.88	0.39	1.16	5.93	0.8	1.24
Chi-Chi 1999 (RSN1504, E)	0.50	92	0.53	0.26	1.22	1.08	0.54	1.08
Duzce 1999 (RSN1605, 270)	0.51	84	0.49	0.25	1.22	1.02	0.53	1.24