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INTERPRETATION AND EVALUATION OF NZS1170.5 2016 PROVISIONS FOR SEISMIC RATCHETING

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ABSTRACT

During seismic events, some structures have a tendency to ratchet and displace more in one direction than in the opposite direction after yielding, resulting in larger peak and residual displacements. Provisions to define the tendency for seismic ratcheting and the resulting displacement amplification are provided in the 2016 amendments of NZS1170.5. This paper presents some insight into the factors causing ratcheting, along with interpretation and evaluation of the proposed provisions. Firstly, the mechanics of seismic ratcheting due to dynamic stability, eccentric gravity loads, and unbalanced structural strengths in the back-and-forth directions are discussed. Afterwards, the new provisions were detailed and demonstrated by working through the NZS1170.5 commentary examples. The authors' interpretation of the provisions is then presented, potential areas of confusion are identified, and wording changes to provide consistency and clarity are proposed. Finally, the displacement amplification factors provided in the 2016 amendments were evaluated using results of an independent study on single-degree-of-freedom reinforced concrete bridge columns subjected to eccentric gravity loading. It was found that the displacement amplification method proposed was reasonable, except when columns designed with a high ductility factor or which exhibit inelastic bilinear response had a significant tendency for ratcheting.

INTRODUCTION

In the Cambridge Dictionary, the term "ratchet" refers to "something that makes a situation change or develop in one direction only". During strong seismic events, several factors such as P-delta effects or different lateral strengths in the forward and reverse directions may contribute together to cause structures to predominantly deform more in one direction than in the other. Thus, this effect is termed seismic ratcheting. Such behaviour can result in (i) greater peak drifts, resulting in greater damage to structural and non-structural drift-sensitive elements (e.g., partitions); (ii) greater residual drifts, resulting in the structure being more susceptible to aftershocks or needing full replacement following a major earthquake [1]; and (iii) a greater probability of the structure collapsing. An example of this can been seen in Figure 1, where a steel bridge column tested by Kawashima et al. [2] predominantly displaced in one direction.

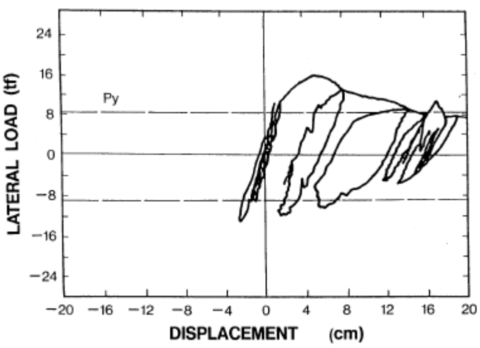

Figure 1: Shaking table test result for a steel bridge pier showing ratcheting behaviour (source: Kawashima et al. [2]).

One notable example of ratcheting due to unbalanced lateral strengths was the Hotel Grand Chancellor (HGC) building shown in Figure 2. To avoid blocking off an existing lane, the eastern bay was cantilevered off the building. The HGC building experienced significant damage during the 22 February 2011 seismic event; including collapse of the stairs and failure of a ground floor wall. One possible reason noted in volume 2 of the Royal Commission report [3] was the cantilevering action of the eastern bay, which could cause the building's lateral strength to decrease in the cantilevered direction and increase in the opposite direction, resulting in earlier yielding and greater displacements in the eastward direction. This was further backed by analyses performed for The Royal Commission shown in Figure 3, where the building had significantly larger response in the eastern direction if the cantilevered effect was considered (Figure 3a) while the response would be more similar in both directions had it been excluded (Figure 3b). The cantilevered effect was observed for all six ground motion records investigated; the four 22 February 2011 recordings closest to the HGC building (REHS, CBGS, CHHC, and CCCC); a composite earthquake comprising of the 4th September 2010, 26th December 2010, and 22 February 2011 recordings from the REHS station applied one after another (COMP); and an earthquake representing an Alpine Fault event. It could also be observed that the predicted response in the west direction decreased due to the cantilevered effect.

Volume 2 of the Royal Commission report [3], which detailed key findings from the investigation into the performance of the HGC building, stated that "the modal response spectrum method of analysis was used in the design without allowance for the eccentric gravity loads acting on the structure." Further in the report, it was explained that "the modal response spectrum and the equivalent static method of analysis are based

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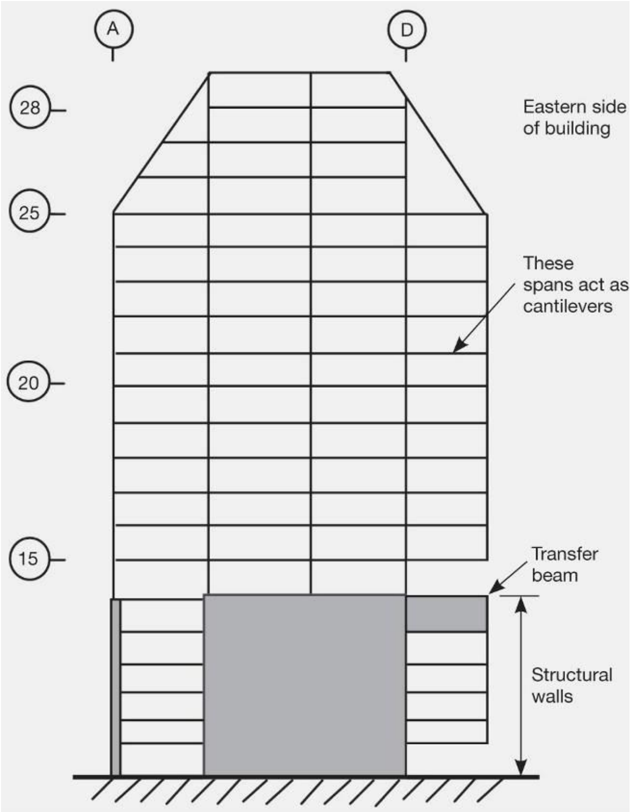
(a) View from south after the 2011 February earthquake.



(b) Schematic sectional elevation on moment resisting frames and structural wall in east-west direction.

Figure 2: The Hotel Grand Chancellor building (source: Cooper et al. [3]).

on the assumption that the strength and stiffness of a structure are equal for both forward and backward displacement. The cantilevering action in the HGC building violates this fundamental assumption so the analytical results based on elastic methods of analysis are incorrect." It was also noted that none of the investigative reports into the performance of the HGC building identified this effect, which "highlights the need for structural engineers to have a clear understanding of the basic assumptions involved in seismic design".

The 2016 amendments of NZS1170.5 [4] introduced two new clauses to address ratcheting effects in seismic design, in addition to clauses related to P-delta effects which already exist in the standard. Clause 4.5.3 identifies buildings with a tendency to seismically ratchet. If the tendency for ratcheting is not negligible, but is not significant enough to require time history analyses, then amplification factors proposed in Clause 7.2.1.3 can be used to modify displacements obtained via equivalent static method of analysis or the modal response spectrum approach However, supplementary publications detailing the derivation and validation of the clauses have not been published. As such, there is a need to evaluate these provisions to ensure that they are conceptually sound and clear in its wording.

In order to address this need, an independent study was conducted by the authors of this paper. In particular, answers are sought for the following questions:

- What are the causes of ratcheting, and which do the 2016 NZS1170.5 [4] provisions address?
- How were the 2016 NZS1170.5 [4] ratcheting provisions developed?
- What is proposed in the new NZS1170.5 [4] provisions, and can any improvements be made?
- How do the displacement modifiers proposed in Clause 7.2.1.3 of NZS1170.5 [4] compare against numerical results and other studies?

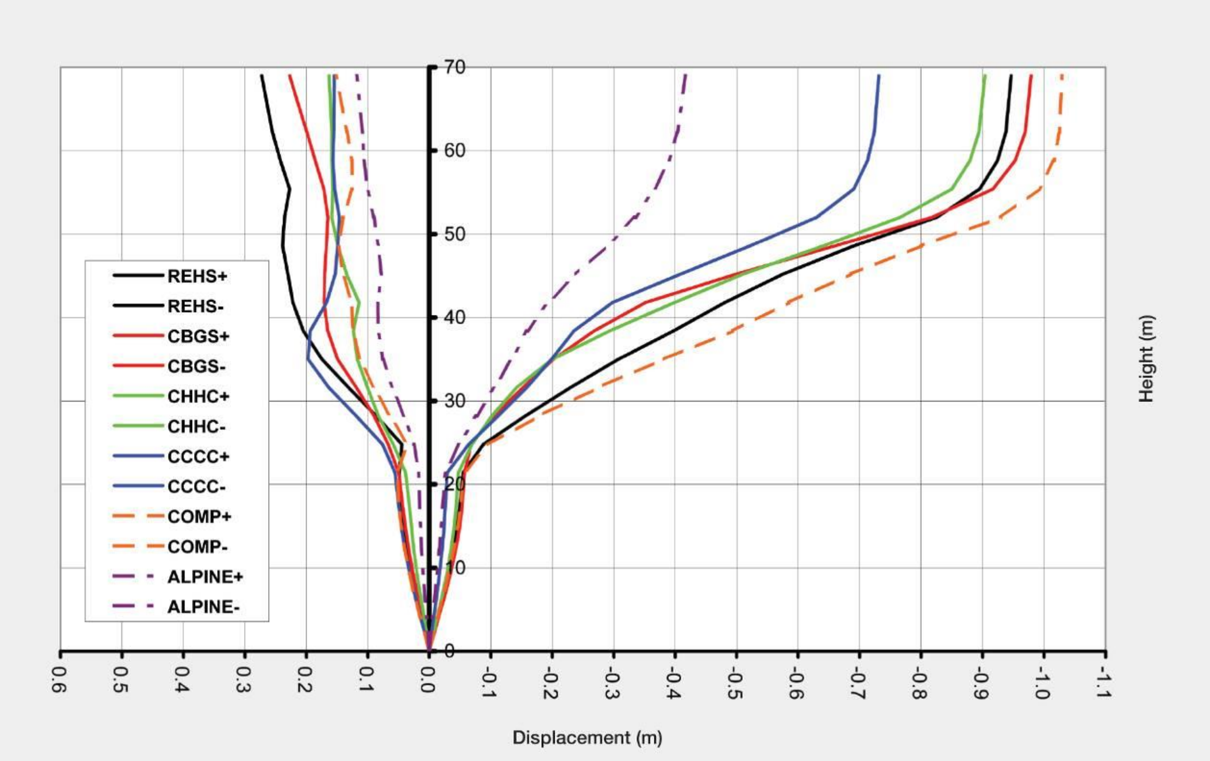
This paper details the causes of ratcheting and describes other studies examining this phenomenon in the "*Literature Review*" section. This is followed by a description of the new clauses in the "*NZS1170.5 Provisions to Address Seismic Ratcheting*" section. The authors' interpretation of the provisions is presented in "*Evaluation of Clause 4.5.3*" and "*Evaluation of Clause 7.2.1.3*" sections. Finally, the potential use of NZS1170.5 provisions to mitigate ratcheting is discussed in the "*Use of NZS1170.5 Provisions to Limit Ratcheting Effects*" section.

LITERATURE REVIEW

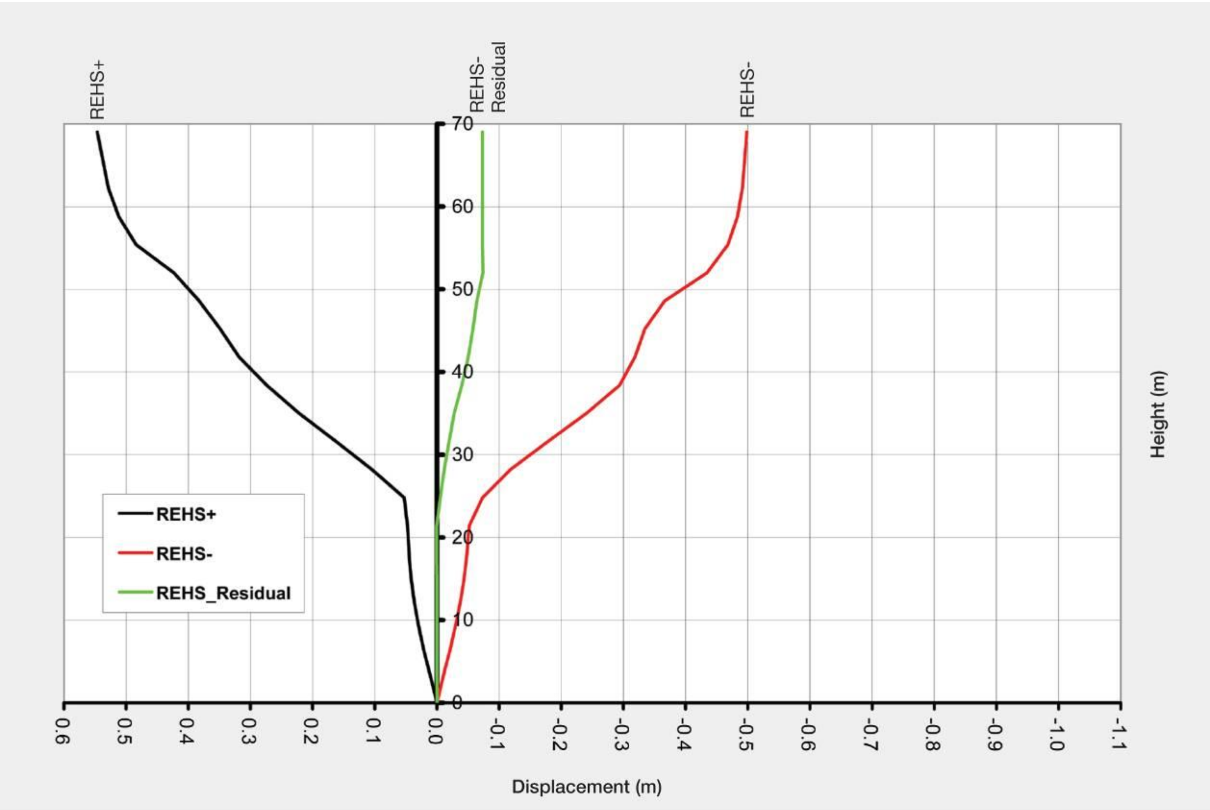
Causes of Seismic Ratcheting

Based on existing literature and from the authors' previous work [5-7], ratcheting can occur due to the following effects:

- **Ground motion effect**: some ground motions, such as pulse-type motions, may cause structures to deform more in one direction than another. Differential lateral loads caused by build-up of ground water pressure may also have an influence.
- **Dynamic stability effect**: structures with a negative post-elastic stiffness due to structural properties or P-delta effects may deform more in the direction of first yield.
- **Eccentric loading effect**: structures subjected to eccentric gravity loading may tend to deform more in the direction of the eccentricity.



(a) With cantilever action



(b) Without cantilever action

Figure 3: Numerical estimates of the maximum displacement relative to ground envelopes for the Hotel Grand Chancellor (source: Cooper et al. [3]).

- **Structural form effect**: structures with different lateral strengths in the forward and reverse directions may predominantly deform more in the weak direction. Having different stiffness in the forward and reverse directions can also have an influence.

The ground motion effect is complicated due to it being heavily dependent on fault type, rupture mechanism, local soil site conditions, epicentral distance, and other factors. However, the tendency for seismic ratcheting due to the ground motion effect can be minimized with appropriate structural form. As such, the focus will be primarily on other causes of ratcheting. Nevertheless, past studies on the effect of near-fault and pulse-type motion on seismic ratcheting of buildings can be found in literature [8-10].

The effect of dynamic stability on seismic ratcheting of structures was first described by MacRae and Kawashima [5]. They considered two different cases of bilinear response which had no initial forces acting on them; one with a positive post-elastic stiffness (Figure 4a), and another with a negative post-elastic stiffness (Figure 4b). The latter may occur due to large P-delta effects or a brittle structural response. If the building has a positive post-elastic stiffness and has a residual displacement at point A (on Figure 4a), the structure would have a lower yield strength in the negative direction. This results in a greater tendency for the building to yield back towards the zero-displacement position. In contrast, the building with the negative post-elastic stiffness would be more likely to yield away from the zero-displacement position, which results in ratcheting. Due to these observations, the first case is considered to be more dynamically stable than the second, though it must be emphasized that this does not eliminate ratcheting effects completely.

Structures subjected to eccentric gravity loads may also be susceptible to ratcheting effects during seismic events. MacRae and Kawashima [5] considered a column with the lateral force-displacement backbone hysteresis shown in Figure 5. They proposed that the effect of a moment applied to the column, M_{ecc}, caused by eccentric gravity loads on the lateral force capacity of the column can be approximated by applying an equivalent static lateral force, $S_{gravity}=M_{ecc}/L$, to the top of the column; where L is the column's height. This causes the baseline of the hysteresis curve to be shifted by M_{ecc}/L compared to a concentrically loaded scenario. Therefore, the strength provided in the forward direction will be increased by M_{ecc}/L and that in the opposite direction will be decreased by the same amount. These final strengths, which are the strengths adjusted considering the effect of the eccentric moment, are referred to in this paper as "effective strengths". This terminology differs from that used in NZS1170.5 [4], and justification for this is provided when evaluating the provisions in later sections of this paper. If the initial strengths for the concentrically loaded case were equal in the forward and reverse directions, then the column would have a smaller effective strength in the reverse direction, and thus only requires a smaller force to yield in the reverse direction compared to the forward direction; resulting in the column ratcheting in the reverse direction. Unequal stiffness in the back-and-forth directions can also affect ratcheting behaviour, though its effect is more complex [7].

Displacement Modification due to Ratcheting

Dupuis et al. [11] investigated the effect of gravity-induced lateral demand (GILD) on the seismic performance of shear wall buildings. In their research, the strength in the weak direction was increased by the moment caused by the GILD, M_{GILD}, as shown in Figure 6. Based on their findings, they proposed actions to address the amplification in building displacements and drifts as shown in Table 1, where α is the ratio between the applied eccentric moment, M_{GILD}, and the yield capacity of the member in the eccentric moment direction. As shown in Table 1, if M_{GILD} is significantly smaller than the yield capacity, Dupuis et al. [11] conclude that the effects of ratcheting can be ignored. If the eccentric moment is sizable, they proposed that non-linear response history analysis should be performed to properly consider the influence of ratcheting. All other cases require displacements to be amplified by a factor of 1.2.

Dupuis et al. [11] proposed that the range of α corresponding to these cases is dependent on the hysteresis loop shapes, and considered two types of structural systems; (i) systems with self-centring characteristics such as shear walls with high axial load, where the hysteresis will be flag-shaped which results in self-centring response, and (ii) other systems which do not exhibit self-centring, such as elastic-perfectly plastic hysteretic behaviour. These findings were used to form Clause 4.1.8.10 in the National Building Code of Canada (NBCC) [12]. However, NBCC does not describe how to determine whether the structure exhibits self-centring or not. Another drawback is that the analyses forming the basis of the provision are for reinforced concrete structures only, which raises questions on how applicable they are for other structural forms.

Mitigation of Ratcheting

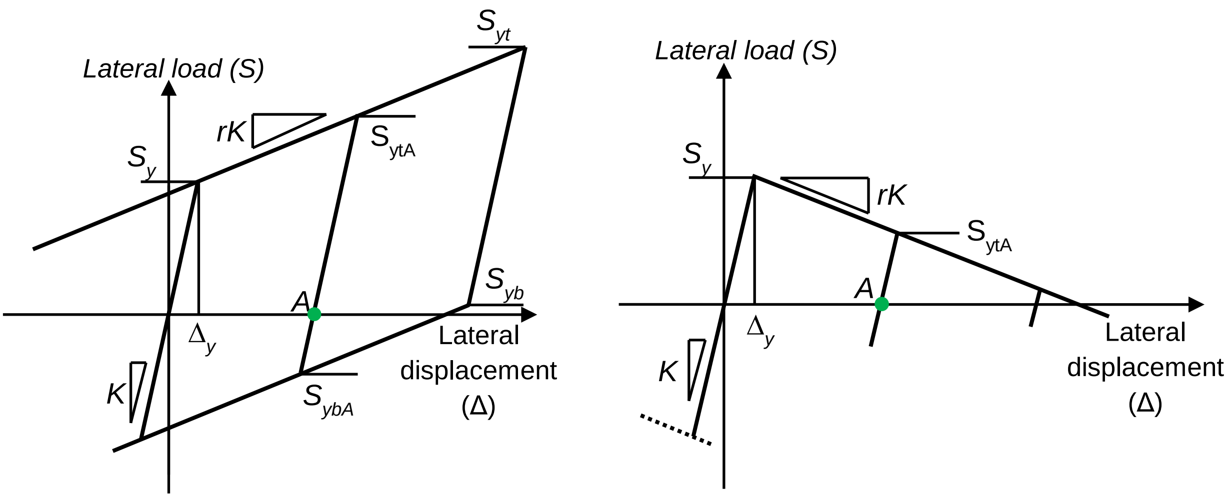
Mitigation of the likelihood of ratcheting of bridge columns subjected to eccentric loading has also been studied. This generally involves a strength increase in the direction of eccentricity as shown in Figure 7. If the direction of eccentricity acts in the "reverse direction", then the relation between the absolute value of design column lateral strength in the forward and reverse directions if gravity loads were concentric (ϕS_{fn} and ϕS_{rn}, respectively) excluding the effect of eccentric moments is as shown in Equation 1.

$$\phi S_{rn} = \phi S_{fn} + \beta S_{gravity} \tag{1}$$

Where $S_{gravity}$ is the equivalent lateral load caused by eccentric loading and β is the multiplier of $S_{gravity}$ by which the strength is increased.

Table 1: Summary of proposed code requirements for new gravity induced lateral demand irregularity (source: Dupuis et al., [11]).

Systems with self-centering characteristics	Other systems	Code requirement
$0.0 \le \alpha \le 0.1$	$0.0 \le \alpha \le 0.03$	No requirements
$0.1 < \alpha \le 0.2$	$0.03 < \alpha \le 0.06$	Multiply displacements by 1.2
$0.2 < \alpha$	$0.06 < \alpha$	Nonlinear response history analysis



(a) Positive bilinear hysteresis *(b) Negative bilinear hysteresis*

Figure 4: Examples of dynamic stability (source: MacRae and Kawashima [5]).

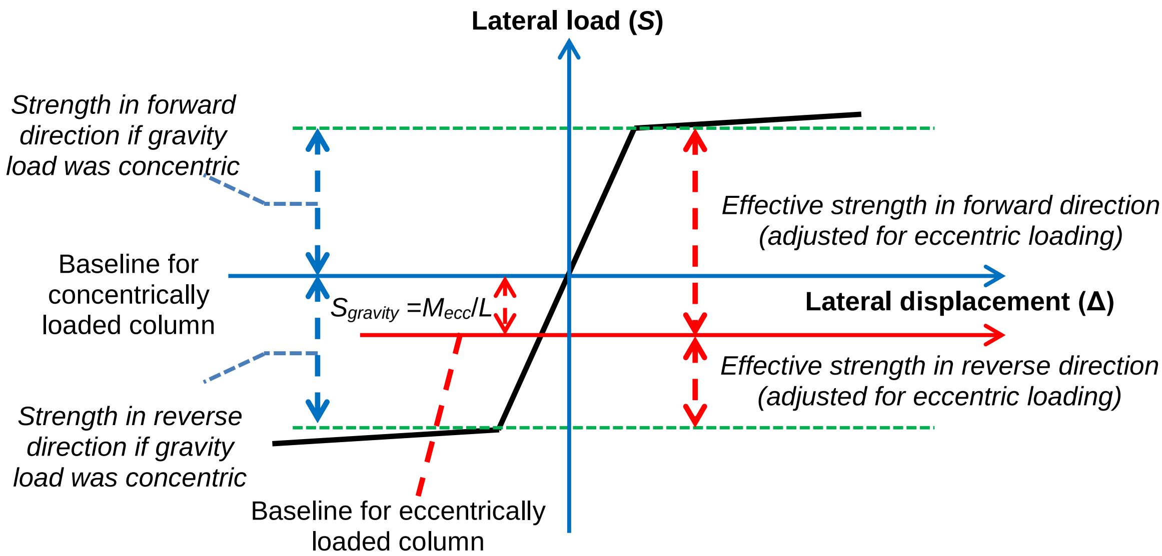


Figure 5: Eccentric loading effect on dynamic stability.

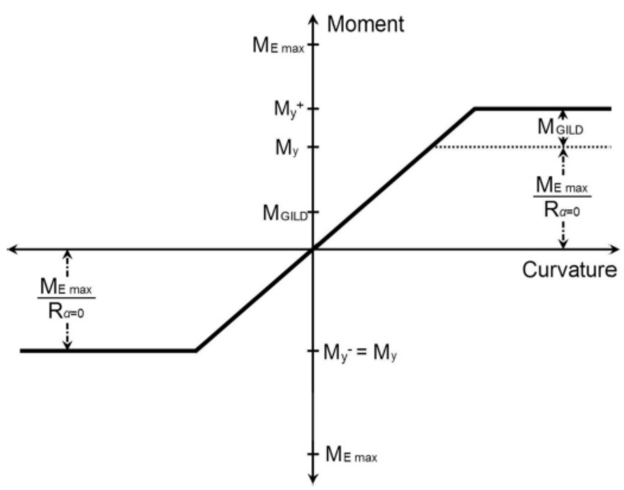


Figure 6: Relationship between flexural demands and capacities for shear wall (source: Dupuis et al. [11]).

The Japanese Road Association [13] proposed using β = 1.0 based on static loading considerations. In contrast, MacRae and Kawashima [5] stated that columns exhibiting bilinear hysteretic response should have equal lateral resistance against seismic actions in both directions considering the effect of loading eccentricity. If S_f and S_r are denoted as the "effective" lateral strengths in the forward and reverse directions, respectively, then these can be calculated from Equations 2 and 3, respectively. For S_f and S_r to be equal, β must be equal to 2.0.

$$S_f = \phi S_{fn} + S_{gravity} \tag{2}$$

$$S_r = \phi S_{rn} - S_{gravity} \tag{3}$$

Yeow et al. [6] showed that for columns with the same stiffness in each direction, the optimal strength increase, β_o, would be influenced by P-delta effects. This is because the yield displacement in the positive and negative directions relative to the original baseline are different, resulting in a different reduction in ϕS_{fn} and ϕS_{rn}. They proposed methods to estimate the β_o required for columns exhibiting inelastic bilinear response subjected to P-delta effect. However, they highlighted that β_o is harder to estimate for systems that have unsymmetric hysteretic behaviour. They explained this by examining the elastoplastic and Takeda [14] hysteretic behaviour with $\beta_o = 2.0$ excluding the effect of P-delta in Figures 8a and 8b, respectively. In the elastoplastic hysteresis case, the potential energy of the column at point A is equal to that required to cause yield in the opposite direction. In the case of Takeda hysteretic behaviour [14], the potential energy at A is smaller than that required to cause yield in the opposite direction. As such, the system is less likely to fully yield back towards the zero-displacement position for this case, and, hence, it requires a higher β. They suggested that β = 2.3 should be used for columns exhibiting Takeda [14] hysteretic behaviour with the P-delta effect included based on time history analysis. This study did not consider elastic stiffness variation in the different directions, which could be important if different quantities of reinforcing were provided in the forward and backward directions as stiffness and strength would have some degree of proportionality as discussed by Priestly and Kowalsky [15].

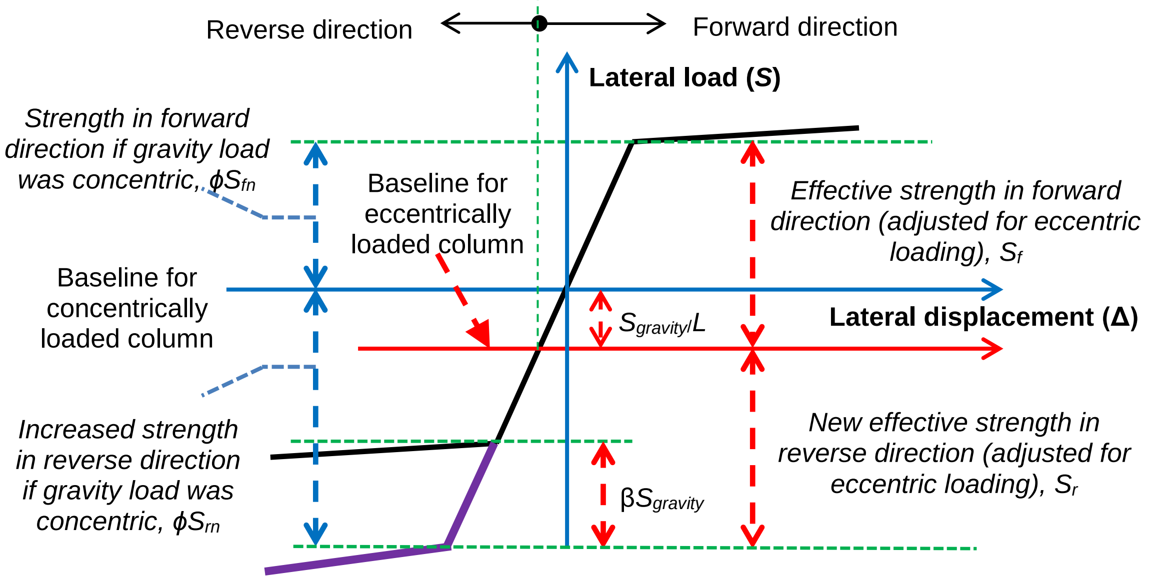
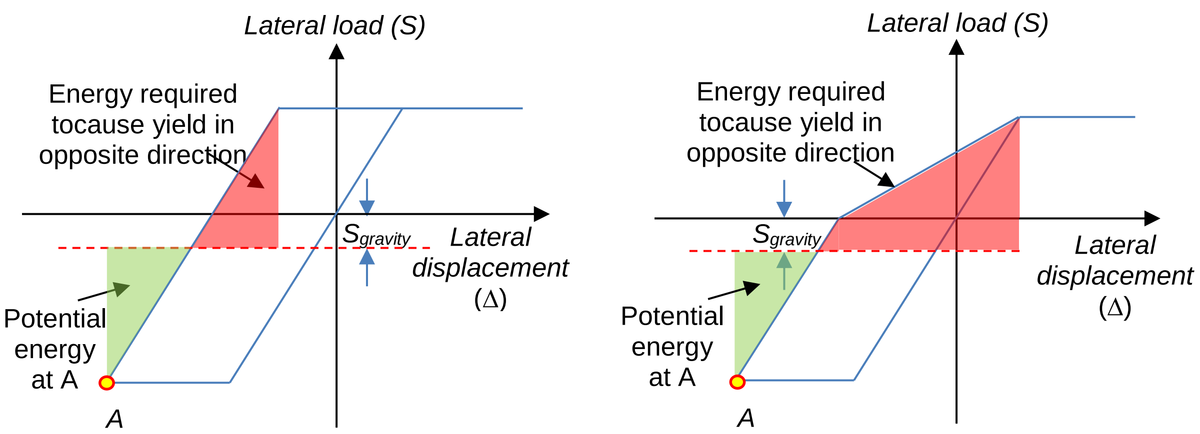


Figure 7: Ratcheting Mitigation.



(a) *(b)*

Figure 8: Energy considerations for columns subjected to eccentric loading; (a) elastoplastic hysteretic curve, (b) Takeda hysteretic behaviour.

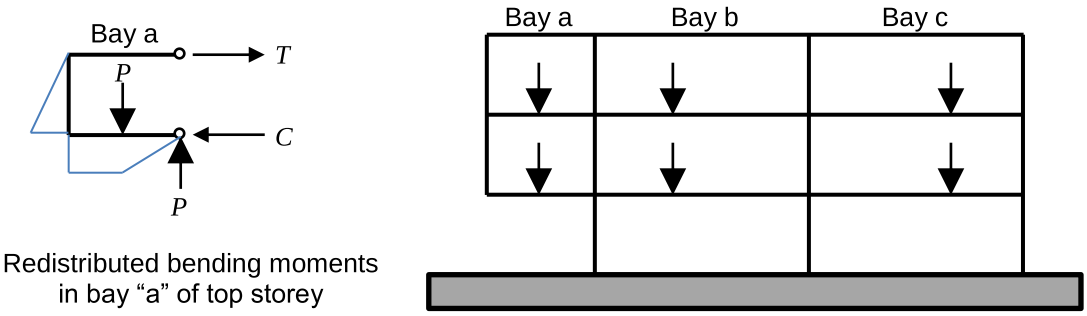


Figure 9: Illustration of additional lateral forces imposed on frame due to eccentric bay loading (source: NZS1170.5:C4.5.3 [16]).

NZS1170.5 PROVISIONS TO ADDRESS SEISMIC RATCHETING

This section summarizes the current ratcheting provisions in NZS1170.5 [4] and the corresponding numerical examples in the commentary [16]. The authors have provided some interpretation of the provisions where they felt it was necessarily, while evaluations of the provisions are detailed in the "Evaluation of Clause 4.5.3" and "Evaluation of Clause 7.2.1.3" sections.

Provision Background and Applicability

The 2016 amendments to Part 5 of the New Zealand Structural Design Actions standards, NZS1170.5 [4], provides provisions to determine the tendency and effect of seismic ratcheting for structures that behave in a ductile manner during strong shaking. Based on the descriptions provided in C7.2.1.3 of the commentary [16], the new ratchetting provisions were based on 1,500 time-history analyses performed on single-degree-of-freedom structures with plastic-hinge behavior being represented by the Takeda [14] hysteretic behaviour. The column's natural period was varied from 0.5 to 2.5 s, ductility values ranged from 1 to 5, and 10 different ground motion records were used. In discussions with the provision writer, P-delta effect and stiffness and strength interactions of reinforced concrete members were not considered in the analyses. The results of these analyses have not yet been published.

It was also noted that not all eccentricity loading effects would necessarily result in ratcheting. C4.5.3 of NZS1170.5's commentary [16] states that "in many cases, particularly in moment-resisting frames, moment redistribution associated with seismic actions and inelastic deformation reduces the difference in lateral strengths". This was illustrated by various examples shown in Figure 9. It was stated that for the case with eccentric gravity loads applied in bay "a", the eccentric gravity load could be converted into lateral forces *C* and *T*, and the rest of the building must be designed to enable the additional actions to be safety transferred to the base of the structure via a valid load path. For gravity loads in bays "b" and "c", the actions may be directly redistributed as axial loads into the columns and thus seismic ratcheting effects may be less prominent for such cases.

Clause 4.5.3: The Ratcheting Index

Clause 4.5.3(a) describes the tendency of a building to exhibit seismic ratcheting by the ratcheting index, r_i, in Equation 4.

$$r_i = r_{i,1} + r_{i,2} \tag{4}$$

According to Clause 4.5.3(ai), $r_{i,1}$ is the ratio of the "design lateral strength" in the forward direction, S_f, to the corresponding strength in the reverse direction, S_r, following Equation 5. The forward direction is taken as the direction of the higher lateral strength.

$$r_{i,1} = \frac{S_f}{S_r} \tag{5}$$

According to Clause 4.5.3(aii), $r_{i,2}$ can be calculated from Equation 6; where S_g is the "change in the lateral strength due to the portion of eccentric gravity load in the forward direction being balanced by a corresponding change in the lateral strengths of the structural elements". Furthermore, it was stated that "in cases where S_g is negative and the resulting value of r_i is less than 1.0 the ratcheting action is predicted to increase in the forward direction. In this situation the ratcheting index should be taken as the inverse of r_i".

$$r_{i,2} = \frac{S_g}{S_r} \tag{6}$$

In Clause 6.1.1, it was described that ratcheting effects should be considered only for limited ductile and ductile structures when r_i is greater than 1.15. If r_i is greater than 1.5, time history analysis must be used to determine the structure's displacements. If r_i is between these two bounds, amplification factors from Clause 7.2.1.3 may be used to modify displacements obtained via equivalent static or modal response spectrum methods of analysis. Clause 6.2.1.2 further mentions that when r_i is greater than 1.15, the design base shear strength in the weak direction has be equal or greater to that given in equation 6.2(1) of NZS1170.5 [4].

Clause 7.2.1.3: Increased Displacements due to Ratcheting

Clause 7.2.1.3(b) details that the displacements should be increased by an amount equal to 0.75(r_i -1) times the lateral deflections obtained using equivalent static or modal response spectrum method of analysis if $1.15 < r_i < 1.5$. It is described in C7.2.1.3 of the commentary [16] that this value was based on the average value obtained from the unpublished analyses which form the basis of the provisions, and was further increased by 20% to consider the potential for increased P-delta loads due to ratcheting actions. Discussions with the provision writer indicated that consideration of P-delta effect from Clause 6.5 is still required in addition to the 20% factor included in the ratcheting clause.

Ratcheting Index Calculation Examples

The following describes the supplementary examples to calculate r_i provided in C4.5.3 of the commentary [16]. The summary example calculations are shown in Table 2. It should be noted that the authors have provided additional figures (Figure 11) based on the authors' own interpretation of the application of the previously discussed clauses to aid in the understanding of the calculations. These figures may not be reflective of the provision writer’s intention.

In case “A”, the reinforcement layout on the “a” and “b” sides of the column are identical, resulting in the lateral strengths in both directions being 100 as shown in Figure 10a when there are no eccentric gravity loads applied. If there is an eccentric gravity load causing an equivalent lateral demand of 15, the lateral strength in the forward direction (S_f) will increase to 115, while the strength in the reverse direction (S_r) will decrease to 85, as shown in Figure 10b. The value of S_g is 0 since none of the eccentric load is balanced by any change in the lateral strength, and hence $r_{i,2}$ = 0. The ratcheting index is therefore r_i = $r_{i,1}$ = 1.35. This implies that seismic ratcheting is not negligible, but not significant enough to warrant time history analysis. Thus, Clause 7.2.1.3 must be used to determine the displacement modification factor required if displacements were estimated using equivalent static or modal response spectrum methods of analysis; though examples for this were not provided. A lateral force-displacement illustration of case “A” is shown in Figure 11a.

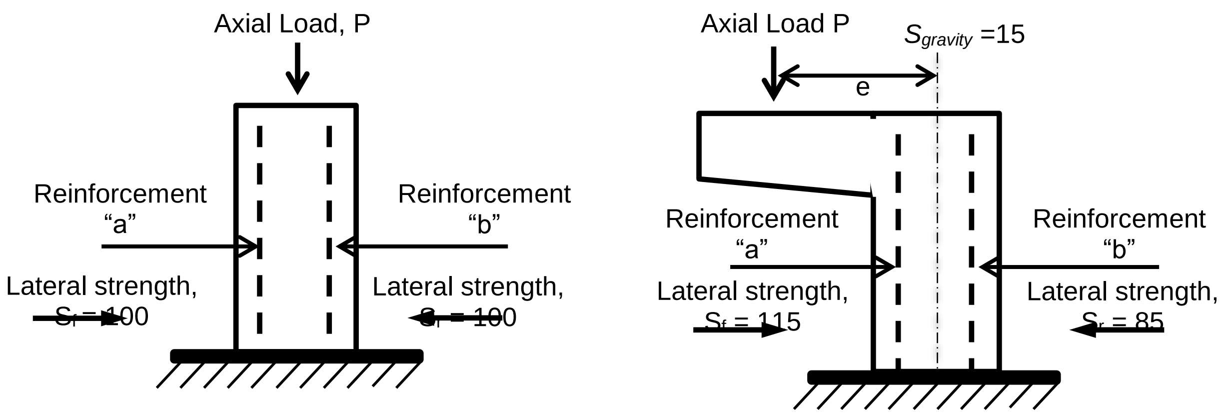
In case “B”, the quantity of reinforcement on side “b” is increased so that S_r increases from 85 to 100. The strength in the forward direction remains 115. Therefore, $r_{i,1}$ = 115/100 = 1.15. As the reinforcement is increased enough to balance the eccentric loading, S_g = 15. Therefore, $r_{i,2}$ = 15/100 = 0.15, and r_i for this case is 1.3. This still requires the use of Clause 7.2.1.3 to determine the displacement modification factor. A lateral force-displacement illustration of case “B” is shown in Figure 11b.

In case “C”, the quantity of reinforcement on side “b” is increased even further so that S_r is now 115, while S_f remains 115. Here, $r_{i,1}$ = 115/115 = 1.0. S_g is again taken as 15. Therefore, $r_{i,2}$ = 15/115 = 0.13, and r_i for this case is 1.13. Ratcheting effects do not need to be considered further in this case. A lateral force-displacement illustration of case “C” is shown in Figure 11c.

In case “D”, the quantity of reinforcement on side “b” is increased so that S_r becomes 130, while the S_f remains 115. The example in the commentary calculates $r_{i,1}$ = 115/130 = 0.88. The value of S_g is kept as 15, and hence $r_{i,2}$ = 15/130 = 0.12 and r_i for this case is 1.0. No ratcheting would be anticipated as mentioned in the NZS1170.5 commentary [16]. A lateral force-displacement illustration of case “D” is shown in Figure 11d. The authors note that this example is inconsistent with the original definition of S_r and S_f, as the direction with the larger lateral load capacity should be treated as the “forward” direction, and thus $r_{i,1}$ should be greater than 1.0. This will be discussed further in the following section.

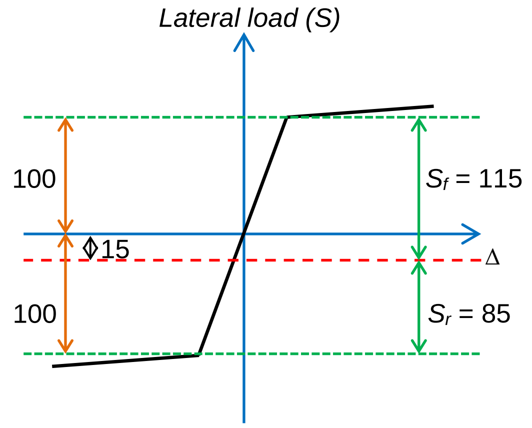
Table 2: Calculation of ratcheting indices (NZS1170.5:C4.5.3 [16]).

Case ID in commentary	S_f	S_r	S_g	$r_{i,1}$	$r_{i,2}$	r_i
A	115	85	0	1.35	0	1.35
B	115	100	15	1.15	0.15	1.30
C	115	115	15	1	0.13	1.13
D	115	130	15	0.88	0.12	1.00



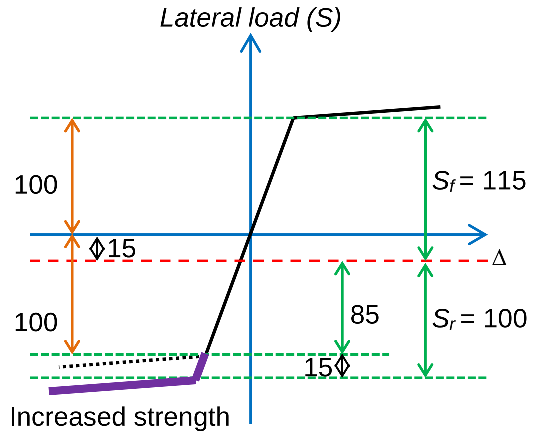
(a) Column without eccentric loading ***(b) C-bent column with eccentric loading***

Figure 10: Ratcheting index example for Case A (source: Standards New Zealand [16]).



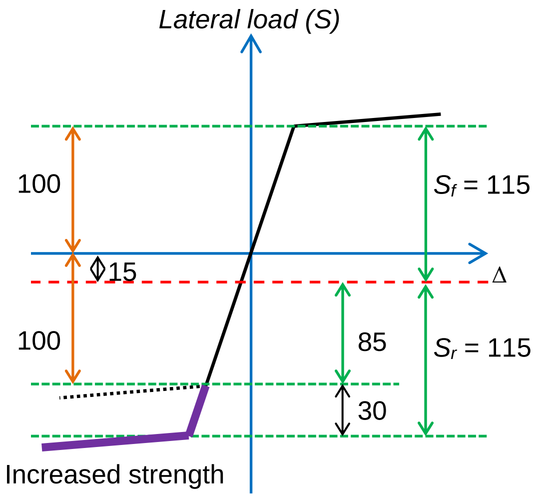
$r_i = S_f/S_r + S_g/S_r = 115/85 + 0/85 = 1.35$

(a) Case "A"



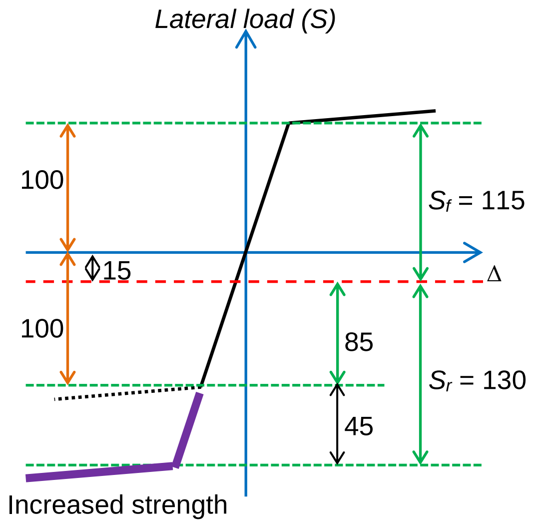
$r_i = S_f/S_r + S_g/S_r = 115/100 + 15/100 = 1.30$

(b) Case "B"



$r_i = S_f/S_r + S_g/S_r = 115/115 + 15/150 = 1.13$

(c) Case "C"



$r_i = S_f/S_r + S_g/S_r = 115/130 + 15/130 = 1.00$

(d) Case "D"

Figure 11: Visual representation of 2016 NZS1170.5 Clause 4.5.3 commentary examples.

EVALUATION OF CLAUSE 4.5.3

General Overview of Clause 4.5.3 Issues

The wording of Clause 4.5.3 is confusing at parts, which may possibly result in different interpretations of the provisions. These issues are elaborated upon in this section.

"Design Lateral Strength" Terminology

One case where different interpretations may exist is regarding the term "design lateral strength" used to describe S_f and S_r. According to Clause 7.2.2 of NZS1170.0 [17], the building's strength must satisfy the following condition at the ultimate limit state:

$$R_d \geq E_d \tag{7}$$

Where R_d is the design capacity (equal to a reduction factor, ϕ, multiplied by a nominal capacity) and E_d is the design action effect.

In reviewing the structural standards for various construction material types, such as NZS3101 for concrete structures [18], ϕ is usually applied at a member level and can be different depending on the action (i.e. axial, shear, flexural). Consider the case of a bridge pier column which lateral strength is governed by its flexural capacity (nominal moment capacity of M_{fn} and M_{rn} in the forward and reverse directions, respectively), and loading combination (f) from Clause 4.2.2 of NZS1170.0 (i.e. permanent (G), imposed (Ψ_EQ), and earthquake (E_u), actions) where the gravity loads were eccentric. The design moment capacities, ϕM_{fn} and ϕM_{rn}, must satisfy the following condition:

$$\phi M_{fn} \geq -(M_G + M_{\Psi EQ}) + M_{EU} \tag{8a}$$

$$\phi M_{rn} \geq M_G + M_{\Psi EQ} + M_{EU} \tag{8b}$$

Where M_G, $M_{\Psi EQ}$ and M_{EU} are moment demands at the base of the bridge pier resulting from G, Ψ_EQ and E_U, respectively.

If M_G and $M_{\Psi EQ}$ were shifted to the left-hand side of the equation as shown in Equation 9, then the left-hand side of the equation can be viewed as the "effective" flexural capacity remaining to resist moment demands caused by seismic actions.

$$\phi M_{fn} + M_G + M_{\psi EQ} \geq M_{EU} \tag{9a}$$

$$\phi M_{rn} - \left(M_G + M_{\psi EQ}\right) \geq M_{EU} \tag{9b}$$

Assuming that the mass of the column were lumped at the top, and that the column height is L, then the remaining lateral load capacity to resist seismic actions are:

$$\frac{\phi M_{fn}+M_G+M_{\psi EQ}}{L} = \phi S_{fn} + S_{gravity} = S_f \geq S_{EU} \tag{10a}$$

$$\frac{\phi M_{rn}-(M_G+M_{\psi EQ})}{L} = \phi S_{rn} - S_{gravity} = S_r \geq S_{EU} \tag{10b}$$

Where $S_{gravity}$ is the effect of gravity loading on the lateral capacity, and S_{EU} are lateral loads imposed by seismic actions.

Recall that the "design capacity" (R_d) is defined as ϕ multiplied by a nominal capacity. In Equations 10a and 10b, this would indicate that ϕS_{fn} and ϕS_{rn} are the "design lateral strengths". However, the authors understand and agree that S_f and S_r must be adjusted to consider $S_{gravity}$ (i.e., left-hand side of Equations 10a and 10b, respectively) before being used in Equation 5, and thus recommend that a different terminology be used to define S_f and S_r. While this approach is based on a simple column example, it is similar to lateral capacities determined from pushover analyses provided that design strengths were adopted in the structural modelling.

Inconsistency in Example Case "D"

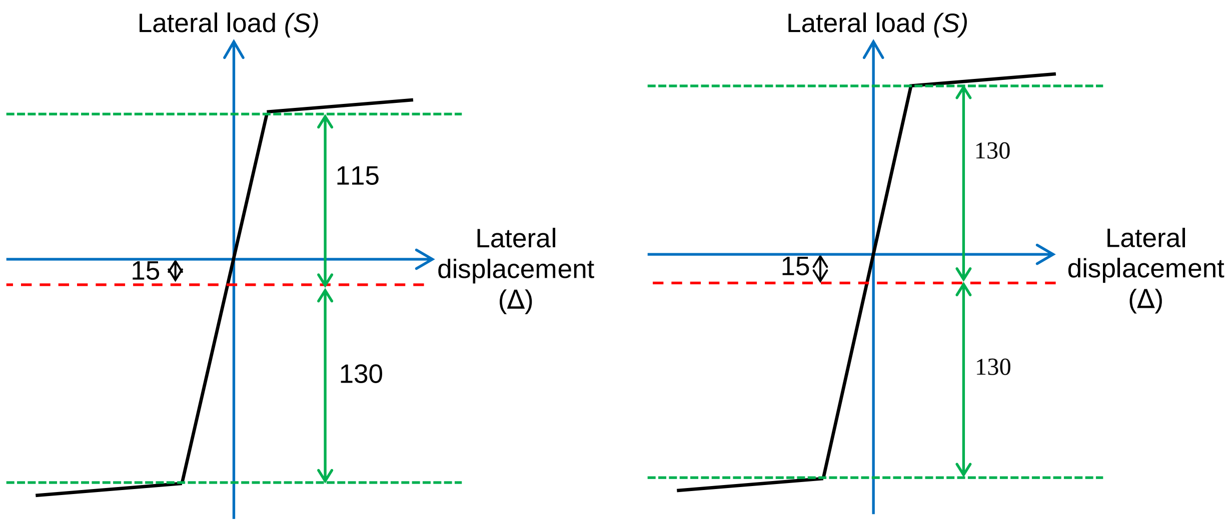
According to Clause 4.5.3, the forward direction corresponds to the direction of the higher lateral strength; i.e., S_f must be greater than S_r. However, example case "D" violates this as S_f is smaller than S_r. If code definitions were followed, then S_f and S_r should have been taken as 130 and 115 in case "D", respectively. In this case, S_g should be taken as a negative value (-15) since it acts in the "forward" direction as the forward and reverse directions have swapped. This would result in $r_{i,1}$ = 130/115 = 1.13; $r_{i,2}$ = -15/115 = -0.13; and r_i = 1.13-0.13 = 1.0. Both approaches (Table 2 and the approach detailed in this paragraph) results in identical r_i values.

Definition of S_g

S_g is defined in Clause 4.5.3(aii) as "the change in the lateral strength due to a portion of the eccentric gravity load in the forward direction being balanced by a corresponding change in the lateral strength of the structural elements". One potential misinterpretation is regarding the words "change in lateral strength", as it is not made clear what strength this change is relative to. Say for example than an engineer's initial design is identical to that in case "D" as shown in Figure 12a, where S_g = 15. If, for example, the strength acting in the opposite direction of the eccentricity was insufficient according to equation 6.2(1) of NZS1170.5 [4] and had to be increased to 130 as shown in Figure 12b, the change in lateral strength is 15 in the direction opposing the eccentric gravity load. One interpretation would be that S_g is now 0 (from 15 initially) if considering the change of strength relative to case "D". As S_f and S_r are the same in this case, $r_{i,1}$ = 1.0 and hence r_i = 1.0. However, this case is identical to example case "C" from NZS1170.5's commentary [16] (see Table 2) in concept (i.e. $S_f = S_r$ with an eccentric load present), but in case "C" r_i is greater than 1.0. It therefore follows that the provision writer of Clause 4.5.3 of NZS1170.5 [4] intend for r_i to be greater than 1.0 in this scenario. It should be noted that in the examples provided, the "change in lateral strength" was only for the reverse direction, S_r, relative to the case where the same amount of reinforcing was applied on both sides of the column. If this is indeed the definition of S_g, this needs to be clarified.

Consideration of P-Delta Effects

Finally, as mentioned previously, discussions with the provision writer indicated that the 20% increase in displacement amplification factor only addresses possible increase in P-delta actions due to ratcheting displacements. It does not replace the need to assess P-delta effect from Clause 6.5. This is not clear based on the wording in C7.2.1.3 of the commentary [16].



(a) *(b)*

Figure 12: Illustration of example demonstrating complication of S_g; (a) case "D", and (b) increase in lateral strength in direction opposing eccentricity.

Proposed Revised Definitions

This section provides recommendations on clarification of the various complications related to Clause 4.5.3 based on the interpretation of the provisions and its application in the examples provided in the commentary. These do not seek to provide derivation or validate assumptions of the ratcheting clauses in NZS1170.5 [4]. They aim to provide a means of ensuring that the NZS1170.5 [4] provisions can be modified in a way that is easier to interpret.

Firstly, the authors propose that different terminology be used to describe S_f and S_r. As previously stated, "design lateral strength" by code definition would indicate ϕS_{fn} and ϕS_{rn}, while the intent of Clause 4.5.3 is to firstly adjust for the gravity loading effect before applying Equation 5. Perhaps "design lateral strength adjusted for eccentric gravity actions", "effective design lateral strength", or "remaining design lateral strength to resist seismic actions" could be more appropriate definitions.

Secondly, two separate alternative approaches are proposed here to address complications regarding the forward/reverse direction. Both alternatives give similar results if they are applied consistently. These are:

- **Alternative 1** – retain the definition that the forward direction corresponds to the direction of larger strength, and change example case "D" to be consistent with this.
- **Alternative 2** – redefine S_f as the direction opposing the eccentricity, which ensures consistency with example case "D". In the event that $S_f + S_g$ is smaller than S_r (i.e., the column is more likely to ratchet in the direction opposite to eccentricity), then the inverse of r_i should be taken to indicate the tendency for ratcheting in the forward direction. The latter condition means that, if no eccentric moments were applied, the selection of S_f and S_r becomes arbitrary.

Thirdly, the absolute value of S_g can be defined by Equation 11, where $S_{gravity}$ is the reduction in lateral capacity due to gravity load effects. If Alternative 1 is used for determining the "forward" and "reverse" directions, then S_g is negative if the eccentric moment acts in the forward direction (Equation 11b) and positive otherwise (Equation 11a). If Alternative 2 is used, then S_g is always positive (Equation 11a) as the eccentric moment is assumed to always act in the reverse direction in this case. In either case, it is the authors' interpretation from the examples provided in the commentary that S_g cannot exceed $S_{gravity}$. This would need further confirmation.

$$S_g = min\begin{bmatrix} S_{gravity} \\ max(S_r - S_f + 2S_{gravity}, 0) \end{bmatrix} \tag{11a}$$

$$S_g = -min\begin{bmatrix} S_{gravity} \\ max(S_f - S_r + 2S_{gravity}, 0) \end{bmatrix} \tag{11b}$$

Finally, it should be made clear in the commentary that the increase of 20% in the displacement amplification factor does not discount the need to perform additional P-delta checks mentioned in Clause 6.5 of NZS1170.5 [4].

Additional Examples to Demonstrate Application of Proposed Revised Definitions

Three further examples proposed by the authors are examined here to demonstrate the recommendations. These examples, together with the original four examples, are listed in Table 3. The first example (case "II" in Table 3) demonstrates the calculation of S_g using Equation 11.

Example 1: Consider case "II" from Table 3, where design lateral strength in the reverse direction including eccentric load effects, S_r, is increased from 85 in case "A" to 95. The design lateral strength in the forward direction including eccentric load effects, S_f, remains the same at 115. Therefore, $r_{i,1} = S_f/S_r$ = 115/95 = 1.21. Using $S_{gravity}$ = 15, $S_g = S_r - S_f + 2S_{gravity}$ = 10. Therefore, $r_{i,2} = S_g/S_r$ = 10/95 = 0.11. The ratcheting index, r_i, for this case is $r_{i,1}+r_{i,2}$ = 1.32. In this case, displacement modifiers from Clause 7.2.1.3 are required if equivalent static or modal response spectrum methods of analysis were used.

The next two examples consider identical column configurations, but are based on the different interpretations of the forward and reverse direction. Alternative 1, where the wording is consistent with Clause 4.5.3, is examined first.

Example 2: Consider case "VI(a)" from Table 3, where the design lateral strength including eccentric load effects is 145 in the direction of eccentricity and 115 in the opposite direction, and $S_{gravity}$ = 15. Based on this, and using Alternative 1 where the forward direction is in the larger strength direction, S_f = 145 and S_r= 115. As the eccentricity acts in the forward direction, S_g = -15 using Equation 7. Therefore, $r_{i,1}$ = 145/115 = 1.26 and $r_{i,2}$ = -15/115 = -0.13. The ratcheting index, r_i, for this case is 1.13, and seismic ratcheting need not be considered further.

Example 3: Case "VI(b)" from Table 3 considers the same scenario as Example 2, but using Alternative 2 where the forward direction is in the direction opposing the eccentricity. Based on this, S_f = 115 and S_r = 145. This is because the eccentricity acts in the direction corresponding with the design lateral strength of 145. Using Equation 7, S_g = 15. Therefore, $r_{i,1}$ = 115/145 = 0.79 and $r_{i,2}$ = 15/145 = 0.10. As the sum of $r_{i,1}$ and $r_{i,2}$ is smaller than 1.0, the inverse of the sum is taken instead. This results in r_i = 1.12, which is near identical to the interpretation based on Alternative 1, which indicates that both approaches are reasonably consistent with each other.

EVALUATION OF CLAUSE 7.2.1.3

In this section, a case study considering the performance of bridge columns with eccentric loadings was examined to evaluate if the displacement increase estimates provided in Clause 7.2.1.3 are reasonable.

Comparisons were also made against recommendations from Clause 4.1.8.10 of the National Building Code of Canada [12], which is based on the study by Dupuis et al. [11] discussed previously. To enable this comparison, the α parameter adopted by Dupuis et al. [11] needs to be converted back to r_i. As seen previously in Figure 6, Dupuis et al. [11] assumed the same moment capacity in both directions ($M_{E,max}/R_{\alpha=0}$) before adjusting for the actions caused by eccentric lateral loads. As such, this can be treated as being akin to ϕS_{fn}. Afterwards, the strength in the direction of eccentricity was increased by M_{GILD}, so $\phi S_{rn} = \phi S_{fn} + S_{gravity}$. S_g can be taken to be equal to $S_{gravity}$, while $S_f = \phi S_{fn} + S_{gravity}$ and $S_r = \phi S_{rn} - S_{gravity} = \phi S_{fn}$. As previously stated, α is the ratio between the applied eccentric moment, M_{GILD}, and the flexural yield capacity of the member in the eccentric moment direction. This can be related to $S_{gravity}$ and S_{fn} as shown in Equation 12. Based on this, the parameters α and r_i can be related as shown in Equation 13, and the ratcheting provisions of the NBCC [12] can be related back to r_i as shown in Table 4.

$$\alpha = \frac{M_{GILD}}{M_{E,max}/R_{\alpha=0}} = \frac{S_{gravity}}{\phi S_{fn}} \tag{12}$$

$$r_i = \frac{S_f}{S_r} + \frac{S_g}{S_r} = \frac{\phi S_{fn} + S_{gravity}}{\phi S_{fn}} + \frac{S_{gravity}}{\phi S_{fn}} = 1 + 2\alpha \tag{13}$$

Methodology

Overview

A case study was examined considering the reinforced concrete cantilever bridge column shown in Figure 13. Three scenarios were considered as follows:

- Retaining the reinforcing layout shown in Figure 13 but increasing degree of eccentricity (i.e. varying α from Equation 12, which in turn varies eccentric moment, M_E);
- Increasing the amount of reinforcing on one side of the column without any eccentric loads (i.e. α = 0); and,
- Varying both reinforcing and eccentricity.

In the first and second scenarios, the influence of eccentricity and unbalanced structural strengths could be evaluated individually. Furthermore, S_g (and hence $r_{i,2}$) would both be zero based on the revised definitions proposed. For these two scenarios, the natural period, T, was set at 1.0 s, while the k_μ/S_p factor from Clause 5.2.1.1 of NZS1170.5 [4], where k_μ is the elastic spectrum scaling factor and S_p is the structural performance factor, was set at 4.0 for the symmetric reinforcing layout shown in Figure 13. As axial loads could have a noticeable influence on the flexural hysteretic response of structural members, an axial load ratio ($P/A_g f_c$, where P is the axial load, A_g is the gross cross-sectional area, and f_c is the peak unconfined concrete strength) of 0 and 0.1 were examined. It should be noted that in the first scenario, because the column strength was not adjusted to account for the eccentricity effect, these may not be code compliant; particularly if the eccentricity effect causes the effective lateral strength in the weaker direction to drop below minimum requirements. However, the authors view this as a necessity to examine the influence of eccentric loading on its own.

In the third scenario, a parametric study was performed by varying α, $P/A_g f_c$, k_μ/S_p, and T to observe the influence of these parameters on ratcheting. The "base case" adopts the same values used in the first two scenarios. Each parameter was then modified individually so that comparisons could be made with the "base case". In this scenario, the column reinforcing was adjusted to ensure that all columns satisfy minimum code requirements. The column reinforcing was then increased above this level in order to obtain different r_i values, until r_i = 1.0 was satisfied.

In all scenarios, the peak displacement obtained was normalized by a case where r_i = 1.0. This ratio is termed the "displacement amplification" factor, and can be used to compare against Clause 7.2.1.3 from NZS1170.5 [4] and the NBCC provisions [12] shown in Table 4.

Table 3: Calculation of ratcheting indices with new examples.

Case ID in paper	Case ID in commentary	S_f	S_r	S_g	$r_{i,1}$	$r_{i,2}$	r_i
I	A	115	85	0	1.35	0	1.35
II	-	115	95	10	1.21	0.11	1.32
III	B	115	100	15	1.15	0.15	1.30
IV	C	115	115	15	1.0	0.13	1.13
V	D	115	130	15	0.88	0.12	1.00
VI(a)	-	145	115	-15	1.26	-0.13	1.13
VI(b)	-	115	145	15	0.79	0.10	1.12

Table 4: Ratcheting provisions in NBCC [12] in terms of ratcheting index (modified using Equation 13).

Systems with self-centering characteristics	Other systems	Code requirement
$1.00 \le r_i \le 1.2$	$1.00 \le r_i \le 1.06$	No requirements
$1.2 < r_i \le 1.4$	$1.06 < r_i \le 1.12$	Multiply displacements by 1.2
$1.4 < r_i$	$1.12 < r_i$	Nonlinear response history analysis

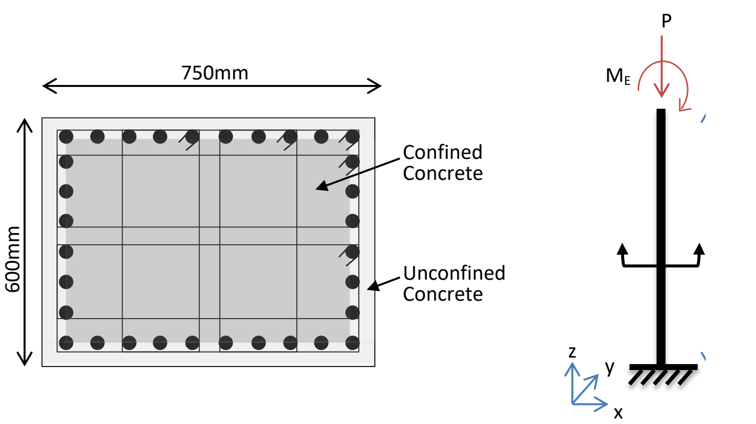


Figure 13: Cantilevered column model.

Table 5: Concrete material properties.

Property	Unconfined concrete	Confined concrete
Peak compressive strength	30 MPa	37 MPa
Strain at Peak compressive strength	0.002	0.010
Initial Young's Modulus	27.3 GPa	30.4 GPa
Peak strain	0.010	0.028

Table 6: Steel material properties.

Property	Steel reinforcing
Yield strength	300 MPa
Young's Modulus	200 GPa
Post-yield stress-strain stiffness ratio	0.01
Peak strain	0.010

Modelling

The reinforced concrete cantilever column fiber model adopted in this study is based off that shown in Figure 13. This was based on a scaled reinforced concrete bridge column used by Chen et al. [10]. The analyses were performed using Opensees [19] with 5% Rayleigh damping with initial stiffness and co-rotational analyses to consider the P-delta effect. A time step of 0.001 s was used in the analyses. This was based on a sensitivity study which found that a time step of 0.001 s or lower produced similar results.

The unconfined and confined concrete materials were modelled using Concrete04 while the steel material was modelled using Steel02 on Opensees [19]. The material properties adopted are shown in Table 5 (for unconfined and confined concrete) and Table 6 (for steel). All steel bars had 16 mm diameters. Note that concrete tension resistance was ignored.

As previously discussed, the key parameters considered were α, P/A_gf_c, k_μ/S_p, and T. These were considered in the modelling through the following means (values adopted in square brackets):

- α – through an eccentric moment applied at the top of the column, M_E, which was equal to α multiplied by the original column strength (i.e. based on reinforcing layout in Figure 13 and applied axial load, P) [0.1, 0.2, 0.3, and 0.4];
- P/A_gf_c – through applied axial load at the top of the column [0, 0.1, 0.15, and 0.2];
- k_μ/S_p – through scaling of ground motions so that the ratio of the elastic moment demand to the flexural capacity of the column is equal to the target k_μ/S_p [1, 2, 4, and 6]; and,
- T – through modifying the building height [0.25 s, 0.5 s, 1.0 s, 1.5 s, 2.0 s].

With regards to the method adopted to consider k_μ/S_p, it may be argued by some that this would result in vastly different ground shaking intensities between the various k_μ/S_p cases. However, since the response obtained from the parametric study will be normalized by a similar column designed so that r_i = 1.0 which was also subjected to the same ground shaking intensity, this scaling approach was deemed to be reasonable. This was verified by performing the opposite approach for a single case (i.e. redesigning columns instead of scaling records) which resulted in similar displacement amplification factors.

The far field record suite provided in Appendix A of FEMA P695 [20] was used in the analyses. This record suite contains 22 sets of ground motion records; each with two horizontal and one vertical component. Each horizontal component was treated as an individual event, and analyses were performed twice for each component; once each in the forward and backward directions to eliminate directionality effects. Vertical components were not considered. This resulted in 88 analyses for each combination of α, P/A_gf_c, k_μ/S_p, and T considered for all three scenarios. The median displacement amplification value for each scenario/combination case was obtained and compared against the corresponding r_i value.

Evaluation of Displacement Amplification Factors

The median displacement amplification factors the first two scenarios (i.e. varying eccentricity effects and amount of reinforcing) are shown in Figure 14a ($P = 0.1A_gf_c$) and Figure 14b (P = 0). The different axial loads applied resulted in the column exhibiting different degrees of self-centring, as shown in Figures 14c and 14d. It can be seen that for these two cases, the NZS1170.5 [4] estimates are slightly larger than that observed from the numerical study, indicating that the provisions provide a reasonable but conservative estimate of the displacement amplification. In contrast, NBCC [12] underestimates the displacement increase in the case where the column's hysteresis curve exhibits self-centring behaviour (labelled as NBCC(SCS) in Figure 14a) when r_i is greater than 1.35; and severely overestimates in cases where the column does not exhibit self-centring behaviour over the range where it may be applied (labelled as NBCC(OS) in Figure 14b).

The median displacement amplification factors obtained from the third scenario are shown in Figure 15. In the case of varying α in Figure 15a, at any given value of r_i the displacement amplification generally decreases with increasing α. This seems counter-intuitive as one would expect the displacement amplification factor to increase with α. However, consider a case where no strength increase was provided in the reverse direction to adjust for the eccentric gravity load effect, similar to that considered in case "A" of the examples. If the eccentricity effect is greater, then there would be a bigger difference between S_f and S_r, resulting in a larger $r_{i,1}$, and hence a larger r_i. This would also be true even if the strength was increased in the reverse direction, provided that the ratio of the strength increase to the eccentricity effect is the same between the various α cases. Based on this, at the same r_i value, a column with greater eccentricity actions would already have a greater relative increase in strength in the reverse direction, which should decrease the tendency for ratcheting and hence result in lower displacement amplification. This does indicate that a given r_i value on its own does not necessarily indicate the extent of ratcheting to be expected.

In the case of varying axial load in Figure 15b, increasing axial load ratio generally resulted in lower displacement amplification factors due to the column exhibiting greater self-centring tendencies at greater axial load ratios. Figure 15c shows that the displacement amplification factor generally increases with increasing k_μ/S_p since inelastic actions are more significant, which causes ratcheting effects to be more pronounced. It was also observed that displacement amplification decreases with increasing T as shown in Figure 15d, though its influence was not as significant.

Overall, Clause of 7.2.1.3 in NZS1170.5 [4] appear to provide a reasonable estimate in most cases. The clear exceptions to this are where the structure exhibits lesser self-centring tendencies or if designed to a higher ductility factor. The authors recommend a more rigorous study be performed to consider a wider range of structural systems and construction materials to verify the proposed displacement amplifications in Clause of 7.2.1.3, particularly for the potentially problematic areas highlighted in this study.

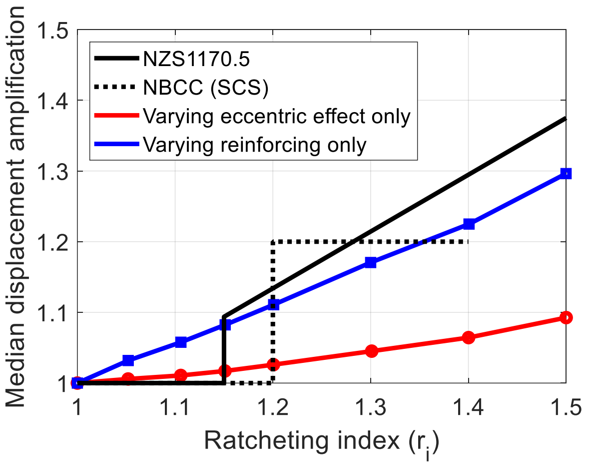
In contrast, Clause 4.1.8.10 of the NBCC [12] appear to be too conservative for systems which do not exhibit self-centring behaviour (i.e. P = 0 in Figure 15b), and is potentially unconservative when r_i > 1.3 to 1.35 for self-centring cases.

USE OF NZS1170.5 PROVISIONS TO LIMIT RATCHETING EFFECTS

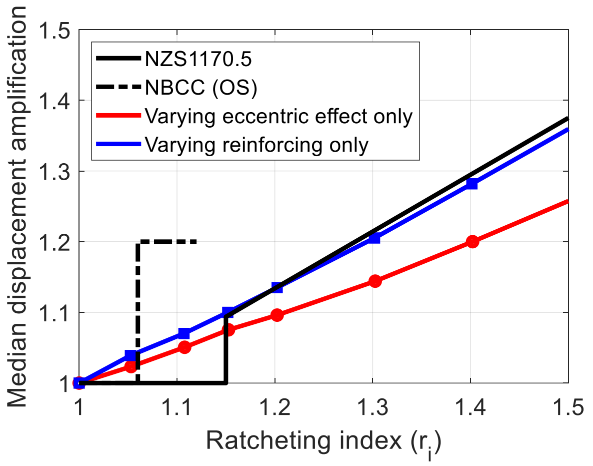
NZS1170.5 [4] allows the effects of ratcheting to be neglected if 1.0 < r_i < 1.15. This limit can be used to develop a strength relationship between the forward and reverse directions, which could then be adopted in design to limit ratcheting effects. Assuming that the eccentric action acts in the reverse direction, the design lateral capacity in the forward direction excluding the effect of eccentricity is ϕS_{fn}, and that the reduction in the lateral loads due to eccentric action is α. ϕS_{fn}, the design lateral strength in the reverse direction excluding the effect of eccentric moment, ϕS_{rn}, can be calculated from Equations 14 and 15. This assumes that the increase in strength required in the reverse direction is greater than $S_{gravity}$, so $S_g = S_{gravity}$.

$$1.15 \leq r_{i,1} + r_{i,2} = \frac{(\phi S_f + \alpha\phi S_{fn}) + \alpha\phi S_{fn}}{\phi S_{rn} - \alpha\phi S_{fn}} \quad (14)$$

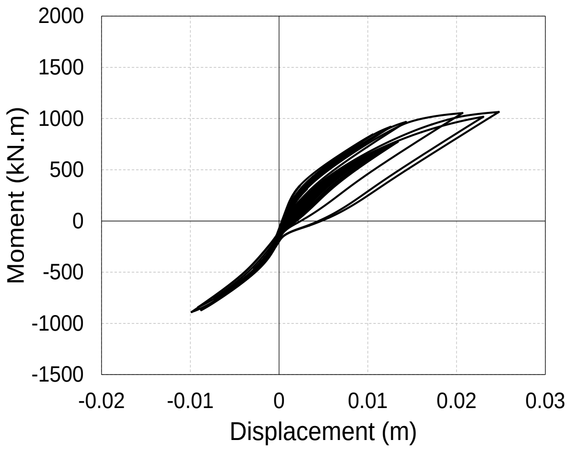
$$\phi S_{rn} = (0.87 + 2.74\alpha).\,\phi S_{fn} \quad (15)$$



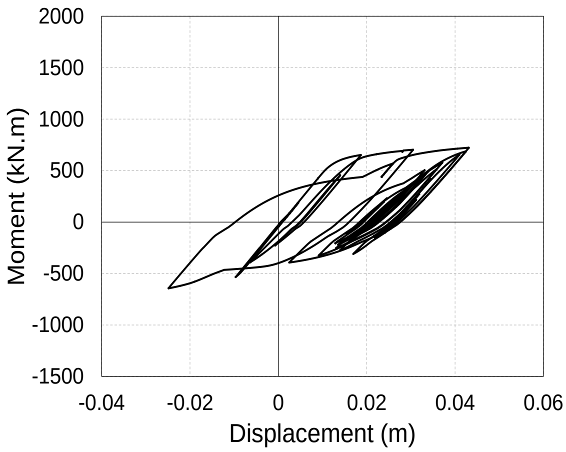
(a) Displacement amplification ($P/A_g f_c$' = 0.1)



(b) Displacement amplification ($P/A_g f_c$' = 0)

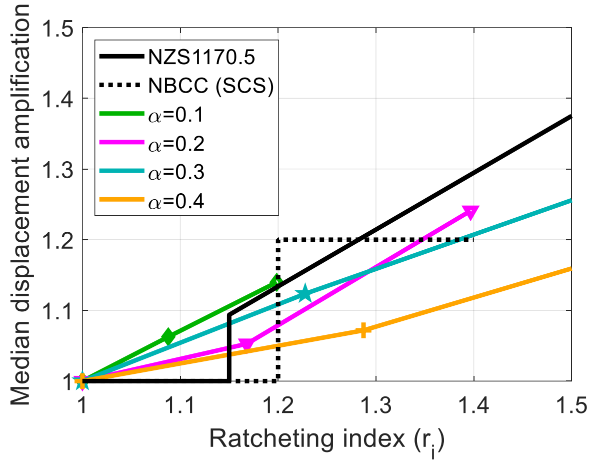


(c) Sample hysteresis shape for $P/A_g f_c$' = 0.1



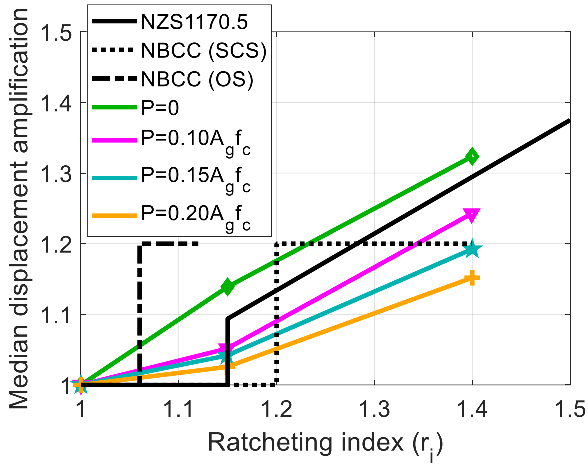
(d) Sample hysteresis shape for $P/A_g f_c$' = 0

Figure 14: Evaluation of displacement amplification factors for case of varying eccentric moment or reinforcing separately (S_g = 0, T=1s, k_μ/S_p =4, damping 5%, and using fiber section).



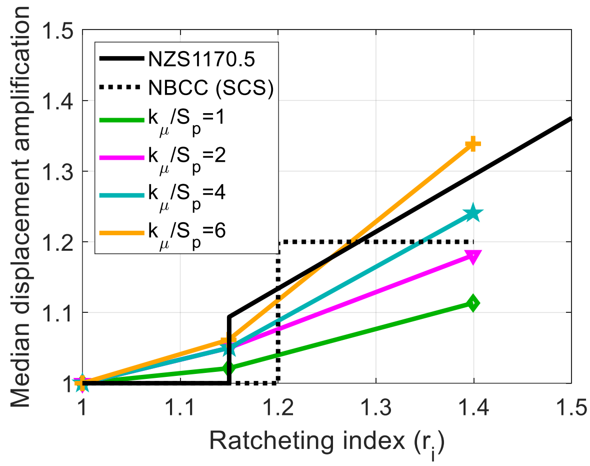
(a) Effect of eccentric moment ratio

(P=0.1A$_g$f$_c$', k$_\mu$/S$_p$ =4, T=1.0s)



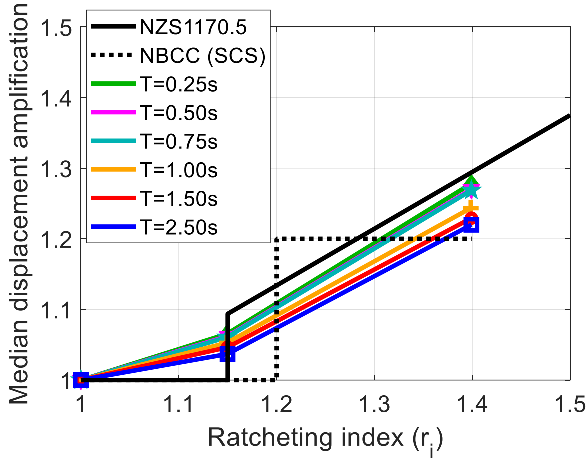
(b) Effect of axial load ratio

(α=0.2, k$_\mu$/S$_p$ =4, T=1.0s)



(c) Effect of k$_\mu$/S$_p$

(P=0.1A$_g$f$_c$', α=0.2, T=1.0s)



(d) Effect of natural period

(P=0.1A$_g$f$_c$', α=0.2, k$_\mu$/S$_p$ =4)

Figure 15: Effect of various parameters on displacement amplification (including P-delta).

In comparison, the approach proposed by MacRae and Kawashima [5] for bilinear hysteretic behaviour, and Yeow et al.'s [6] approach for Takeda hysteretic behaviour including P-delta effect as discussed previously in section 2.3, are shown in Equations 16 and 17, respectively.

$$\phi S_{rn} = (1.00 + 2.00\alpha).\phi S_{fn} \tag{16}$$

$$\phi S_{rn} = (1.00 + 2.30\alpha).\phi S_{fn} \tag{17}$$

AVENUES FOR FUTURE WORK

In this study, it was found that the NZS1170.5 [4] displacement amplification factors proposed seem to be reasonable for most of the RC columns considered. However, displacement amplification factors for structures with lesser self-centring tendencies (e.g., RC columns with low axial force ratios) or ones with high ductility factors (i.e., k_μ/S_p = 6.0) were underestimated in some cases. In fact, similar analyses considering steel columns [21] showed that Clause 7.2.1.3 may severely underestimate the displacement amplification factor for structures exhibiting inelastic bilinear hysteretic behaviour, and thus modifications to the displacement amplification estimate is required to properly consider such cases. Furthermore, the applicability of such provisions to multi-storey buildings needs consideration. It is also desirable for future provisions to be written in a way such that the background to the approach is simple and clear, where the parameters are all clearly defined, the steps to apply the method are unambiguous, and where P-delta effect are considered directly (rather than as an add-on). In addition to estimating displacement amplification, methods to mitigate it should be further explored.

CONCLUSIONS

This paper evaluates the new provisions included in NZS1170.5 [4] to address seismic ratcheting. In conclusion:

1. Ratcheting can occur due to the effect of (i) ground motion, (ii) dynamic stability, (iii) eccentric loading, and (iv) structural form. The ratcheting provisions in NZS1170.5 [4] address seismic ratcheting caused by latter two effects.
2. The 2016 provisions were developed for reinforced concrete single storey structures with Takeda hysteretic

behaviour [14]. They consider ratcheting tendencies due to eccentric gravity loading and unbalanced strength in the forward and backward directions. P-delta effect was not included in the analyses used in the clause derivation, although ratcheting displacements were increased by 20% to approximate the potential increase in P-delta actions due to ratcheting displacements. While not clearly stated in the provisions, the effect of P-delta alone also needs to be added to the displacements. The effect of stiffness-strength dependency was also not considered.

3. The new ratcheting provisions cover three clauses. The first is Clause 4.5.3, which provides a procedure to derive a ratcheting index, r_i. If r_i is greater than 1.5, then time-history analysis is required. If r_i is less than 1.15, then no further displacement modifications are required. If r_i is in between these values, then the displacements can be modified according to Clause 7.2.1.3. The third Clause is 6.1.1 which describes how ratcheting depends on the ductility level of the structure. This paper proposes wording modifications for Clause 4.5.3 to provide clarity. Additional design examples are provided.
4. The time history analyses of a bridge column conducted in this study provide displacement amplifications consistent with NZS1170.5 [4] provisions for the ratcheting effect for most cases. However, where the structure exhibits lesser degree of self-centring, or where it was designed to a high ductility factor, the estimates may potentially be unconservative.

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