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Overstrength and ductility of limited ductile RC walls: from the design engineers perspective

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ABSTRACT: Force-based seismic analysis of a structure requires knowledge of its overstrength and displacement ductility. Local earthquake standards provide typical values for both of these parameters for common types of construction in that region. This paper presents the findings of a recent study to determine the overstrength and displacement ductility of limited ductile reinforced concrete (RC) walls in Australia using the static non-linear pushover analysis method introduced in the latest version of the Australian earthquake code. This activity was performed from the 'perspective of a design engineer', i.e. by using simple analysis methods presented in well-known texts without having to rely on the use of specialised finite element packages, while following and complying with the Australian earthquake loading and material standards. For this purpose a mean stress-strain curve of Australian and New Zealand reinforcement was determined by analysing test data from an independent industry materials testing laboratory. A proposed model for the actual stress-strain curve of 500 MPa L, N and E grade reinforcement is presented in this paper. The long term mean in-situ strength of concrete is also discussed and recommendations put forward.

1 INTRODUCTION

Force-based seismic analysis of a structure requires knowledge of its overstrength and displacement ductility. Local earthquake standards provide typical values for both of these parameters for common types of construction in that region. The Australia earthquake loading standard, AS 1170.4 (Standards Australia 2007) suggests a set of ductility factors ranging from 1 to 4 for different types of structural systems. AS 1170.4 suggests concrete and steel structures designed and detailed in accordance with the 'main body' of their respective material standards – i.e. AS 3600 (Standards Australia 2009) and AS 4100 (Standards Australia 1998) respectively – result in a 'limited ductile' structure and warrants the use of a ductility factor of 2. Furthermore it suggests concrete and steel structures designed and detailed in accordance with the earthquake design clause of each respective material standard result in a 'moderately ductile' structure and warrants the use of a ductility factor of 3. To achieve what AS 1170.4 refer to as a 'fully ductile' structure with a ductility factor of 4, designers are directed to use the New Zealand earthquake loading standard, NZS 1170.5 (Standards New Zealand 2004). Additionally, it must also be used in conjunction with the New Zealand concrete structures standard, NZS 3101 (Standards New Zealand 2006) or the New Zealand steel structures standard, NZS 3404 (Standards New Zealand 1997) for concrete and steel structures respectively.

AS1170.4 accounts for the level overstrength in the structure by the use of the structural performance factor (S_p); where the overstrength (Ω) of the structural system equals $1/S_p$ (Wilson and Lam 2007). The structural performance factor is either taken as 0.77 for limited ductile structures or 0.67 for moderately and fully ductile structures. Meaning the overstrength factor for limited ductile structures is 1.3 and for moderately and fully ductile structures it is 1.5. In a subtle move towards displacement-based seismic design and assessment methodologies, AS 1170.4 allows as an alternative that the ductility and structural performance factor be determined using a non-linear static pushover analysis.

There has been much criticism of the force-based seismic design approach in recent times, particularly with regards to the assumption that all structures of a similar basic structural form and level of detailing possess the same level of overstrength and ductility (Priestley 2013; Priestley, Calvi and Kowalsky 2007). The non-linear static pushover method presents a superior alternative for determining overstrength and ductility factors. It is unrealistic to assume that all concrete structures designed and built in Australia, which are detailed in accordance with the main body of AS 3600, have the same level of overstrength and ductility.

While this is a superior alternative, performing non-linear analyses of RC structures adds extra layers of complexities, which includes required knowledge of the mean material properties of reinforcement and concrete. AS 3600 requires all non-linear analysis methods to use "mean values of all relevant material properties" and AS 3600 Supp1 (Standards Australia 2014) states that for "non-linear and other refined methods of analysis, actual stress-strain curves, using mean rather than characteristic values, should be used." While AS 3600 provides guidance to what the mean strength of concrete is, no guidance is provided as to what the mean properties of D500L and D500N reinforcement are for non-linear analysis purposes. To the author's best knowledge, at the time of this study, this information had not been presented elsewhere in literature.

This paper will propose mean stress-strain curves for D500L, D500N and D500E reinforcement and standard grades of concrete. These curves will be used to calculate the overstrength and ductility values for a set of limited ductile RC walls by performing a non-linear static pushover analysis. The walls used in this study were selected from a set of 31 case study buildings used by the authors while undertaking a recent reconnaissance survey of the Australian RC construction industry.

2 MEAN STRENGTH OF REINFORCEMENT

The Australian concrete standard, AS 3600 (Standards Australia 2009) requires reinforcing to comply with the joint Australian and New Zealand standard, AS/NZS 4671 (Standards Australia and Standards New Zealand 2001). AS/NZS 4671 specifies three classes of ductility: class L, i.e. low ductility; class N, i.e. normal ductility; and class E, i.e. earthquake. Various strength grades are available for each ductility class. In Australia the common grades of reinforcing are D500L and D500N, which both have a characteristic yield stress of 500 MPa.

The mean strength of reinforcement was determined using the tensile test results from a materials testing laboratory contracted by industry suppliers to assess the code compliance of their reinforcing bar. The database of test results developed includes tests on bars from multiple suppliers over a period of 5 years from 2011 to 2015. A summary of the reinforcing bars tested by the industry testing laboratory is presented in Table 1. The test results include values for the yield stress (f_{sy}), ultimate stress (f_{su}) and ultimate strain (ε_{su}) of the reinforcement tested. The ultimate strain is taken as the uniform elongation, i.e. the point corresponding to the onset of necking.

In addition to the test results discussed above, a series of tensile tests of D500N reinforcement was performed in the Smart Structure Laboratory at Swinburne University of Technology. These tests were undertaken to further confirm the values obtained from the materials testing laboratory. 193 samples of N grade reinforcing bar were tested, and were of various sizes being: N12, N16, N20, N24 and N28. The reinforcement was purchased from four suppliers in an attempt to get an unbiased sample set. In addition to the properties described above, the yield plateau strain (ε_{sp}) was also recorded.

Table 1. Summary of tensile tests performed by independent materials testing laboratory.

Grade	Bar sizes tested	Number of samples
D500L	SL82, SL92, SL102	2128
D500N	N10, N12, N16, N20, N24, N28, N32	3979
D500E	E12, E16, E25, E32	150

The test results obtained from the materials testing laboratory only supplied enough information for deducing a bilinear approximation of the mean stress-strain curve of reinforcement. Whereas the testing performed at Swinburne also recorded the yield plateau strain, and hence a more refined approximation of the mean stress-strain curve of reinforcement could be deduced using the (Priestley et al. 2007) model, expressed by Equations 1 to 3.

$$f_s = E_s \varepsilon_s \qquad \text{where: } \varepsilon_s \leq \varepsilon_{sy} \qquad (1)$$

$$f_s = f_{sy} \qquad \text{where: } \varepsilon_{sy} < \varepsilon_s \leq \varepsilon_{sp} \qquad (2)$$

$$f_s = f_{su} - \left(f_{su} - f_{sy}\right)\left[\frac{\varepsilon_{su}-\varepsilon_s}{\varepsilon_{su}-\varepsilon_{sp}}\right]^2 \qquad \text{where: } \varepsilon_{sp} < \varepsilon_s \leq \varepsilon_{su} \qquad (3)$$

Proposed values for constructing bilinear stress-strain curves for the mean response of D500L, D500N and D500E reinforcement are summarised in Table 2 and are based off the results from the independent materials testing laboratory's data. A comparison of the characteristic (i.e. AS/NZS 4671) and mean stress-strain curves of these grades of reinforcement is presented in Figure 1. The results of the Swinburne tests are also presented in this figure; good correlation with the independent laboratory's tests was observed. A summary of the mean value and coefficient of variation of each parameter from the test results is presented in Table 3 and Table 4. A typical histogram plot of the yield stress results for D500N and D500L reinforcement is presented in Figure 2. The strict quality control of class E reinforcement, driven by the tighter restrictions AS/NZS 4671 stipulates compared to that of class L and N reinforcements, is apparent in Figure 1.

Table 2. Recommended mean properties of reinforcement for non-linear analysis.

Grade	f_{sy}	f_{su}	ε_{su}
D500L	585 MPa	620 MPa	3.3 %
D500N	550 MPa	660 MPa	9.5 %
D500E	530 MPa	660 MPa	13 %

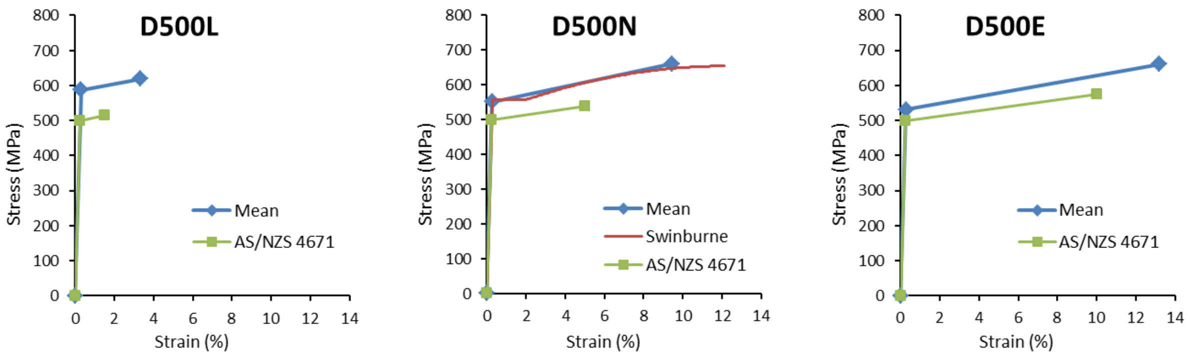


Figure 1. Mean and characteristic stress-strain curves of D500L, D500N and D500E reinforcement.

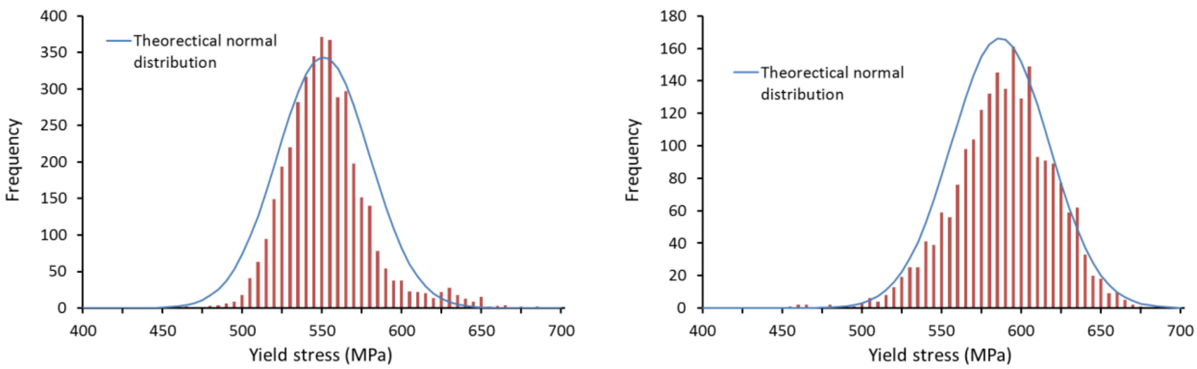


Figure 2. Histogram plots of the yield stress test results. LEFT: D500N rebar. RIGHT: D500L rebar.

Table 3. Mean values of different reinforcement properties.

Grade	f_{sy}	f_{su}	f_{su}/f_{sy}	ε_{sp}	ε_{su}	Number of samples
D500L*	586.7 MPa	619.4 MPa	1.056	–	3.31 %	2128
D500N*	551.0 MPa	660.5 MPa	1.201	–	9.46 %	3979
D500N†	557.0 MPa	654.2 MPa	1.175	1.97 %	12.14 %	193
D500E*	531.4 MPa	661.0 MPa	1.245	–	13.19 %	150

* Denotes results obtained from independent industry test laboratory.
† Denotes results obtained from testing performed at Swinburne University of Technology.

Table 4. Coefficient of variation of different reinforcement properties.

Grade	f_{sy}	f_{su}	f_{su}/f_{sy}	ε_{sp}	ε_{su}	Number of samples
D500L*	0.053	0.050	0.024	–	0.254	2128
D500N*	0.053	0.057	0.063	–	0.307	3979
D500N†	0.056	0.059	0.043	0.483	0.132	193
D500E*	0.046	0.045	0.037	–	0.107	150

* Denotes results obtained from independent industry test laboratory.
† Denotes results obtained from testing performed at Swinburne University of Technology.

A statistical analysis was performed to determine if the theoretical characteristic values of each set of test results were in compliance with AS/NZS 4671. The results are summarised in Table 5. All the characteristic values were within the limits set by the standard except the lower characteristic yield stress of the D500E bars and the lower characteristic f_{su}/f_{sy} ratio of the D500L bars. The non-code compliance of the D500L bars is concerning as the f_{su}/f_{sy} ratio is important, as if it is too low the yielding region of the bar will not propagate along the length causing localised strain concentrations, as discussed by Allington and Bull (2003).

Table 5. Code compliance to AS/NZS 4671.

Property	D500L		D500N		D500E		Type of specified value
	Actual	Limit	Actual	Limit	Actual	Limit	
$f_{sy.L}$ (MPa)	536	≥ 500	503	≥ 500	491*	≥ 500	C_{vL}: p = 0.95
$f_{sy.U}$ (MPa)	637	≤ 750	599	≤ 650	572	≤ 600	C_{vU}: p = 0.05
$[f_{su}/f_{sy}]_L$	1.02†	≥ 1.03	1.10	≥ 1.08	1.19	≥ 1.15	C_{vL}: p = 0.90
$[f_{su}/f_{sy}]_U$	–	–	–	–	1.30	≤ 1.40	C_{vU}: p = 0.10
$\varepsilon_{su.L}$ (%)	2.2	≥ 1.5	5.7	≥ 5.0	11.4	≥ 10.0	C_{vL}: p = 0.90

* Denotes non-code compliance for the lower characteristic yield stress of the D500E bars.
† Denotes non-code compliance for the lower characteristic ratio of ultimate stress to yield stress of the D500L bars.

It should be noted that the standard deviation of the test results can be determined by multiplying the mean value (i.e. Table 3) by the coefficient of variation (i.e. Table 4). Also worth noting is that there were no trends with respect to bar size and the mean of the dataset for each respectively bar size was approximately equal to or less than one standard deviation away from the overall mean of that grade.

3 MEAN IN-SITU STRENGTH OF CONCRETE

The mean in-situ strength of concrete is somewhat more complicated to determine than the mean strength of reinforcement. Concrete is generally specified by its 28 day lower characteristic cylinder strength. The mean cylinder strength is then related to the characteristic cylinder strength by the standard deviation of the mix design, i.e. Equation 4 (Warner et al. 1998). The in-situ strength of concrete is then somewhat lower than the cylinder strength. Furthermore, structures would rarely be subject to 'ultimate limit state' loading conditions at or around 28 days of age and as such long term strength development of concrete should also be taken into consideration.

$$f_{cm} = f_c' + 1.65s \tag{4}$$

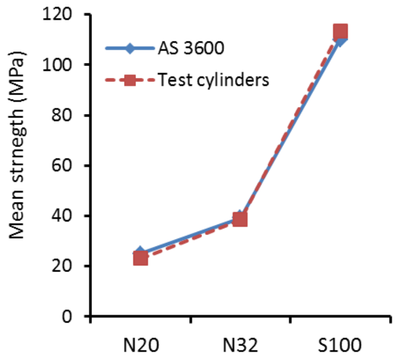
Where: f_{cm} is the mean strength; f_c' is the lower characteristic strength; and s is the standard deviation of the concrete mix.

The standard deviation of a concrete mix is dependent on many variables, for example concrete grades which plants sell high volumes of (i.e. N32 or N40) typically have a lower standard deviation, and separately, central (i.e. city) plants tend to work on lower standard deviations than country plants. For these reasons, plus many others, it becomes very difficult to undertake an experimental study to determine the mean strength of concrete without achieving biased results. For this project the equation suggested in AS 3600 Supp1 for the mean cylinder strength (i.e. Equation 5) was used. Equation 5 was compared against a limited set of test data received from an independent materials testing laboratory contracted by industry companies to test concrete cylinders. Good correlation was observed between the two (Table 6). The in-situ mean strength of concrete was taken to be 90 per cent of the mean cylinder strength, i.e. $f_{cmi} = 0.9f_{cm}$, in line with recommendations by AS 3600.

$$f_{cm} = (1.2875 - 0.001875f_c')f_c' \tag{5}$$

Table 6. Comparison of AS 3600 Supp1 mean strength equation and experimental tests.

	Concrete grade: N20	Concrete grade: N32	Concrete grade: S100
f_c'	20 MPa	32 MPa	100 MPa
f_{cm} (AS 3600)	25.0 MPa	39.3 MPa	110.0 MPa
s (Equation 4)	3.0 MPa	4.4 MPa	6.1 MPa
f_{cm} (test results)	23.1 MPa	38.5 MPa	113.5 MPa
s (test results)	2.4 MPa	4.2 MPa	8.5 MPa
Number of samples	8	18	16



Under the correct conditions the strength of concrete continuously increases at a logarithmic growth rate with respect to time. It is commonly thought that this is the case for concrete generally, however for in-situ RC structures this is not necessarily the case. In-situ concrete typically does not continue to strengthen with age. Unlike continuously moist cylinder samples, which continue to gain strength almost indefinitely; the strength of in-situ concrete can slightly decrease with age. A thorough discussion on the long term strength development of concrete is provided in Neville (1996), including the overview of a study looking at the strength development of core samples over a one year period, which shows no long term net strength gain of the in-situ concrete after 28 days (Figure 3). In-lieu of this, the long term in-situ mean strength of concrete was taken to be the same as the 28 day in-situ mean strength, i.e. f_{cmi}.

The stress-strain curve of concrete presented in AS 3600 Supp1 is being proposed for performing non-linear analysis on limited ductile RC walls. This stress-strain curve is a modified version of the Thorenfeldt, Tomaszewicz and Jensen (1987) curve, which has been calibrated for Australian concrete. It is suitable for modelling the behaviour of normal and high strength unconfined concretes. Stress-strain curves for standard grades which are used in RC walls are presented in Figure 3.

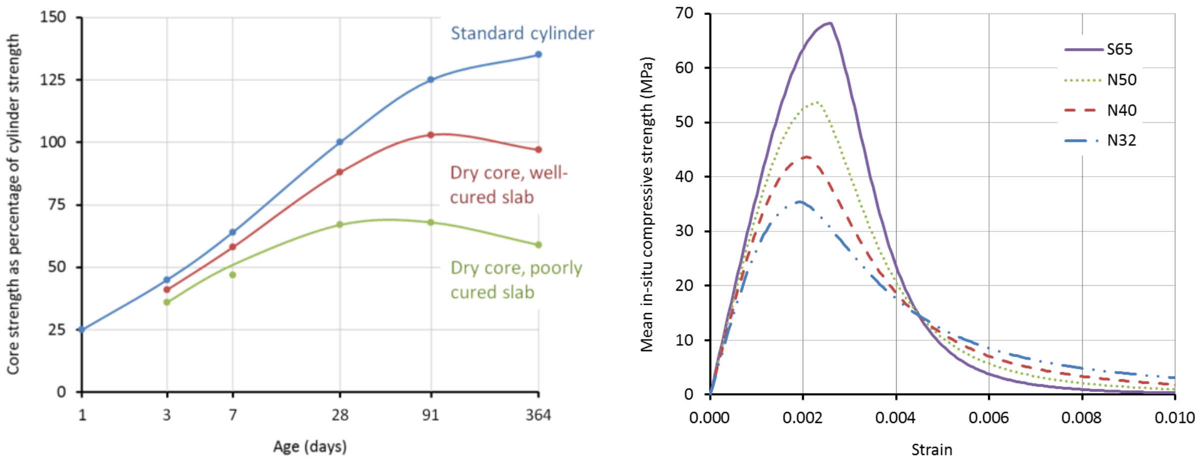


Figure 3. LEFT: Strength development of concrete cores made with type 1 (i.e. GP) cement expressed as a percentage of the 28 day cylinder strength (38 MPa) – redrawn from Neville (1996). RIGHT: AS 3600 Supp1 stress-strain curves for standard grades of concrete.

4 NON-LINEAR STATIC PUSHOVER ANALYSIS

The non-linear static pushover analysis was performed by initially undertaking a moment-curvature analysis of each wall section using a fibre-element analysis program written by the authors, similar to the one presented by (Lam, Wilson and Lumantarna 2011). The mean stress-strain curves of D500N reinforcement and concrete used in the analysis were discussed previously (i.e. Figure 1 and Figure 3 respectively).

A bilinear approximation of the moment-curvature performance of each wall was constructed, in line with the recommendations of Priestley et al. (2007). This was done by projecting a line from the origin through the point of the response curve corresponding to the notional yield curvature (ϕ_y') and up to the point corresponding to the yield curvature (ϕ_y). The corresponding moment value at this location is the nominal moment capacity (M_n). A straight line is then projected from this point to the point corresponding to the ultimate curvature of the wall (refer Figure 4).

The notional yield curvature (ϕ_y') was taken as the point corresponding to first yield of the extreme tensile reinforcement (i.e. $\varepsilon_s = \varepsilon_{sy}$) or the maximum compressive stress of the concrete being reached in the extreme compressive fibre (i.e. $\varepsilon_c = \varepsilon_{co}$), whichever occurs first. The yield curvature (ϕ_y) was taken as the point corresponding to the strain in the extreme tensile reinforcement reaching 0.015 or the maximum compressive stress of the concrete being reached in the extreme compressive fibre (i.e. $\varepsilon_c = \varepsilon_{co}$), whichever occurs first. The concrete strain limit for the yield curvature and the notional yield curvature are the same because it was assumed there was no confinement reinforcement in the walls, in line with the construction trends of Australia. The ultimate curvature (ϕ_u) was taken as the point corresponding to the tensile reinforcement reaching the tensile strain limit ($\varepsilon_{s.uls}$) determined by Equation 6 Sullivan, Priestley and Calvi (2012) or the strain in the extreme compressive fibre of the concrete equalling 0.003 (Standards Australia 2009), whichever occurs first. The slope of the elastic branch of the bilinear relationship is equal to $E_c I_{eff}$ and as such the effective second moment of area of the wall can be calculated using Equation 7.

The bilinear moment-curvature response was converted to force-displacement using the process and equations proposed by Priestley et al. (2007). That is, the yield displacement (Δ_y) is calculated using the yield curvature while assuming a linear curvature distribution up the height of the wall. The ultimate displacement (Δ_u) is calculated using the concept of a plastic hinge, where it is assumed at the base of the wall there is a region of constant curvature and strain occurring over a plastic hinge length (L_p). This process is performed using Equations 8 to 11. A typical force-displacement curve can be seen in Figure 4.

$$\varepsilon_{s.uls} = \min[0.6\varepsilon_{su}; 0.05] \tag{6}$$

$$I_{eff} = \frac{M_n}{E_c \phi_y} \quad \text{OR} \quad I_{eff} = \frac{M_y}{E_c \phi_y'} \tag{7}$$

$$F = \frac{M}{H_e};\ F_y = \frac{M_y}{H_e};\ F_n = \frac{M_n}{H_e};\ F_{max} = \frac{M_{max}}{H_e} \tag{8}$$

$$\Delta_y = \frac{\phi_y (H_e + L_{sp})^2}{3} \tag{9}$$

$$\Delta_u = \Delta_y + (\phi_u + \phi_y) L_p \left[H_e - \left(\frac{L_p}{2} - L_{sp}\right)\right] \tag{10}$$

$$L_p = kH_e + 0.1L_w + L_{sp} \tag{11}$$

Where: H_e is the effective height of the wall and can be taken to equal 70 per cent of the overall height of the wall; L_{sp} is the strain penetration and can be taken to equal $0.022 f_{sy} d_b$; L_w is the wall length; and $k = 0.2(f_{su}/f_{sy} - 1) \leq 0.8$. (Priestley et al. 2007).

In addition to the non-linear static pushover analysis, the ultimate moment capacity (ϕM_u) and ultimate force capacity (i.e. $\phi M_u / H_e$) of each wall was calculated strictly in accordance with AS 3600 (i.e. using characteristic strengths) – note in this instance that ϕ is a capacity reduction factor used for ultimate design in Australia. The overstrength of each wall was determined by dividing the maximum force capacity from the pushover analysis by the AS 3600 force capacity, i.e. $\Omega = F_{max}/(\phi F_u)$.

Two ultimate displacements, $\Delta_{u.L}$ and $\Delta_{u.M}$ respectively, were calculated using the lower characteristic and mean ultimate strains (i.e. 5.7 and 9.5 per cent respectively) of D500N reinforcement. The corresponding ductility factors for each scenario were calculated in accordance with the recommendations in Wilson and Lam (2007), i.e. $\mu = \Delta_u / \Delta_{yu}$. The results of the analyses are presented in Table 7 and Table 8. Δ_{yu} is the displacement of the wall corresponding to the AS 3600 level of perform and can be determined using the effective stiffness of the wall: $\Delta_{yu} = (F_n / \Delta_y) \phi F_u$.

Table 7. Summary of RC walls used in non-linear static pushover analysis.

#	Cross section	L_w (mm)	b_w (mm)	t_w (mm)	H_e (m)	p_v	f'_c (MPa)	Axial load ratio
1	Rectangular	5800	–	300	17.640	0.0141	50	0.029
2	Rectangular	4000	–	300	17.640	0.0158	50	0.073
3	Rectangular	5700	–	250	15.470	0.0251	65	0.055
4	Building core	2800	2800	200	19.250	0.0084	32	0.013
5	Building core	3000	7500	250	18.200	0.0172	65	0.025
6	Building core	3000	5200	200	19.600	0.0105	32	0.051

Table 8. Non-linear static pushover analysis results.

#	ϕF_u (kN)	ϕ	F_{max} (kN)	Δ_{yu} (m)	Ω	$\Delta_{u.L}$ (m)	μ_L	$\Delta_{u.M}$ (m)	μ_M
1	1674	0.78	2247	75	1.34	150	2.0	150	2.0
2	1018	0.75	1359	107	1.33	186	1.7	186	1.7
3	2672	0.76	3590	62	1.34	108	1.8	108	1.8
4	485	0.79	705	126	1.45	442	3.5	580	4.6
5	3187	0.78	4544	106	1.43	387	3.6	512	4.8
6	1243	0.76	1743	123	1.40	428	3.5	563	4.6

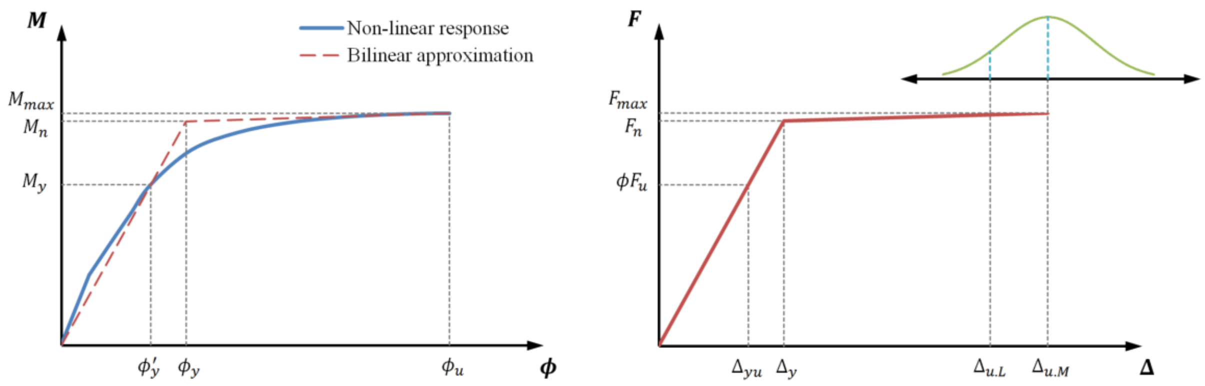


Figure 4. LEFT: Typical moment-curvature diagram showing bilinear approximation. RIGHT: Typical force-displacement curve from non-linear static pushover analysis.

5 DISCUSSION AND CONCLUSION

This paper has presented the statistical representation of the actual mechanical properties, i.e. mean values and standard deviations, of D500L, D500N and D500E reinforcement. These properties were determined using the tensile test results obtained from an independent materials testing laboratory, whom is engaged by industry suppliers to test for code compliance of their reinforcing bar. Bilinear stress-strain curves for the mean response of these grades of reinforcement have been presented. These curves are based off the test results of 2128, 3979 and 150 samples of D500L, D500N and D500E reinforcement respectively, tested over a period of 5 years (2011 to 2015). A discussion on the long term mean in-situ strength of concrete is also included.

A non-linear static pushover analysis was performed using the mean stress-strain curves, presented in this paper, in accordance with AS 1170.4 and AS 3600 for assessing the overstrength and displacement ductility of RC walls. This analysis was performed for three rectangular walls and three box-shaped building cores (i.e. lift shafts or stairwells) selected from actual case study buildings in Australia. The overstrength of each wall was between 1.33 and 1.45. The recommended value in AS 1170.4 for these types of structures is 1.3, indicating good performance.

The ductility of each was assessed based on the lower characteristic and mean ultimate strain of reinforcement. The three rectangular walls have equal ductility for both of these scenarios as the limiting factor for these walls was the ultimate compression strain limit being reached in the extreme compressive fibre of the wall. These walls achieved a ductility factor of 2, 1.7 and 1.8. The recommended value in AS 1170.4 for these types of structures is 2. While the calculated ductility factor is less than the value recommended by the code, the ultimate displacement of each wall was greater than the peak displacement demand of a typical ultimate limit state earthquake in Melbourne or Sydney – a 1 in 500 year return period event earthquake where the maximum response spectrum displacement is 102 mm – which on the surface indicates a code compliant structure. It is noted that a limiting compressive strain of 0.003, as stipulated by AS 3600, is very conservative. Increasing the compressive strain limit would likely increase the ductility of these walls further. Sullivan et al. (2012) has proposed a compressive strain limit of 0.004 for unconfined concrete at this performance level.

The ductility of the three buildings cores was calculated to be approximately 3.5 and 4.5 for when the tensile strain limit is taken with regards to the lower characteristic and mean ultimate strain respectively. The large ductility in the building cores, relative to the rectangular walls, is partly due to the large compression flange area of the section. Meaning a large compressive force can be generated while simultaneously limiting the maximum compressive strain and resulting in the neutral axis being closer to the sections extreme compressive fibre. This causes the reinforcement to undergo large plastic deformations and become the limiting criteria for terminating the analysis. The tensile flange of the wall generates large tensile strains, where it is predominately in pure tension. This situation could possibly result in significant tension stiffening and reduce the level of ultimate curvature, and hence ultimate displacements the wall is able to develop (Menegon, Wilson and Lam 2015).

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