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Seismic Behaviour of Slender Rectangular Reinforced Concrete Walls based on analytical methods

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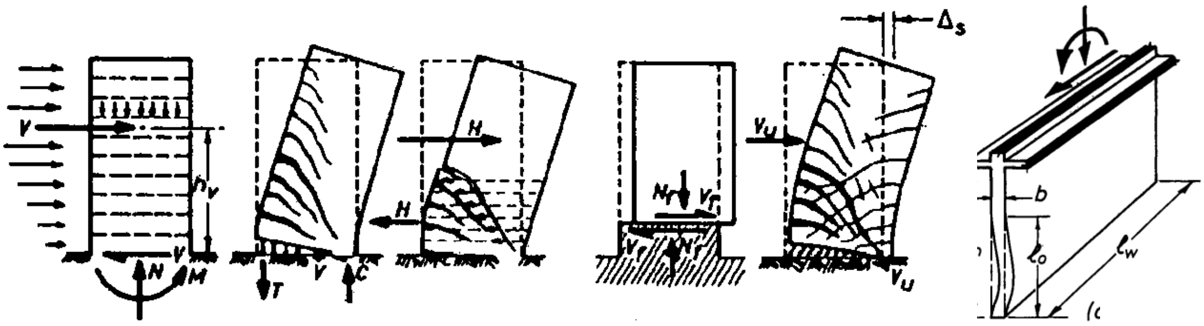
ABSTRACT

In the seismic design and assessment of structures, one of the key parameters is the displacement capacity of the structural elements. Slender reinforced concrete (RC) structural walls are one of the more commonly used lateral resisting elements in seismic regions. The drift capacity of a structural wall is commonly determined from evaluating the ultimate plastic hinge rotation at its base utilising a moment-curvature analysis of the section. While there are currently several plastic hinge length equations available in literature for different RC structural elements, it is unclear which is the most appropriate for slender walls. In this study, a database of wall specimens that exhibited flexural failure modes has been collected to assess the accuracy of determining wall drift capacities using the moment curvature method. For this purpose, the accuracy of some of the commonly referred to equations for determining the plastic hinge length of RC structural elements has been evaluated and the most appropriate for use with slender walls identified. Furthermore, the displacement capacities of slender walls derived from the moment-curvature approach are compared with those obtained from a direct rotation method approach (both Section C5 of the NZ Technical Proposal to Revise the Engineering Assessment Guidelines and ASCE41-17). The moment-curvature approach is shown to provide a better match to the test data compared to the to the direct rotation approach and can provide further insight into the seismic behaviour of the walls.

1 INTRODUCTION

RC walls are widely used as lateral load resisting elements in the seismic design of structures. Considering the height to length ratio of the walls, they can be classified as squat or slender. The seismic behaviour of squat walls with height to length ratio often smaller than one is dominated by shear deformations and therefore various forms of shear failures such as diagonal compression/web crushing, diagonal tension, and sliding shear (Paulay et al. 1982). However, in the case of slender walls with height to length ratio larger than two, the seismic performance is usually governed by their flexural deformations (Priestley et al. 2007, Beyer et al. 2011, Krolicki et al. 2011). Based on observations from damaged buildings in past earthquakes (Connor 1985, Wood 1991, Muguruma et al. 1995, Kam et al. 2011, NIST 2014, Sritharan et al. 2014), as well as experimental studies (Oesterle et al. 1976, Oesterle et al. 1979, Paulay and Goodsir 1985, Thomsen and Wallace 2004, Dazio et al. 2009, Dashti et al. 2017, Segura and Wallace 2018, Tripathi et al. 2019, Shegay et al. 2020, Niroomandi et al. 2021), different modes of failure including flexural (i.e. yielding of the longitudinal reinforcement as shown in Figure 1a), concrete crushing, bar buckling, diagonal tension (Figure 1b) and compression, sliding shear along the construction joints (Figure 1c), hinge sliding (Figure 1d), single crack behaviour (Lu et al. 2018), lateral instability (Figure 1e) and out-of-plane shear-axial (Niroomandi et al. 2022a, Niroomandi et al. 2022b) failures have been observed in slender rectangular walls.



(a) (b) (c) (d) (e)

Figure 1. Failure modes in cantilever walls; (a, b, c & d) (Paulay and Priestley 1992) and (e) (Paulay and Priestley 1993)

In the seismic design and assessment of RC walls, identifying the failure mode and its corresponding displacement capacity is crucial. Furthermore, determining the force-displacement or moment-rotation behaviour of the elements is necessary for nonlinear static (pushover) or time history analysis of structures. There are two main school of thoughts for obtaining such curves. One is by performing a "section analysis" to capture the moment-curvature curve and then to convert it to a force-displacement curve using the equivalent plastic hinge method as discussed in Blume et al. (1961) and Park and Paulay (1975) and is shown in Figure 2.

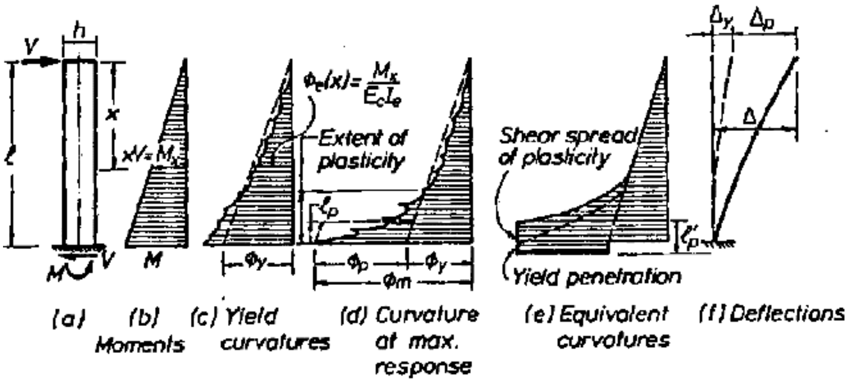


Figure 2. Moment, curvature, and deflection relationships for a reinforced concrete cantilever element subjected to a point load (Paulay and Priestley 1992)

An alternative method that has been recently developed is the so called "direct rotation method" which has been implemented in seismic assessment guidelines such as ASCE41-17 (2017) and recently added to the revised Section C5 of the New Zealand Engineering Assessment Guidelines for RC structures (MBIE 2017)).

In this method, the displacement capacity of RC elements can be determined using empirical equations that have been developed by fitting equations to available test results.

For the section analysis approach, an equivalent plastic hinge length is required to convert the curvature to displacement. There are several equations proposed by various researchers for the equivalent plastic hinge length of different RC elements (Thomsen and Wallace 2004, Priestley et al. 2007, Bae and Bayrak 2008, Berry et al. 2008, Bohl and Adebar 2011, Kazaz 2013, Takahashi et al. 2013). There have been a few studies on the accuracy of the available equivalent plastic hinge equations on capturing the displacement of the walls such those by Cordero et al. (2015) and Puranam et al. (2018). However, these studies were limited in respect to the plastic hinge length equations considered and the database adopted. Therefore, it is unclear which of the available equivalent plastic hinge equations is more appropriate to adopt to capture the displacement capacity of RC walls.

The focus of this study is to investigate the accuracy of the two alternative approaches for assessing the displacement capacity of RC walls. For this purpose, a relatively large database of cyclic tests on rectangular RC slender walls is prepared, limited to walls exhibiting flexural failure modes. The displacement capacities of slender walls derived from the moment-curvature approach utilising the various plastic hinge equations is derived and the equation providing the best match to the test data is identified. The results of the moment-curvature approach will also be compared with those based on the direct rotation method approach (of Revised Section C5 and ASCE41-17).

2 RELATIONSHIP BETWEEN CURVATURE AND DISPLACEMENT

To guide the nonlinear section behaviour of a reinforced concrete structural element a moment-curvature curve can be developed. There are certain assumptions behind these curves such as: (i) plane section remains plane, (ii) perfect bond between concrete and steel reinforcement exists, and (iii) the tension capacity of concrete is often ignored (this is not a necessary assumption but is common). Essential information such as the neutral axis depth, section moment capacity, curvature at different limit states and the elastic stiffness can be obtained from a section analysis. At the member level, the moment-curvature curve can be transferred into a force-displacement curve knowing the plastic hinge length and equivalent shear span. Furthermore, the lateral displacement of a structural element can be determined as the sum of the flexural and shear displacements.

To convert curvature to displacement, a simplified approach based on the concept of an equivalent plastic hinge length, as shown in Figure 2, can be used. For a cantilever wall, the displacement due to the flexural deformations of a structural element can be written as the sum of the yield and plastic displacements as shown in Equation (1) and Equation (2) (Priestley et al. 2007, Beyer et al. 2011, Krolicki et al. 2011). It should be noted that only the flexural portion can be determined using this method and the shear contribution needs to be added separately.

$$\Delta'_y = \phi'_y \left(H_e + L_{sp}\right)^2 / 3 \tag{1}$$

$$\Delta = \Delta'_y \frac{M}{M_y} + \left(\phi - \phi'_y \frac{M}{M_y}\right) L_p \left(H_e + L_{sp} - 0.5 L_p\right) \tag{2}$$

Where, ϕ'_y is the curvature at first yield defined as the curvature at the concrete compressive strain of $\varepsilon_c = 1.8 f'_c / E_c$ (FIB25 2003) or the yield tensile strain of the steel reinforcement whichever occurs first, H_e is the effective height of the wall which can be assumed as $0.7H_w$ as suggested by Priestley et al. (2007) for multi-storey wall buildings, H_w is the total height of the wall, $L_{sp} = 0.022 f_y d_b$ is the strain penetration length according to Paulay and Priestley (1992) and L_p is the equivalent plastic hinge length.

The shear component of the lateral deformation of RC walls can be estimated using a number of proposed methods (Priestley et al. 2007, Beyer et al. 2011, Krolicki et al. 2011). In this study, the method outlined in Priestley et al. (2007) was used. The detail of this method has not been repeated here for brevity and readers should refer to Priestley et al. (2007) noting that for slender RC walls, shear deformation will be small relative to flexural deformations.

3 ASSESSING RC SLENDER WALLS USING A MOMENT-CURVATURE ANALYSIS

As stated in Section 2 and shown in Equation (2), to convert the moment-curvature from a section analysis to a force-displacement curve, an equivalent plastic hinge length is required. As can be seen in Figure 2d, a distinction must be made between the equivalent plastic hinge length, L_p in Equation (2) and the distance from the column base to the cross-sectional level with zero plastic strain, L_{pz}. In this study, the accuracy of the equivalent plastic hinge equations is assessed by comparing predicted and observed estimates of the displacement capacity of the walls (and not the length over which the strain in the member exceeds the yield strain). Various equations are available in the literature for estimating the equivalent plastic hinge length of RC elements, some of which were specifically developed for RC walls. The ultimate drift of the test specimens was used as the main indicator for assessing the accuracy of each plastic hinge length equation. To assist in comparing the performance of each equivalent plastic hinge length equation, the mean ratio of the analytical drift capacity to the ultimate experimental one as well as the standard deviation and COV were also calculated for each alternative.

3.1 Available equivalent plastic hinge length models

Seven equations for estimating the equivalent plastic hinge length of RC walls were evaluated in this study. A summary of these methods is presented in Table 1. For more information on each method, refer to the provided references.

Table 1. Equivalent plastic hinge length models used

Reference	Method
Thomsen and Wallace (2004)	$L_p = 0.5L_w$
Priestley et al. (2007)	$L_p = kH_e + 0.1L_w + L_{sp}$
Berry et al. (2008)	$L_p = 0.05H_e + 0.1\dfrac{f_y d_b}{\sqrt{f_c'}}$
Bae and Bayrak (2008)	$L_p = H_e\left(\dfrac{0.3P}{P_0} + \dfrac{3A_{st}}{A_g} - 0.1\right) + 0.25L_w \geq 0.25L_w$
Bohl and Adebar (2011)	$L_p = (0.2L_w + 0.05H_e)\left(1 - \dfrac{1.5P}{A_g f_c}\right) \leq 0.8L_w$
Kazaz (2013)	$L_p = 0.27L_w\left(1 - \dfrac{P}{A_g f_c'}\right)\left(1 - \dfrac{f_y \rho_{sh}}{f_c'}\right)(\dfrac{H_e}{L_w})^{0.45}$
Takahashi et al. (2013)	$L_p = 2.5t_w$

3.2 Selected experimental tests on RC slender walls

For this part of the study, a database of 72 slender rectangular walls tested under in-plane cyclic loading was used. The key geometrical and material properties of the test specimens are listed in Table 2. One of the challenges with evaluating the accuracy of the available equivalent plastic hinge length equations was choosing the right specimens to be included in the database. The chosen specimens had the following criteria: 1) Tested under common cyclic loading regimes, 2) Shear span ratio greater than two, 3) Rectangular or barbell shape, 4) Their behaviour was governed by a flexural failure mode, 5) Doubly reinforced, 6) No anchorage or lap issues. It is worth noting that these specimens were selected from a database of more than 300 specimens, from which the walls that did not meet the above criteria were removed. Walls exhibiting bar buckling and global buckling have been included in the database.

Table 2. Database of the RC walls used for the equivalent plastic hinge length investigation

Reference	Wall ID	$\frac{M}{VL_w}$	$\frac{P}{A_g f'_c}$	H_e mm	L_w mm	t_w mm	d_b mm	ρ_s	$\frac{\delta_u}{H_{eff}}$ %
Oesterle et al. (1976)	R1	2.4	0	4572	1905	102	9.5	0.008	2.3
	R2	2.4	0	4572	1905	102	12.7	0.021	2.9
	B1	2.4	0	4572	1905	305	12.7	0.002	3.3
	B3	2.4	0	4572	1905	305	12.7	0.021	4.39
Oesterle et al. (1979)	B10	2.4	0.08	4572	1905	305	15.9	0.020	3.33
Pilakoutas & Elnashai (1995)	SW4	2	0	1200	600	60	12	0.009	1.79
	SW6	2	0	1200	600	60	12	0.005	1.83
	SW8	2	0	1200	600	60	10	0.004	2
	SW9	2	0	1200	600	60	10	0.005	2.1
Thomsen & Wallace (1995)	RW1	3.13	0.1	3810	1219	102	9.5	0.012	2.2
	RW2	3.13	0.07	3810	1219	102	9.5	0.013	2.3
Tupper (1999)	W3	3.75	0.11	3750	1000	152	19.5	0.010	3.04
Oh et al. (2002)	WR20	2	0.1	3000	1500	200	12.7	0.012	2.7
	WR10	2	0.1	3000	1500	200	12.7	0.023	2.9
	WR0	2	0.1	3000	1500	200	12.7	0.000	2.2
	WB	2	0.1	3000	1500	240	9.5	0.011	2.8
Mobeen (2002)	W-1	2.74	0.15	3283	1200	250	16	0.021	3.78
Han et al. (2002)	W3	3	0.1	4500	1500	200	12.7	0.012	2

Liu (2004)	W1	3.13	0.076	3750	1200	200	19.5	0.025	3
	W2	3.13	0.035	3750	1200	200	19.5	0.025	2.9
Ghorbani-Renani et al.(2009)	A2C	2.08	0	2700	1300	200	25	0.027	3.18
Dazio et al. (2009)	WSH1	2.28	0.051	4560	2000	150	10	0.012	1.04
	WSH2	2.28	0.057	4560	2000	150	10	0.012	1.38
	WSH3	2.28	0.058	4560	2000	150	12	0.011	2.03
	WSH4	2.28	0.057	4560	2000	150	12	0.000	1.6
	WSH5	2.28	0.128	4560	2000	150	8	0.013	1.36
	WSH6	2.26	0.108	4520	2000	150	12	0.017	2.07
Tran (2012)	S63	2	0.073	2438	1219	152	19.1	0.018	3
	W1	2.03	0	1500	740	100	10	0.080	2.99
	W7	2	0	1500	750	125	10	0.000	3.48
	W9	2	0	1500	750	125	12	0.000	2.97
Christidis et al. (2013)	W11	2	0	1500	750	125	12	0.000	3
Villalobos (2014)	W-MC-C	2.175	0.1	3314.7	1524	203.2	25.4	0.013	3
	W-MC-N	2.175	0.1	3314.7	1524	203	25.4	0.008	2.5
Hube et al. (2014)	W4	2.5	0.15	1750	700	75	10	0.008	1.6
christidis et al. (2016)	W13	2	0	1500	750	125	12	0.002	1.7
Lu et al. (2017)	C1	2	0.035	2800	1400	150	10	0.004	2.6
	C2	4	0.035	5600	1400	150	10	0.004	2.5
	C3	6	0.035	8400	1400	150	10	0.004	2.6
	C4	2	0	2800	1400	150	10	0.000	1.5
	C5	2	0.066	2800	1400	150	10	0.008	2.5
	C6	4	0.035	5600	1400	150	10	0.012	2.5
Zhu & Guo (2017)	MW	2.04	0.08	3460	1700	200	16	0.012	2.95
Abdulridha & Palermo (2017)	W1-SR	2.4	0	2400	1000	150	11.3	0.038	4.07
Zhi et al. (2017)	SW1-1	2.07	0.054	3312	1600	200	14	0.009	2.23

	SW2-1	2.08	0.063	3536	1700	200	16	0.013	2.6
Shegay et al. (2017)	C10	4.6	0.092	10350	2250	200	16	0.015	3.8
	A10	4.6	0.092	10350	2250	200	16	0.013	3.7
	A14	4.6	0.14	10350	2250	200	16	0.017	3.1
	A20	4.6	0.21	10350	2250	200	16	0.019	2.7
Dashti et al. (2017)	RWB	3	0.042	6000	2000	125	12	0.025	2
	RWT	3	0.047	6000	2000	135	12	0.023	2
	RWL	3.75	0.063	6000	1600	125	16	0.024	3
Lu et al. (2018)	M1	4	0.035	5600	1400	150	10	0.012	2.5
	M2	4	0.035	5600	1400	150	12	0.012	3.5
	M3	4	0.035	5600	1400	150	12	0.012	2.5
	M4	4	0.035	5600	1400	150	16	0.012	3.5
Segura & Wallace (2018)	WP1-1	3.74	0.1	8560	2286	152.4	15.9	0.018	2.02
	WP1-2	3.74	0.1	8560	2286	152.4	15.9	0.018	2.56
	WP2-1	3.74	0.1	8560	2286	152.4	15.9	0.029	2.16
	WP2-2	3.74	0.1	8560	2286	152.4	15.9	0.029	2.46
	WP3-1	3.74	0.1	8560	2286	152.4	15.9	0.021	2.1
	WP3-2	3.74	0.1	8560	2286	152.4	15.9	0.021	1.99
	WP6	3.57	0.1	8170	2286	191	15.9	0.025	4.08
	WP7	3.51	0.1	8030	2286	229	15.9	0.022	3.54
Yuan et al. (2018)	SW-1	2	0.13	2560	1280	200	12	0.021	3.05
	SW-2	2	0.13	2560	1280	200	12	0.021	2.69
Zhu & Guo (2019)	W-CIS	2.07	0.1	3525	1700	200	16	0.014	2.56
Tripathi et al. (2019)	SWD-1	3	0.055	6000	2000	150	12	0.010	2.5
	SWD-2	3	0.055	6000	2000	150	12	0.008	2
	SWD-3	3	0.055	6000	2000	150	12	0.011	2.5
Niroomandi et al. (2021)	SP1-Uni	3.75	0.046	6000	1600	125	16	0.015	2.5

3.3 Evaluation of the equivalent plastic hinge length models against experimental results

To evaluate the accuracy of the evaluated equations for the equivalent plastic hinge length in slender RC walls, the force-displacement curves of the walls and their ultimate drift capacities were determined utilising each of the equations presented in Section 3.1 and compared with the experimental results. In both the analytical and experimental results, the displacement corresponding to a 20% drop off in strength was considered as the ultimate displacement capacity of the walls as per common practice. To determine the moment-curvature curve of the rectangular walls, a section analysis has been performed using CUMBIA (Montejo and Niroomandi 2021). For walls with a barbel section, SAP2000 (2019) was used. The Mander et al. (1988) model was adopted for the unconfined and confined behaviour of concrete. The ultimate strain of concrete was defined based on Equation (3) below proposed by fib (2003) which is a revised version of that originally proposed by Paulay and Priestley (1992). For steel reinforcement, the King et al. (1986) model was used.

$$\varepsilon_{cu} = \left(0.004 + \frac{0.6\rho_v f_{yh}\varepsilon_{su}}{f'_{cc}}\right) > \varepsilon_{spall} \tag{3}$$

Where,

$\rho_v = \rho_{ax} + \rho_{ay}$ is the volumetric transverse reinforcement ratio and,

$\rho_a = A_{sv}/(h_{core} \times s)$ is the transverse reinforcement ratio of the boundary element,

h_{core} is the length of the core of the boundary element,

f_{yh} is the yield strength of the transverse reinforcement,

ε_{su} is the ultimate strain of the transverse reinforcement,

f'_{cc} is the concrete strength of the confined section and ε_{spall} is the strain at concrete cover spalling assumed to be 0.0064 according to Priestley et al. (2007).

The key properties of the steel reinforcement and concrete were adopted from those reported in the tests. Otherwise, if unavailable, the values recommended in literature were used. Based on the recommendation in Kowalsky (2000), a maximum strain capacity of 6% was assumed for steel reinforcement to take into account effects such as bar buckling and cyclic degradation (even if higher values had been reported from the uniaxial tensile tests).

It is worth noting that the force-displacement curves obtained from the analytical approach adopted here take into account cyclic degradations. 1) Cyclic degradation effects are considered at the material level in both the Mander et al. (1988) model for concrete, and by limiting the tensile strain of the steel reinforcement to 6%, 2) Since the equivalent plastic hinge lengths are assessed against specimens tested under a cyclic loading regime, therefore, the effects of cyclic degradation is considered at the member level as well.

3.4 Analytical vs experimental results

Figure 3 shows the force-displacement curves of selected specimens comparing the experimental results with the analytical ones derived from the alternative equivalent plastic hinge length equations. The observed discrepancies shown in Figure 3 reinforces the need for such investigations on the equivalent plastic hinge length equations.

In Figure 4, the predicted drift capacity of the specimens derived using the alternative equivalent plastic hinge length equations is plotted against the experimental drift capacity of the test specimens. The diagrams show the level of accuracy and variability of each prediction method. To better compare the results, the mean, standard deviation, and coefficient of variation of the estimated drift capacity to the experimental drift capacity of the walls corresponding to alternative equivalent plastic hinge length equations were determined and presented in Table 3. Where the mean is greater than unity, this implies that the model overestimates the drift capacity of the wall. The higher values of standard deviation show a higher dispersion in the results of the analytical method. Therefore, to evaluate the efficiency of plastic hinge length models, both the mean and the standard deviation of results need to be considered.

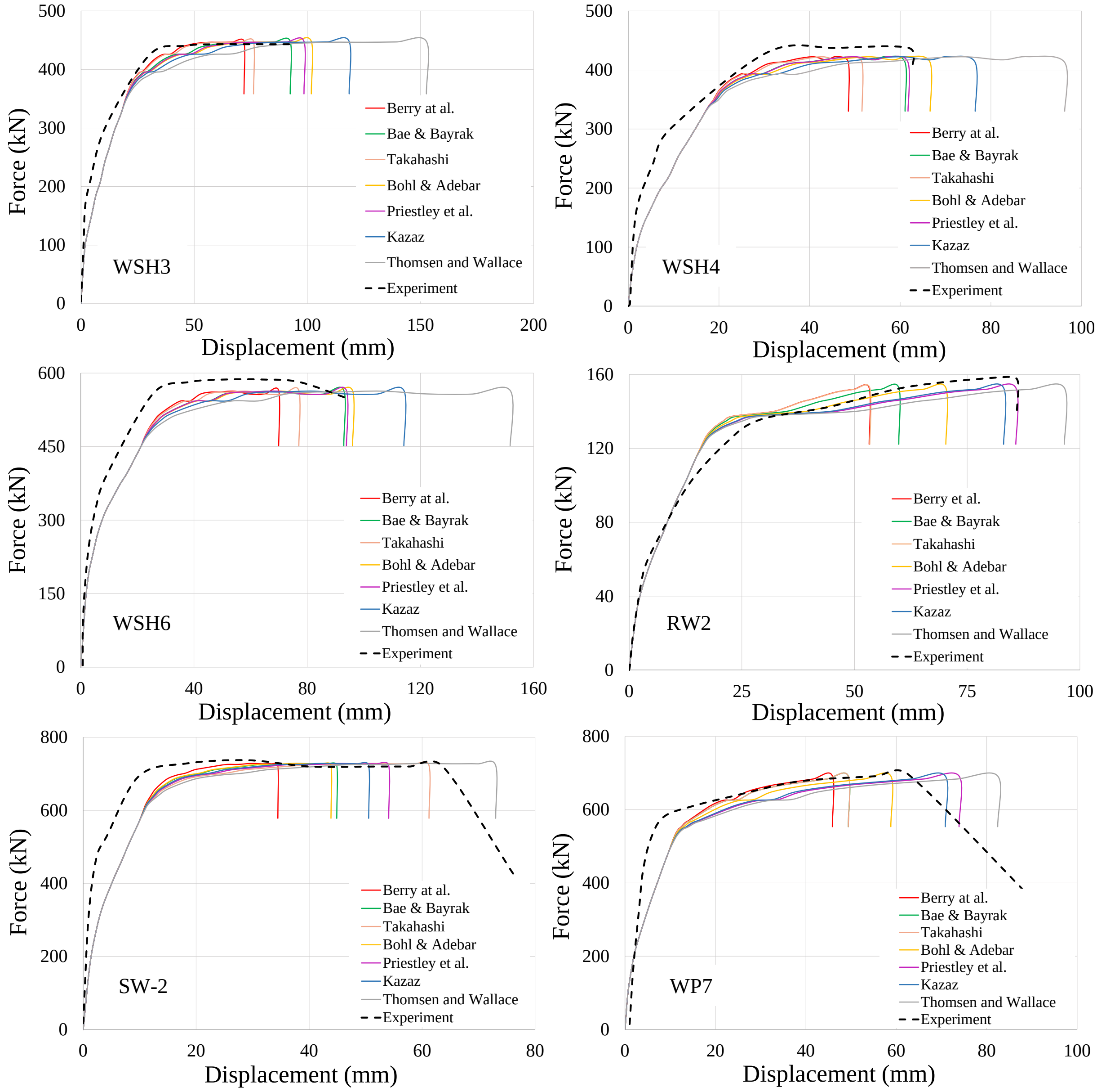


Figure 3. Force-displacement curves of selected specimens – Analytical vs Experiential results

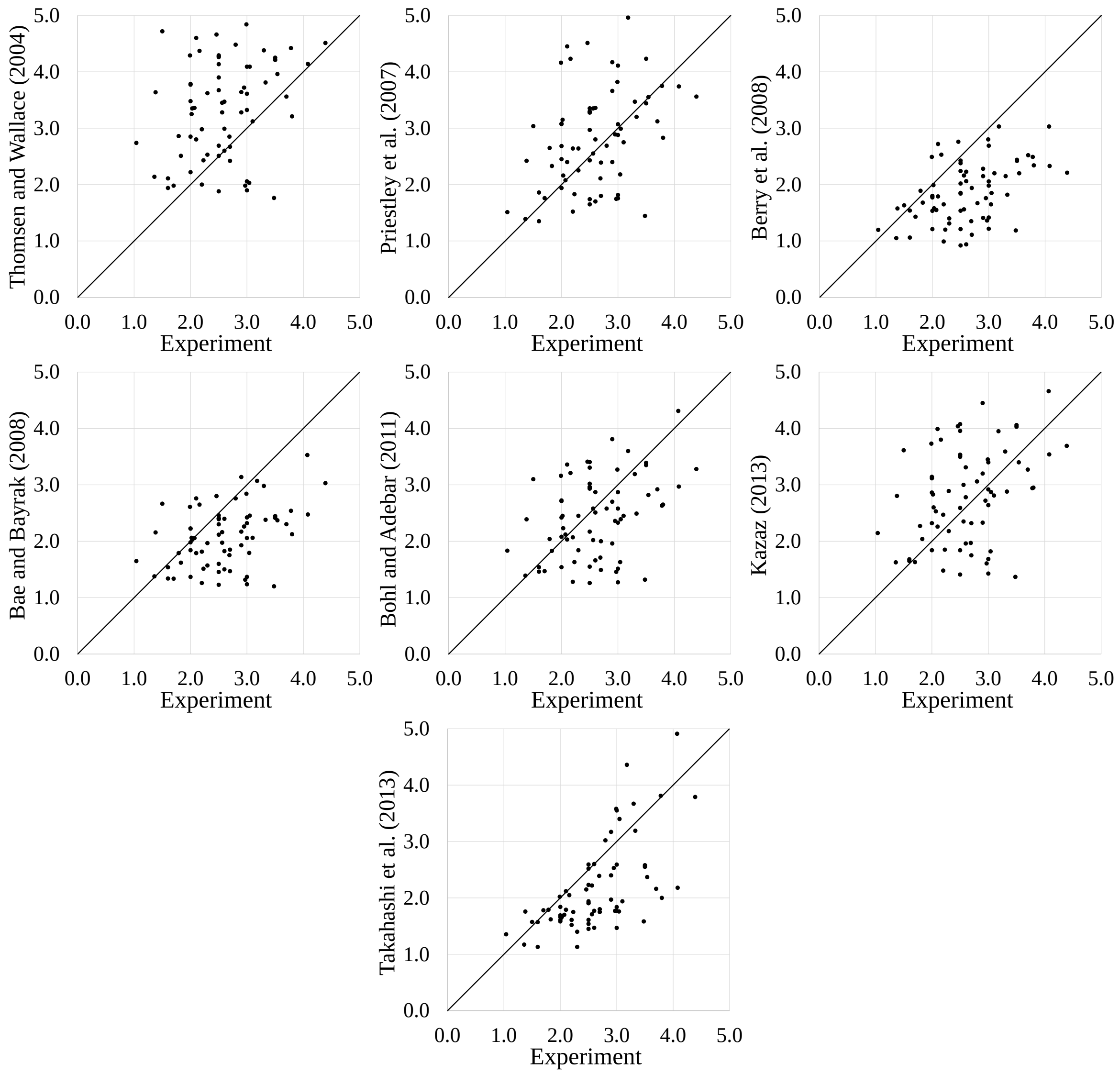


Figure 4. Comparison of the analytical vs. experimental drift capacities of the test specimens

Table 3. Evaluation of the analytical drift vs experimental results

Method	**Analytical/Experimental drift estimation**		
	Mean	**Standard Deviation**	**COV**
Thomsen and Wallace (2004)	1.38	0.475	0.345
Priestley et al. (2007)	1.13	0.367	0.325
Berry et al. (2008)	0.74	0.221	0.298
Bae and Bayrak (2008)	0.84	0.265	0.315
Bohl and Adebar (2011)	0.97	0.331	0.340
Kazaz (2013)	1.13	0.395	0.351
Takahashi et al. (2013)	0.83	0.212	0.254

3.5 Discussion of the results

Based on the results shown in Table 3, for seismic assessment purposes in which the accuracy of the drift capacity is important, the equation proposed by Bohl and Adebar (2011) appears to be the more appropriate. However, it has a large variation. The equations proposed by Priestley et al. (2007), Kazaz (2013) over estimates the displacement capacity. That proposed by Bae and Bayrak (2008) and Takahashi et al. (2013) have similar levels of accuracy whilst underestimating the results. The one by Takahashi et al. (2013) has less variation. If the standard variation (or COV) of the results is key, then Takahashi et al. (2013) method may be considered most suitable one. It is worth noting, according to Takahashi et al. (2013), for their equation to work ($L_p = 2.5t_w$), the depth of the neutral axis should be longer than the wall thickness and the transverse reinforcement of the boundary elements should be less than half of that required by the seismic provisions of ACI 318-08 (2008). Otherwise, the equation tends to underestimate the capacity. This might explain the underestimated results when using this method as not all test specimens met those requirements.

The equations proposed by Thomsen and Wallace (2004) as well as ASCE41-17 (2017) considerably overestimates the displacement capacity. Therefore, it may not be suitable for assessment or design purposes. The equation suggested by Berry et al. (2008) that was originally developed for RC columns considerably underestimates the displacement capacity.

4 ASSESSING RC SLENDER WALLS USING THE DIRECT ROTATION METHOD

An alternative method to that based on a moment-curvature analysis is the direct rotation method. In this section, the accuracies of the methods suggested in the Revised Section C5 (yellow edition) of the NZ Engineering Assessment Guidelines (MBIE 2017)) and ASCE41-17 (2017) for estimating the drift capacity of slender RC walls have been evaluated.

4.1 NZ seismic assessment guidelines

According to NZ seismic assessment guidelines (MBIE 2017), the ultimate rotation capacity of a slender wall (shear span ratio greater than or equal to 2) can be estimated using Equation (4).

$$\theta_u = \frac{2\beta_v \varepsilon_y H_e}{3L_w} + (K_d - 1)\phi_y L_p \tag{4}$$

Where, β_v is a dimensionless parameter that accounts for the influence of shear span ratio on the contribution of the flexure component to total yield deformation. For slender walls with shear span ratio between 2 to 4, β_v can be taken as the values shown in Table 4, otherwise $\beta_v = 1$ should be used. The yield strain, ε_y shall not be taken greater than 0.002, $K_d = 15 - 20c/L_w$, c is the neutral axis depth, and for plastic hinge length, L_p, the equation proposed by Priestley et al. (2007) should be used (see Table 1). Figure 5 shows the accuracy of the ultimate rotation capacity estimated by Equation (4).

Table 4. β_v values for Equation (4)

$M/(VL_w)$	2	3	4
β_v	1.43	1.33	1.25

4.2 ASCE41-17 (Table 10.9)

According to ASCE41-17 (2017), the plastic rotation capacity of structural walls with behaviour controlled by flexure, can be determined using Table 10-9 of this standard. Walls with boundary elements that have transverse reinforcement spacing greater than $8d_b$ should be assumed unconfined. For confined boundary elements, ASCE41-17 (2017) refers to the requirements of ACI318 (2019). However, it is unclear which requirements of ACI318, including the level of ductility, that should be considered here. Therefore, in this study any wall with boundary elements with transverse reinforcement spacing smaller than $8d_b$ has been assumed confined. As the plastic rotation capacities determined using Table 10-9 of ASCE41-17 led to conservative results (see Figure 5), the results can therefore be considered as upper bound in respect to application of ASCE41-17 approach. Any of the walls not meeting the requirements of ACI318 will further reduce the mean plastic rotation capacity when using the ASCE41-17 provisions.

4.3 Discussion of the results

Figure 5 illustrates the greater accuracy achieved utilising the moment-curvature analytical approach compared with utilising the direct rotation methods of the Revised Section C5 Engineering Assessment Guidelines and ACE41-17. The moment-curvature approach is shown to provide a better match to the test data compared to the direct rotation approach. The direct rotation method proposed in Revised Section C5 achieved better accuracy and less variation compared to that proposed in ASCE41-17.

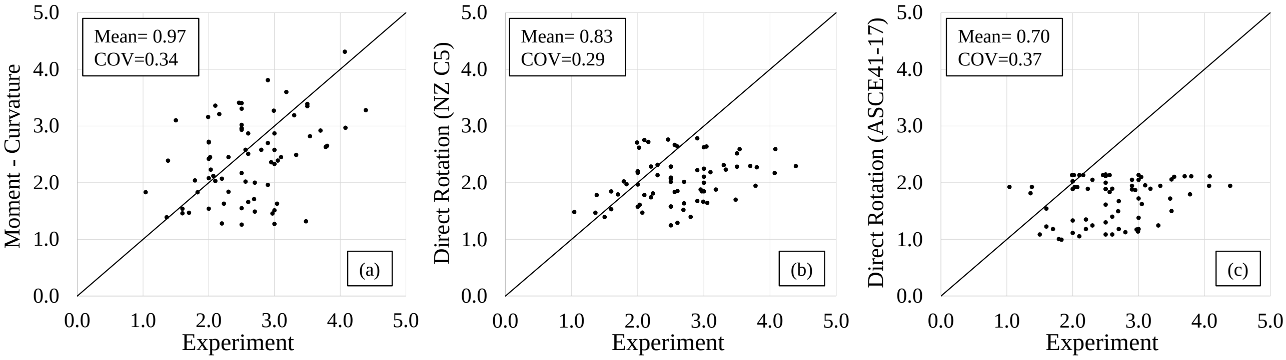


Figure 5. Moment-curvature vs direction rotation, (a) Moment-Curvature, (b) C5, and (c) ASCE41-17

5 CONCLUSIONS

In this paper, the seismic behaviour of slender RC walls with exhibiting flexural failure modes as one of the main failure modes has been investigated based on moment-curvature analysis.

For this purpose, first, a database of tests on slender RC walls with prominent flexural mode behaviour was collected. Then the force-displacement of the test specimens were determined by means of moment curvature analyses considering plastic hinge rotations. Seven well-known equivalent plastic hinge length equations were adopted. Subsequently, the ultimate drift capacity of the walls obtained from the moment curvature approach were compared with those obtained from the test data.

The moment-curvature approach adopting the plastic hinge length equation proposed by Bohl and Adebar (2011) is shown to provide the best match to the test data and considered the most appropriate equation for estimating the drift capacity of RC walls. It is worth noting the method proposed by Takahashi et al. (2013) has the least variation. However, it underestimates the results compared to the test data.

Furthermore, it is shown that the moment-curvature approach is more accurate for estimating the drift capacity of slender RC walls than the approach utilising the direct rotation method proposed in Revised Section C5 of the NZ Engineering Assessment Guidelines and ASCE41-17, provided that an appropriate expression for the plastic hinge length is adopted.

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