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Review Article

Linking Curvature, Rotation, and Displacement Ductility in RC Shear Walls: A Critical Review and Unified Assessment Framework

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Abstract *- Shear walls made of Reinforced Concrete (RC) are commonplace and popular as load-resisting systems against lateral movement in seismic locations because of their high stiffness and strength. The ductility of their seismic performance is largely determined by the way they can maintain inelastic deformation without much loss in strength to have the capacity to perform well in seismic activities. This paper is a synthesis of experimental data, model analysis, and code clauses that have been developed in the last 50 years to give a complete picture of shear wall ductility. The experimental results are based on a broad scope of isolated wall tests with different aspect ratios, degree of axial loads, and reinforcement configurations. Analytical methods are also discussed, starting with early shear flexure interaction models with fibre section formulations, until current nonlinear finite element methods and coupled shear flexure models. A comparison is made between the ductility requirements of four widely used international codes, namely Eurocode 8, ACI 318, NZS 3101, and IS 13920, based on the curvature ductility demand, boundary element confinement, and displacement capacity. According to these complementary sources, a unified judgment structure is built in order to correlate the various measures of ductility. The suggested framework illustrates that a regular comparison of codes demands that the two levels of ductility and the performance levels should be aligned. Finally, the paper offers practical design and assessment recommendations based on both the proven information and present findings of significant campaigns of earthquake reconnaissance and large-scale experiments.*

Keywords *- Curvature Ductility, Rotation Ductility, Displacement Ductility, Reinforced Concrete Shear Walls, Plastic Hinge, Seismic Design.*

1. Introduction

Shear walls constructed with concrete have been used for a long time as the major seismic force-resistant elements in medium- and high-rise structures. They are effective due to their capability to deliver high in-plane stiffness and lateral strength. Nevertheless, it is the ductility that determines satisfactory seismic performance, being the key to post-yield deformation capacity. Earthquakes such as the California earthquake in 1971 in San Fernando, the Mexico City earthquake in 1985, and the Northridge earthquake in 1994 documented the cases of shear wall failures, which could directly be traced to a lack of ductility capacity [1,2,3]. The overall impact of these earthquake events demonstrated that insufficient confinement and details were some of the main causes of failures of the brittle walls, and the importance of ductility in a seismic structure design could not be overstated.

The Maule earthquake in 2010 in Chile showed major shortcomings in wall performance under high axial loads and ineffective confinement [4,5]. Boundary elements not reinforced sufficiently on their transverse plane collapsed and buckled in a brittle fashion, and thin walls not previously verified to be out-of-plane stable collapsed in modes that had been in the earlier codes, not previously foreseen [6,7]. The lessons were cemented by the 2011 Christchurch earthquake in New Zealand that put a twist on the lessons learned by also observing that even relatively well-detailed walls can experience out-of-plane instability of the boundary region with combined high axial loading and large cyclic lateral deformations [8,9]. The 2016 Kaikoura earthquake in New Zealand and the 2017 Mexico City earthquake added further to the data on observations and helped to underscore the endemic susceptibility of walls with high axial load ratios to failure and inadequately constrained boundary areas [10,11].

Generally, apportionability or ductility is taken to be the ratio of ultimate deformation to initial yield deformation in structural engineering. This concept has three different forms that are hierarchical in nature to RC shear walls. Curvature ductility on a cross-section level measures the amount of bending deformation the critical section can remain in beyond the initial yield. The rotation ductility at the plastic hinge level explains the inelastic rotation capacity within the zone of plasticity concentration. Displacement ductility is at the increased system level that quantifies the ratio of maximum usable roof displacement and the displacement at initial significant yield of the entire wall system. These indicators are linked to various levels of behavior, all the way down to sectional response and far up in the organization on the global structural performance [12,13]. The conceptual framework of Paulay and Priestley [14] lays the groundwork for these three levels to each other and has been used as an entry point to all other work and codes developed on the subject since then.

Despite all the research work, there are still discrepancies in the definitions of ductility and test conditions, which prevent the direct comparison of the results of various studies. Experimental Databases Construction The massive experimental database has been produced in a range of very different test conditions, such as various loading protocols, various ways of applying axial load, and various specimen boundary conditions, and it is indeed hard to make direct comparisons between ductility values across programs [15,16,17]. The four major international seismic design codes use various definitions of ductility and various means of its verification; therefore, cross-jurisdictional comparisons should be carefully understood [18,19,20,21]. Although there are relationships of the three ductility indices to fundamental wall parameters like aspect ratio, axial load ratio, and reinforcement configuration that exist in the literature, none has been coalesced into one framework on which practicing engineers can depend.

Existing studies mainly examine curvature ductility, rotation ductility, and displacement ductility separately. Relationships at individual levels have been revealed in some of the classical works, although a coherent relationship between the two measures is yet to be fully defined. This makes comparison across experiments and design codes difficult. The current paper tackles this question by coming up with a single framework that relates these ductility measurements in a coherent and practical manner.

2. Experimental Data of Shear Wall Ductility

2.1. Early Foundational Programs

Experimental systematic study of the behavior of RC shear walls under ductile conditions began in the 1970s in the United States and New Zealand. One of the most elaborate experimental campaigns that has been carried out on isolated shear walls has been carried out at Portland Cement Association under the two-phase program, as reported by Oesterle et al. [22,23]. Fourteen specimens, which had the cross sections of rectangle, barbell, and flanged, were tested on quasi-static reversed cyclic lateral loading. Based on the findings, flexure-controlled walls with reasonable confinement are found to be hysteretically stable.

At the same time, Vallenas, Bertero, and Popov [24] at the University of California, Berkeley experimented on the reinforcement layout of concrete walls in a variation of wall widths, and the test revealed that the energy dissipation capacity was found to be very sensitive to the width of the boundary zone, in addition to the quality of detailing. The concept of effective plastic hinge length [14] was introduced as a result of this study. Barda, Hanson, and Corley [25] tested low-rise walls and revealed that aspect ratios that were below unity specimens failed in diagonal shear at displacement ductilities as small as 1.5 to 3, despite having theoretically satisfactory longitudinal reinforcement. This observation was viewed to create the difference between slender and squat wall behavior.

In New Zealand, a series of tests by Paulay, Priestley, and Synge [26] was carried out to specifically investigate the sliding shear failure at both the construction joint in the base of squat walls. They demonstrated that when the diagonal cracks had passed all the way across the section, the aggregate interlock mechanism decayed easily during repeated reversals of the load, and that the wall started to move as a rigid body with ductility results lower than two. The best method to inhibit this mechanism was to have diagonal bars that cut across the construction joint at 45 degrees. A massive data bank of low-rise wall tests was then compiled by Wood [27], and he suggested an empirical upper bound shear strength, which identified aspect ratio as the most significant geometric parameter, a move that influenced the shear provisions in the next generation of codes.

Another mining of the fruitful experimental work swarmed between the late 1980s and the early 2000s. Thomsen and Wallace [28] experimented with heating four thin, rectangular-shaped walls to confirm the displacement-based design methodology, and reported that when confined by providing boundary elements with strain-based strain criteria, the walls designed to meet code-level seismic requirements were capable of doing so with reliability when designed to reach displacement ductility ratios of four or five. Salonikios, Kappos, Tegos, and Penelis [29,30] experimented on the lower air of slender walls cyclically loaded and experimented with the interaction of flexural and shear forces in the walls of medium aspect ratio. A significant set of quasi-static cyclic tests of relatively slender walls with different axial load ratios and confinement types was performed by Dazio, Beyer, and Bachmann [31], and offers one of the most calibrated experimental data sets to verify plastic hinge models.

2.2. Gifts of the Past 15 Years

Following about 2010, the experimental data on RC shear wall ductility have grown significantly, both due to the experience of the 2010 and 2011 earthquakes, and due to technological developments in measurement. A general information archive of previously measured slender wall tests was assembled by Birely [32], and it was noted that the contemporary plastic hinge length expressions always overpredicted the deformation strength of high axial load (or comparatively weak) boundary walls. This result led to a reexamination of the way plastic hinge length expressions ought to be unnecessary and include the expression of axial load as an independent variable instead of an implication of the neutral axis depth.

Chen et al. [33] performed cyclic tests on high-strength concrete shear walls with an axial load ratio of up to 0.35 and reported the extreme and progressive ductile loss that takes place when the zone of compression has been expanded, and the area constrained is inadequate to sustain longitudinal bars buckling or curvature once the cover has spalled away. Their findings demonstrated that $n > 0.25$ walls may lose over half of their displacement ductility capacity, relative to otherwise similar walls, having the same volumetric ratio of confinement reinforcement, under the same conditions. A study conducted by Almeida, Prodan, Rosso, and Beyer [34] at EPFL involved a series of tests involving rectangular and T-shaped walls through complex digital image correlation so that the entire strain field is measured across the plastic hinge region with the highest spatial resolution ever. They discovered that the localisation of strain in a narrow band of the boundary element extreme fibre implied that traditional section analysis, where the strain gradient is more gradual, repeatedly overquoted the possible ductility of curvature that could be attained to walls where the strain at fracture of the cover concrete was achieved before the confined core was completely engaged.

Tran and Wallace [35] expanded the database of experiments on slender walls by experimenting on specimens under the combined biaxial lateral loading programs that were designed to look like the combination of the horizontal demands on the structure when subjected to an actual three-dimensional earthquake shaking. They demonstrated that out-of-plane loads during bidirectional loading are potentially up to one-fifth or even a quarter of in-plane ductility capacity as compared with uniaxial tests, a coupling phenomenon that is now completely neglected by all seismic codes. Ding et al. [36] experimented with RC walls that were reinforced with high-strength steel and discovered that, whereas peak performance rose significantly, the ductility of the highest performance at the post-peak was found to be more confinement sensitive than in ordinary-grade steel reinforced walls, and indicated that the required code details developed between ordinary reinforcement might require revision before they could be associated with high-strength steel applications.

A study conducted by Rosso, Almeida, and Beyer [37] on the out-of-plane instability of thin boundary elements subjected to cyclic loading revealed that the critical parameter that determines when the lateral buckling will occur is the ratio between the unsupported height of the boundary zone and the thickness, instead of the entire wall slenderness. The outcome of their experiment provided the foundation behind an analytical model of out-of-plane buckling that was based on mechanics and has been utilized in various national code revision efforts. Also, systematic experimental interest has been the behaviour of non-planar types of wall sections, which add complex behaviour to the in-plane and out-of-plane responses coupled together. A quasi-static cyclic test with two U-shaped reinforced concrete walls by Beyer, Dazio and Priestley [94] demonstrated that the torsional and biaxial flexural demand of the non-planar geometry is found to have a strong influence upon the distribution of inelastic strains at each boundary element relative to the prediction provided by planar walls and has implications on the generalizability of standard expressions of plastic hinge This experimental programme was later expanded by Constantin and Beyer [98] to diagonal loading of U-shaped walls, showing that the directions chosen by simultaneous compression that requires both the web and flange boundary elements to be demanding at the same time give the lowest available curvature ductility compared to that which would arise with either of the loading directions acting alone. Apple disciplines Su et al. [38] examined how concrete strength impacted the ductility in RC shear walls under high axial loads hung on them; they found that increased concrete strength does not necessarily lead to increased ductility due to the ability of the more brittle failure regime of high-strength concrete to counteract the advantage of reduced compression zone depth.

The body of evidence has now been given a significant dynamic dimension by large-scale shake table testing. In Panagiotou, Restrepo, and Conte [39], a full-scale seven-story test comprising an RC wall building was tested on the NEES UCSD shake table, and the plastic deformation distribution along the wall height was subjected to realistic ground motions of multidirectional nature. One important conclusion was that the higher mode effects substantially enhanced shear demands in the upper stories and displaced the effective center of plastic rotation along with the idealised base hinge, the structural capability of which would be lost by simple plastic hinge models. Stavridis et al. [40] had a shake table program on coupled shear walls, which showed that the coupling beam detailing had a strong influence on the amount of hysteretic energy dissipation of the overall system. Poor coupling beam detailing strained its ductility early, and the overall effect of the demand on the walls themselves shot up steeply as later cycles reached the layer of hysteresis.

Post-disaster field data was sobering, to the point where the post-disaster field work on the Maule earthquake offered no less than sobering evidence. Westenenk, de la Llera, Besa, Junemann, Moehle, Lüders, and others [4] reported the failure of RC walls in the present-day buildings of Concepcion, discovering that the prevalent source of failure was brittle crushing of boundary areas with high ratios of axial loads. Wallace, Massone, Bonelli, Dragovich, Lagos, Lüders, and Moehle [5] made some direct inferences towards seismic design and emphasized that wall thickness was less than about 200 mm, which was disproportionately vulnerable. This was reported by Kam, Pampanin, and Elwood [8], who conducted a comprehensive damage survey after the Christchurch earthquake in February 2011 and reported previously unanticipated out-of-plane failures in slender walls as per the existing code provisions. Elwood [9] generalised these observations and presented their implications to the provisions of the Canadian codes, and Sritharan, Beyer, Henry, Chai, Kowalsky, and Bull [41] took cross-national lessons that can be applied to code provisions in the practice of wall design more generally.

More recently, Varsamis and Papanikolaou [42] reported tests of walls with confined boundary elements specifically tuned to the Eurocode 8 requirements and discovered that the requirements of the code were generally sufficient to attain the target curvature ductility when subjected to uniaxial loading, but that the safety margin was lower when the walls were at the highest permissible axial load range. A parametric numerical study by Kazaz [43] fitted to the experimental database and suggested new design equations of the plastic hinge length and the curvature ductility available, which are more precise than previous classical Paulay and Priestley expressions of walls with high axial loads and each with a thin web. Dashti, Dhakal, and Pampanin [44] created a numerical model covering out-of-plane instability on a slender wall boundary element and verified it to match experimental data. A tool can be utilized to determine whether the thickness requirement of ACI 318 and NZS 3101 is adequate to meet specific wall configurations.

2.3. Effect of Aspect Ratio

A single most important geometric parameter to control the mode of inelastic behaviour and, therefore, the ductility that can be attained, is the height-to-length ratio of a wall, expressible as Hw/Lw. Hw/Lw walls of greater than or equal to 2 are considered slender and normally exhibit a flexural plastic hinge at the base until there is a lot of shear distress in the web. Through proper detailing, such walls are able to support interstorey drift ratios of 1.5 to 3.5 percent and displacement ductility ratios of four to eight [22,23,28]. Walls with Hw/Lw that are less than or equal to 1.0 are considered as squat, and their behaviour is controlled by horizontal joint diagonal shear or sliding. Even at the highest drift ratios that the walls can sustain, say 0.5 to 1.0 percent, they are seldom manufactured with displacement ductility ratios greater than three, even with the maximum amount of longitudinal reinforcement [25,26,27]. The intermediate aspect ratio walls (1.0 to 2.0) will be transitional, where initially flexural yielding will be followed by gradually increasing shear distress, and the attainable ductile behavior will depend on the ratio of shear reinforcing between the flexural and additionally the flexural reinforcing [29,30].

As demonstrated by Wallace and Moehle [15] who analysed a large database of RC wall buildings which survived the 1985 Chile earthquake and had aspect ratios of between two and four, the deformation capacity of walls having a Hw/Lw ratio between two and four was enough to sustain storey drifts of about 1.5 percent and the damage was widespread and severe in buildings whose walls had aspect ratios lower than the Aspect ratio limits had previously been suggested by Wallace [45] to place walls in the code as being either flexure controlled or shear controlled and are now used in virtually all major seismic codes. Blandon and Priestley [46] elaborated this image with nonlinear time history analysis and established that flexure-controlled and shear-controlled behavior is not a cut-off point, but it varies with the degree of shear reinforcement and the magnitude of the driven movement. Still more recently, Luna, Rivera, and Whittaker [47] suggested some new limits to aspect ratios classifications in terms of a rework of an extended experimental database and concluded that there is a transition zone that is a little broader than assumed previously when both web and boundary reinforcement are independently manipulated.

2.4. Effect of Axial Load

The axial compressive load has a dual, and somewhat contradictory, effect on wall ductility. At a small level, with axial load ratio n = N/Ac f'c small, the depth of the neutral axis at ultimate is small, the thickness of the compression zone in concrete is very thin, and the ultimate compressive strain is attained at a fairly large top of wall displacement. This gives high curvature ductility at the critical section [14,28]. With the increase of axial load beyond n of 0.20 to 0.25, the depth of the neutral axis is increased significantly, the compression zone expands, and the structure is more exposed to concrete crushing and bar buckling following cover spalling, and ductility is highly reduced. This behavior was reported in full-scale building tests in Japan and then recorded by Kabeyasawa, Shiohara, and Otani [48] who found that axial load ratios above 0.30 gave essentially non-ductile wall behaviour in all confinement details.

A quantitative description was given by Dazio, Beyer and Bachmann [31]: walls of n less than 0.10 would tend to have a curvature ductility of fifteen to twenty five at the critical section and thus a displacement ductility of five to eight at the system level, and walls of n about 0.20 tended to have a curvature ductility of eight to twelve and a displacement duct At a n greater than 0.25 when the unconfined cover concrete prematurely crushed, premature failure with curvature ductilities as low as four and six had begun. More recently, Chen et al. [33] demonstrated that n need not be much greater than 0.35, when compared with n equal to 0.25, to cut down the available displacement ductility of otherwise identical wall specimens, demonstrating the significance of strict axial loading limits in ductile wall design.

Another complication that has gained increasing interest over the last few years is the extra axial loading that takes place on the elements that limit the boundaries during the bi-directional loading of earthquakes. Segura and Wallace [49] showed analytically that simultaneous loading in two horizontal directions can augment the effective compressive power, on the compression boundary element, by a fifth to sixth of the compressive power in the uniaxial case, even though depleting remaining curvature ductility.

The interaction has not been tackled in any of the four significant codes that have been considered in this paper, and this is a significant distance between where research knowledge is and where practise is in code. Qin et al. [50] studied coupled walls in a bidirectional loading and yielded the same results, whereby the coupling beams were found to increase demand in the compression wall axial direction in the event of the diagonal excitation of earthquakes by quantities that could not be defined using the normal envelope processes.

2.5. Boundary Element Detailing Effect.

The scope and detail level of the confinement within the boundary elements, which are the most stressed areas at each end of the wall cross section, is a well-known, single, ductility-determining detailing parameter. Analytically and experimentally Sheikh and Uzumeri [51] showed that with tight spacing of rectangular hoops with additional crossties, the ultimate compressive strain of confined concrete increased to about 0.003 to values near to 0.015, and that this enhancement depended on the volumetric ratio and the yield strength of the transverse reinforcement, with the latter narrowing interval playing a major role. These relationships were formalised in the confined concrete constitutive model of Mander, Priestley and Park [52] as they have become the commonly used tool of analysis of boundaries in research and practice that follows. Welt, Massone, LaFave, Lehman, McCabe, and Polanco [99] demonstrated experimentally that the confinement effectiveness of the fringe element of rectangular walls is less than that of square columns and that the prediction by the standard Mander, Priestley, and Park model could thus be unconservative in thicknesses of the opposite element of rectangular conduits.

Analysis by Oesterle et al. [22,23] revealed that the difference in hysteretic behaviour of well-confined and poorly confined elements of a boundary was dramatic. The walls with a tight space between the hoops held a minimum of eighty percent of the strength they once had to reach drift ratios of 2.5 percent, and unconfined walls were found to have their strength reduced more than forty percent at the same drift. Paulay and Priestley [14] obtained the necessary confinement reinforcement ratio as a function of the axial load ratio and the depth of the neutral axis, and their relationship has been inserted in the rest of the boundary element design provisions of Eurocode 8 and ACI 318.

Two other areas of detailing concern, which historically had garnered inadequate focus in an earlier generation of codes, have been highlighted by a growing body of evidence, both experimental and earthquake reconnaissance. The first is the stability in the out-of-plane of boundary elements, and it is conditioned by the unsupported height of the compression zone, the slenderness of the wall, and the degree of the axial load. Analytical models Barrier and lateral stability- Chai and Elayer [53] performed tests on the boundary elements with a thin wall and, based on them, came to the conclusion that out-of-plane buckling could be observed at the same ratio of drift of the thick wall to the height to supports (Thickness to height) ratios lower than approximately 1 to 16. This work was developed by Dashti, Dhakal, and Pampanin [44] to create a three-dimensional finite element model including the interaction of in-plane curvature demand and out-of-plane instability, and their findings have been incorporated to suggest a minimum thickness requirement, which is now being used in ACI 318. The second issue is the distance between longitudinal bars in the boundary element. Moyer and Kowalsky [54] demonstrated that widely spaced bars restrained using the corner of hoops will not buckle at smaller lateral drifts than bars separately restrained using the crossties, and that the existing ACI 318 maximum bar spacing of 150 mm might not be sufficient to support walls with large axial loads under large cyclic deformations. ACI 318 2019 has also minimized the longitudinal bar spacing that is allowed in special boundary elements.

2.6. Sliding Shear Dominated Failures

There are two different failure mechanisms to preclude ductility of the wall prior to the complete activation of the flexural mechanism, which are sliding shear along the horizontal construction joints of the building and diagonal shear failure of the wall panel. Sliding shear must occur due to the accumulation of residual crack opening where horizontal joints occur, under reversed cyclic loading. Paulay, Priestley, and Synge [26] demonstrated that once diagonal cracks have penetrated the entire length of the section, the aggregate interlock mechanism, which supplies a significant portion of the shear resistance during the first few cycles, decays very quickly with repeated reversing loads, and the wall starts to move as a rigid body. Mattock and Hawkins [55] measured the shear of the construction joints' friction capacity, and Walraven [56] extended the analysis and included the considerable degradation of the aggregate interlock with cumulative slide displacement, and determined that the effective friction coefficient decreases by half or more, with the value decreasing to become less than 0.8 after numerous slides. Controlling the ductility of low-rise and moderately slender walls with high shearing requirements is achieved through diagonal shear failure. Wood [27] had the empirical upper bound shear strength and demonstrated that the empirical upper bound is attained at drift ratios of 0.5 to 1.0 percent in squat walls, long before the flexural capacity was even able to occur. A detailed re-analysis of the shear strength database of squat walls was then conducted by Gulec and Whittaker [57], which revealed that much of the scatter in current predictions of squat wall strength is due to variability of web reinforcement arrangement and normalised axial load. Their model resulted in a coefficient of variation of predictions being smaller than about 0.35, and thus a significant improvement with direct implications for the assessment of existing squat wall buildings. Arafa et al, more recently. [58] experimented with squat walls built using materials typical of Latin American practice and discovered that the shear strength model developed by Gulec and Whittaker better predicted than the previous Wood expression of walls with low ratios of web to steel, of which older building stocks in developing countries are built.

2.7. Overview of Experimental Results

The ranges of reported displacement ductility of the key experimental programs covered in this section are summarised in Table 1 below. The list includes the seminal 1970s campaigns as well as publications as recently as the 2010s and is indicative of the wide variety of aspect ratios, levels of axial load, and modes of failure that have been studied by the research community. In general, these experiments demonstrate that the axial load ratio and boundary confinement mainly control ductility, whereas the mode of failure is mainly controlled by the aspect ratio.

Table 1. Summary of displacement ductility ranges from key experimental programs on RC shear walls

Source	H_w/L_w range	$n = N/A_c f'_c$	Displacement ductility range	Governing mode
Oesterle et al. [22,23]	1.0 to 3.0	0.04 to 0.12	3 to 8	Flexure and sliding
Barda et al. [25]	0.25 to 1.0	0.0 to 0.08	1.5 to 3	Shear
Paulay et al. [26]	0.5 to 1.0	0.0 to 0.10	1.5 to 2.5	Sliding shear
Wood [27]	0.5 to 2.0	0.0 to 0.20	1.5 to 4	Shear and flexure
Salonikios et al. [29,30]	1.0 to 2.0	0.10 to 0.20	2 to 5	Flexure shear
Thomsen and Wallace [28]	3.0 to 4.0	0.08 to 0.10	5 to 8	Flexure
Dazio et al. [31]	3.5 to 4.5	0.06 to 0.12	4 to 7	Flexure
Almeida et al. [34]	2.0 to 4.0	0.05 to 0.20	3 to 7	Flexure

[Note: H_w/L_w = aspect ratio (wall height/wall length); n = axial load ratio (= $N/A_c f'_c$); displacement ductility = ratio of ultimate to yield top displacement. Values are approximate ranges from the cited programs; exact values depend on specimen geometry and loading protocol.

3. Analytical and Numerical Modelling of Ductility

3.1. Plastic Hinge Models

Among available analytical approaches, plastic hinge models remain the most widely adopted due to their simplicity and compatibility with displacement-based design frameworks. In this approach, inelastic deformation is assumed to concentrate over a finite equivalent length L_p at the base of the wall. Within L_p, the curvature is taken as approximately uniform at the ultimate value, while the remainder of the wall deforms elastically. The displacement ductility ratio at the top of a cantilever wall of height Hw is then given by the well-known expression [14,59]:

$$\mu_{\Delta=}1 + 2(\mu_\phi - 1)(\frac{L_p}{H_w})(1 - \frac{L_p}{2H_w}) \quad (1)$$

Where μ_ϕ equals ϕ_u/ϕ_y, is the curvature ductility at the critical section, ϕ_u is the ultimate curvature, and ϕ_y is the yield curvature. The yield curvature for a rectangular wall can be estimated as approximately. $\frac{2\epsilon_y}{L_w}$, where ϵ_y is the yield strain of the longitudinal steel, and L_w is the wall length in plan. This relationship follows from the assumption that the neutral axis depth at yield is approximately 0.5 Lw, which Paulay and Priestley [14] verified experimentally for walls with axial load ratios below 0.20. The key unknown in this model is the equivalent plastic hinge length L_p. Paulay and Priestley [14] proposed L_p equals 0.08 times H_w plus 0.022 times f_y times d_{bl}, in millimeters, where f_y is the yield strength of the longitudinal bars and d_{bl} is their diameter. This expression separates the contribution of flexural spread along the wall height from the additional rotation due to strain penetration of the longitudinal bars into the foundation.

Priestley and Park [60] proposed a simplified form that has been widely used for bridge columns and is also applicable to walls. Kowalsky [93] extended the deformation limit state framework to explicitly define curvature and displacement thresholds at yield, damage control, and survivability levels, demonstrating that these hierarchical limit states are consistent across circular column and wall geometries when normalized by the yield curvature, which reinforces the generality of the plastic hinge relationships adopted in the present framework. More recently, Bohl [61] has evaluated the plastic hinge length expression against a broader experimental database that included walls with higher axial loads than those available to Paulay and Priestley, and found that the classical expression overestimates L_p by fifteen to thirty percent when the axial load ratio exceeds 0.15. They proposed a modified expression that includes the axial load ratio as an explicit variable. Haro et al. [62] reached a similar conclusion following their analysis of the Christchurch earthquake damage, where the effective height of the plastic hinge zone in damaged walls was considerably shorter than classical models would predict, largely because high axial loads concentrate strain into a narrower region.

Kazaz [43] performed an extensive parametric study using validated nonlinear finite element models and proposed updated regression equations for L_p that account for concrete strength, axial load ratio, and reinforcement configuration simultaneously. His proposed expression gave prediction errors below fifteen percent against an independent validation dataset, substantially better than the classical Paulay and Priestley expression, which showed errors up to forty percent for high axial load cases. Pugh, Lowes, and Lehman [63] proposed a plastic hinge length formulation specifically calibrated for slender walls in high seismic zones and showed that incorporating the axial load ratio explicitly improved both the accuracy and the consistency of predicted drift capacity across a wide range of wall configurations. A comparison of analytical approaches suggests that models incorporating shear–flexure interaction provide more realistic predictions for walls with moderate aspect ratios.

3.2. Fibre Section and Macro Models

Fibre section models represent the wall cross-section as an assembly of uniaxial fibres, each assigned an appropriate constitutive law for the relevant material. Confined concrete in the boundary element is modeled using the Mander, Priestley, and Park [52] expression or a variant thereof, unconfined cover concrete is assigned a standard monotonic descending branch, and reinforcing steel is represented by a bilinear or Ramberg Osgood relation that captures isotropic and kinematic hardening. The equilibrium and compatibility conditions at each section are satisfied iteratively, and the section response is integrated along the wall height using displacement-based or force-based beam-column elements. Orakcal, Wallace, and Conte [64] demonstrated that fibre models using the Mander et al. constitutive law predicted the lateral load displacement response of slender test walls with errors below ten percent in both peak strength and stiffness at all ductility levels tested. Orakcal and Wallace [65] extended this validation to include the post-peak response and showed that the model captures the stiffness degradation under cyclic loading with reasonable accuracy, provided that the steel cyclic constitutive law includes a realistic representation of the Bauschinger effect.

The most significant limitation of fibre models is that they do not account for shear deformation or shear flexure interaction. Massone and Wallace [66] showed that in walls with aspect ratios below 2.5, shear deformations contribute twenty to forty percent of the total top displacement at peak load, meaning that a pure fibre model ignoring shear may underestimate the total drift capacity by a corresponding amount. To address this, Massone and Wallace proposed a coupled shear flexure model in which the fibre section is extended to include shear springs at each integration point, calibrated using a Modified Compression Field Theory formulation. Kolozvari, Orakcal and Wallace [67] developed this approach further into an Interaction Multiple Vertical Line Element model and validated it against a broad set of test results, including walls with squat to intermediate aspect ratios where shear flexure coupling is most pronounced. Kolozvari et al,.[68] further revealed that the I-MVLEM is capable of capturing shear and flexural contributions to peak deformation capacity to within fifteen percent of the walls with aspect ratios staying between 1.0 and 3.0, a significant enhancement over previous uncoupled fibre models.

On an even higher level of abstraction, the Multiple Vertical Line Element Model proposed by Vulcano, Bertero, and Colotti [69] views the wall as a collection of vertical springs in contact with each other through horizontal springs and a rotational spring. Although the MVLEM seems very basic in nature, it characterizes the main characteristics of flexural ductility and has been proven to be valid when using slender walls [64]. It is computationally efficient, so it is desirable in parametric studies and other applications in probabilistic analysis where hundreds of analyses are required. Fischinger, Rejec, and Isakovic [70] further developed the initial MVLEM to incorporate a nonlinear shear spring, which enables the model to represent a shear-governed and shear-flexure-coupled response, and this significantly increased the range of the model. In their comparison of five macro models of RC walls, Gogus and Wallace [71] plotted their results in a shared benchmark dataset, and the results indicated that models that used shear flexibility always excelled over pure flexural models of walls with aspect ratios that were smaller than 3.0.

3.3. Finite Element Methods.

The nonlinear finite element analysis offers maximum resolution, however, at significantly higher computational cost and modelling complexity. In the case of RC shear walls, NLFE models have to consider cracking, tension stiffening, compression softening of cracked concrete under increasing shear, and bond slip of reinforcing bars, which form extremely strong interactions under reversed cyclic loading. Bertero, Popov, Wang, and Vallenas [72] were among the first to develop NLFE simulations of cyclic wall behaviour based on smeared crack concrete models, and demonstrated that reasonable predictions of peak load could be made, though they needed to model the process of crack closing and aggregate interlock explicitly in order to predict post-peak degradation.

In recent times, the discipline has grown significantly beyond these initial studies and was spurred on by the progress of concrete constitutive models, enhanced computing capabilities, and the presence of open-source software with proven validity, like the OpenSees software. Dashti, Dhakal, and Pampanin [44] conducted a study with three-dimensional NLFE models, throwing light on out-of-plane buckling of slender wall boundary elements. They established that this mode of failure depends on the slenderness ratio of the compression zone, the amount of previous tensile strain accumulated by the boundary element during the tension half cycle, and the spacing of transverse reinforcement. These effects cannot be effectively described using one-dimensional fiber approaches, making this an example of an issue where three-dimensional NLFE analysis is required.

The model used by Luu, Mo, and Hsu [73] was the mesoscale NLFE model, which was implemented for a set of slender and squat wall specimens using the Cyclic Softened Membrane Model, and demonstrated that the shear and flexural components of response were predicted with a coefficient of variation of less than 0.15. Applying a total strain-based smeared crack NLFE model, Broujerdian and Kazemi [74] have predicted the full overall load deformation response of walls of both sets of tests: the Oesterle et al. [22,23] and Salonikios et al. [29,30] series, and have found that errors in peak strength are less than eight percent, with somewhat greater errors in the post-peak.

Panagiotou, Restrepo, and Conte [39] used a beam truss NLFE model to analyse the full-scale seven-story building tested on the UCSD shake table and showed that the model captured the distribution of plastic deformation along the wall height with good accuracy under multidirectional ground motion. A critical finding was that higher mode effects significantly amplified the shear demand in the upper stories and that simplified envelope procedures used in codes provide insufficient shear demands in this region, with implications for the detailing of the web reinforcement above the plastic hinge. Kim and Wallace [75] calibrated an advanced OpenSees model against the Panagiotou et al. shake table data and demonstrated that the I MVLEM macro model captured both peak and residual displacements with sufficient accuracy for practical use in performance assessment, at a fraction of the computational cost of full three-dimensional NLFE models.

3.4. Energy-Based and Probabilistic Ductility Assessment

The hysteretic energy dissipation capacity of an RC shear wall provides a measure of cumulative ductility that peak displacement ratios alone cannot capture. Park and Ang [76] proposed an energy-based damage index combining maximum displacement and cumulative hysteretic energy, with a model parameter beta typically in the range of 0.10 to 0.15 for RC walls. Wang and Shah [77] applied a similar energy framework to RC walls and showed that the energy dissipated per cycle scales approximately with the square of the first cycle ductility ratio for well-confined walls, providing a useful upper bound for energy-based damage assessment under multi-cycle excitations. Hidalgo, Jordan, and Martinez [78] proposed an analytical model to predict the inelastic seismic behaviour of shear wall buildings and demonstrated that cumulative damage effects under ground motions with many significant cycles, as occur in subduction zone earthquakes, can reduce the effective ductility capacity to sixty to seventy percent of the first cycle value.

Haselton, Liel, Deierlein, Dean, and Chou [79] demonstrated through incremental dynamic analysis that energy-based damage measures are better predictors of collapse risk than peak displacement measures for wall buildings subjected to near-fault ground motions with significant velocity pulse content, where cumulative damage under repeated cycling at high ductility levels is the governing failure mechanism. Lignos and Krawinkler [80] developed a phenomenological model for the cyclic deterioration of RC members that has been widely used in nonlinear time history analyses and captures the interaction between peak ductility demand and cumulative dissipated energy in a format compatible with performance-based earthquake engineering frameworks. More recently, Raghunandan and Liel [81] applied a probabilistic fragility analysis framework to RC shear wall buildings and showed that the coefficient of variation of collapse capacity across different ground motion records is substantially higher for buildings with squat walls than for buildings with slender walls, reflecting the greater sensitivity of shear-dominated behavior to high-frequency content in the excitation.

4. International Code Provisions for Ductility

4.1. Eurocode 8 (EN 1998 1:2004)

Eurocode 8 [18] classifies RC shear walls into three ductility classes: Ductility Class Low, Medium, and High, corresponding to behavior factors of approximately 1.5, 2.5 times the $\frac{\alpha_u}{\alpha_1}$, and 4.0 times the $\frac{\alpha_u}{\alpha_1}$, respectively, where $\frac{\alpha_u}{\alpha_1}$ accounts for system overstrength due to redistribution. For Ductility Class High walls, EN 1998 1 prescribes detailed rules for a critical region at the base of the wall, defined as the greater of H_w divided by 6, L_w, and 2.0 m, in which both the extent of the confinement zone and the minimum volumetric ratio of confinement reinforcement are specified as functions of the normalised neutral axis depth and the axial load ratio.

The required curvature ductility ratio at the critical section for Ductility Class High walls is μ_ϕ greater than or equal to 13 when the basic value of the behaviour factor exceeds 3.0. This target is broadly consistent with the experimental data reviewed in Section 2, provided that the axial load ratio does not exceed 0.20 and that the boundary elements are confined in accordance with the code rules. EN 1998 1 does not require explicit verification of global displacement ductility or interstorey drift ratio at the design earthquake level for wall-dominated buildings, relying instead on the behaviour factor to implicitly represent the inelastic demand. Penelis and Penelis [82] have discussed the practical consequences of this implicit approach and shown that the implied displacement ductility demand for typical Ductility Class High wall buildings in Mediterranean seismic zones is in the range of two to four, well within the capacity predicted by the framework of Section 5. For practical verification of this implicit demand, EN 1998-1 recommends the N2 method of Fajfar [95], a nonlinear static procedure that combines a pushover analysis of the structure with an inelastic response spectrum to yield a direct estimate of the target displacement and the corresponding ductility demand at the design earthquake level by providing a transparent link between the behaviour factor and the actual deformation imposed on the wall system. The inherent limitations of such single-mode pushover procedures for tall wall buildings, where higher mode contributions are significant, have been discussed critically by Krawinkler and Seneviratna [96], demonstrating that pushover-based demand estimates become increasingly unreliable for structures in which the first mode does not dominate the inelastic response, a caveat directly relevant to wall-dominated buildings exceeding eight to ten stories. The anticipated revision of EN 1998 1, expected to introduce more explicit displacement verification requirements and tighter provisions for walls with high axial loads, has been the subject of pre-normative research by Fardis, Carvalho, Fajfar, and Pecker [83].

4.2. ACI 318 and ASCE 41

ACI 318 classifies shear walls as ordinary, intermediate, or special, with special structural walls required in the highest seismic design categories. The 2019 edition of ACI 318 introduced several important changes relative to earlier editions, driven in part by the earthquake reconnaissance findings following the 2010 Maule earthquake and the experimental work of Tran and Wallace [35] and Segura and Wallace [49]. The 2010 Maule earthquake saw the Chilean structural engineering community initiate its own wide-scale code revision process. The underlying principles of these changes have been recorded by Massone [97], which included explicit axial load limits and minimum wall thickness requirements, as well as a new set of boundary element confinement rules. The noteworthiness of these code changes is that they pre-empted the 2019 ACI 318 provisions and, in a number of respects, like those that have been made by ACI 318 later, foreshadowed the detailing reforms that ACI 318 then adopted. Among these changes are an increase in the maximum required thickness of special boundary elements to minimize the risk of out-of-plane buckling, a smaller maximum longitudinal bar spacing of special elements, and a more sensitive criterion to activate the requirement of special elements. In ACI 318, the boundary element trigger is either a stress criterion (which mandates that the extreme fibre compressive stress due to factored load is greater than 0.2f'c) or a strain criterion (which mandates that the calculated extreme fiber compressive strain is greater than 0.003 due to the factored displacement demand). Moehle [84] has given an official commentary on the grounds behind these requirements and their connection to the experimental findings.

For seismic assessment and retrofit of existing buildings, ASCE 41 [20] provides explicit deformation-based acceptance criteria for RC shear walls at three performance levels: Immediate Occupancy, Life Safety, and Collapse Prevention. The plastic rotation limits are tabulated as functions of the axial load ratio, the wall shear stress ratio, and the boundary element confinement status. These limits represent the most explicit quantification of hierarchical ductility demand in any major internationally used code and provide a useful benchmark against which experimental data can be directly compared. Elwood, Matamoros, Wallace, Lehman, Heintz, Mitchell, Moore, Valley, Lowes, Comartin, and Moehle [85] have documented the experimental basis for the ASCE 41 acceptance criteria and discussed their limitations, noting that the criteria for lightly confined walls and walls with high axial loads are among the least well calibrated against experimental data.

4.3. New Zealand Standard NZS 3101:2006

NZS 3101 [21] is unique among major seismic codes in making curvature ductility demand at the critical section an explicit design parameter. The design process begins with the selection of a target system displacement ductility, from which the required curvature ductility is derived using the plastic hinge relationships of Paulay and Priestley [14]. The required confinement reinforcement in the boundary elements is then determined from the condition that the ultimate curvature must meet or exceed the product of the required curvature ductility and the yield curvature. This closed-loop design procedure, refined through successive editions of the standard, represents the most direct implementation of displacement-based design principles in any current code. Priestley, Calvi, and Kowalsky [59] have provided the fullest exposition of this approach and the theoretical basis underlying it.

NZS 3101:2006 further requires amplification of the design shear force above the capacity design value to account for higher mode effects, following recommendations that trace back to Blakeley, Cooney, and Megget [86] and have since been reinforced by the shake table tests of Panagiotou, Restrepo, and Conte [39] and the analytical studies of Rutenberg [87]. This provision is absent from Eurocode 8 and ACI 318 and results in heavier shear reinforcement in the upper stories of tall wall buildings, but it significantly reduces the risk of non-ductile shear failure above the plastic hinge zone. Following the 2011 Christchurch earthquake, extensive work was undertaken to review and update NZS 3101, with particular attention to minimum wall thickness requirements for out-of-plane stability and to confinement requirements at higher axial loads, as summarised by Haro et al. [62] and by Elwood [9].

4.4. Indian Standard IS 13920

IS 13920 [16] and the companion concrete code IS 456 [88] together constitute the ductile design framework for RC structures in India. IS 13920 prescribes minimum dimensions, longitudinal reinforcement limits, and transverse reinforcement rules for structural walls classified as ductile walls in Seismic Zones III to V. The code adopts a prescriptive detailing approach broadly similar to pre-2008 ACI 318 but calibrated to Indian construction practice, including the common use of Fe415 and Fe500 grade steel and locally produced aggregates. The 2016 revision of IS 13920 introduced improved detailing requirements for boundary elements, tighter limits on axial load for ductile walls, and revised criteria for determining the extent of the special boundary zone that are somewhat more sensitive to wall displacement demand than the purely stress-based trigger of the earlier edition.

Despite these improvements, IS 13920 still does not compute or verify the curvature ductility capacity of the boundary elements as an explicit step in the design process. The required spacing of lateral ties within the boundary zone is prescribed without direct reference to the target displacement ductility or the axial load ratio, and there is no equivalent to the closed-loop verification chain of NZS 3101. Murty, Goswami, Vijayanarayanan, and Mehta [89] have documented this gap and proposed draft provisions that would introduce an explicit curvature ductility verification analogous to that of NZS 3101, arguing that this change is urgently needed for walls in high seismic zones where axial load ratios from gravity loads alone can reach 0.25 to 0.30 in Indian residential high rises. Jain [90] has provided a broader context by comparing the seismic vulnerability of RC building stock in India with international benchmarks and identifying wall ductility capacity as one of the principal risk drivers for the existing building stock.

4.5. Comparative Summary

Table 2 below summarises the key ductility-related provisions of the four codes reviewed in this section, allowing the similarities and differences to be seen at a glance.

Table 2. Comparison of ductility-related provisions in major international seismic codes

Feature	EN 1998 1 [18]	ACI 318 19 [19]	NZS 3101 [21]	IS 13920 [16]
Ductility classification	DCL, DCM, DCH	Ordinary, Intermediate, Special	μ_Δ= 1.25, 3, 6	Ordinary, Ductile
Boundary element trigger	Strain-based ($\varepsilon_{cu} \geq$ 0.0035)	Stress or strain (revised 2019)	Curvature demand	Stress-based ($\sigma_c \geq 0.2f_{ck}$)
Target curvature ductility (highest class)	$\mu_\phi \geq$ 13	Implicit only	Explicit	Not specified
Confinement model used	Tabulated and formulae	ACI 318 Section 18	Mander et al.	Prescriptive spacing
Higher mode shear amplification	Partial (Cl 5.4.2.4)	Not required	Required (Blakeley et al.)	Not required
Out-of-plane stability check	Not explicit	New in ACI 318 2019	Under revision post 2011	Not explicit
Explicit drift limit	Yes (0.5 to 2.0%)	Implicit via R factor	Yes, DDBD-based	Limited

[Note: Notes: DCL/DCM/DCH = Ductility Class Low/Medium/High (EN 1998-1); μ_φ = curvature ductility ratio; μ_Δ = displacement ductility ratio; DDBD = Direct Displacement-Based Design]

5. A Unified Ductility Assessment Framework

5.1. Rationale and Structure

A comparison of experimental findings and code provisions reveals a key challenge in consistent ductility evaluation. Direct comparison between experimental results and code requirements remains difficult due to differences in definitions and assumptions. The objective of this section is to derive these relationships in closed form and to calibrate them against the experimental database of Table 1. The model is based on the pioneering work of Paulay and Priestley [14] and Priestley, Calvi, and Kowalsky [59] and builds upon it with the plastic hinge length correction equation suggested by Bohl [61] and Kazaz [43] at increased axial loads and the confined concrete model of Mander, Priestley, and Park [52] and the energy-based degradation correction of

5.2. Critical Section Curvature Ductility

The yield curvature of a continuous under combined axial load and bending is 0y = – and ultimate curvature 0u = - where is the ultimate compressive strain on the extreme fiber and cu is the neutral axis depth at ultimate. In confined concrete in the boundary element, the ratio is calculated using the Mander, Priestley, and Park [52] constitutive model, which depends on the volumetric ratio of confinement reinforcement, the yield strength of the confinement reinforcement, the fracture strain of the steel, and the confined compressive strength of the concrete. In the case of the commonly European or New Zealand practice yield strength, of 500 MPa, volumetric ratio rate of 1.5 percent, steel fracture strain of 0.10, and concrete strength of 30 MPa, the model approximated to 0.015, which is in agreement with the measures associated with the stated condition by Oesterle et al. [22,23]. The plastic hinge length itself is estimated using the neutral axis depth cu, which is obtained from section equilibrium under the given axial load N using the simplified rectangular stress block. For walls with axial load ratios below 0.20 and longitudinal steel ratios between 0.005 and 0.02, the normalised neutral axis depth ranges from approximately 0.10 to 0.20 of L_w [14]. The curvature ductility ratio then becomes:

$$\mu_\phi = \frac{\phi_u}{\phi_y} = \frac{\varepsilon_{cu} L_w}{2\varepsilon_y c_u} \tag{2}$$

This expression makes clear that curvature ductility increases with increasing confined concrete ultimate strain and decreases as the neutral axis depth grows, confirming that both improved confinement and reduced axial load enhance section-level ductility. For the walls tested by Thomsen and Wallace [28] with concrete strength 42 MPa, axial load ratio 0.09, longitudinal steel ratio 0.025, and confinement ratio 1.1 percent, this expression predicts μ_ϕ16.4, compared with the experimentally measured value of approximately 14 to 18. For the walls tested by Almeida, Prodan, Rosso, and Beyer [34] at higher axial loads, the expression overestimates $\mu_\phi$by up to 25 percent when strain localisation is pronounced, consistent with the observation of those authors that a localisation correction factor is needed for thin walls. The modified expression of Kazaz [43] incorporates an empirical localisation factor calibrated against a broad dataset and reduces this overestimation to below 12 percent for the cases covered by that study.

5.3. Rotation Ductility at the Plastic Hinge

For the lumped plasticity idealisation, the rotation ductility at the plastic hinge, defined as $\frac{\theta_u}{\theta_y}$, is numerically equal to the curvature ductility at the critical section. This is a direct consequence of the assumption that curvature is approximately uniform over the plastic hinge length, so both the yield and ultimate rotations scale with the same length. The plastic hinge rotation at ultimate is θ_u equal to L_p, which is the deformation parameter used directly in the ASCE 41 [20] acceptance criteria and which can be measured in wall tests as the slope of string potentiometer recordings over the plastic hinge zone.

The length of the plastic hinge itself is approximated by the expression of Paulay and Priestley [14], and the correction of Bohl [61] is used in cases of walls with axial load ratios exceeding 0.15. Plugging the Paulay and Priestley expression, using a typical slender wall, Hw/Lw 3.0, yield strength of 500 MPa, and bar diameter of 20 mm, Lp is approximated at 0.30 Hw. The Bohl correction decreases this by about a fifth of the axial load ratios of 0.20, which is more consistent with the experimental data of Dazio, Beyer, and Bachmann [31] and the more recent data of Chen et. al. [33].

5.4. Displacement Ductility at the System Level

Substituting the plastic hinge length into the fundamental relationship of Equation (1), the system displacement ductility is obtained as an explicit function of the section curvature ductility and the wall geometry. For walls with H_w/L_w equal to 2.0 and curvature ductility equal to 15, the framework gives displacement ductility of approximately 5.1. For the same section ductility, but H_w/L_w equal to 4.0, the displacement ductility falls to approximately 3.8. This counterintuitive result, that taller walls of the same cross section achieve lower displacement ductility, follows from the fact that the plastic hinge length becomes a smaller fraction of the wall height as the wall grows taller, and it is consistent with the experimental evidence of Table 1 for slender walls.

For squat walls where the plastic hinge model is not applicable, the displacement ductility is estimated from the empirical shear drift relationships of Gulec and Whittaker [57], which give a displacement ductility of approximately 2.2 for walls with H_w/ L_w equal to 0.5 and an axial load ratio of 0.05. This is consistent with the range of 1.5 to 3 reported in Table 1 for shear-governed walls. The framework of Hidalgo, Jordan, and Martinez [78] provides a complementary estimate for walls of intermediate aspect ratio, and the two approaches give results within 15 percent of each other for walls with H_w/L_w between 0.8 and 1.5, which is within the uncertainty of the underlying experimental data.

The framework also permits direct comparison of the ductility demands implicit in the four codes. For a Ductility Class High wall designed to EN 1998 1 [18] with behaviour factor of 4.0 and a spectral acceleration ratio of 2.5, the implicit displacement ductility demand is approximately 2.7. The corresponding required section curvature ductility, for H_w/L_w equal to 3.0, is approximately 9. The EN 1998 1 requirement of curvature ductility at least 13, therefore, provides a ductility margin of approximately 44 percent above the implicit demand, a safety factor that accounts for record-to-record variability and modelling uncertainty. An equivalent exercise for an NZS 3101 [21] wall designed for displacement ductility of 6 yields a required curvature ductility of approximately 18, which is consistent with the confinement reinforcement typically required by that code using the Mander, Priestley and Park [52] model.

5.5. Framework Validation

Table 3 compares the displacement ductility values predicted by the framework against selected experimental results from the database of Table 1. The prediction error is defined as the predicted value minus the measured value, expressed as a percentage of the measured value.

The framework systematically over-predicts displacement ductility by eight to fifteen percent for flexure-controlled slender walls and under-predicts by five to twelve percent for shear-controlled squat walls. The overprediction for slender walls reflects the limitation of the plastic hinge model in not capturing the reduction in effective L_p caused by strain localisation, as identified by Almeida, Prodan, Rosso, and Beyer [34] and Kazaz [43].

The underprediction for squat walls reflects the limitation of the empirical shear drift relationships in not fully capturing the influence of boundary reinforcement on the post-peak response. The energy-based correction of Park and Ang [76], as updated by Haselton, Liel, Deierlein, Dean, and Chou [79], provides a complementary assessment of cumulative ductility degradation that the peak-based framework does not capture and should be applied in assessments of buildings subject to near-fault or subduction zone ground motions.

Table 3. Validation of the unified framework: predicted versus experimental displacement ductility

Test programme	H_w/L_w	Axial load ratio n	Measured displacement ductility	Predicted displacement ductility	Error (percent)
Oesterle et al. [22,23]	2.4	0.07	6.2	7.1	+14.5
Thomsen and Wallace [28]	3.3	0.09	5.8	6.5	+12.1
Dazio et al. [31]	3.8	0.10	5.4	5.9	+9.3
Almeida et al. [34]	2.8	0.12	5.1	5.5	+7.8
Chen et al [33]	3.0	0.28	2.8	2.6	-7.1
Salonikios et al. [29,30]	1.5	0.15	3.6	3.4	- 5.6
Barda et al. [25]	0.5	0.04	2.1	1.9	-9.5
Rosso et al. [37]	3.0	0.16	4.2	4.6	+9.5

[Note: Error = (predicted − measured)/measured × 100%. Positive error = framework over-prediction; negative error = framework under-prediction. All predictions use the Bohl [61] axial load correction for n > 0.15.]

6. Discussion

6.1. Synthesis of Key Findings

The proposed framework can be used to support future research in displacement-based design and performance assessment of shear walls. It also provides a basis for comparing different design codes and improving consistency in ductility evaluation. Further studies may extend this framework to more complex wall systems and loading conditions. The experimental, analytical, and code-based evidence reviewed in this paper converges on several conclusions that are now firmly established and have been reinforced by the research of the last fifteen years. Aspect ratio plays a governing role in determining the ductility mode and capacity of shear walls. Slender walls can achieve displacement ductility of five to eight with proper detailing, while squat walls are limited to displacement ductility below three regardless of detailing quality [22, 23, 25, 27]. The axial load ratio is the most sensitive material geometric parameter. An increase of 0.10 in the axial load ratio reduces curvature ductility by thirty to fifty percent for typical wall proportions, a sensitivity that justifies the axial load limits of all major codes and that has been powerfully demonstrated by earthquake damage surveys following the 2010 and 2011 events [4 ,5, 8, 9]. Boundary element confinement, quantified through the volumetric ratio and yield strength of transverse reinforcement, is the key controllable variable governing the ultimate compressive strain and hence the curvature ductility [51, 52]. Inadequate confinement was the primary cause of non ductile failure in the foundational experimental programs [22, 23] and in real earthquake damage [4, 5, 8].

Two additional failure modes that have received considerably greater attention in the last decade deserve emphasis. Out-of-plane buckling of slender boundary elements, shown experimentally by Rosso, Almeida, and Beyer [37] and modeled analytically by Dashti, Dhakal, and Pampanin [44], can occur at drift levels within the range expected during design-level seismic events for walls with high axial loads and thin sections. The interaction between in-plane curvature demand and out-of-plane instability is not captured by any one-dimensional model and requires either three-dimensional NLFE analysis or the use of the simplified stability criteria proposed by Dashti, Dhakal, and Pampanin [44] and now incorporated in ACI 318 2019 [19]. Biaxial loading interaction, investigated by Wallace [49] and Qin et al. [50], can increase the effective compressive demand on boundary elements by twenty to forty percent relative to the uniaxial design assumption, a coupling effect that remains unaddressed in all four codes reviewed.

The unified framework of Section 5 demonstrates that the three ductility indices are not independent: given the material properties, geometry, and reinforcement details, curvature ductility, rotation ductility, and displacement ductility are fully determined by the chain of expressions derived in that section. This chain can be used in two directions. In design, one starts from a target displacement ductility and works back to determine the required curvature ductility and hence the necessary confinement reinforcement. In assessment, one starts from the as-built details and works forward to determine the available displacement ductility and compares it with the seismic demand. Priestley, Calvi, and Kowalsky [59] have provided the fullest exposition of this approach in the context of displacement-based design, and Moehle [84] has discussed its application in the context of ACI 318 special wall provisions. Future research may consider the implications of shear wall ductility on the safety and resilience of buildings in seismic regions. The results can also assist in improving the building codes as they will be developed and improved to become more performance-based in nature. These studies will help in creating more dependable and stable structural systems subjected to earthquake loading.

6.2. Shortcomings of the Framework.

The hinge model of plasticity on which the unified model is based is an idealisation, which increasingly loses its truth as the failure mode moves to a different pattern than the pattern described by a concentrated flexural hinge at the base. In the case of squat walls, the idea of a lumped plastic hinge is inherently inappropriate since shear deformation is spread all over the panel and cannot be modeled by a rotation at a given cross-section. In cases where the axial ratio of load is larger than 0.25, a uniform curvature over the plastic hinge length is assumed and thus overestimates the actual rotation capacity, as indicated by both the experimental results of Chen et al. [33] and the analytical work by Bohl [61]. In walls with large openings, discontinuities, or irregularly shaped plans, the effective location and length of the plastic hinge should not be determined by empirical formulae, but by NLFE analysis, as argued by Panagiotou, Restrepo, and Conte [39] and by Fischinger, Rejec, and Isakovic [70].

The cumulative damage effects of multi-cycle loading are also not included in the framework, but they decrease the effective ductility compared to that at the first cycle. Estimates by Park and Ang [76] put the effective ductility of the 1985 Mexico City ground motion at about seventy and eighty percent of the first cycle ductility of RC walls with moderate confinement. In near-fault ground motions that have pulse characteristics, the reduction may be more extreme, and Haselton, Liel, Deierlein, Dean, and Chou [79] suggest an explicit cumulative ductility test in performance-based evaluation in the described conditions. This framework does not examine the problem of interplay between axial load variation and lateral loading that takes place within walls of a coupled wall system, where the axial load reverses sign during the earthquake loading portion and has the potential to induce tension on the nominally compression-only walls. Qin et al [50] and Fox et al have discussed this issue. [91].

6.3. Code Harmonisation

Table 2 reveals that the four codes are broadly aligned on the physical mechanisms they seek to control but differ significantly in how they quantify and verify ductility. EN 1998 1 [18] and NZS 3101 [21] both compute an explicit target curvature ductility. ACI 318 [19] relies on prescriptive detailing without an explicit ductility chain. IS 13920 [16] lacks both the explicit target and the ductility chain. The unified framework of Section 5 could serve as a practical basis for harmonisation, because it provides the analytical relationships needed to translate the prescriptive rules of ACI 318 and IS 13920 into equivalent curvature ductility targets, and conversely to express the explicit curvature ductility targets of EN 1998 1 and NZS 3101 as equivalent detailing prescriptions. Murty, Goswami, Vijaya Narayanan, and Mehta [89] have illustrated this translation for IS 13920 and found that the implicit curvature ductility targets are significantly lower than those of EN 1998 1 for walls with axial load ratios above 0.15, suggesting that IS 13920 may be non-conservative for this class of wall.

The most practically significant discrepancy between the codes is in the treatment of higher-mode shear demand. NZS 3101 [21] requires amplification of design shear forces above the plastic hinge region, reflecting recommendations that trace back to Blakeley, Cooney, and Megget [86] and have since been reinforced by the shake table tests of Panagiotou, Restrepo, and Conte [39] and the analytical studies of

Rutenberg [87] and of Wallace [92]. Neither EN 1998 part 1 nor ACI 318 requires this amplification for wall-dominated systems, creating a vulnerability in tall wall buildings designed to these codes when they are subjected to ground motions with significantly higher mode content. Fardis, Carvalho, Fajfar, and Pecker [83] have argued for the introduction of explicit higher mode checks in the forthcoming revision of Eurocode 8, pointing to the consistent evidence from analytical studies and the Panagiotou et al. [39] shake table experiment.

6.4. Recommendations for Practice

Based on the full body of evidence reviewed in this paper, the following recommendations are offered for the design and assessment of RC shear walls in seismic regions. Axial load ratios should be limited to 0.20 or below for ductile walls wherever possible. Above this level, the curvature ductility capacity degrades rapidly, and the required confinement reinforcement increases steeply, making it difficult and expensive to achieve displacement ductility above three [31, 33]. Buildings that must carry high axial loads should use enlarged boundary elements with increased confinement reinforcement ratios, and the adequacy of the available curvature ductility should be verified using the Mander, Priestley, and Park [52] constitutive model with the modified plastic hinge length of Bohl [61].

Boundary element confinement should be designed using the Mander, Priestley, and Park [52] model to achieve a target ultimate compressive strain of at least 0.010. For thin walls or walls with axial load ratios above 0.20, the out-of-plane stability of the boundary element should be checked using the procedures of Dashti, Dhakal, and Pampanin [44], which are now reflected in ACI 318 2019 [19]. The maximum longitudinal bar spacing within special boundary elements should follow the more restrictive limits of ACI 318 2019 rather than the older limits still present in some national codes, based on the experimental findings of Moyer and Kowalsky [54].

Lap splices must be kept out of the potential plastic hinge zone, which extends at least Lw above the base of the wall. Splices placed within this zone under cyclic loading are susceptible to bond failure before the longitudinal steel reaches its yield strain, as documented experimentally by Paulay and Priestley [14] and confirmed by more recent studies, including those of Birely [32] and Tran and Wallace [35].

Squat walls in high seismic hazard regions should be provided with diagonal reinforcement crossing the base construction joint, following the recommendation of Paulay, Priestley, and Synge [26], to suppress sliding shear failure. The shear strength should be verified using the updated model of Gulec and Whittaker [57] rather than the earlier Wood [27] expression, which is less accurate for walls with web reinforcement ratios below 0.5 percent.

For performance-based assessment, the ASCE 41 [20] plastic rotation limits provide the most explicit and experimentally calibrated criteria currently available and are recommended as a benchmark for checking walls designed to any code, including those like IS 13920 [16] that do not internally provide such limits. Site cumulative ductility correction that has been found to be described by Haselton, Liel, Deierlein, Dean, and Chou [79] should be introduced in case of near-fault or subduction zone ground motion sites. Effects of biaxial loading of wall systems when orthogonal walls are placed side by side should be considered based on the method of Segura and Wallace [49]. To ensure the final verification of drift capacity, the empirically developed expressions of Abdullah and Wallace [100], whose expressions are calibrated using a detailed database of special tests of the boundary element wall, should be used to ensure that the component of the drift capacity at the operating displacement level is consistent with the level of detailing. Their research indicates that the best method of estimating the drift capacity is through the relationship between drift capacity and the depth ratio of the neutral axis and the aspect ratio of the wall. It also concludes that axially loaded walls with a ratio of axial load over neutral axis depth greater than 0.20, and neutral axis depth exceeding 20 percent of the wall length, are much more likely to fail to meet the minimum drift capacity that is presupposed in current codes. This concurs with the conservative limit of axial load suggested in this paper.

7. Conclusion

The paper is an integrated discussion of the ductility behavior of RC shear walls. The most important conclusions taken from this work are as follows.

RC shear walls have a displacement ductility of about 1.5 to 8, but the range greatly depends on the aspect ratio, the ratio of the axial load, and the quality of boundary element detailing. Walls with the narrowest boundary elements are the ones that have well-controlled flexure and attain maximum values. The inherent limitation of squat, shear-controlled walls is that they have a displacement ductility of less than three, no matter the amount of longitudinal reinforcement supplied. This finding, already determined by pioneering investigations in the 1970s and 1980s, has been regularly reaffirmed by the more recent experimental efforts, and by observations after the event of large-scale seismic events.

The most sensitive design parameter to ductility is the axial load ratio. A decrease in the ratio of the axial load to 0.10/0.20 results in an increase in the curvature ductility ratio of the critical section to approximately twice that with a 0.20/0.10 ratio. Axial load ratio walls (at the boundary elements) that exceed 0.25 are susceptible to brittle failure, even where the boundary elements are nominally constrained within code requirements. The extra axial load on boundary elements that occurs in a unidirectional earthquake loading, which can be twenty to forty percent greater than the uniaxial design value, is not taken into consideration at this stage of any of the four codes reviewed, and is a significant frontier in developing codes.

Boundary element confinement, measured by the volumetric ratio and yield strength of transverse reinforcement, is a direct factor of ultimate compressive strain and thus curvature ductility. All confinement design level predictions should be accomplished using the Mander, Priestley, and Park confined concrete model, which can be easily used to predict the correct confinement level on rectangular hoops. Out-of-plane stability of slender boundary elements, which are now treated in ACI 318 2019, also needs to be explicitly checked for conditions of thin walls or high-axial load walls, and is also an important refinement compared to earlier versions of the code, which did not include this check.

These three ductility indices, each being curvature ductility, rotation ductility, and displacement ductility, have a plastic hinge geometry associated with them in close-form relationships verified by a wide range of experimental data. The integrated framework obtained in this paper allows consistent cross-code comparison and offers a full chain of analysis between material properties, reinforcement particulars, and system-level deformation capacity. Expressions of modified plastic hinge lengths that explicitly consider axial load enhance the validity of the predicted displacement ductility of axial load ratios in a wall where both are above 0.15 and are better than the classical expression where they are.

The reviewed four codes (EN 1998 1), ACI 318, NZS 3101, and IS 13920 are broadly consistent on the approach to confinement of the boundary element and the limits on the axial loads, but have wide variations in their explicit treatment on the subject of demand on the ductility of the curvature, higher mode amplification of shear, and out-of-plane stability. IS 13920 is the least fully implemented ductility design chain, and could be materially enhanced by the introduction of explicit curvature ductility targets and verification procedures similar to those provided in NZS 3101. The new 2019 update of ACI 318 has filled many of the gaps, especially in the behaviour of thin-walled structures and the stability out of plane, and is much better than previous versions.

The same framework acknowledges the plastic hinge model as the sole constraint to non-slender walls, as well as multi-cycle loading. Future formulations to consider are coupled shear flexure, out-of-plane stability, correction of cyclic degradation of the framework to broaden the range of wall configurations, and seismic input features. The overall influence of the in-plane seismic demand, as well as the out-of-plane seismic demand, on the ductility capacity of the wall that has recently been identified as a key failure mode during major earthquakes is the most urgent gap between the level of research knowledge and what current international codes present.

The main implication of this review in the practice of structural engineering is that the ductility of RC shear walls is not one property, but it is a hierarchy of three related properties, i.e., curvature, rotation, and displacement ductility. All of which should be clearly checked on the right level of the design or assessment chain. It is done consistently with the analytical tools offered by the unified structure of Section 5. The next most important task in the case of practitioners to IS 13920 is to propose clearly defined curvature ductility testing of walls in which the ratio of the axial load exceeds 0.15; the framework indicates that such walls will tend to exhibit implicit ductility levels far below the goals of EN 1998-1 and NZS 3101 when subjected to the same level of seism To EN 1998-1 practitioners, the first action which is the most urgent is the implementation of higher mode shear amplification in tall wall buildings which exists in NZS 3101 but not at the current Eurocode—executive summary. The overarching framework offers the analytic foundation of both these advances, as well as a work tool to the research community and code-drafting organizations working in this area.

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