Examination of code performance limits for shear walls

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SUMMARY
Recently proposed changes to modeling and acceptance criteria in seismic regulations for both flexure and shear dominated reinforced concrete structural walls suggest that a comprehensive examination is required for improved limit state definitions and their corresponding values. This study utilizes a well calibrated modeling tool to investigate the deformation measures defined in terms of plastic rotations and local concrete and steel strains at the extreme fiber of rectangular structural walls. We compare requirements in ASCE/SEI 41, Eurocode 8 and the Turkish Seismic Code. This way, a critical evaluation of the requirements embedded in these documents becomes possible. It is concluded that the performance limits must be refined by introducing additional parameters. Significant recommendations are provided for Eurocode 8 and the Turkish Seismic Code.

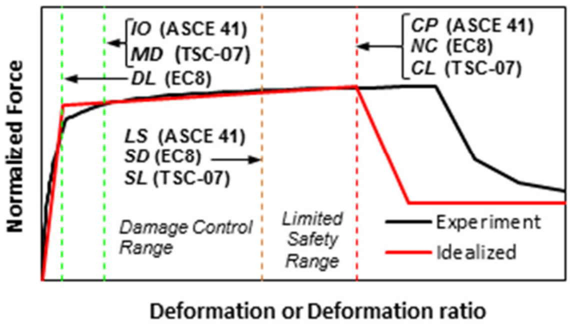
Keywords: Performance limit, plastic rotation, shear wall, codes, analytical response.

1. INTRODUCTION

Provisions for performance assessment of reinforced concrete structures, such as FEMA356 (2000), Eurocode 8 (EC8-3, 2005) and ASCE/SEI 41 (2006) include deformation limits for both flexure and shear controlled wall members at specific limit states to estimate the performance of components and structures. The criteria are defined in terms of plastic hinge rotations and total drift ratios for the governing behavior modes of flexure (ductile members) and shear (brittle members), respectively. Recently, strain limits are defined for concrete in compression and steel in tension at serviceability and damage-control limit states as a vital component of direct displacement-based design procedures (Priestley et al., 2007). The recently revised Turkish Seismic Code (TSC-07, 2007) specifies limiting strain values associated with different performance levels. On one side deformations are specified in relation to global parameters, and on the other side local damage indicators in terms of strain limits are used to determine the expected performance. For results of nonlinear pushover analyses to be evaluated according to either of the acceptance criteria, whether the local and global response will imply similar performance states is a matter that must be established. Another criticism raised against the rotations associated with different limit states is that they may turn out to be lower than the actual rotations expected to develop in reinforced concrete sections, so they are unduly conservative. In this study the adequacy of the limits specified by codes and guidelines is investigated. We employ nonlinear finite element analysis for reinforced concrete structural walls that has been thoroughly verified by the benchmark problems in Kazaz (2010). The absence of comprehensive experimental data due to limitations in the experimental setups and accuracy of the analytical procedures in predicting reinforced concrete local response under varying stress conditions because of the inherited inadequacy of plane section hypothesis for walls necessities such study.

2. CODE PERFORMANCE LIMITS

Fig.1 shows the conceptualized force versus deformation curve used in ASCE/SEI 41, TSC-07 and EC8-3 to specify member modeling and acceptance criteria for deformation-controlled actions. Three discrete Component Performance Levels and two intermediate Component Performance Ranges are defined as shown in Fig. 1 to identify the performance level of a member. As indicated in Fig. 1, at the Collapse Prevention level (CP) member deformation capacities are taken at ultimate strength or at lateral displacement demand at which capacity begins to rapidly degrade for primary components. At the Life Safety level (LS), member deformation capacities are reduced by a (safety) factor of 4/3 over those applying at Collapse Prevention. For the Immediate Occupancy (IO) two definitions arise in reference to Fig. 1. While ASCE/SEI 41 and TSC-07 anticipate some degree of nonlinear deformation beyond the global yield for the immediate occupancy level and minimum damage, respectively, EC8-3 adopts the global yield point as the limit state for the damage limitation on the member.



Abbreviations:

- **ASCE/SEI 41**
  IO: Immediate Occupancy
  LS: Life Safety
  CP: Collapse Prevention
- **Eurocode 8 (EC8)**
  DL: Damage Limitation
  SD: Significant Damage
  NC: Near Collapse
- **TSC-07**
  MD: Minimum Damage Limit
  SL: Safety Limit
  CL: Collapse Limit

Figure 1. Component performance levels

2.1. ASCE/SEI 41 performance limits

ASCE/SEI 41 basically adopts the same performance limits proposed in the wall provisions of FEMA 356 for the seismic assessment and rehabilitation of existing buildings. For walls deforming inelastically under lateral loading governed by flexure, the rotation (θ) over the plastic hinging region at the base of member is used. For shear walls whose inelastic response is controlled by shear, the deformation limits are expressed in terms of the lateral drift ratios. For multi-story shear walls the drift shall be the story drift. Table 1 gives the ASCE/SEI 41 plastic rotation limits for members controlled by flexure where P/P_o is the axial load ratio and v is the maximum average shear stress in the member normalized with respect to concrete compressive strength $\sqrt{f_c}$. Here V_{max} is the maximum shear force carried by the member. ASCE/SEI 41 (2006) adopts the ACI 318-02 (2002) requirements for the definition of a confined boundary.

Table 1. Plastic rotation limits for shear wall members controlled by flexure in ASCE/SEI 41

			Acceptable Plastic Hinge Rotation (radians)		
			Performance Level		
$\frac{(A_s - A_s')f_y + P}{t_w L_w f_c'}$	$v = \frac{V_{\max}}{t_w L_w \sqrt{f_c'}}$	Confined boundary	IO	LS	CP
≤ 0.10	$\leq 0.25\ (3)^*$	Yes	0.005	0.010	0.015
≤ 0.10	$\geq 0.50\ (6)^*$	Yes	0.004	0.008	0.010
≥ 0.25	$\leq 0.25\ (3)^*$	Yes	0.003	0.006	0.009
≥ 0.25	$\geq 0.50\ (6)^*$	Yes	0.0015	0.003	0.005

*The values in parentheses are in psi

2.2. Eurocode 8

The deformation capacity of beam-columns and walls is defined as the chord rotation θ, *i.e.*, the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span ($L_v = M/V$ = moment/shear), *i.e.*, the point of contra-flexure. The chord rotation is also equal to the element drift ratio, *i.e.*, the deflection at the end of the shear span divided by the length. The state of damage in a member is defined in EC8-3 by three Limit States.

Limit State of Near Collapse (NC): The value of the plastic part of the chord rotation capacity at ultimate θ_{um} of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um}{}^{pl} = \frac{1}{\gamma_{el}} 0.0145\left(0.25^{\nu}\right)\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.3} f_c^{\,0.2}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.275^{100\rho_d}\right) \tag{2.1}$$

where γ_{el} = 1.8 for primary elements and 1.0 for secondary elements, h = depth of cross-section, $\nu = N/bhf_c$ (b width of compression zone, N axial force positive for compression), ω and ω'= reinforcement ratio of the longitudinal tension (including the web reinforcement) and compression reinforcement, respectively, f_c is the concrete compressive strength (MPa), $\rho_{sx}=A_{sx}/s_h b_w$ = ratio of transverse steel parallel to the direction x of loading (s_h = stirrup spacing), ρ_d = steel ratio of diagonal reinforcement (if any), in each diagonal direction, α= confinement effectiveness factor. In walls the value given by Eqn. 2.1 is multiplied by 0.625.

Limit State of Significant Damage (SD): The chord rotation capacity corresponding to significant damage θ_{SD} may be assumed to be 75% of the ultimate chord rotation θ_{um} given by Eqn. 2.1.

Limit State of Damage Limitation (DL)*:* The deformation capacity for this limit is given by the chord rotation at yielding θ_y, evaluated for walls using the following equation

$$\theta_y = \phi_y \frac{L_v + \alpha_v z}{3} + 0.002\left(1 + 0.125\frac{h}{L_v}\right) + 0.13\phi_y \frac{d_b f_y}{\sqrt{f_c}} \tag{2.2}$$

where ϕ_y is the yield curvature and $\alpha_V z$ is the tension shift of the bending moment diagram, d_b is the (mean) diameter of the tension reinforcement, z is the internal lever arm, taken equal to $0.8L_w$ in walls with rectangular section, α_v should be set equal to 1 if shear cracking is expected to precede flexural yielding, otherwise α_v=0.0. The first term in the above expressions accounts for flexure, the second term for shear deformation and the third for anchorage slip of bars.

2.3. Turkish Seismic Code limit states

In a perplexing divergence from either of these two approaches the Turkish Seismic Code specifies strain limits to evaluate the performance of reinforced concrete members. Concrete and steel strain limits at the fibers of a cross section for minimum damage limit (MD) are given as

$$(\varepsilon_{cu})_{\mathrm{MD}} = 0.0035\ ; \qquad (\varepsilon_s)_{\mathrm{MD}} = 0.010 \tag{2.3}$$

Concrete and steel strain limits at the fibers of a cross section for safety limit (SL) are

$$(\varepsilon_{cg})_{\mathrm{SL}} = 0.004 + 0.0095\ (\rho_s/\rho_{sm}) \le 0.0135\ ; \qquad (\varepsilon_s)_{\mathrm{SL}} = 0.040 \tag{2.4}$$

and for collapse limit (CL) they are specified as

$$(\varepsilon_{cg})_{\mathrm{CL}} = 0.004 + 0.013\ (\rho_s/\rho_{sm}) \le 0.018\ ; \qquad (\varepsilon_s)_{\mathrm{CL}} = 0.060 \tag{2.5}$$

In Eqns. 2.3 to 2.5, ε_{cu} is the concrete strain at the outer fiber, ε_{cg} is the concrete strain at the outer fiber of the confined core, ε_s is the steel strain and (ρ_s/ρ_{sm}) is the ratio of existing confinement reinforcement at the section to the confinement required by the Code. The limits utilized in TSC-07 are mostly based on the studies and proposals of Priestley et al. (2007), where strain limits for tension and compression in relation to serviceability and damage-control limit states to be used in moment-curvature analysis are listed.

3. ANALYTICAL FRAMEWORK

Idealized cantilever models were used in the computations. Instead of an inverted triangular load distribution mimicking the first mode response, a point load applied at the effective height (~$2H_w/3$) was used as shown in Fig. 2a. The effective height in such case is also referred as shear span (L_v). Trial analysis of cantilever walls under monotonically increasing uniform and inverted triangular load patterns demonstrated that even when cracking may extend up to mid-height of the wall, significant steel yielding extends over only lower one or two stories. The upper stories can be effectively treated as a cracked beam. Using this analogy the finite element model displayed in Fig. 2b was developed in the general purpose finite element code ANSYS (2007) to reduce the computation time. The first two stories of the cantilever wall were discreticized with solid continuum elements whereas the upper stories are modeled with Timoshenko beam elements. The nonconformance between the nodal degrees of freedom of beam and solid elements was overcome by providing the transition with constraint elements. To define the behavior of beam elements generalized nonlinear section properties were used. The load deformation behavior of beam elements was assigned in the form of bilinear force-distortion angle (F-γ) and moment-curvature (M-ϕ) relation. The initial flexural rigidity was taken as $0.5EI_w$.

At the solid part of the model, the confined and unconfined stress-strain curves of the concrete are calculated with Saatcioğlu and Razvi (1992) model. Uniaxial behavior of longitudinal and transverse steels was modeled with a bilinear isotropic hardening using von Mises yield criterion. Modulus of elasticity of the steel material was taken as 200 000 MPa. The yield stress and tangent modulus at the strain hardening was taken as 420 MPa and 1500 MPa, respectively. Using a strain hardening stiffness helps achieving a better convergence. Buckling of longitudinal reinforcement in the compression boundary element is modeled according to Dhakal and Maekawa (2002). Further details of the modeling approach and verification test cases are given in Kazaz (2010).

The variables of the parametric study are summarized below. The parameters are

- Wall length (L_w): 3 m, 5 m and 8 m.
- Effective shear span (L_v): 5 m, 6 m, 9 m, 15 m, 24 m.
- Wall boundary element longitudinal reinforcement ratio (ρ_b): 0.5, 1, 2, 4 percent.
- Wall axial load ratio at the base ($P/f_c/A_w$): 0.02, 0.05, 0.1, 0.15, 0.5.

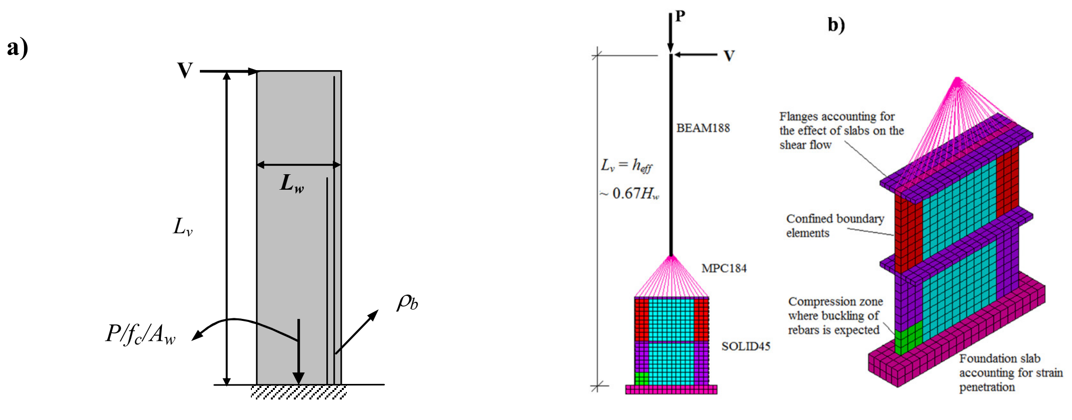


Figure 2. a) Illustration of variables of the parametric study; b) Finite element model.

The walls were designed according to TSC-07 specifications. Concrete strength was taken as 25 MPa for all cases. Wall boundary elements were assumed to extend over a region of $0.2L_w$ at the edges according to TSC-07. For any given combination of above parameters, such as wall length (L_w), ratio of boundary element longitudinal reinforcement area to the boundary region cross section area (ρ_b) and axial load ratio (P/P_o), the wall yield moment (M_y) is calculated. In the following step, using the specified shear span length (L_v) the design shear force was calculated ($V_d=M_y/ L_v$). The ratio of the horizontal and vertical web reinforcement is assumed to be nominally 0.0025. If the factored shear force ($V_e=\lambda V_d$) exceeds the shear safety limit calculated with $V_n=A_w(0.65f_{ctd}+\rho_t.f_{yd})$ according to TSC-07, the required amount of web horizontal reinforcement is recalculated employing the same equation. Since codes specify that the amount of vertical reinforcement should not be less than the horizontal reinforcement in the web, the same steel ratio of web reinforcement is used in the vertical direction. As transverse reinforcement ϕ8/100 mm is used at the boundaries. If the ACI 318-02 had governed the design, ϕ8 hoops at 85 mm spacing would have been required as confinement steel at the boundary elements. In conclusion wall boundaries can be considered as well confined for TSC-07 and adequately confined for ACI 318-02. Obviously confinement should be considered among the variables of the parametric study, but since this would increase the analysis permutations significantly, the study will be limited to confined members.

4. RESULTS OF ANALYSES

The method that is employed to evaluate the length of the plastic zone, L_{pz}, can be described as follows. Curvature profile computed from element strains calculated at the same height at the two wall ends are used to determine the spread of plasticity along the wall. The limiting yield curvature to determine the spread of plasticity along the wall was calculated with the expression $\phi_y=2\varepsilon_y/L_w$, where ε_y is the yield strain of the reinforcement (Priestley et al., 2007). The sketch in Fig. 3a illustrates the calculation of the base section curvature and rotation in a way that is consistent with ASCE/SEI 41. The rotations (θ_b), which are assumed to represent the rotation of the base section, are calculated just above the plastic zone length by using the vertical displacements calculated at tensile and compressive edges in the same row. The base curvature ϕ_b is calculated by fitting a best line to the curvature profile along the plastic zone length. The intercept of the best fit line equation is adopted as ϕ_b.

The response quantities of wall models are presented at the three damage levels given above namely, global yield, ultimate and an intermediate damage level, i.e. life safety, defined as the percentage of ultimate. In the analysis the ultimate point is determined on the basis of one of the criteria defined as the point on the load-deformation curve where strength drops abruptly or degrades to 85% of the ultimate strength ($V_{\max}$), or the steel strain at the tension side exceeds $\varepsilon_s = 0.1$, or the reinforcing bars at the compression side buckles (accompanied by significant crushing of concrete). On the other hand, the degrading effect of cyclic loading regimes on the stiffness and strength of reinforced concrete was not considered in the analyses carried out in this study. Vallenas et al. (1979) proposed that as a general rule the overall deformation capacity under a realistic ground motion could be expected to be over 75 percent of the deformation capacity under monotonic loading conditions. Typical load-deformation curve validating this assumption is obtained from analyses of the RW2 wall specimen tested by Thomsen and Wallace (1995) as displayed in Fig. 3b. 75 percent of the ultimate displacement capacity of the analytical model analyzed statically agrees quite well with the ultimate displacement capacity of experimental specimen tested under cyclic loading regime. In conclusion, it is assumed that collapse prevention performance level is taken at the 75 percent of the ultimate point of the finite element analyses in this study.

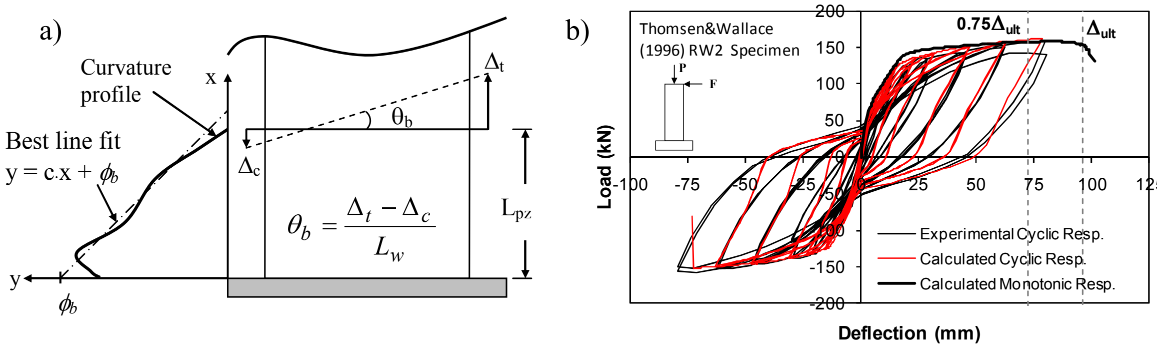


Figure 3. a) Schematic descriptions of base curvature and rotation calculation, b) Typical load-deformation curves of analyzed wall models

Figs. 4 to 6 display the comparison of the calculated plastic rotation limits with the limits specified in these documents. Data in each plot is also classified with respect to axial load ratio as well. Figs. 4(a-b-c) compare ASCE/SEI 41 limits with the calculated plastic rotation limits. At the immediate occupancy performance level the ASCE/SEI 41 limits yield conservative estimations for medium and high axial load ratios, but for low axial load ratios the limits are on the unsafe side. This situation contradicts expectations, yet it is the consequence of utilized procedure in the calculation of plastic rotations. Analysis results indicate that while yielding initiates at a concentrated region near the base of the wall under low axial load ratios, it has a distributed pattern in walls subjected to high axial load ratios. Consequently the plastic region length is larger in high axial load cases yielding larger plastic rotations. At the collapse prevention performance level, ASCE/SEI 41 limits are below 0.02 rad yielding conservative estimates. However, it appears that a cap has been applied for greater values. The capped data falls into the region characterized by low shear stress and flexural response. Since $\phi_y=2\varepsilon_y/L_w$ seems to yield good estimation of yield curvature the yield rotation according to ASCE/SEI 41 can be obtained as $\theta_y=(2\varepsilon_y/L_w)0.5L_w=0.0021$ rad assuming $L_p=0.5L_w$.

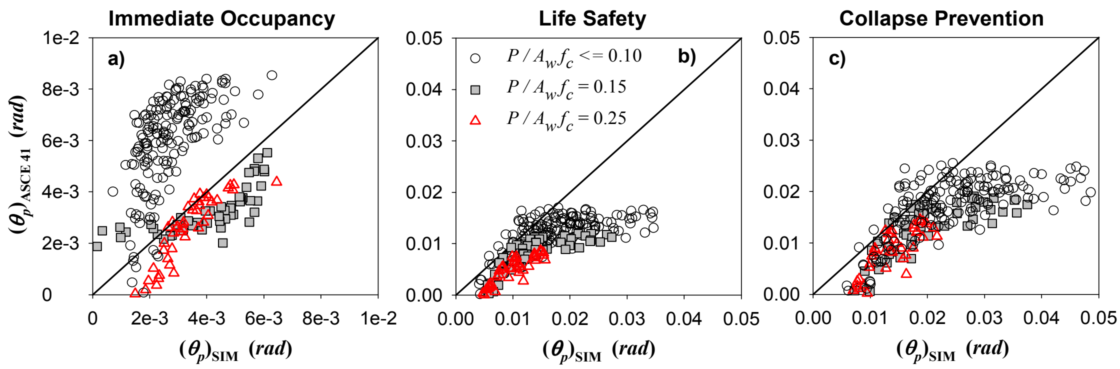


Figure 4. Comparison of calculated plastic rotation limits with the ASCE/SEI 41 limits

The rotation limits presented in EC8-3 that are defined in terms of chord rotation differ from the rotations advocated in this study. In this study it is intended to obtain rotation limits that account for the member deformation characteristics within a story. The chord rotation cannot be representative of base rotation of shear walls under higher mode effects and walls interacting with frames, especially with the strong ones. In order to be consistent with EC8-3 definitions, chord rotations were calculated and compared with EC8-3 limits in Fig. 5. The chord rotation is calculated as the tip drift ratio using the flexural displacement component. EC8-3 adopts the yield point as the damage limitation point. Fig. 5a displays the correlation of the yield rotation calculated using Eqn. 2.1 with the element drift ratio calculated at the tip (roof) of cantilever finite element model. The inference of Fig. 5a is that the yield rotation given by Eqn. 2.1 significantly overestimates the analysis results, especially as the shear-span-to-wall-length ratio increases. No pronounced effect of axial load ratio is observed on the yield rotation. The order of yield rotations calculated according to EC8-3 ranging from 0.003 to 0.009 rad is high for stiff shear wall elements. The yield rotation limits proposed by EC8-3 are unconservative for shear walls especially when the shear-span-to-wall-length ratio is high.

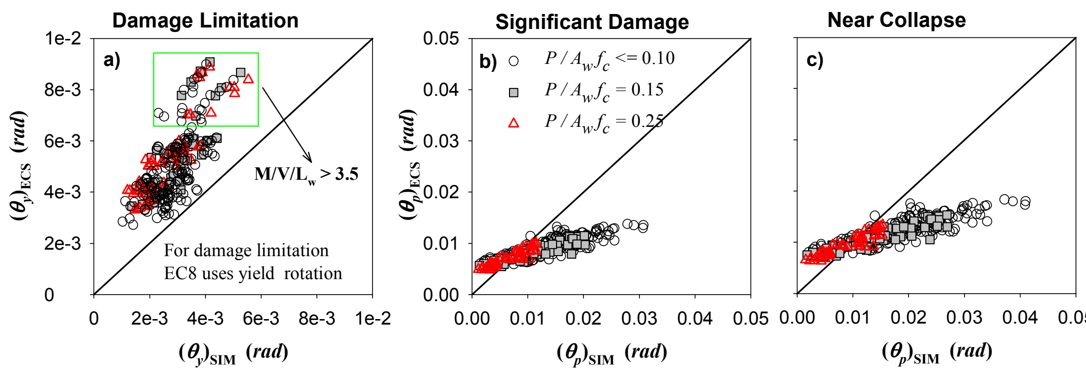


Figure 5. Comparison of calculated plastic rotation limits with the plastic chord rotation limits in EC8-3

Eqn. 2.2 proposed in EC8-3 used to calculate the ultimate limit state plastic rotation yields conservative, yet unrealistic limits as shown in Fig. 5c. The equations seem to be insensitive to the most of the design parameters, except the shear-span-to-wall-length ratio and axial load ratio. The resulting plastic rotation limits vary between 0.006 rad to 0.019 rad in average. The plastic rotation limits calculated according to EC8-3 are observed to be smaller than the ones given in ASCE/SEI 41, which is contrary to the expectations. EC8-3 defines chord rotation with respect to shear-span (M/V) as opposed to the plastic rotation in ASCE41 defined over the plastic hinge region. The life safety limits are calculated as ¾ of collapse prevention limits as shown in Fig. 5b.

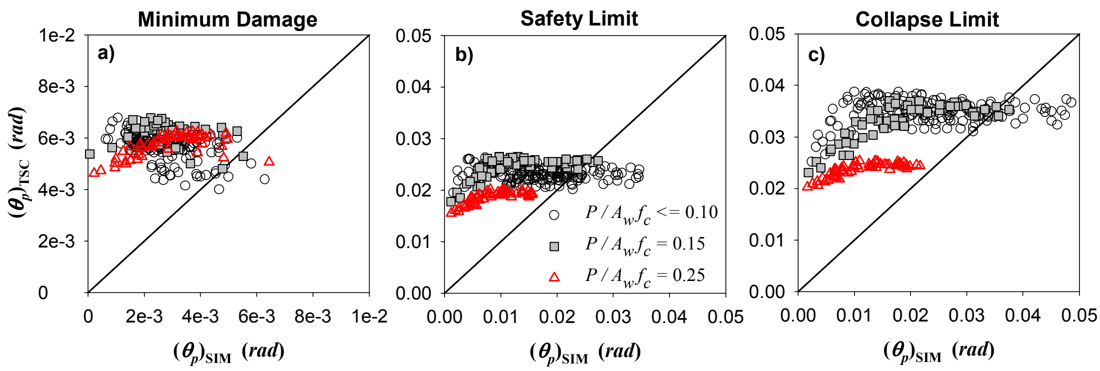


Figure 6. Comparison of calculated plastic rotation limits with the TSC-07 limits

Fig. 6 displays the correlation of plastic rotation limits calculated according to strain limits given in TSC-07 using section based moment-curvature analysis with the plastic rotations calculated from the analytical model. The plastic hinge length was taken as $0.5L_w$ as proposed by TSC-07 in the calculation of plastic rotations from the curvatures from section analyses. For all damage states (or performance levels) TSC-07 overestimates the rotation significantly without showing remarkable variation. The horizontally extending trend in Figs. 6(a-b-c) reflects that the given TSC-07 limits fall short of predicting the variation in the rotation due to varying design parameters. As discussed previously the same is also valid for EC8-3. The average limit curvatures in dimensionless form for serviceability ($\phi_s L_w$) and damage–control ($\phi_{dc} L_w$) states were proposed as 0.0175 and 0.072, respectively, by Priestly et al. (2007). The procedure implemented in TSC-07 for the seismic assessment results in rotations as θ_{MN}=$0.5\phi_s L_w$=0.00875 rad for minimum damage and $\theta_{GÇ}$=$0.5\phi_{dc} L_w$=0.036 rad for collapse limit. These limits are consistent with what is presented in Fig. 6, yet insufficient to calculate the actual rotations for a range of walls.

5. DISCUSSION

The analysis results suggest that the deformation capacity of structural walls with confined boundary elements is larger than the limits given in ASCE/SEI 41 provisions. It is seen that ASCE/SEI 41 yields conservative estimations of the structural performance. On the other hand, if the strain based performance criteria defined in TSC-07 or as suggested by Priestley et al. [4] is used in the determination of structural performance, unconservative estimations of performance are obtained for reinforced concrete rectangular walls. Equations given in EC8-3 for calculating performance based rotation limits lead to unconservative values at yield. The plastic rotation limits show insignificant variation and appear to be inadequate in estimating the finite element results. It is worth noting that similar forms of equations are used for beams, columns and walls despite significant differences in their behavior within a structure.

Fig. 7 displays the lower bound plastic rotation limits of the data obtained from analysis at different performance levels. In this figure, ASCE/SEI 41 limits are also displayed for comparison. As seen here, the major differences are for limits in the immediate occupancy level and at low and high shear stress ranges for the other two performance levels. ASCE/SEI 41 limits are specified at $0.33\sqrt{f_c}$ and

$0.50\sqrt{f_c}$, where intermediate values can be obtained by linear interpolation. However, outside these limits we have little basis about what the extrapolated trend may be. The proposed limits for the plastic rotations of walls controlled by flexure are tabulated in Table 2. These limits are given as alternative values to Table 6.18 of ASCE/SEI 41 for conforming members summarized in Table 1. The limits are derived as a function of normalized shear stress (ν) and axial load level (P/P_o) for different ranges of these variables in order to obtain more accurate representation of plastic rotation limits at the specified performance levels. Limits in relation to mid-range axial load levels ($P/P_o = 0.15$) are also introduced to increase the accuracy of the assessment procedure. In case the ASCE/SEI 41 format, i.e. numerical limit values at specific ν and P/P_o, the under lined number corresponds to the existing ASCE/SEI 41 limit and the left side number is the value proposed by this study.

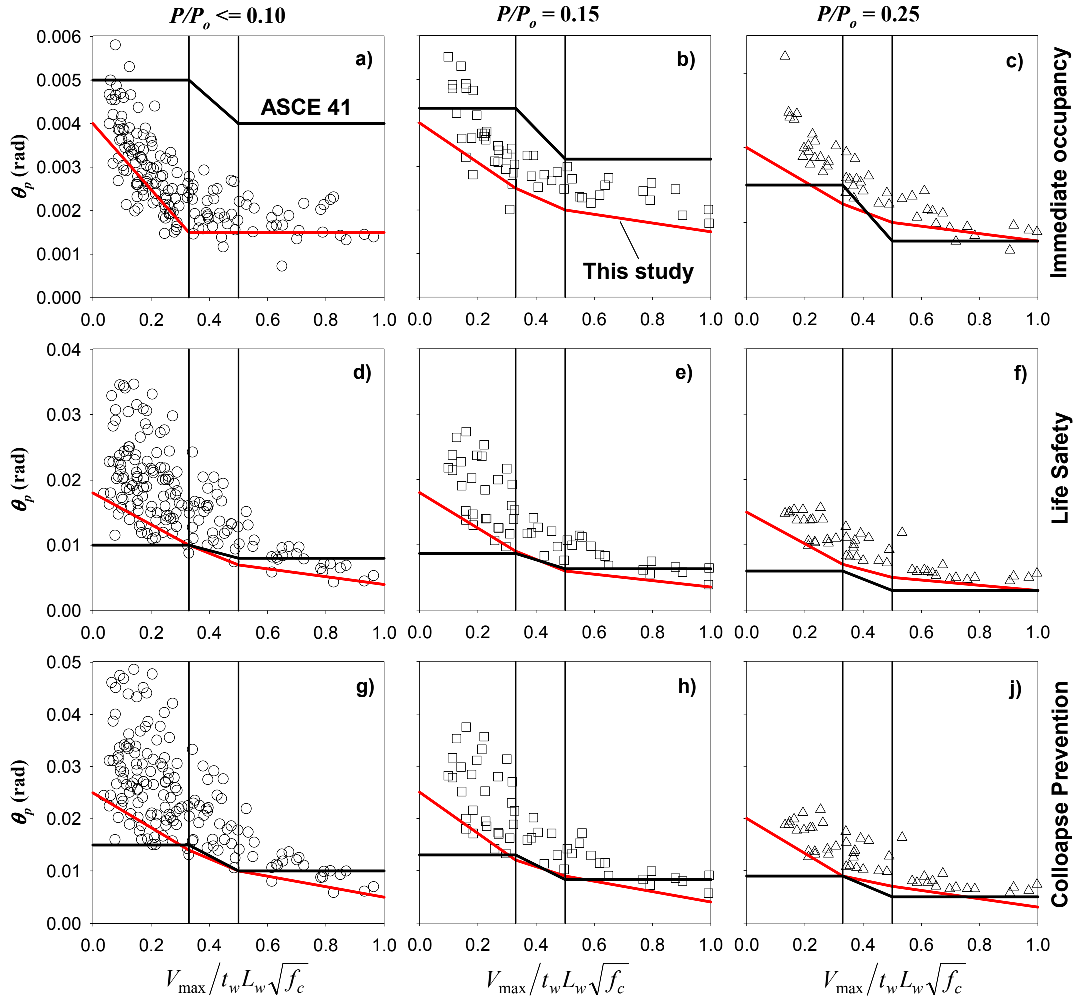


Figure 7. Lower bound plastic rotation limits at different performance levels

Alternative to limits proposed in Table 2, the lower bound plastic rotation limits can also be expressed with a more general exponential expression as given in Eqn. 7.1.

$$\theta_p = \begin{cases} (0.031 - 0.053P/P_o).e^{-1.75\nu} & if\ P/P_o \le 0.10 \\ (0.031 - 0.053P/P_o).e^{-(1.90-1.47P/P_o)\nu} & if\ P/P_o > 0.10 \end{cases} \tag{7.1}$$

As seen in Fig. 7 the proposed values correspond to the lower bound limits of the collective results. Using the parameters that govern the response of shear walls a general equation is derived to estimate the rotation capacity of shear walls with moderately confined boundary elements at ultimate. The equation from a regression analysis reads as

$$\theta_p = A(\rho_b)^B . e^{-(Cv+DL_w)} \mp \sigma(\theta_p) \quad where \quad \sigma(\theta_p) = \begin{cases} 0.1429\theta_p + 0.0005 & if\ \theta_p \le 0.014\,rad \\ 0.0025 & if\ 0.014\,rad < \theta_p \le 0.03\,rad \\ 0.8125\theta_p - 0.0219 & if\ 0.03\,rad < \theta_p \le 0.038\,rad \\ 0 & if\ 0.038\,rad < \theta_p \end{cases} \tag{7.2}$$

where A, B, C and D are coefficients defined in Table 3 as a function of axial load ratio. $\sigma(\theta_p)$ is the standard deviation of the calculated plastic rotation limits defined as a function of plastic rotation. The following set of equations can be employed in the calculation of $\sigma(\theta_p)$. The limit obtained through Eqn. 7.2 is greater than the limits given in Table 2. The predictions disregarding standard deviation are compared with the analytical results in Fig. 8. The predicted values agree quite well with the computational results. If the predicted values are reduced by 0.75 the limits for life safety performance level is obtained.

Table 2. The proposed plastic rotation limits for shear wall members controlled by flexure

		IO	LS	CP
$P/P_o \le 0.10$	$\nu < 0.33$ $\nu = 0.33$	0.005 - 0.0106 ν 0.0015 / 0.005§	0.018 - 0.0242 ν 0.01 / 0.01	0.025 - 0.0303 ν 0.015 / 0.015
	$0.33 < \nu < 0.50$	0.0015	0.0139 - 0.0118 ν	0.0247 - 0.0294 ν
	$\nu > 0.50$ $\nu = 0.50$	0.0015 0.0015 / 0.004	0.012 - 0.008 ν 0.008 / 0.008	0.015 - 0.01 ν 0.01 / 0.01
$P/P_o = 0.15$	$\nu < 0.33$ $\nu = 0.33$	0.005 - 0.0045 ν 0.0025 / 0.0043	0.018 - 0.0273 ν 0.009 / 0.0087	0.025 - 0.0394 ν 0.012 / 0.013
	$0.33 < \nu < 0.50$	0.0035 - 0.0029 ν	0.0148 - 0.0176 ν	0.016 - 0.012 ν
	$\nu > 0.50$ $\nu = 0.50$	0.0025 - 0.001 ν 0.002 / 0.0032	0.0085 - 0.005 ν (0.006 / 0.0063	0.016 - 0.012 ν 0.01 / 0.0083
$P/P_o \ge 0.25$	$\nu < 0.33$ $\nu = 0.33$	0.004 - 0.0045 ν 0.0025 / 0.003	0.015 - 0.0242 ν 0.007 / 0.006	0.02 - 0.0333 ν 0.009 / 0.009
	$0.33 < \nu < 0.50$	0.0035 - 0.0029 ν	0.0118 - 0.0109 ν	0.012 - 0.009 ν
	$\nu > 0.50$ $\nu = 0.50$	0.0025 - 0.001 ν 0.002 / 0.0015	0.007 – 0.004 ν 0.005 / 0.003	0.012 – 0.009 ν 0.0075 / 0.005

§(This study / ASCE/SEI 41) (underlined values corresponds to ASCE/SEI 41 limits)

Table 3. Coefficients of Eqn. 7.2 to calculate the ultimate plastic rotation limit of structural walls

P/P_o	A	B	C	D
≤ 0.10	0.138	0.220	1.814	0.071
= 0.15	0.087	0.148	1.779	0.066
= 0.25	0.034	0.037	1.485	0.037

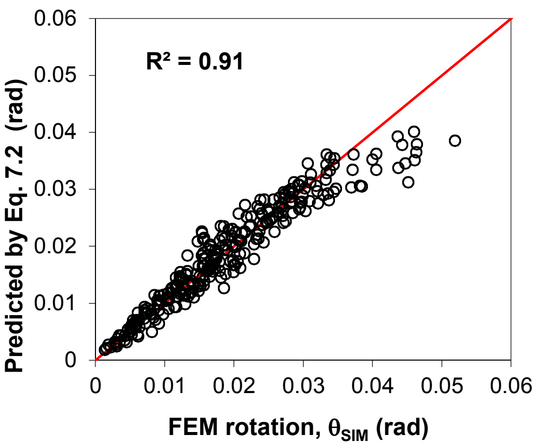


Figure 8. Correlation of predicted plastic rotations with analysis results

6. CONCLUSIONS

Nonlinear static or response history analysis procedures are the tools with which the deformation response of structural components are estimated in displacement based procedures. Regardless of the method of analysis employed, local and global quantities in terms of internal forces and deformations form the basis of judgment. These are then used to assess performance of structural assemblies. The most challenging part of the displacement based assessment procedures is the determination of the deformation limits that strongly influences the results. Therefore, the primary objective of the study carried out was to evaluate the limits recommended by the most widely used codes and guidelines. The results of this study that were obtained from comprehensive parametric analyses of the walls provide the data to adequately test performance based deformation limits specified in different documents. Among the documents evaluated, ASCE/SEI 41 limits were observed to be the most accurate ones yielding conservative results at all levels except the low axial load levels. It was shown that neither EC8-3 nor TSC-07 specifies adequately consistent deformation limits. TSC-07 suggests unconservative limits at all performance levels, and it appears to fall short of capturing the variation reflected in the calculated values. Likewise EC8-3 seems to fall short of representing the variation with unconservative estimations at life safety and collapse prevention levels.

This study primarily focused on the plastic hinge rotation limits in three different seismic codes. Accurate mathematical expressions defining the lateral drift, base curvature and rotation limits at yield and ultimate damage states can be found in Kazaz et al. (2012).

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