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REDUCING DESIGN DRIFT LIMITS

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Abstract*: A simple and cost-effective method of improving the seismic performance of RC structures is explored in this paper. Twelve archetypal RC moment frames were developed following the New Zealand design code. The number of stories ranged from 3 to 10. The buildings were designed to different drift limits (2.5, 2.0, 1.5, 1.0, and 0.5%) to evaluate the effect of increasing structural stiffness on their seismic performance. Incremental dynamic analyses were conducted for these structures and the intensity associated with 4% storey drift was computed as a means to assess their vulnerability to collapse. The results showed that for a ground motion with a peak ground velocity of 0.7m/s (comparable with the 2011 Christchurch earthquake), reducing the drift limit from 2.5 to 1.0% led to a 20-fold reduction in the likelihood of collapse. The increase in overall building cost associated with reducing the drift limit from 2.5% to 1.0% was less than 1%. An increase in floor acceleration demands related to reducing the drift limits was observed, however, the current guidelines for acceleration sensitive non-structural components result in the design capacity exceeding the acceleration demands. The results suggest that the effect of reducing drift limits on acceleration-sensitive non-structural components is negligible. Reducing design drift limits is a cost-effective way of improving the seismic performance of the structure as well as partitions, facades and finishes.*

1. Introduction

It is well understood that structural damage can be controlled by limiting drift. Algan (1982) has also shown that reducing drift demands limits damage to drift-sensitive non-structural components such as partitions, facades and finishes. In recent decades, novel “low-damage” structural systems have been proposed to reduce structural damage (Kelly et al., 1980; Nims et al., 1993; Smith and Willford., 2007). All the while, conventional reinforced concrete (RC) wall structures in Japan and Chile have consistently performed well in previous earthquakes by limiting drift and damage through stiffness (Lagos et al., 2021; Riddell et al., 1987; Shiga., 1968). This paper looks at the potential of lowering drift limits to reduce seismic damage. Twelve hypothetical RC frame structures between 3 and 10 storeys in height were designed to drift limits ranging from 2.5% to 0.5%, and nonlinear dynamic analyses were conducted to quantify the improvement in performance related to reduced drift limits.

The idea of limiting drift is not new. As early as 1923, T. Naito recommended the use of shear walls to make structures as stiff as possible (Naito, 1923). Since then, extensive research and field evidence has demonstrated the benefits of controlling drift (Cecen, 1979; Algan, 1982; Sozen, 1989; Hassan & Sozen, 1997; Pujol et al., 2021). Nevertheless, there is a concern within research and industry that reducing drift limits will lead to 1) a large increase in building cost, and 2) increased floor acceleration demands resulting in damage to suspended nonstructural components. In an informal poll at the 2023 New Zealand Society for Earthquake Engineering (NZSEE) conference, 48% (70/148) of voters identified cost as the “biggest barrier to improving the resilience of our (NZ) building stock”, however, this consensus seems to be unsubstantiated. In fact, Garcia et al. (1996) illustrated that reducing the drift limit from 2.5% to 1.0% by increasing structural stiffness would produce cost increases on the order of 5% to 10% of the cost of the structure. Freeman (1932) and later, Aslani & Miranda (2005) have shown that for a typical office or apartment building, the structure accounts for roughly 20% of the building cost. These results would suggest that a reduction in the design drift limit would lead to a trivial increase in the cost of the building. To strengthen the argument, the cost of each of the mentioned hypothetical RC frames was quantified and compared for the different design drift limits. The floor spectral demands for each structure were also examined to investigate the effect of reducing drift limits on nonstructural component demands.

2. Design of Archetypal RC frames

Twelve RC space frame structures were designed for a site in Wellington, New Zealand. A three, six, and 10 storey building were designed according to current New Zealand seismic design guidelines (NZS1170.5) with a standard drift limit of 2.5%. The structures were then redesigned for lower drift limits, including 2.0%, 1.0%, and 0.5%. Typical floor heights were 3.6 m with a first-floor height of 4.5 m. Each frame had three 6.0 m wide bays in each plan direction. The concrete strength (30 MPa) and yield strength of reinforcement were not varied (500 MPa) from frame to frame. Soil-structure interactions were ignored and columns were assumed to be fixed at the base. The buildings were designed for a site in Wellington with soil class C (shallow soil) and a hazard factor, Z = 0.4 (PGA ~ 0.4 g). Table 1 provides a summary of the general building properties.

Table 1: General building properties for the designed RC frame structures.

General building parameters	
Hazard factor, Z (PGA)	0.4 g
Soil Class, C, (Shallow soil)	C
T_g (Characteristic period of the ground)	0.7 s
Ductility factor, μ	4
Structural performance factor, Sp	0.7
Bay width	6 m
Number of bays	3
Floor area	324 m^2
1st floor height	4.5 m
Typical storey height	3.6 m
Average seismic unit weight	8 kPa

A force-based approach was used for the design. The equivalent static method outlined in NZS1170.5 was used to apply reduced lateral forces to the structure. The force reduction factor is calculated as μ/Sp ~ 5. To estimate the design drift demands and period of the structure the storey stiffness was estimated using Equation 1 proposed by Schultz (1992).

$$K_s = \frac{24E_c}{H^2}\frac{1+C_s}{\left(\frac{2}{\sum k_c}+\frac{1}{\eta_a \sum k_{ga}}+\frac{1}{\eta_b \sum k_{gb}}\right)} \tag{1}$$

E_c, is the elastic modulus of concrete and H, is the storey height. $\sum k_c$ is the relative flexural stiffness of the columns in that storey, equal to the sum of the uncracked second moment of area, I_c, divided by the length of the element, H, for the number of columns. $\sum k_{ga}$ is the relative flexural stiffness of the girders above the storey, equal to I_b/L, where L is the length of the beam and I_b is the uncracked second moment of area of the girder and slab calculated as a T-section with a width equal to twice the beam depth. Similarly, $\sum k_{gb}$ is the relative flexural stiffness of the girders below the storey. η_a and η_b are correction factors to account for changes in storey height from one floor to the next. η_i can be calculated using Equation 2 where H is the height of the storey being considered, and H_i is the height of the storey above (H_a) or below (H_b).

$$\eta_i = \sqrt{\frac{H}{H_i}} \tag{2}$$

When the flexural stiffness of the columns is larger than that of the girders, $\sum k_c/\sum k_g > 1$, the error in the boundary storey stiffness increases. C_s, is a correction factor used to approximate the stiffness of boundary stories, including the first and second storeys and the roof of the frame (Equation 3a-c).

$$C_1 = \frac{\sum k_C}{22\sum k_{ga}} \tag{3a}$$

$$C_2 = \frac{\eta_b \sum k_C}{32\sum k_{gb}} \tag{3b}$$

$$C_R = \frac{-\sum k_C}{55\sum k_{ga}} \tag{3c}$$

Displacements demands were calculated based on the lateral forces and estimated storey stiffness from Equation 1, accounting for the various modification factors within NZS1170.5 for ductility, P-delta effects, and building height. Rayleigh's method (Equation 4) was used to calculate the period of the structure. KE_r, is the relative kinetic energy, calculated as half the sum of the storey mass multiplied by the floor displacement squared for each storey. PE, is the potential energy, calculated as half the sum of the products of each force in the linear force distribution and the associated floor displacement. Column and beam sections were initially assumed and floor displacements were calculated. The column and beam sections were then resized to achieve the desired drift limit in an iterative process. Table 2 provides a summary of the varied parameters of the frame structures:

$$T = 2\pi\sqrt{\frac{KE_r}{PE}} \tag{4}$$

$$KE_r = \frac{1}{2}\sum_{i=1}^{n} mass_i \cdot \delta_i^{\,2} \tag{4a}$$

$$PE = \frac{1}{2}\sum_{i=1}^{n} F_i \cdot \delta_i \tag{4b}$$

Table 2: Varied building properties for the designed RC frame structures.

Varied building parameters	
Design drift limit	2.5 to 0.5%
No. of stories, N	3 to 10
Initial period, T	0.3 to 1.9 sec.
Relative stiffness, N / T	4 to 20
Base shear strength coefficient	0.07 to 0.60
Σ M_c / Σ M_b	1.7 to 4.0
Column height to depth ratio	3 to 8
Beam length to depth ration	7 to 15
Longitudinal reinf. ratio	1.1 to 2.4 %
Transverse area reinf. ratio	0.4 to 1.1 %

Columns and beams were designed to reach their flexural strengths, and not fail in shear or bond, for an assumed ultimate steel strength of 1.35 fy. The buildings were designed to have a strong-column weak-beam mechanism such that Σ M_c / Σ M_b was always greater than 1.2. Column and beam sections remained constant up the height of the building. The effective stiffness, k_e, is used as a metric to compare all 12 RC frame structures, defined as

$$k_e = \frac{N}{T} \tag{5}$$

where, N, is the number of stories and T is the initial period. It has been shown that RC shear wall structures in Chile and Japan have a period proportional to N/20 where $k_e = 20$, while steel frame structures in the US have a period of approximately N/5 ($k_e = 5$) (Goel & Chopra, 1997; Midorikawa, 1990) (Figure 1). As the drift limit reduces, the effective stiffness, k_e, increases. It was also observed that taller buildings designed to the same drift limit have a larger effective stiffness. Table 3 provides a summary of key building parameters for each of the hypothetical RC frames. The building ID is denoted as 'xSy' where 'xS' refers to the number of stories, and 'y' refers to the design drift limit (i.e. 3S2.0 is a 3-storey frame with a 2.0% drift limit). Designing a 3-storey frame to a 2.5% drift limit led to the structure having Σ M_c / Σ M_b < 1.2, failing to meet the code requirements for strength. As a result, an additional 3-storey frame was included designed to a 1.5% drift limit.

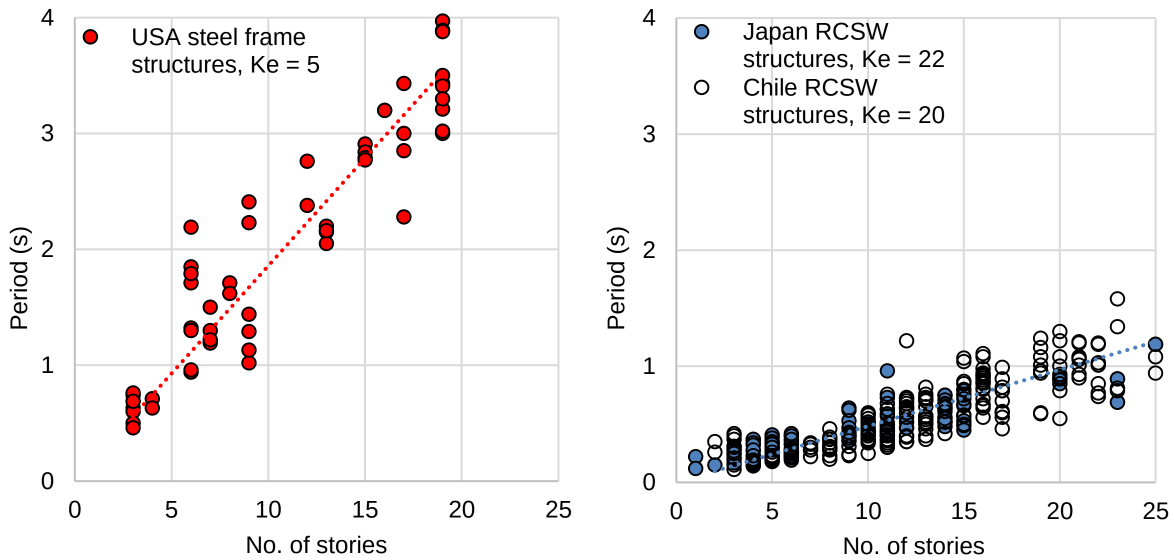

Figure 1: Relationship between the measured period, T, and the no. of stories, N, for (a) steel frame buildings in the US (Goel & Chopra, 1997), and (b) RC shear wall structures in Japan and Chile (Midorikawa, 1990)

Table 3: Seismic design characteristics of 12 archetypal reinforced concrete frame buildings.

No. of stories	Design drift limit	Building ID:	Initial period (s)	Base shear coefficient, C_y	Effective stiffness, k_e
3	2.0%	3S2.0	0.80	0.21	3.8
3	1.5%	3S1.5	0.61	0.28	5.0
3	1.0%	3S1.0	0.46	0.37	6.7
3	0.5%	3S0.5	0.28	0.60	10.8
6	2.5%	6S2.5	1.35	0.10	4.5
6	2.0%	6S2.0	1.19	0.12	5.1
6	1.0%	6S1.0	0.64	0.26	9.6
6	0.5%	6S0.5	0.36	0.60	16.8
10	2.5%	10S2.5	1.89	0.05	5.4
10	2.0%	10S2.0	1.46	0.07	7.1
10	1.0%	10S1.0	0.90	0.17	11.5
10	0.5%	10S0.5	0.51	0.50	20.1

*k_e is equal to the number of stories divided by the initial period for uncracked sections.

3. Numerical modelling of RC frames

Nonlinear structural models of the hypothetical RC frame structures were developed in OpenSeesPy (Mazzoni et al. 2000). Beams and columns were modelled as elastic beam-column elements and their nonlinear response was simulated using concentrated plastic hinges at beam and column ends. The models are based on centreline dimensions, and the use of rigid offsets was ignored. The hysteretic behaviour of the plastic hinges was modelled using the modified Ibarra-Medina-Krawinkler (IMK) peak-oriented material model which incorporates in-cycle and cyclic deterioration of component strength and stiffness. The plastic hinge parameters are estimate based on empirical equations derived from cyclic tests of columns (Lignos and Krawinkler, 2011). For the hypothetical RC frames designed to code, strength decay did not occur until after 4% drift based on the empirical equations. The RC frame structures were designed as space frames, with all columns resisting lateral loads and as such, modelling of a leaning column was not needed to simulate destabilising P-Δ effects. 2% Rayleigh's damping was applied to the first and third-mode period of the structure using the committed tangent stiffness matrix.

4. Incremental dynamic analyses

To study the performance of the hypothetical RC frame structures, incremental dynamic analyses were conducted. The numerical models of the designed hypothetical frames were analysed using the FEMA P-695 Far-Field ground motion set (FEMA, 2009), simulating the behaviour of the structure for a recorded ground motion. The ground motion set contains 22 ground motions with components in both horizontal directions (44 ground motion records). The epicentral distances ranged between 8km and 100km. The records were obtained from earthquake events with moment magnitudes between M6.5 and M7.6. PGV for the ground motion set varied between 15 cm/s and 115 m/s. PGA ranged from 0.2g and 0.8g. The duration of the ground motions varied from 20s to 100s. The significant duration of the ground motions ($Ds_{5\text{-}95}$) ranged from 4 to 51 seconds with a median $Ds_{5\text{-}95}$ of 11s. The ground motions were scaled until global instability occurred. Storey drift demands of 2% and 4% were also explored in the following sections. Previous research has shown a clear linear relationship between drift demands, the fundamental period, and PGV, in the absence of strength decay and P-Δ effects (Sozen, 2003 and Shah, 2021). Ongoing work by the authors has shown that PGV leads to less scatter in IDA results and a smaller coefficient of variation (standard deviation divided by mean) compared with other intensity measures; PGA, Sa(T1), and SaRatio. PGV is also structure-independent, and can therefore be used to more easily compare fragility curves of the structures. As a result, PGV was used to quantify the ground motion intensity during IDA.

The IDA curve for the 6-storey RC frame designed to a 2.5% drift limit is plotted (Figure 2a) illustrating the relationship between PGV and the peak storey drift, estimated from nonlinear dynamic analysis. There is considerable scatter in the plot, attributed to the record to record variation in the ground motions. The general trend shows an increase in the measured peak storey drift as the ground motion intensity (PGV) increases. P-Δ effects occur shortly after 4% storey drift with the slope of the IDA curves reducing. Figure 2b shows the comparison between Equation 6, the median IDA curve, and the median IDA curve plus one standard deviation for the 6-storey 2.5% drift limit RC frame. Equation 6 is derived from the "velocity of displacement" equation proposed by Sozen (2003) to estimate drift demands. T_i is the initial period for uncracked sections, Γ is the participation factor, and H is the height of the structure. The factor of 1.3 is an estimate of the storey drift to roof drift ratio, and the $\sqrt{2}$ factor was derived empirically. The equation was intended to provide a conservative estimate of the storey drift (Sozen, 2003).

$$SDR = \frac{PGV \cdot T_i}{\sqrt{2}} \cdot \Gamma \cdot \frac{1.3}{H} \quad (6)$$

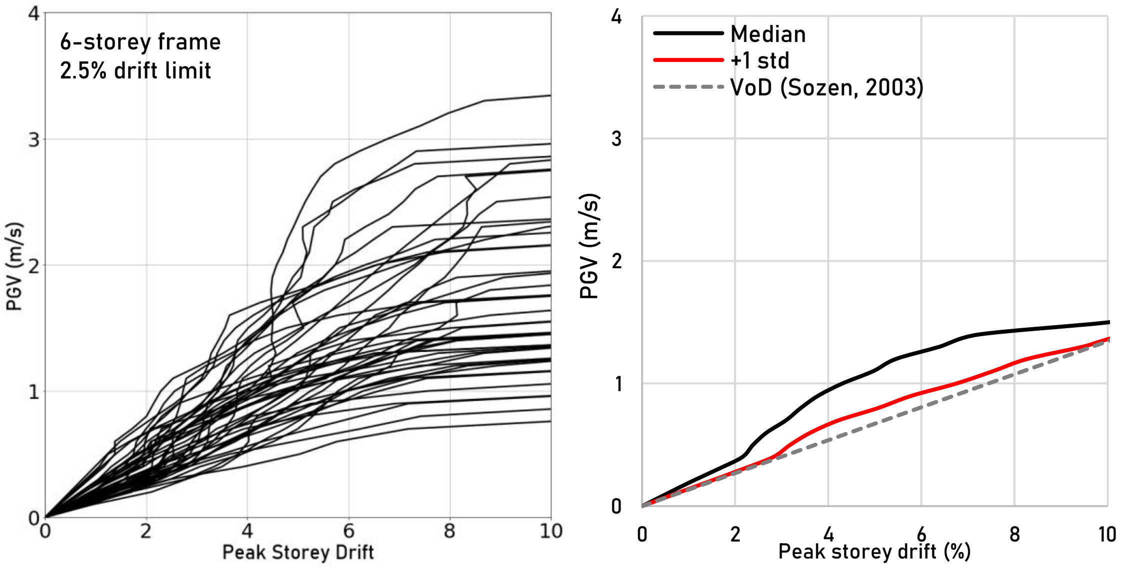

Figure 2: (a) IDA curve for 6-storey RC frame designed to 2.5%, and (b) comparison between the median IDA curve and drift demands estimated using the "Velocity of Displacement" (Sozen, 2003)

The results illustrate a strong similarity between the velocity of displacement and the upper bound of the IDA curve, highlighting the effectiveness of the empirical equation at estimating drift, even for a complex multi-degree-of-freedom system incorporating strength decay and P-Δ effects. To provide a comparison between all 12 hypothetical RC frames, fragility curves were plotted in Figure 4 and Figure 3 for exceeding 2% and 4% storey drift respectively. The 2% storey drift is related to total loss of partitions and facades (Algan, 1982), and 4% storey drift is related to the commencement of instability and nonlinearity in the relationship between PGV and drift suggested by Figure 2.

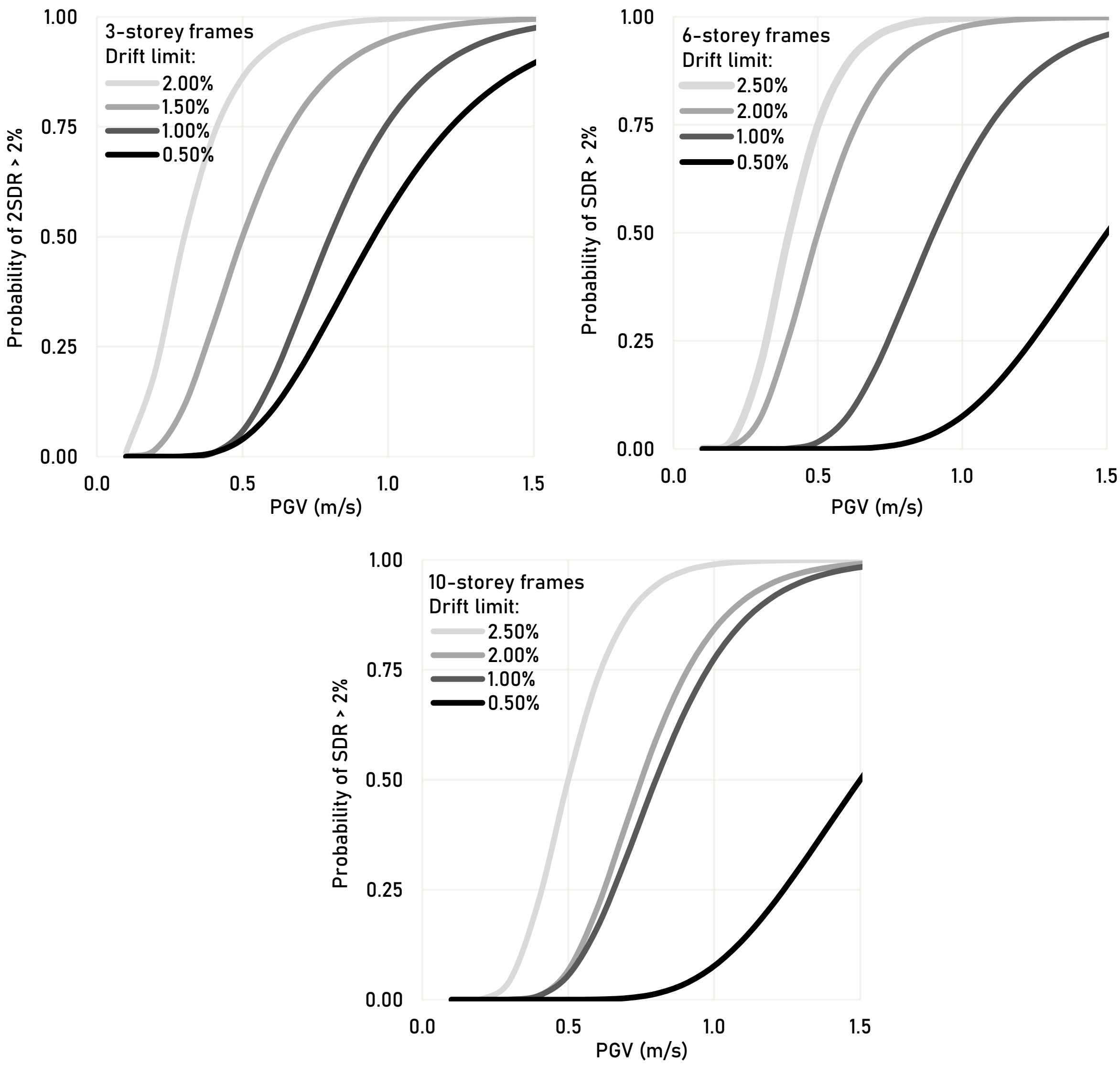

Figure 3: Fragility curves for RC frames (a) 3-storeys, (b) 6-storeys, and (c) 10-storeys showing the probability of collapse, defined as exceeding 2% storey drift vs the ground motion intensity (PGV).

Interestingly, the taller buildings tend to perform better when designed to the same drift limit as shorter buildings, with the median PGV for exceeding 2% or 4% storey drift being larger. The taller buildings designed to the same drift limit had a larger effective stiffness (Table 3). Figure 5 plots the effective stiffness of each frame and the design drift limit against the median ground motion intensity required to produce a storey drift demand of 4%. Figure 5 shows a strong linear relationship between the median PGV and effective stiffness (R^2 = 0.98) while the trend is less clear for PGV and the design drift limit (R^2 = 0.64). The results suggest that to control drift demands, focus should be placed on controlling the effective stiffness of the structure, rather than the design drift limit.

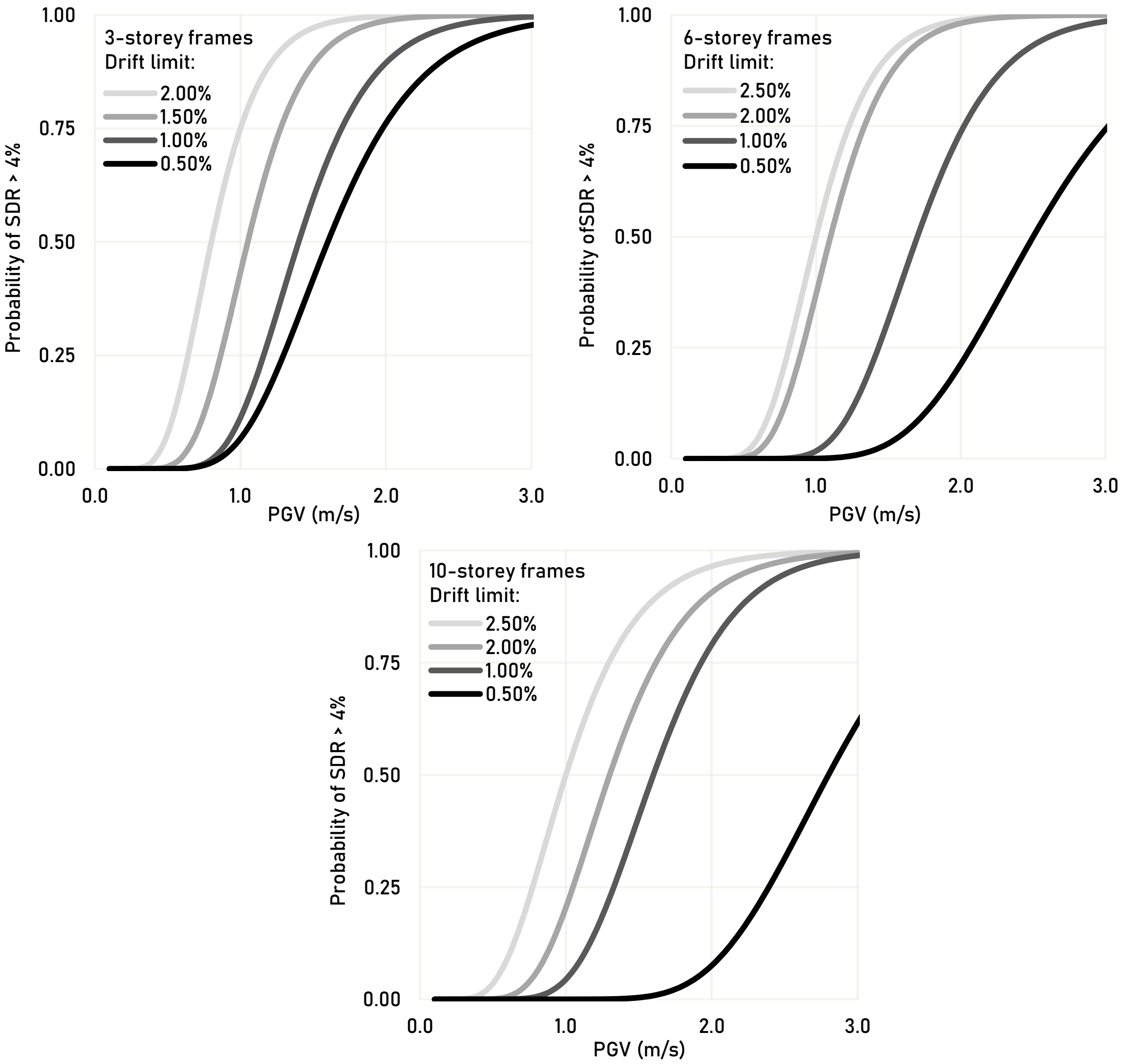

Figure 4: Fragility curves for RC frames (a) 3-storeys, (b) 6-storeys, and (c) 10-storeys showing the probability of collapse, defined as exceeding 4% storey drift vs the ground motion intensity (PGV).

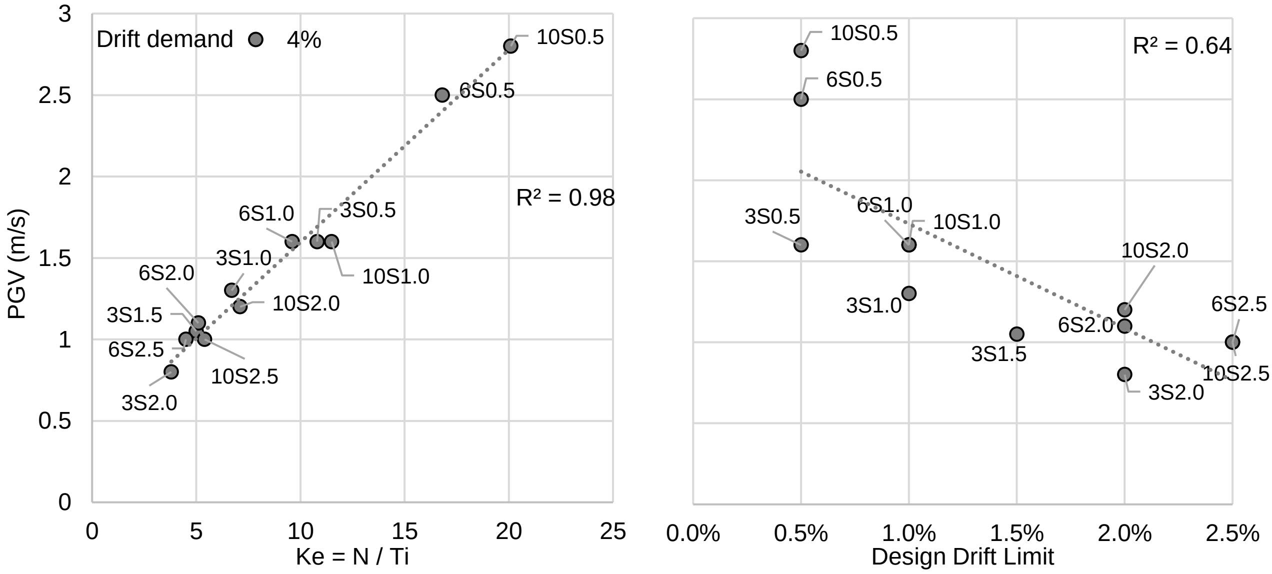

Figure 5: Relationship between PGV and (a) the effective stiffness, Ke, and (b) the design drift limit for a storey drift demand of 4%.

To provide a further comparison between the performance of the considered frames, the fragility curves illustrated in Figure 4 and 4 were used to estimate drift demands of the hypothetical structures for the 2011 Christchurch Earthquake. The PGV recorded at the CBGS station and around the CBD region was approximately 0.7 m/s. Figure 6 shows the likelihood of the hypothetical structures exceeding 2% and 4% storey drift for a PGV of 0.7 m/s.

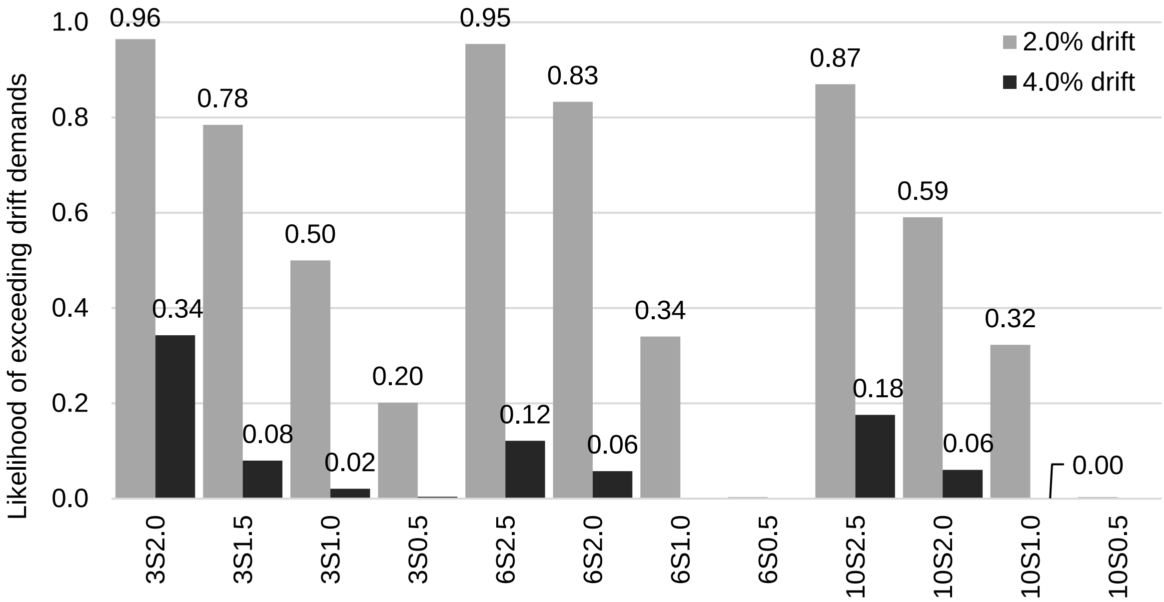

Figure 6: Likelihood of exceeding 2% or 4% drift for a PGV of 0.7m/s, representative of the ground motion intensity in the 2011 Christchurch earthquake.

None of the buildings designed to a 0.5% drift limit reached 4% drift. The likelihood of exceeding 2% drift showed a sharp decline as the drift limit was reduced. As the number of stories increased, drift demands decreased for the same design drift limit, likely due to the increase in effective stiffness. The 3-storey 2.0%-drift-limit structure (3S2.0) was 17 times more likely to exceed 4% drift compared with the 3-storey 1.0%-drift-limit structure. A similar trend is observed for the 6 and 10 storey structures. On average, reducing the drift limit from 2.5% to 1.0% led to a reduction in the likelihood of exceeding 2% storey drift by a factor of two, and a reduction in likelihood of exceeding 4% storey drift by a factor of greater than 20. For the hypothetical structures designed to a drift limit of 2.0% or 2.5%, the likelihood of exceeding 2% drift for a PGV of 0.7 m/s was 84%. It is expected that exceeding 2% drift would lead to considerable damage to drift-sensitive non-structural components (Algan, 1982). Furthermore, earthquake reconnaissance following the 2016 Kaikoura Earthquake has shown that extensive damage to pre-cast flooring systems occurred at storey drift demands of approximately 1 – 1.5% (Par et al., 2019). These numerical results suggest that a reduction in the design drift limit to 1.0% or less can lead to a large reduction in drift demands, as well as damage to the structure and drift-sensitive non-structural.

5. The cost of reducing drift limits

The idea of limiting drift is not new (Naito, 1923), and neither is our understanding of the associated cost (Garcia et al. 1996). Hare (2019) noted that resistance to change to design better buildings may be attributed to "a reluctance of the profession to accept that what has historically been done may not be adequate". To reinforce the idea that improving the building stock by reducing drift limits leads to a trivial increase in cost, the cost of the 12 hypothetical RC frames was quantified using QV Cost Builder (Version 2023-Q3), a quantity surveying catalogue which provides estimates of material, labour, and element costs for archetypal buildings. Table 4 shows the approximate base building costs for low-rise (3-5 storeys) and high-rise (6-15 storeys) office buildings per square metre. The base building cost is defined as the building cost, ignoring contents and outfitting of the building such as HVAC systems, furniture, fittings and equipment. The overall building cost is defined as the base building cost plus the cost of contents, legal fees, and land. Based on QV Cost Builder data, the estimated cost of the structure is approximately 20% of the base building cost for an archetypal office building. The cost of each frame was calculated using up to date costs for concrete, reinforcement, and formwork (Table 5). The unit costs provided in Table 5 account for labour and material costs, including delivery to site, wastage and loss, handling and placing.

Table 4: Cost breakdown of base building costs for a site in Wellington for low and high-rise office buildings.

	Low Rise Offices. 3 to 5 storeys.		**High Rise Offices. 6 to 15 storeys.**	
Element/Element Group	**$/m2**	**%**	**$/m2**	**%**
Substructure	145	5.6	204	5.4
Frame	194	**7.4**	238	**6.3**
Upper Floors	199	7.6	281	7.5
Structure	538	**20.6**	723	**19.2**
External Fabric	450	17.2	492	13.1
Internal Finishing	402	15.4	666	17.7
Services	826	31.7	1,339	35.6
External Works & Sundries	29	1.1	26	0.7
Prelims & Contingency	364	14.0	519	13.8
Total	2,600	100	3,800	100

It was assumed during design that the flooring systems and foundations remained unchanged for all of the buildings. As a result, the only change in cost was associated with the frame itself, the size of the beams and columns, and the quantity of reinforcement. The material and labour costs provided in Table 5 were used to quantify the change in cost attributed to the frame as it was designed to progressively smaller drift limits. For the low-rise office buildings, the frame accounts for approximately 7% of the base building cost. To estimate the change in cost associated with each building, it was assumed that the 7% was indicative of a building designed to the code minimum strength requirements and a 2.5% drift limit. As the drift limit was reduced and section sizes increased, the associated increase in cost was quantified. For example, if the frame costs 50% more for a drift limit of 1.0% compared to the conventional 2.5% drift limit, then the increase in the base building cost would be 7.0% x 50% = 3.5%.

Table 5: Unit costs for construction of cast in-situ RC frame structures accounting for labour and material.

Description	Unit	$ for a site in Wellington
Formwork to sides of:		
Beams or lintels	m2	233
Square and/or rectangular	m2	201
Reinforcement		
25mm diameter	tonne	5,500
32mm diameter	tonne	5,400
10mm diameter (stirrups)	tonne	7,300
Standard Ready-Mixed Concrete Delivered to site		
25 MPa, 19mm aggregate	m3	311

Table 6 provides a summary of the cost for each of the 12 frames and the increase in cost related to the base building cost. The largest increase in cost was for the 10-storey frame designed to a 0.5% drift limit (12.2%). Reducing the drift limit to 1.0% led to an increase in the base building cost of less than 4%. The results indicate that a reduction in the drift limit could be achieved without radical influence on the base building cost.

Table 6: Increase in base building cost associated with reducing the seismic drift limit.

No. of stories	**Drift limit**	**Frame Cost ($)**	**Inc. in frame cost**	**Inc. in base building cost**
3	2.0%	2.27E+05	-	-
3	1.5%	2.63E+05	16%	1.1%
3	1.0%	3.01E+05	33%	2.3%
3	0.5%	4.29E+05	89%	6.4%
6	2.5%	4.87E+05	-	-
6	2.0%	5.12E+05	5%	0.3%
6	1.0%	7.80E+05	60%	3.7%
6	0.5%	1.20E+06	146%	9.0%
10	2.5%	9.28E+05	-	-

10	2.0%	1.04E+06	12%	0.7%
10	1.0%	1.58E+06	70%	4.3%
10	0.5%	2.76E+06	197%	12.2%

To provide an estimate of the increase in cost relative to the overall building cost, the cost of land, contents, and professional & legal fees were accounted for (Figure 7). The land value for 20 addresses within the Wellington Central Body District (CBD) were assessed to estimate land costs using QV Cost Builder. The average cost for land in the Wellington CBD per square meter was approximately $9500.00 /m2. When quantifying the land value for the hypothetical RC frame structures, it was conservatively assumed that the land area was equal to the floor area of the building where in reality, the land area would be larger. Fit-out costs were provided within QV cost builder as $870 /m2 for a typical office building in Wellington. Fit-out costs are approximated to include sub-divisional partitions, amenities, furniture and loose fittings, and any special requirements of the tenant/s or owner/s. Professional fees were approximated as 10% of the base building cost plus fit-out costs. The professional fees include the fees of the consultants including: structural engineers, architects, electric engineers, and quantity surveyors.

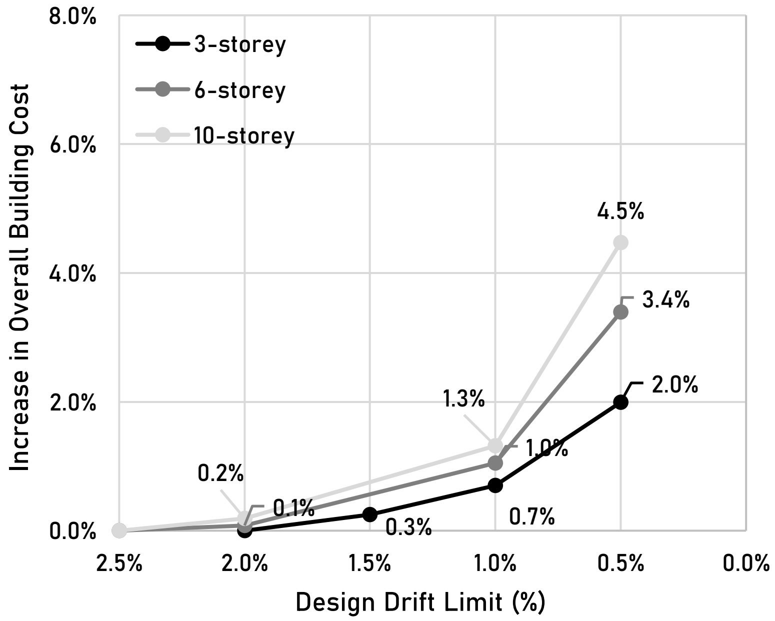

Figure 7: Estimated increase in the overall building cost associated with a reduction in the seismic drift limit for 3, 6, and 10-storey RC frame structures.

The most significant factor which affected the overall building cost was the land value. The assumed land value was approximately three times greater than the base building cost. The increase in the overall building cost attributed to a reduction in the design drift limit was calculated for each frame structure (Figure 7). The results show that a reduction in the drift limit from 2.5% to 0.5% would lead to an increase in the overall building cost of approximately 2.0% for a 3-storey RC frame, and 4.5% for a 10-storey RC frame. Designing to a drift limit of 1.0% would lead to an increase in overall building cost of approximately 1.0%. As shown by Garcia et al. (1996), it is expected that the increase in cost related to reducing drift limits for shear wall structures would be less. It is also expected that reducing drift limits would lead to a similar increase in cost for steel frame or braced frame structures.

6. The effect of reducing drift limits on floor acceleration demands

Reducing drift limits will lead to an increase in the stiffness and strength of a structure, subsequently causing larger acceleration demands. To quantify the change in floor acceleration demands related to reduced drift limits, floor demands were computed for the expected design level earthquake for a site in Wellington (PGA ~ 0.4 g) using the scaled far-field FEMA P-695 ground motion set. The 5% damped median floor acceleration response spectra were plotted for the 2nd storey of each of the 3-storey frames, along with the elastic design spectra for nonstructural components based on the New Zealand design standards (Figure 8).

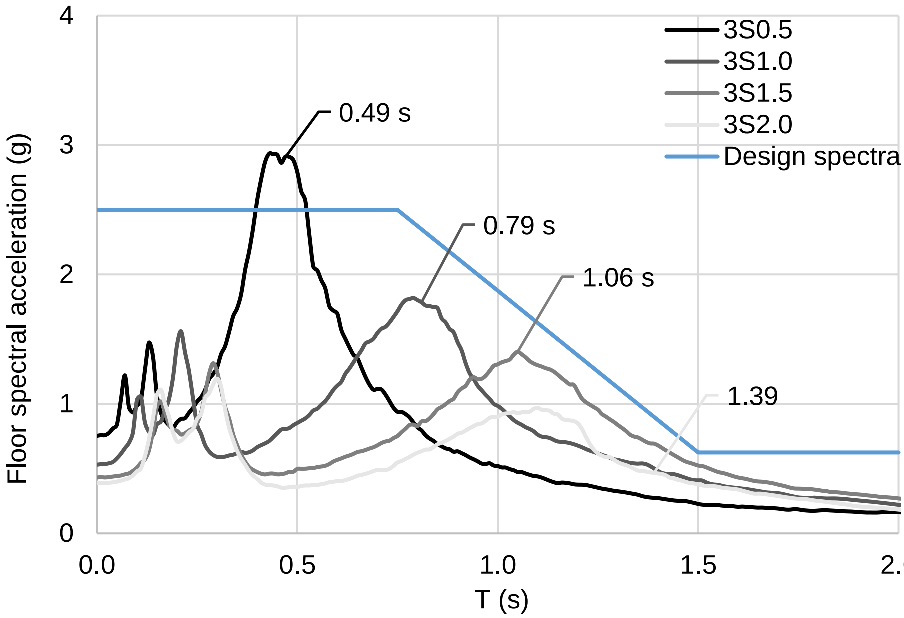

Figure 8: 5% damped floor response spectra for the 2nd storey of the 3-storey RC frame structures based on the design level earthquake with PGA ~ 0.4g. The NZ spectra for design of NSCs is plotted in blue.

The peak floor acceleration demands (at T = 0 s) increase as the drift limit reduces. Peaks in the floor spectra are observed at the fundamental period as well as the 2nd and 3rd modal period. The median spectral demands fall below the elastic design spectra for all of the frames except the 0.5% drift limit structure. For the RC frame designed to a 0.5% drift limit, there is a peak in the floor acceleration spectra at approximately 0.49 s related to the fundamental period of the frame for cracked sections. The sharp peak at 0.49 s is slightly above the elastic design spectra, suggesting that non-structural components designed with a period between 0.4 and 0.5 s may undergo resonance and experience damage as the spectral demands exceed the elastic design demands. Nevertheless, spectral acceleration demands can easily be reduced by 1) designing NSCs outside of resonance with the structure, and 2) accounting for ductility in the NSCs. Haymes (2023) has shown that a small increase in ductility from R = 1 (elastic) to R = 1.5, can lead to a reduction in spectra demands by up to a factor of 4. Furthermore, simply designing for the increased demands would lead to a negligible increase in cost when compared to the cost of the structure and overall building cost, as illustrated in section 5. These results suggest that elastic design of acceleration sensitive non-structural components is easily achievable, even for structures with a design drift limit of 0.5%.

7. Conclusions

A simple approach to improving the seismic performance of structures by reducing design drift limits was explored. Twelve hypothetical RC frames were designed to drift limits ranging from 2.5% to 0.5%. The number of stories ranged from 3 to 10. Nonlinear numerical models of the structures were developed and incremental dynamic analyses were conducted using the FEMA P695 far-field record set. A peak ground velocity of 0.7 m/s, representative of the recorded ground motion intensity for the 2011 Christchurch Earthquake, was used to compare the performance of the different structures. The likelihood of collapse, approximated as the probability of exceeding 4% storey drift, was on average 15% for structures designed to a 2.0% or 2.5% drift limit. The likelihood of collapse for a structure designed to a 1.0% drift limit was on average 0.7%, approximately 20 times less. The relationship between the effective stiffness, K_e, and the estimated drift demands from dynamic analyses was approximately linear, suggesting that the period to storey ratio (N/T_i) is more important than the drift limit when designing to limit drift. Controlling the effective stiffness of a structure in design may lead to more reliable estimates of drift demands and performance when compared the design drift limit. Recognising that designing to a lower drift limit requires larger beam and column sections, the cost of each structure was also quantified. Reducing the drift limit from 2.5% to 1.0% led to an increase in the overall building cost of approximately 1.0% when accounting for the cost of land, contents, and labour costs. Furthermore, it has been shown that the effect of reducing drift limits on acceleration sensitive non-structural components is negligible. Reducing design drift limits is a simple, cost-effective way of improving the seismic performance of the structure as well as partitions, facades and finishes.

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