COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

DEVELOPMENT OF A SIMPLIFIED DISPLACEMENT-BASED PROCEDURE FOR THE SEISMIC ASSESSMENT OF RC WALL BUILDINGS

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Keywords: Simplified assessment, RC walls, low-moderate seismicity, displacement-based assessment

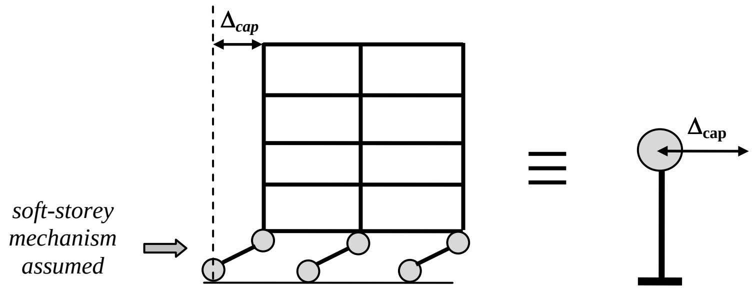
Abstract. *In regions of low or moderate seismicity there may be cases in which a rapid, simplified and conservative seismic assessment may be sufficient to demonstrate that a building satisfies certain seismic risk requirements. In recognition of this, a simplified displacement-based seismic assessment procedure, initially formulated for RC frame structures, is formulated to permit the rapid seismic assessment of reinforced concrete (RC) wall buildings. The key aspect of the procedure is the simplified evaluation of the displacement demand as the maximum spectral displacement across all periods. The displacement capacity, shear capacity and shear demand are also estimated simply, using newly developed equations that are a function of wall geometry and material properties. By adopting such formulations for the displacement and shear, the need to evaluate the period of vibration, stiffness and flexural strength of the walls is eliminated. The proposed approach is evaluated through the design and assessment of several case study buildings. Although the procedure is likely to be conservative in most cases, it is foreseen that it would be used in an initial screening process whereby a pass would require no further assessment and a fail would trigger a more detailed investigation.*

1 INTRODUCTION

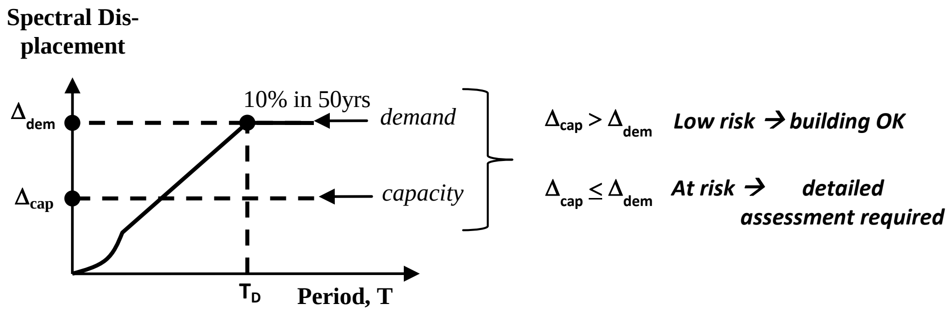
Seismic assessment of a building is traditionally aimed at establishing whether the building satisfies a minimum level of acceptable risk, which is often measured by considering whether code-specified levels of seismic intensity can be sustained. Rigorous seismic assessment requires identification of the structural system, estimation of material properties, and examination of a range of possible failure mechanisms, with an engineer undertaking careful analyses and calculations to quantify the likely capacity and compare this with expected demands. To this extent, a number of guidelines and proposals are available in the literature [1, 2, 3] for the detailed seismic assessment of buildings. In regions of low or moderate seismicity, however, there may be cases in which a rapid, simplified and conservative seismic assessment may be sufficient to demonstrate that a building satisfies minimum seismic risk requirements. In addition, regional building authorities may be interested to assess, with limited resources, groups of buildings in order to identify cases that should be subject to more rigorous seismic assessment and possibly retrofit. In recognition of this, a simplified displacement-based seismic assessment (DBA) procedure has been proposed by Pinho *et al.* [4] and recently developed by Piazza and Sullivan [5] to permit the rapid seismic assessment of RC frame buildings.

Figure 1 illustrates the basis of the simplified procedure proposed by Pinho *et al.* [5]. The approach compares a conservative estimate of the displacement demand, taken equal to the peak spectral displacement demand as shown in Figure 1a, with a conservative estimate of displacement capacity, estimated assuming a column-sway mechanism in the case of RC frame buildings. By assuming a column-sway mechanism the approach intends to achieve a conservative estimate of displacement capacity, and if this capacity is greater that the peak spectral displacement demand, the seismic risk is deemed to be acceptably low. Inputs required to apply the approach are limited to the geometry of the building (member dimensions and the number of storeys) and an estimate of material properties. Note that estimates of the mass and stiffness are not required owing to the fact that the procedure conservatively takes the peak spectral displacement demand, independent of the building period, and nor are estimates of strength required since performance is gauged solely via deformation considerations. The approach has been tested by Piazza and Sullivan [5] for a range of RC frame buildings and was found to perform reasonably well. As such, the objective of this paper is to extend the approach to permit the simplified displacement-based seismic assessment of RC wall buildings. To achieve this, a number of simplified equations, useful for the estimation of displacement capacity, shear capacity and shear demand will first be derived. Subsequently, the approach will be applied to a series of case study wall buildings and results compared with those obtained from more rigorous assessment procedures.

In order to extend the simplified DBA procedure to the case of RC wall buildings, it will be assumed in this work that (i) walls are continuous, with limited openings over the full height of the building; (ii) walls possess aspect ratios (wall height divided by length) greater than 2.5; (iii) walls are at least 200mm thick (this is intended to provide some protection against the likelihood of wall buckling, even though it is recognized [6] that the wall thickness alone is not sufficient to ensure this); (iv) the building possesses stiff, strong foundations, (v) the building is relatively regular in-plan and elevation, and (vi) the building is no more than 20 storeys high and is in reasonable condition (e.g. no signs of concrete cover spalling due to reinforcement corrosion). The guidelines developed in the subsequent sections are not considered applicable to cases where these conditions are not met.



(a) Assessment of displacement capacity for an assumed mechanism (left) & transformation into an equivalent SDOF system displacement capacity (right)



(b) Demand-capacity ratio identified from displacement spectrum

Figure 1. Overview of the displacement-based assessment procedure of Pinho *et al.* [4].

2 SIMPLIFIED EXPRESSIONS FOR THE DISPLACEMENT CAPACITY OF RC WALL SYSTEMS

Slender RC walls can typically be expected to yield in flexure, with a plastic deformation component that will be a function of available confinement reinforcement and the tensile strain capacity of the longitudinal reinforcement. This section develops a simplified approach to estimating the plastic deformation capacity of RC walls, which can then be added to the yield displacement to get the total displacement capacity. It should be kept in mind though that lessons learnt from recent earthquakes have demonstrated [6] that there are a range of other phenomena that can affect the failure mechanism in RC wall structures and this underlines that there will always be considerable uncertainty in any simplified estimate of displacement capacity.

2.1 Simplified estimation of the deformation capacity of RC walls yielding in flexure

The flexural displacement capacity, Δ_{cap}, of a RC wall can be estimated [7] as the sum of the displacement at yield, Δ_y, with a plastic displacement component, Δ_p, as shown in Figure 2 and in Equation 1:

$$\Delta_{cap} = \Delta_y + \Delta_p = \Delta_y + \theta_p f_h H_n \tag{1}$$

The right side of above equation also proposes that the plastic displacement component can be computed as the plastic rotation capacity, θ_p, of a plastic hinge at the base of the wall, multiplied by the total height of the building, H_n, and an effective height factor, f_h, used to relate the total height to an equivalent SDOF system height, noting that the displacement capacity provided by Equation 1 corresponds to an equivalent SDOF system representation of the wall

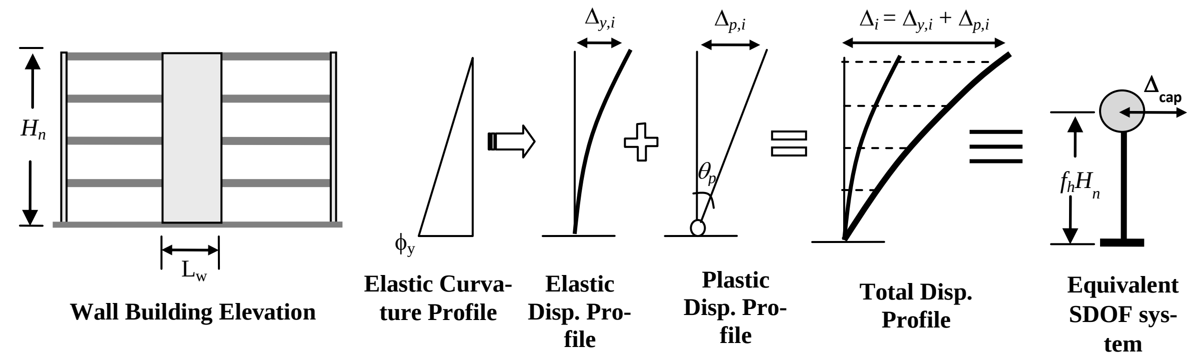


Figure 2. Displacement profile of a cantilever RC wall responding with a flexural plastic hinge (adapted from [8]).

system. In line with the recommendations of [8], the effective height factor can be approximated well by the following expression:

$$f_h = 0.7 + \frac{\sqrt{n} - 0.7}{n^2} \tag{2}$$

where n is the number of storeys in the building.

Subsequently, the flexural yield displacement of a RC wall can be computed using the following expression (adapted from the work of Priestley *et al.* [7]):

$$\Delta_y = \frac{\phi_y \left(f_h H_n \right)^2}{2} \left(1 - \frac{f_h}{3} \right) \tag{3}$$

where ϕ_y is the nominal yield curvature for the wall, obtained using the following expression for rectangular wall sections (similar expressions are provided for U-shaped walls by Beyer *et al*. [9] and T-shaped walls by Smyrou *et al.* [10]):

$$\phi_y = 2.0 \frac{\varepsilon_y}{L_w} \tag{4}$$

where ε_y is the yield strain of the wall's longitudinal reinforcement, obtained by dividing an estimated reinforcement yield strength, f_y, by the Young's modulus for steel, E_s, and where L_w is the length of the wall (also shown in Figure 2).

For what regards the plastic deformation capacity, part 3 of Eurocode 8 (EC8) [11] states that the plastic rotation capacity of an existing RC wall can be estimated using the following expression:

$$\theta_p = \frac{0.6}{\gamma_{el}} 0.0145 \cdot (0.25^{v}) \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega)} \right]^{0.3} f_c^{0.2} \cdot \left(\frac{L_v}{L_w} \right)^{0.35} 25^{\left(\alpha \rho_{sx} \frac{f_{yw}}{f_c} \right)} (1.275^{100\rho_d}) \tag{5}$$

where γ_{el} is equal to 1.8, v is the axial load ratio, ω' and ω are, respectively, the mechanical reinforcement ratios of the compression and tension longitudinal reinforcement, L_v is the shear span (approximated as the base moment divided by the base shear according to EC8 part

3), L_w is the length of the wall, f'_c and f_y are the mean concrete compressive strength (MPa) and the stirrup yield strength (MPa), respectively, ρ_{sx} is the transverse reinforcement ratio parallel to the direction x of loading, ρ_d is the diagonal reinforcement ratio (if present) and α is the confinement effectiveness factor.

In order to apply the expression it is evident that a considerable amount of information is required; information that is not likely to be available to someone interested in a rapid seismic assessment. As such, in order to obtain a simplified estimate of the plastic rotation capacity, a number of values are assumed for various parameters in order to provide what is considered a reasonable, albeit conservative, estimate of the plastic deformation capacity. In particular, it is assumed that f'_c=25MPa, ω'/ω = 1.0, v = 0.25, α =0.0, ρ_d =0, and that the shear span can be computed as the effective eight which leads to the following expression:

$$\theta_p = 0.006\left(\frac{f_h H_n}{L_w}\right)^{0.35} \tag{6}$$

where all the symbols have been defined earlier.

At this point, it is clear that one can obtain a quick estimate of the flexural deformation capacity of a RC wall by summing the yield displacement from Equation 3 with a plastic displacement component obtained by multiplying the plastic rotation capacity of Equation 6 by the effective height (f_hH_n).

In order to gauge the performance of this simplified approach, displacement capacity estimates obtained using the equations above have been compared with experimental values of the displacement capacity of walls, as reported in the literature. Table 1 lists the experimental test specimens examined together the with reference details from which additional information on the experimental testing could be obtained. Note that while some of the wall specimens listed in in Table 1 actually failed in shear, their displacement capacity values are still of interest here and as such, Figure 3 shows the ratio of the experimentally observed to predicted values of displacement capacity for all specimens.

From the results seen in Figure 3 it is apparent that the new expressions perform reasonably well. In some cases the expressions appear very conservative, and this was expected for walls with good confinement. There are four cases in which the observed displacement capacity is less than the estimated value, but considering the simplicity of the approach and the fact that it is for the large majority of cases conservative, the results above are deemed acceptable.

2.2 Accounting for *P*-Δ effects

In cases where the displacement demand is high and the ratio of strength to system weight is low, P-Δ effects may play a significant role in seismic response. To account for P-Δ effects it is proposed that an upper limit is placed on Δ_{cap}, based on an allowable stability index of 0.3.

As localised stability is unlikely to be an issue in an RC wall system it is assumed that the stability index can calculated based on an equivalent single-degree-of-freedom (SDOF) system as per Equation 7:

$$\theta_{P-\Delta} = \frac{m_e g}{V_{base}} \frac{\Delta}{H_e} \tag{7}$$

where m_e and H_e are respectively the effective mass and effective height of an equivalent SDOF system, and V_{base} is the base shear strength of the system. Assuming now that the base shear strength is 10% of the effective weight and taking H_e as equal to $f_h H_n$, an upper limit for the displacement capacity is obtained:

$$\Delta_{cap} < 0.03 f_h H_n \tag{8}$$

TEST NUMBER	SPECIMEN	REFERENCE
1	R1	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
2	R2	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
3	B1	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
4	B3	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
5	B2	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
6	B5	Oesterle, R.G., *et al*, (1976). "Earthquake Resistant Structural Walls-Tests of Isolated Walls"
7	B6	Oesterle, R.G., *et al*, (1979). "Earthquake Resistant Structural Walls-Tests of isolated Walls-Phase II"
8	B7	Oesterle, R.G., *et al*, (1979). "Earthquake Resistant Structural Walls-Tests of isolated Walls-Phase II"
9	B8	Oesterle, R.G., *et al*, (1979). "Earthquake Resistant Structural Walls-Tests of isolated Walls-Phase II"
10	B9	Oesterle, R.G., *et al*, (1979). "Earthquake Resistant Structural Walls-Tests of isolated Walls-Phase II"
11	B10	Oesterle, R.G., *et al*, (1979). "Earthquake Resistant Structural Walls-Tests of isolated Walls-Phase II"
12	W1	Su, R.K.L., Wong, S.M., (2007). "Seismic behavior of slender reinforced concrete shear walls under high axial load ratio"
13	W2	Su, R.K.L., Wong, S.M., (2007). "Seismic behavior of slender reinforced concrete shear walls under high axial load ratio"
14	W3	Su, R.K.L., Wong, S.M., (2007). "Seismic behavior of slender reinforced concrete shear walls under high axial load ratio"
15	RW-A20-P10-S63	Tran, T.A, Wallace, J.W., (2012). "Experimental Study of Nonlinear Flexural and Shear Deformation of Reinforced Concrete Structural Walls",
16	SHW1	Tasnimi, A.A., (2000). "Strength and deformation of mid-rise shear walls under load reversal"
17	SHW2	Tasnimi, A.A., (2000). "Strength and deformation of mid-rise shear walls under load reversal"
18	SHW3	Tasnimi, A.A., (2000). "Strength and deformation of mid-rise shear walls under load reversal"
19	SHW4	Tasnimi, A.A., (2000). "Strength and deformation of mid-rise shear walls under load reversal"
20	SW1-1	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
21	SW1-3	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
22	SW1-4	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
23	SW2-3	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
24	SW4-1	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
25	SW4-2	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
26	SW4-3	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"
27	SW5-1	Zhang, H., (2008). "Database on Static Tests of Structural Members and Joint Assemblies"

Table 1. List of RC wall test specimens used to compare experimental and analytical values of displacement capacity.

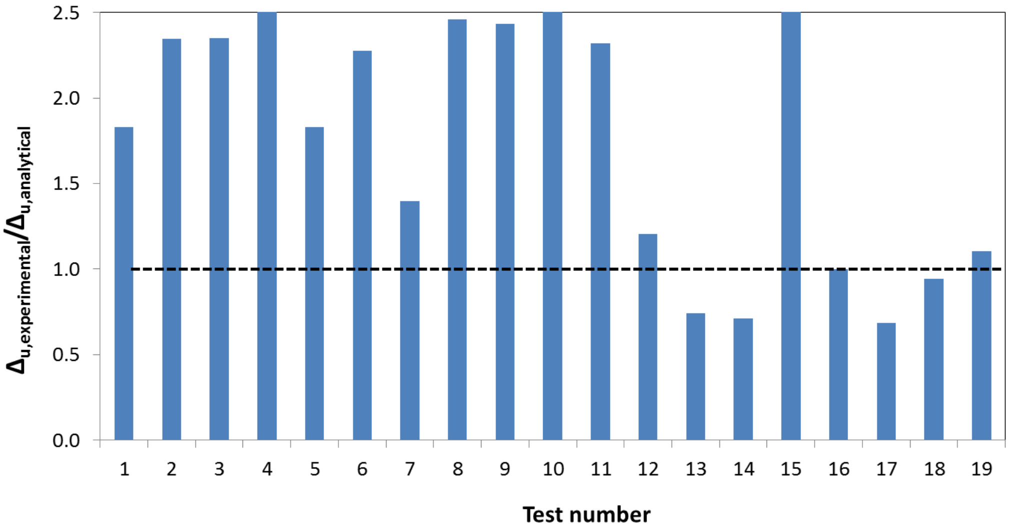


Figure 3. Ratio of the experimentally observed ultimate displacement capacity to the displacement capacity estimated using the simplified analytical approach.

3 SIMPLIFIED EXPRESSIONS FOR SHEAR CAPACITY AND DEMAND

Even if a RC wall building is evaluated and shown to have sufficient displacement capacity there is still the possibility the other failure modes may occur. The most likely of these is shear failure and therefore an additional check is required. The following section develops simplified expressions for estimating both shear capacity and shear demand. It is assumed that the critical region is most likely to be at the base of the wall and therefore the check of shear capacity versus demand is conducted for this location only.

3.1 Simplified estimation of the shear capacity of RC walls

In order to estimate the likelihood of shear failure in RC walls, a simplified estimate is first required of the likely shear resistance. The shear capacity of a RC wall, V_{cap}, is typically estimated in seismic codes by summing a shear contribution, V_c, associated with the concrete tensile strength, together with a contribution, V_s, offered by transverse reinforcing steel, as shown below in Equation 9. Priestley *et al.* [7] explain that the shear resistance of a wall (or column) will also tend to be increased by the axial compression force, but this component is conservatively ignored in formulating a simplified assessment procedure.

$$V_{cap} = V_c + V_s \tag{9}$$

Priestley *et al.* [7] point out that the concrete component of shear resistance tends to degrade as curvature ductility demands increase, in accordance with the modified UCSD shear resistance model. In this work, the concrete shear resistance component of RC walls as predicted by the modified UCSD model is simplified to Equation 10. These simplifications, aimed at avoiding the need for knowledge of reinforcement quantities or material properties, have assumed a concrete compressive strength, f'_c, of 25MPa, and a longitudinal reinforcement content of 0.5% (chosen to be conservatively low). It has further been assumed that the aspect ratio of the critical walls is greater than or equal to 2.0. To avoid the need to calculate the curvature ductility of the wall, it is assumed that the value of γ in the UCSD model is equal to 0.25. This assumption assumes a moderate curvature ductility and therefore will result in overly conservative estimates of shear capacity for walls responding elastically.

$$V_c = 700 L_w t_w \tag{10}$$

where V_c is in units of kilonewtons when the wall length, L_w, and the wall thickness, t_w, have units of meters.

In order to arrive at a simplified estimate of the shear resistance offered by the presence of transverse reinforcement, it is assumed that a typical RC wall section will have at least two transverse reinforcing bars of 10mm diameter every 300mm. In addition, a reinforcement yield strength of f_y=300MPa, a concrete compression zone of approximately 0.2 times the wall length and an inclined strut angle of 30° are assumed appropriate for simplified assessment purposes. Using these values, the traditional expression for transverse reinforcement component of resistance, V_s, simplifies down to:

$$V_s = 220 L_w \tag{11}$$

where V_s is in kilonewtons when the wall length has units of meters.

By making the above simplifications, it is evident that the combination of Equations 10 and 11 permit the total shear resistance of a wall to be estimated with knowledge only of the wall section dimensions, as shown:

$$V_{cap} = 700 L_w t_w + 220 L_w \tag{12}$$

where V_{cap} is in kilonewtons when the wall length and the wall thickness have units of meters.

3.2 Simplified estimation of the shear demands on RC walls

Having obtained an estimate of shear capacity, it is then necessary to obtain an estimate of shear demand. Shear demands in RC walls are likely to be a function of the seismic intensity, the flexural strength of the wall (for cases in which the earthquake intensity is sufficient to make the walls yield in flexure), the periods of vibration of the wall (recognizing that higher modes will contribute significantly to shear demands in multi-storey RC wall buildings) and the ductility demand (or better, the spectral intensity and spectral shape) as explained by Priestley *et al.* [7], Sullivan *et al.* [12] and Sullivan [13] amongst others, who also demonstrated that good estimates of peak shear demands can be obtained using modified modal analysis approaches. Building on the proposal of Pennucci *et al.* [14], Fox *et al.* [15] recently demonstrated that for coupled RC wall buildings, simplified closed-form solutions for total shear demands could be obtained by using a uniform cantilever beam analogy for the walls together with conservative assumptions regarding where the higher mode periods lie in relation to the spectral acceleration plateau. Adapting this approach for simplified assessment purposes here, the total shear demand at the base of a wall can be estimated as:

$$V_d = \sqrt{\left(\frac{3000 \cdot t_w L_w^2}{f_h H_n}\right)^2 + 40\left(H_n A_{f,total} \cdot \frac{t_w L_w}{\sum_{i=1}^{N} A_{w,i}} \frac{\Delta_{dem}}{T_D}\right)^2} \tag{13}$$

where $A_{w,i}$ is the sectional area of wall i and N represents the total number of walls at the ground floor level, t_w is the thickness and L_w the length of the longest wall in the direction of loading under consideration and $A_{f,total}$ the total floor area in the building (such that $H_n A_{f,total}$ gives the building volume), Δ_{dem} is the spectral displacement demand at the corner period, T_D, (which is also taken as the displacement demand, see Figure 1) and the other parameters have been defined earlier. This equation has been formulated assuming that first mode shear demands can be related to the wall flexural strength (which has been approximated using the wall dimensions together with estimated values of reinforcement content and axial load ratio) and that higher mode shear demands can be obtained as described above with reference to the work of Fox *et al.* [15].

3.3 Summary of proposed simplified assessment procedure for RC walls

The previous sections have laid out the ingredients for a simplified displacement-based assessment of RC wall structures. The final recommended procedure is shown in Figure 4. The first step is to obtain information on the building location (to establish seismic hazard information), geometry (building height, number of storeys, total floor area, wall section dimensions) and any other pertinent information that may impact the seismic performance. Secondly, the effective height factor should be found from Equation 2, the flexural yield displacement for the longest walls should be obtained using Equation 3 and the plastic rotation capacity

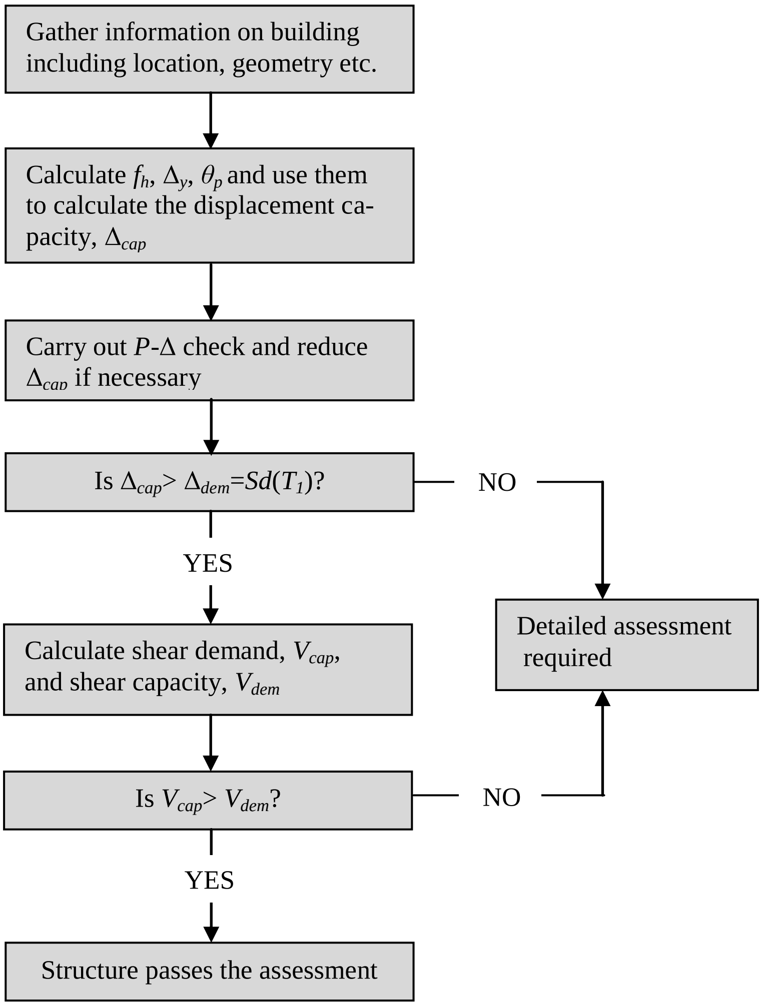


Figure 4. Flowchart of simplified assessment procedure.

from Equation 6. These values are then used in Equation 1 to estimate the displacement capacity, Δ_{cap}. Next the influence of P-Δ should be accounted for by reducing the displacement capacity in accordance with Equation 8 if necessary. The displacement capacity is then evaluated against displacement demand. If the demand is larger than the capacity this indicates the need for a detailed assessment, if not, then the procedure continues to the next step, which is to evaluate the shear capacity and shear demand using Equations 12 and 13 respectively. If the shear demand is greater than the shear capacity then detailed assessment is required, if not, then the structure has passed the assessment and is assumed to be of acceptably low seismic risk.

4 APPLICATION TO CASE STUDY WALL BUILDINGS

4.1 Case study buildings

This work examines various eight storey RC wall buildings all possessing the same generic layout indicated in Figure 5 (from Fox *et al.* [16]) and assumed to lie on rigid foundations. A total of 20 cases for the building are examined, with differences in wall length, reinforcement contents and confinement factors, as shown in Table 2. The buildings are assumed to lie in one of four different regions of seismicity: two regions of low seismicity, *a* and *b*, in which it is assumed that the reference ground acceleration (a_g) associated with a 475 year return period event is 0.10g but with spectral displacement corner periods of T_D=2s and 4s for sites *a* and *b* respectively, and two regions of moderate seismicity, *c* and *d*, in which the 475 year event a_g is taken as 0.20g and with spectral displacement corner periods of T_D= 2s and 4s for sites *c* and *d* respectively. The corner period spectral displacement demands, $Sd(T_D)$, for each of the four sites are also indicated in Table 2. Note that in all case study buildings the walls are given a thickness of 0.25m and walls 5 and 6 were assumed to possess lengths of 6.5m.

		L_w (m)				ρ_v (%)										
Case	Configuration description	W1	W2	W3	W4	W1	W2	W3	W4	C*	Mass (t)	T_1 (s)	Site	T_D (s)	a_g (g)	$Sd(T_D)$ (m)
1	Simple cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	588	2.47	a	2	0.1	0.086
2	Simple cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	588	2.47	b	4	0.1	0.172
3	Simple cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	588	2.47	c	2	0.2	0.172
4	Simple cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	588	2.47	d	4	0.2	0.342
5	Heavy cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	1176	2.47	a	2	0.1	0.086
6	Heavy cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	1176	2.47	b	4	0.1	0.172
7	Heavy cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	1176	2.47	c	2	0.2	0.172
8	Heavy cantilever	5	5	5	5	0.73	0.73	0.73	0.73	1.13	1176	2.47	d	4	0.2	0.342
9	Large ρ_v cantilever	5	5	5	5	1.5	1.5	1.5	1.5	1.22	588	2.03	a	2	0.1	0.086
10	Large ρ_v cantilever	5	5	5	5	1.5	1.5	1.5	1.5	1.22	588	2.03	b	4	0.1	0.172
11	Large ρ_v cantilever	5	5	5	5	1.5	1.5	1.5	1.5	1.22	588	2.03	c	2	0.2	0.172
12	Large ρ_v cantilever	5	5	5	5	1.5	1.5	1.5	1.5	1.22	588	2.03	d	4	0.2	0.342
13	Unequal lengths	6	3	6	3	0.72	0.96	0.72	0.96	1.2	588	2.76	a	2	0.1	0.086
14	Unequal lengths	6	3	6	3	0.72	0.96	0.72	0.96	1.2	588	2.76	b	4	0.1	0.172
15	Unequal lengths	6	3	6	3	0.72	0.96	0.72	0.96	1.2	588	2.76	c	2	0.2	0.172
16	Unequal lengths	6	3	6	3	0.72	0.96	0.72	0.96	1.2	588	2.76	d	4	0.2	0.342
17	Torsional	6.5	6.5	3.5	3.5	0.59	0.59	0.68	0.68	1.2	588	2.62	a	2	0.1	0.086
18	Torsional	6.5	6.5	3.5	3.5	0.59	0.59	0.68	0.68	1.2	588	2.62	b	4	0.1	0.172
19	Torsional	6.5	6.5	3.5	3.5	0.59	0.59	0.68	0.68	1.2	588	2.62	c	2	0.2	0.172
20	Torsional	6.5	6.5	3.5	3.5	0.59	0.59	0.68	0.68	1.2	588	2.62	d	4	0.2	0.342

Table 2. Properties of case study building configurations.

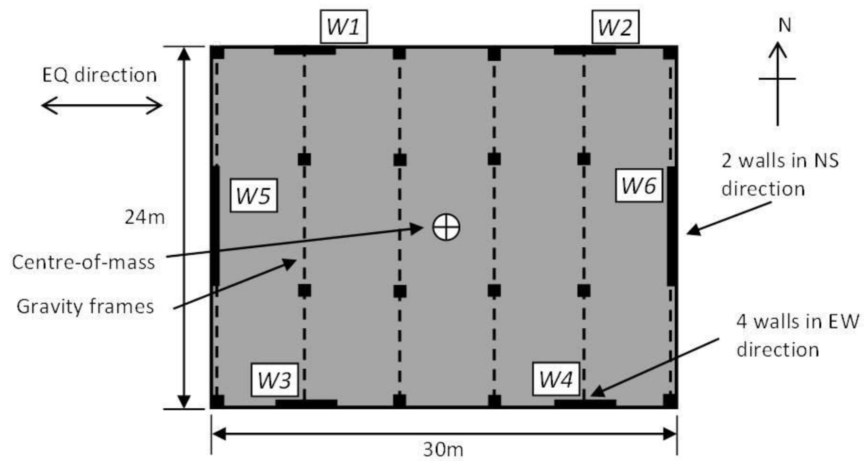


Figure 5. Plan view of case study building (from [17]).

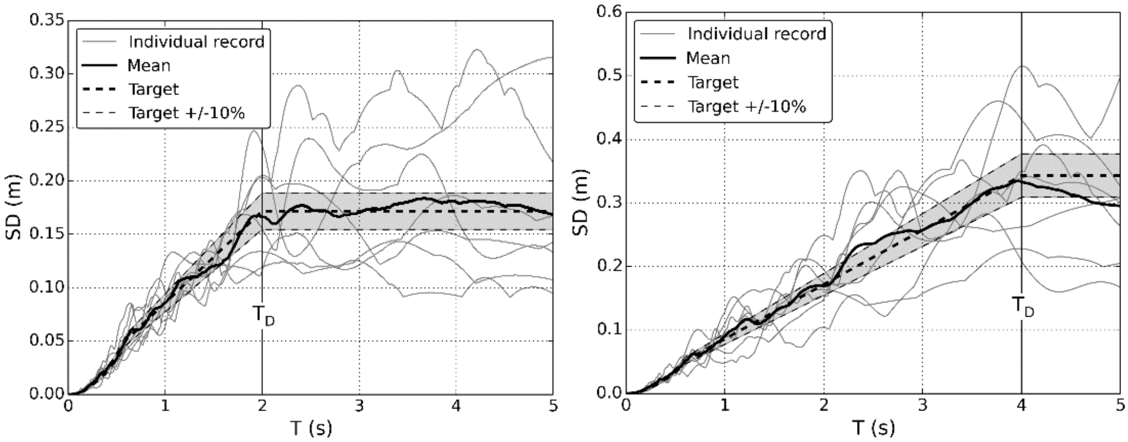


Figure 6. Displacement response spectra of ground motions used in NLRHA (a) Site *a*, and (b) Site *b* (from [17]).

4.2 Modelling and analysis

To rigorously evaluate the performance of the case study buildings they are analysed using nonlinear response-history analyses (NLRHA). All buildings are modelled using a lumped plasticity approach with the finite element program Ruaumoko3D [17]. The frame elements (one per wall per storey) used to model the walls are all linear elastic above level one. Between level one and the foundation Giberson one component frame elements are used, which allow inelastic deformations over a pre-defined plastic hinge length, L_p, which is calculated in accordance with [7]. The 'Takeda thin' hysteresis rule (i.e. a Takeda hysteresis with parameters α=0.5 and β=0.0. Refer Carr [19]) is used to model the hysteretic behaviour in the plastic hinge region. To include P-Δ a 'large displacement' analysis regime is used and additional gravity loads not supported by the walls are applied to a 'dummy column'. All walls are connected by rigid diaphragms.

Two sets of seven ground motions were used for the NLRHAs one. Both sets of ground motions (when appropriately scaled) are compatible with the type 1 spectrum from EC8 [11] for ground-type C; however, the first set is selected to be compatible with a spectral displacement corner period of T_D= 2s while the latter is for T_D= 4s. The target spectra and displacement response spectra of the selected ground motions are shown in Figure 6. Further information on the specific ground motions can be found in [16].

4.3 Results

The performance of the proposed assessment procedure is evaluated through comparison of results against those found using equivalent rigorous approaches. In the first instance the demands evaluated using the simplified procedure are compared to the demands obtained from NLRHA. This is shown in Figure 7 for both displacement and shear, and is expressed as a ratio of simplified demand to NLRHA demand, thus making results larger than one conservative. In general the demand predictions using the simplified method are very good. For shear, all values fall between one and 1.8. For displacement the majority of results are between one (or very close to one) and two. There are two exceptions for this in the torsional building, which are due to displacement demands being estimated at the centre-of-mass in the simplified method while NLRHA is able to capture the larger demands at the flexible edge of the building.

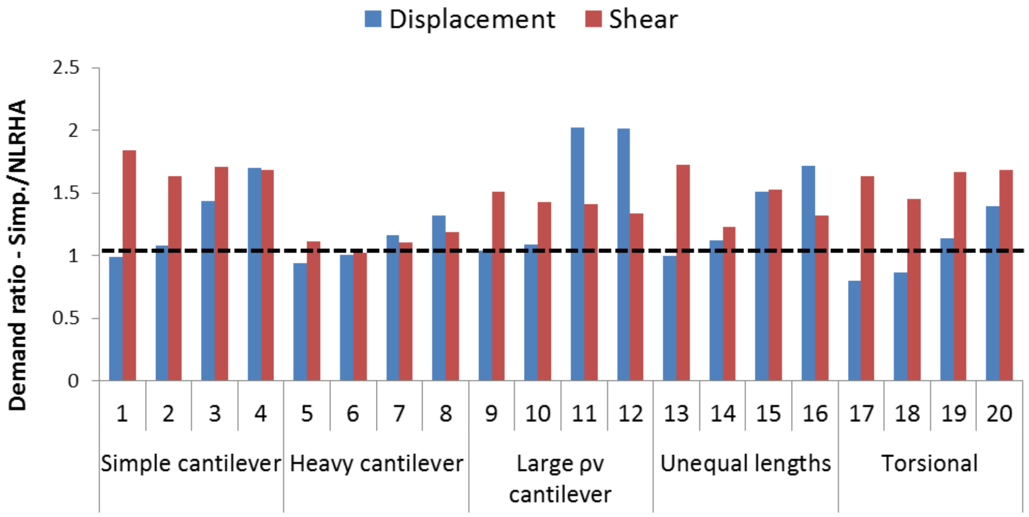


Figure 7. Ratios of demands obtained using the simplified procedure and from NLRHA.

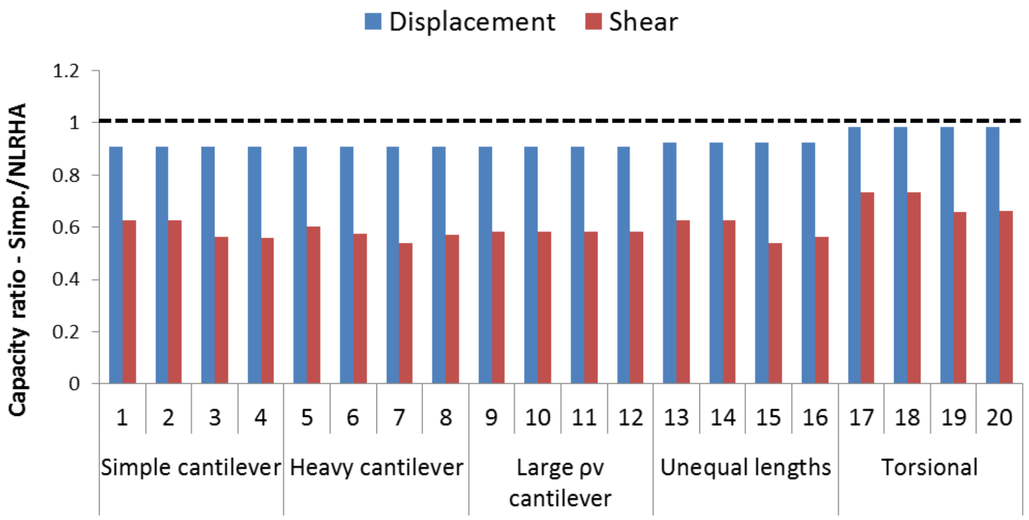


Figure 8. Ratios of capacities obtained using the simplified procedure and equivalent detailed procedures.

Figure 8 shows the same ratios as Figure 7 but this time for capacity. In this case a value less than one is conservative. It can be seen that for both displacement and shear all ratios are less than one. Displacement ratios tend to average around 0.9, while for shear the simplified approach is more conservative giving predictions equal to around 60% of those obtained using the UCSD shear model in full.

Figure 9 shows ratios of demand to capacity calculated using the simplified assessment procedure and using a detailed assessment procedure (with detailed capacity calculations and demand estimates obtained from NLRHA). In each case the ratio is the larger of those obtained from considering displacements or shear. From the detailed assessment results it can be seen that all configurations of the case study building achieve an acceptable level of performance. In fact they tend to have a fairly large reserve capacity against the limits for which they are being assessed. It can be seen that for all cases the simplified procedure produces more conservative demand/capacity ratios (in most cases significantly more conservative). In 10 out of the 20 cases the demand/capacity ratios are large than one, which would invoke the need for a detailed assessment.

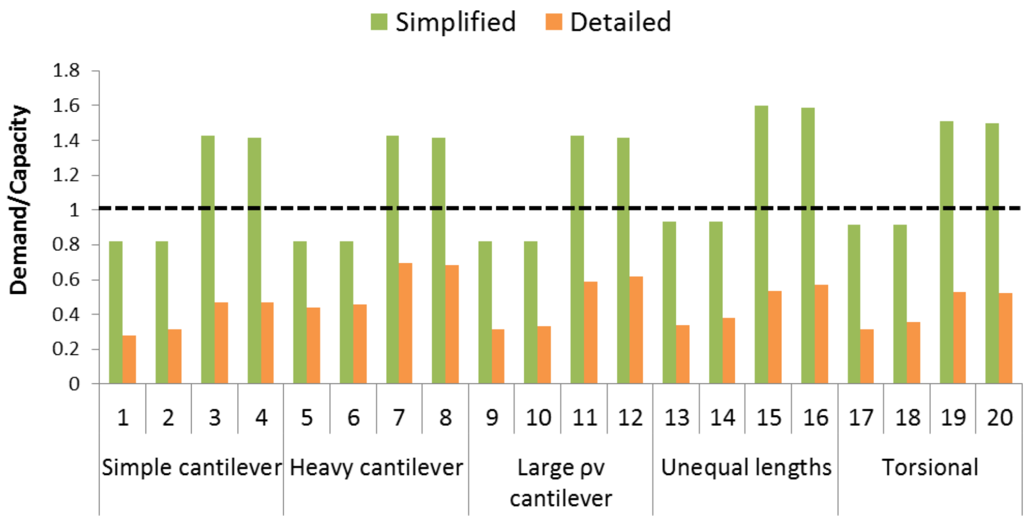


Figure 9. Demand to capacity ratios obtained using the simplified procedure and detailed assessment.

4.4 Discussion

The examination of the case study buildings has shown that the proposed assessment procedure works reasonably well. Predications of demand/capacity ratios are in most cases very conservative; however, this is in-line with the philosophy of the procedure, which is that conservative assumptions can be made and in doing so accuracy is sacrificed for expediency. For the cases considered, 50% would require no further seismic assessment. This reflects the fact that in general the buildings perform very well, as they are in regions of only low and moderate seismicity. Of the two sides of the demand versus capacity equation it appears that the simplified procedure is better at estimating demands, and future work in simplified capacity predictions appears to be the best approach to approving the procedure over all. The results presented in Figure 9 do have to be interpreted with some caution as in all cases shear was critical. It therefore seems prudent that more rigorous evaluations be undertaken on a wider range of case study buildings with widely varying properties.

5 CONCLUSIONS

A simplified procedure has been presented for the seismic assessment of RC wall buildings. The procedure is based on the idea that a number of conservative assumptions can be made and thus the procedure simplified significantly. This has included the development of simplified equations for estimating displacement capacity, shear capacity and shear demand. Also of particular note is the assumption that displacement demand can be approximated by spectral displacement at the corner period of the displacement spectrum (*i.e.* $Sd(T_D)$).

Through a number of case study applications it was shown that the procedure performs reasonable well. As expected it give predictions of demand to capacity that are always more conservative than those obtained using a more detailed assessment with nonlinear response-history analyses. To improve the procedure it was identified that future works should focus on improved shear capacity predictions and the procedure should be tested on a more diverse set of case study buildings.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge that part of the work leading to this research received funding from the Italian Department of Civil Protection (DPC) as part of the EUCENTRE-DPC 2014-2015 agreement.

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