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# Capacity design considerations for RC frame-wall structures

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**Abstract.** There are several important considerations that need to be made in the capacity design of RC frame-wall structures. Capacity design forces will be affected by material overstrength, higher mode effects and secondary loadpaths associated with the 3-dimensional structural response. In this paper, the main issues are identified and different means of predicting capacity design forces are reviewed. In order to ensure that RC frame-wall structures perform well it is explained that the prediction of the peak shears and moments that develop in the walls is particularly important and unfortunately very challenging. Through examination of a number of case study structures it is shown that there are a number of serious limitations with capacity design procedures included in current codes. The basis and potential of alternative capacity design procedures available in the literature is reviewed, and a new simplified capacity design possibility is proposed. Comparison with the results of 200 NLTH analyses of frame-wall structures ranging from 4 to 20 storeys suggest that the new method is able to predict wall base shears and mid-height wall moments reliably. However, efforts are also made to highlight the uncertainty with capacity design procedures and emphasise the need for future research on the subject.

**Keywords:** capacity design; frame wall; dual system; seismic design.

## 1. Introduction

In 1975, Park and Paulay (1975) concisely described the purpose of capacity design as follows, "In the capacity design of earthquake-resistant structures, energy-dissipating elements of mechanisms are chosen and suitably detailed, and other structural elements are provided with sufficient reserve strength capacity, to ensure that the chosen energy-dissipating mechanisms are maintained at near their full strength throughout the deformations that may occur." If capacity design is not undertaken correctly, the well-detailed plastic mechanism envisaged by the designer will not be ensured and alternative mechanisms, inevitably possessing less deformation capacity, will develop and result in partial or full collapse of the building at seismic intensities much lower than those considered during design. For RC frame-wall structures, also known as dual system structures, a commonly desired inelastic mechanism, typically referred to as a beam-sway mechanism, is one in which the beams yield in flexure at their ends and the columns and walls yield in flexure at their bases, as indicated in Fig. 1(a). It has been observed by Paulay and Goodsir (1986) and Lopes and Bento (2001) that a column-sway mechanism, shown in Fig. 1(b), can also be

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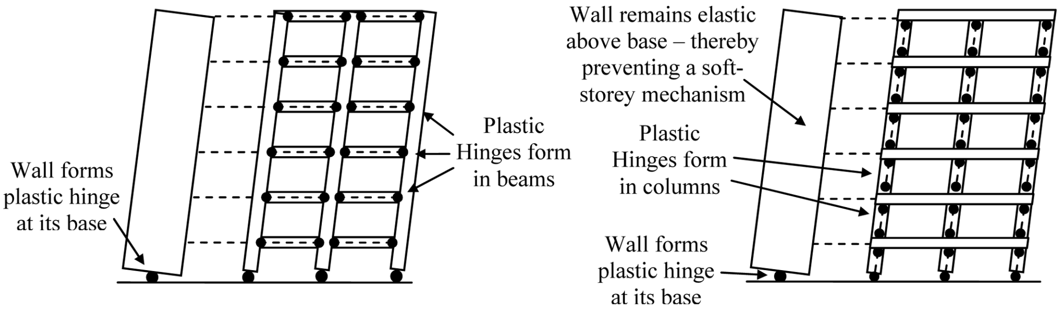



(a) Beam-sway mechanism (b) Column-sway mechanism

Fig. 1 Potentially desirable plastic mechanisms for the seismic design of RC frame-wall structures

detailed to provide good deformation capacity for frame-wall structures. While column-sway mechanisms are not suitable for frame structures alone because they inevitably lead to soft-storey collapse (see, for example, Paulay and Priestley 1992), in dual frame-wall structures the column-sway mechanism should be acceptable because a soft storey will be avoided provided that the walls remain elastic above their base plastic hinge. This observation highlights the importance of the RC walls to the response of frame-wall structures. Unfortunately, as will be shown in this paper, the capacity design procedures currently used to predict capacity design forces for RC walls in frame-wall structures appear to be non-conservative and improved capacity design methods are required.

In order to undertake capacity design correctly, allowance must be made for higher mode effects as well as overstrength in plastic hinge regions due to higher than specified material strengths and strain hardening. In principle, the overstrength of plastic hinge zones can be adequately predicted with knowledge of the probable material strengths and expected curvature ductility demands. However, unforeseen load paths in frame-wall structures can alter the expected strengths of the design plastic mechanism. In addition, higher mode effects are difficult to predict, being dependent on the period and damping of the structure and the intensity of the higher mode excitation relative to the fundamental mode response. The next section describes the main challenges associated with these capacity design tasks.

Note that this paper will focus principally on the challenges associated with the correct prediction of capacity design forces. However, there is an equally important challenge not addressed in this paper, which is to improve the expressions currently available for the prediction of resistances, and in particular shear resistance estimates of RC walls which is outside the scope of this paper but should be included in future research initiatives.

## 2. Challenges for the capacity design of RC frame-wall systems

### 2.1 Plastic hinge overstrength

The term "overstrength" principally refers to the effects that higher than specified material strengths and strain hardening have on the forces that develop in plastic hinges. Overstrength in plastic hinge zones causes an overall increase in the forces associated with the 1$^{st}$ mode sway-mechanism, and consequently increases the capacity design forces that must be resisted elastically to ensure that the intended plastic mechanism develops. As such, it is important that a reasonable estimate of the maximum hinge strength is made. This is a challenge for RC structures since it is known that there is significant variability in material properties used in construction and that concrete resistance tends to increase with the passage of time. In order to safely evaluate the overstrength of RC plastic hinges, Priestley *et al.* (2007) recommend that the following upper-bound material strengths are used in the evaluation of the plastic hinge resistance.

$$f'_{co} = 1.7f'_c \tag{1}$$

$$f_{yo} = 1.3f_y \tag{2}$$

Where $f'_c$ is the characteristic concrete compressive strength, $f'_{co}$ is the upper bound (overstrength) concrete compressive strength, $f_y$ is the characteristic yield strength of the longitudinal reinforcement and $f_{yo}$ is the upper bound (overstrength) yield strength of the longitudinal reinforcement. As such, to establish the overstrength of a RC plastic hinge, moment-curvature analyses can be run in which the material strengths indicated in Eqs. (1) and (2) are used and curvatures corresponding to maximum expected deformations are imposed. Provided that the constitutive model used for the reinforcement in the moment-curvature analyses includes strain-hardening, such moment-curvature analyses can be an effective means of establishing the plastic hinge overstrength.

In the estimation of plastic hinge overstrength, it is important to stress that the evaluation be undertaken considering the actual reinforcement quantities proposed for construction. This is particularly important for regions of moderate seismicity in which the overstrength hinge forces, and consequently the seismic design shears, may be associated with the development of member strengths that were governed by gravity load combinations. Another important but uncertain source of overstrength for RC beams can be the contribution of slab reinforcement to the flexural resistance. Similarly, for columns and walls, there is likely to be uncertainty in the expected axial loads, which can increase the plastic hinge strength significantly, as discussed further in Section 2.3. Overstrength effects related to axial loads in beams may also be expected, with beam axial forces generated not only by seismic accelerations but also potentially from the axial forces that develop as a result of beam elongation (refer Fenwick and Megget 1993).

### 2.2 Higher mode effects

The evaluation of higher modes effects is particularly important and can be very challenging for both medium and high rise frame-wall structures, particularly with regard to capacity design forces in the walls. To illustrate the fact that wall forces are affected more by higher modes than frame forces, consider the shears shown in Fig. 2 for an 8-storey case study RC frame-wall structure subject to non-linear time-history (NLTH) analyses by Sullivan *et al.* (2006). The figure plots the wall shear and frame shear up the height of the building at different instances in time during the seismic response to the magnitude 6.7 Erzican record (scaled in intensity by a factor of 1.2 in order to be spectrum compatible). The case study structure was designed using a Direct DBD procedure (refer Priestley *et al.* 2007) and the NLTH analyses were undertaken using the program Ruaumoko (Carr 2004). For details of the NLTH modelling and anlaysis assumptions, readers should refer to Sullivan *et al.* (2006).

One can observe that the wall shears plotted in Fig. 2 vary significantly over a period of 0.25s, due to response of the second mode of vibration (note that the fundamental mode of vibration of the

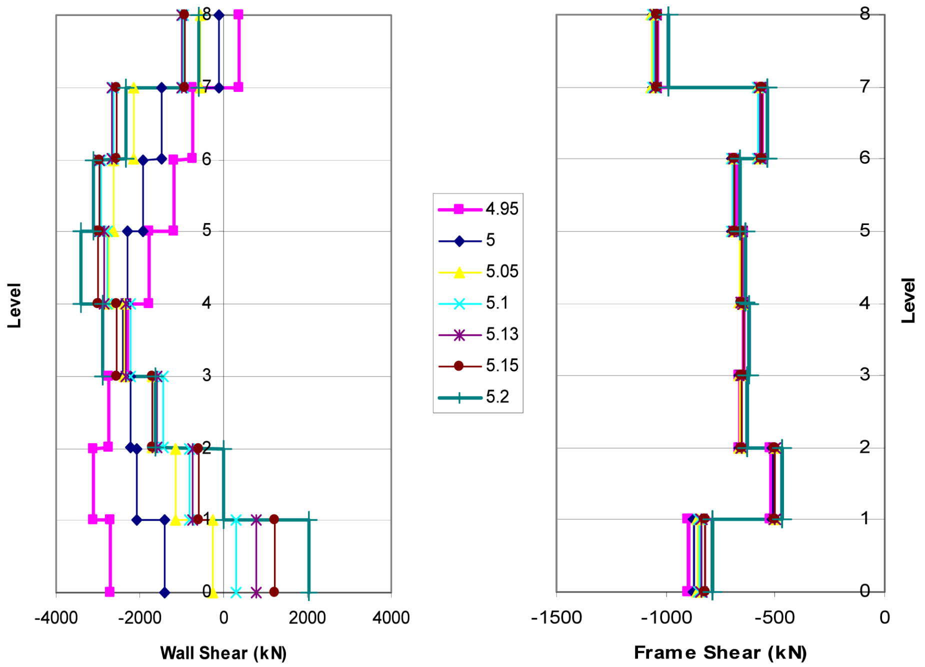



Fig. 2 Variation of shear forces in (a) the walls and (b) the frames of an 8-storey RC frame-wall structure over a 0.25s interval (4.95s to 5.20s) of the Erzican record (Sullivan *et al.* 2006)

building was close to 2s). However, on the contrary, the frame shears remain fairly constant for the same interval, supporting the notion that frame forces are relatively insensitive to higher mode actions. The reasons for the irregular frame shear profile that can be observed over the lower two storeys will be discussed later in Section 3.2.

The fact that wall shears are greatly affected by higher mode response, as seen in Fig. 2, has driven international standards to include capacity design procedures that aim to account for such effects. These procedures will be reviewed in Section 3.1, where it will be shown that the current capacity design procedures may seriously underestimate the magnitude of such forces.

## *2.3 Three dimensional structural response*

In addition to structural overstrength and higher mode effects, another factor that should be considered in the capacity design of frame-wall structures is related to 3D effects. Consider the 3D sketch of Fig. 3, presenting a frame-wall structure deforming under seismic actions.

The main lateral resistance of frame-wall structures is known to be offered by the in-plane stiffness of the frames and walls. However, as indicated in Fig. 3, torsion in transverse beams and coupling of slabs between walls and (gravity) columns may also offer seismic resistance to the frame-wall system. Bertero *et al.* (1985) observed such 3D effects experimentally as part of a US-Japan Cooperative research program into frame-wall structures. They noted damage to floors of a 1/ $5^{th}$-scale model subject to shake table testing and considered this an indication of the restraint offered by the slabs to uplift and bending of the walls. Due to this restraint, the maximum compression in the central wall increased by approximately 50%. As shown in Fig. 4, for typical RC walls an increase in the axial force acting on the section will lead to an increase in flexural

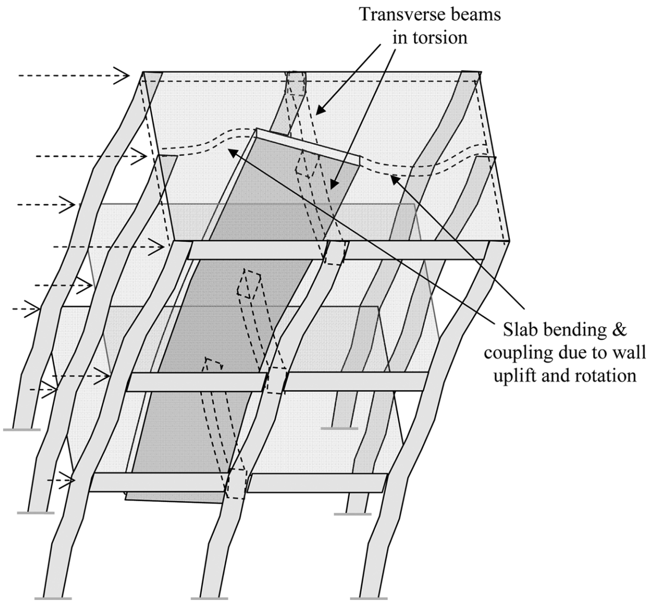



Fig. 3 Sketch of deformed frame-wall structure, highlighting potential 3D effects

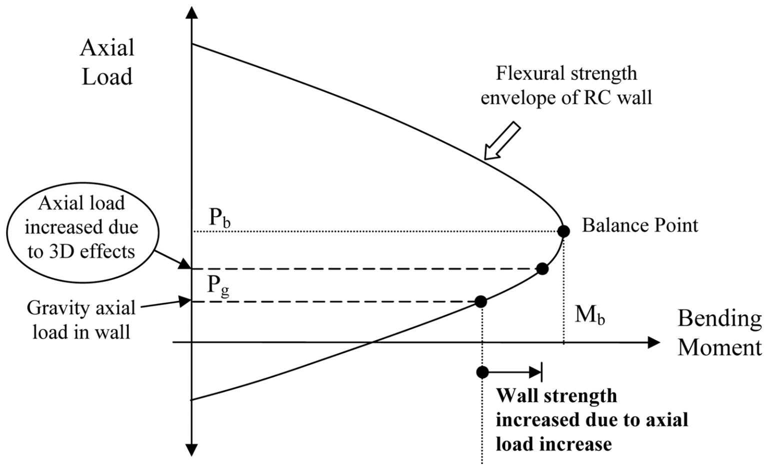



Fig. 4 Axial load-bending moment interaction diagram, illustrating the manner in which an increase in axial load due will typically lead to increased flexural strength

resistance. An increase in flexural resistance is clearly important for capacity design as it implies that the shear forces associated with the first mode plastic mechanism will also increase.

In addition to 3D effects, one should recognise that even gravity axial loads in RC walls can be difficult to establish due to uncertainties associated with shrinkage and construction sequence (see Aktan *et al.* 1983). Furthermore, vertical ground motion components can also be expected to alter the axial forces in the walls during the seismic response. As argued by Papazoglou and Elnashai (1996) and more recently by Elnashai *et al*. (2006), the performance of concrete members can be significantly affected by the vertical component of excitation, and this appears to be an important area for future research.

While major sources of uncertainty in wall axial loads are therefore clear, the best means of quantifying such effects for capacity design is not so clear. Certainly, considerable effort would be required to build 3D models capable of properly reproducing the 3D effects described above, and the accuracy of such analyses would themselves be sensitive to difficult modelling assumptions. As such, until better guidance can be provided through future research, it appears that designers must simply consider the implications of such uncertainties in setting the shear resistance of the structure. Fortunately, as pointed out by Bertero *et al.* (1985), while the fundamental mode shear forces in the walls will increase with increasing axial load, the shear-friction capacity of the walls will also increase. This point does alleviate some of the concern associated with uncertainties in axial loads, but it is clear that additional research is required to establish practical means of accounting for such effects.

## 3. Capacity design methods in current codes

### 3.1 Different code procedures

Capacity design methods in different international codes vary rather significantly. In the US, the ASCE 7-05 recommends the use of an overstrength factor approach, in which it appears that all the sources of seismic force amplification described in the previous section are lumped into a single overstrength factor. For well detailed RC frame-wall structures, the ASCE 7-05 specifies that this overstrength be generally taken as 2.5. In Europe, a more refined approach is proposed in the EC8 (CEN EC8 2004). The recommendations included in the EC8 for frame-wall structures appear to stem from the work of Goodsir and Paulay (Goodsir 1985, Paulay and Goodsir 1986) the findings of which are also reported in Paulay and Priestley (1992). The New Zealand concrete structures standard, NZS3101:Part 1:2006 (2006), also advocates the Paulay and Goodsir approach, directing readers to the recommendations found in Paulay and Priestley (1992). Fig. 5 presents the recommendations of Paulay and Goodsir (1986) for the capacity design wall moments and shears,

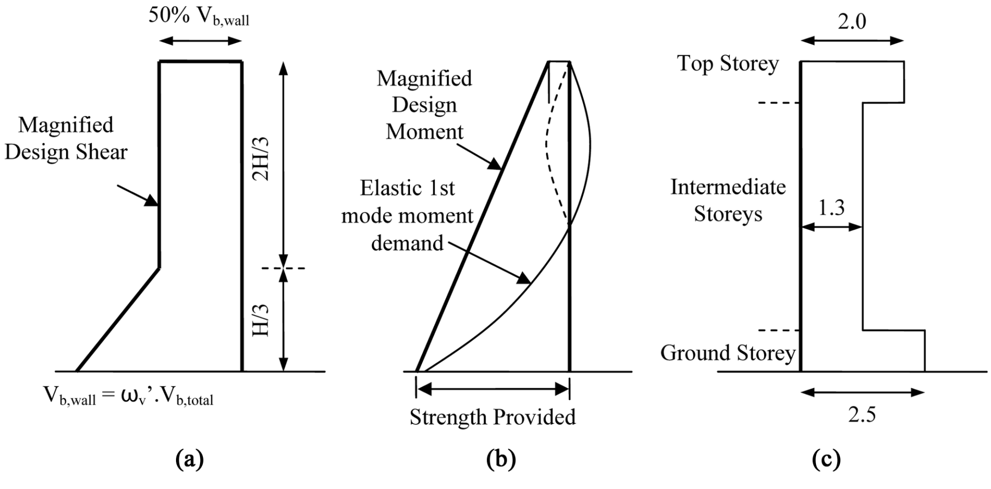



Fig. 5 Paulay and Goodsir (1986) capacity design profiles for RC frame-wall structures: (a) wall shears, (b) wall moments, (c) column shears

as well as the capacity design column shears in frame-wall structures.

The approach proposed by Paulay and Goodsir is to define design force envelopes that aim to account for dynamic magnification due to higher mode effects, in addition to the material overstrength of hinging elements. The approach uses dynamic magnification factors (similar to the overstrength factor approach of the ASCE7-05) that vary up the height of the building. Dynamic magnification factors are essentially scaling factors used to magnify 1st mode forces obtained from elastic analysis, up to force levels expected during a fully dynamic non-linear response. The magnification factors are typically obtained empirically by firstly finding the fundamental mode design strengths of a number of structures and then submitting models of these structures to non-linear time-history analyses using a number of different ground motions. They are used in conjunction with overstrength factors to set design strengths of elements that are intended to remain elastic for a given seismic event, in accordance with a capacity design philosophy.

To obtain the wall shear magnification factor using Paulay and Goodsir's recommendations it is first necessary to calculate a "free standing wall" magnification factor, $\omega_v$, in accordance with the appropriate form of Eq. (3).

$$\omega_v = 0.9 + n/10 \qquad \text{when } n \leq 6 \quad (3a)$$

$$\omega_v = 1.3 + n/30 \leq 1.8 \qquad \text{when } n > 6 \quad (3b)$$

where $n$ is the total number of storeys. The dynamic shear magnification factor appropriate for a wall in a frame-wall structure, $\omega_v'$, is then calculated for the base of the wall using the proportion of design base shear resisted by the walls, $V_{b,wall}/V_{b,total}$, as shown in Eq. (4).

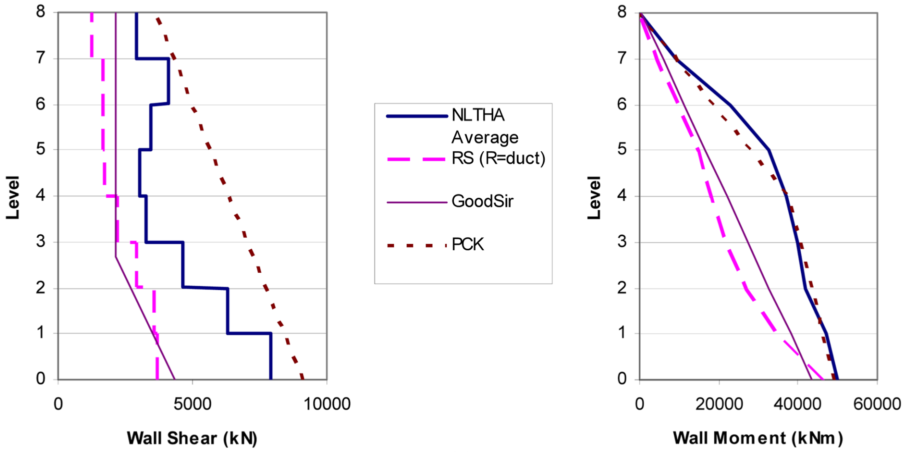
$$\omega_v' = 1 + (\omega_v - 1) \cdot \frac{V_{b,wall}}{V_{b,total}} \quad (4)$$

In the EC8 this process is simplified by stating that the base shear forces in the wall may be taken as being 50% higher than the forces obtained from seismic analysis.

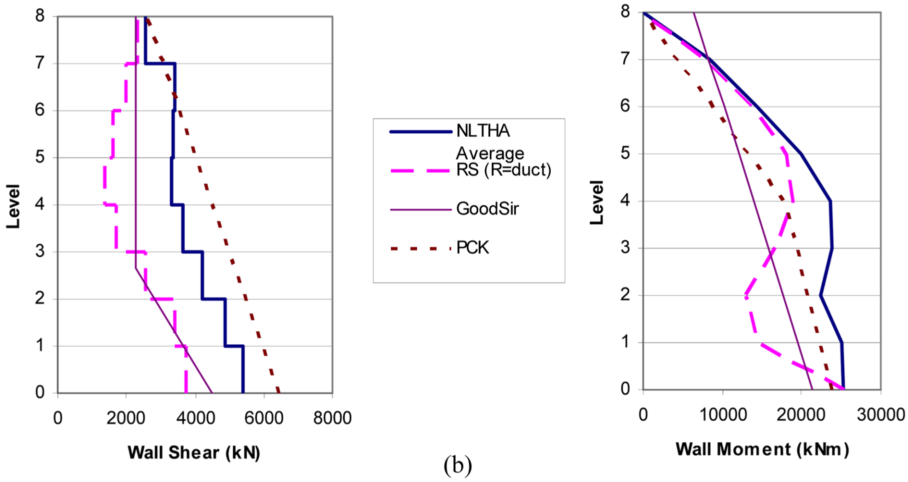
### *3.2 Limitations of current procedures*

Previous work by Priestley and Amaris (2002), amongst others, has highlighted the fact that RC wall forces can be considerably underestimated by capacity design procedures included in current codes. To highlight this point for walls in frame-wall structures, Fig. 6 compares wall shears and moments predicted by different capacity design approaches with the forces evaluated through non-linear time-history (NLTH) analyses of two 8-storey frame-wall structures. Details of the case study structures and the modeling and analysis approach can be found in Sullivan *et al.* (2006). The main distinction between the two case study buildings relates to the proportion of overturning resisted by the frames: the frames of the building shown in Fig. 6(a) resist only (approximately) 25% of the overturning whereas those in Fig. 6(b) resist 70%. As base shear proportions can be significantly affected by a change in the base column plastic hinge strengths (without significantly affecting the total frame overturning resistance), it is considered that the overturning resistance provides a better indication of the relative importance of the frames and walls to dual systems. However, as modern codes tend to refer to base shear proportions, it may be of interest to note that the frames of the case study buildings shown in Figs. 6(a) and (b) were assigned first mode base-shear proportions of (approximately) 20% and 50%, respectively. The three prediction curves included in the comparison correspond to the Paulay and Goodsir (1986) approach, the response spectrum (RS) method in which modal forces were combined using an SRSS rule and then reduced by a behaviour factor corresponding to the system displacement ductility, and a simplified "PCK" capacity design procedure recently proposed by Priestley *et al*. (2007) and that will be described in more detail in Section 4.1.

Reviewing Fig. 6, it can be seen that the Paulay and Goodsir (1986) approach and the response spectrum (RS) method both significantly underestimate the wall shears up the height of the building and the wall moments around the mid-height of the structure. Wall moments at the base appear to be slightly underestimated by the Paulay and Goodsir approach, but this difference only reflects the NLTH analysis overstrength due to strain-hardening. Alarmingly, the wall base shear obtained from non-linear time-history analyses of the strong-wall building (Fig. 6(a)) are twice as large as those




(a)




(b)

Fig. 6 Capacity design predictions of wall shears (left) and wall moments (right) compared with average of peak forces obtained from NLTH analyses for 8-storey frame-wall structures: (a) 25% overturning resisted by frames, (b) 70% overturning resisted by frames predicted by the Paulay and Goodsir (1986) approach, which is the approach advocated by both the Eurocode 8 (CEN EC8 2004) and the New Zealand standard (NZS3101 2006). The approach of Priestley *et al*. (2007), labelled “PCK” in Fig. 6, appears to work well for the 8-storey case study buildings presented here. One might argue that the results indicate that the PCK approach would provide insufficient flexural resistance around the mid-height of the walls shown in Fig. 6(b) but Priestley *et al*. (2007) report that an effort has been made to ensure that the design flexural envelopes are not too conservative since minor flexural ductility demands above the wall base do not have serious consequences (see, for example, Sullivan *et al*. 2006 for analytical evidence of this).

Another important aspect for the capacity design of frame-wall structures is the prediction of shear demands in the columns of the RC frames. Overall, as illustrated earlier in Fig. 2, the shear demands in columns are not significantly affected by higher modes. However, as reported by Goodsir (1985), an increase in shear demands can be expected at the ground storey and also at the top storey of frames and this is reflected in the column capacity design shear profiles of Fig. 5(c). This form of shear profile can be seen in Fig. 7(a), where the maximum frame shears recorded in a 16-storey frame-wall building studied by Sullivan *et al*. (2006) are reported for five different ground motions. In part, the apparent shear magnification at the ground storey can be attributed to the

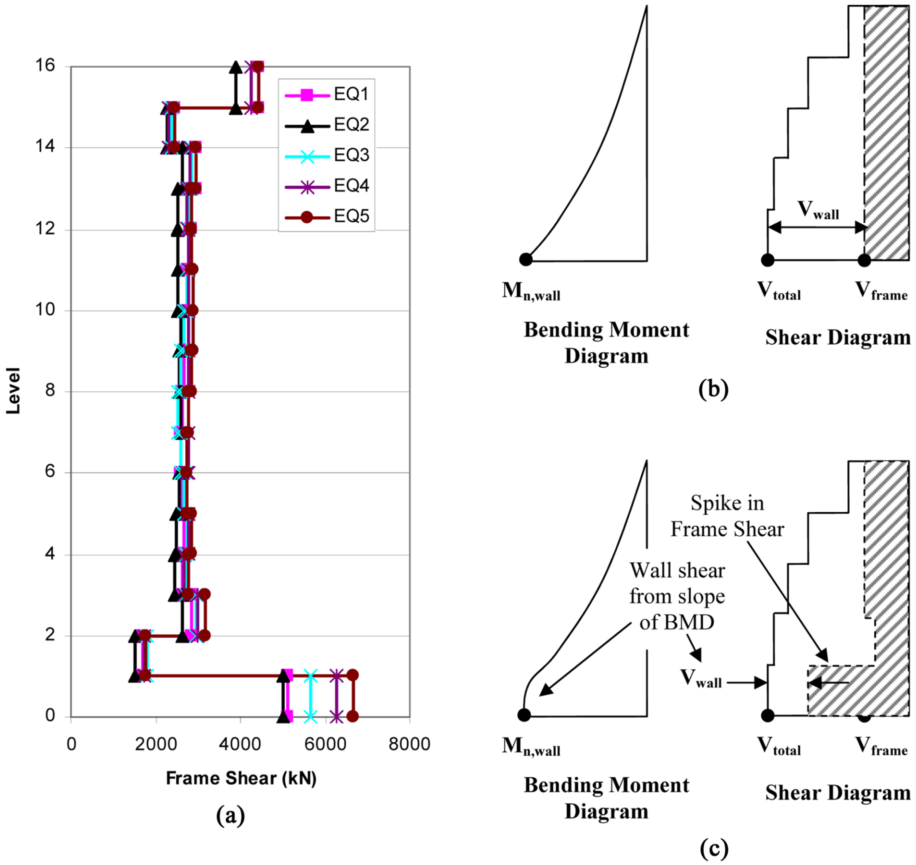



Fig. 7 (a) Frame shears recorded from NLTH analyses, (b) bending moment diagram in walls at yield and corresponding shear forces in frames and walls, (c) bending moment diagram in walls as demands increase beyond yield and corresponding shear forces in frames and walls development of a full plastic mechanism in the frame. As indicated earlier in Fig. 1, column bases are expected to yield in the classic beam-sway mechanism. Considering this, and noting that columns will be considerably stronger than beams, it is likely that the slope of the bending moment diagram in the first floor and therefore the column shears at the first floor are greater than those obtained from a standard elastic analysis. However, the frame shear demands over the lower storey are also affected by the moment profile in the walls, and the change in the wall moment profile that occurs as wall yielding develops can imply a large transfer of shear to the frame at the ground storey, as will be explained with reference to Figs. 7(b) and (c).

As shown in Fig. 7(b), initially, at the initiation of wall yield, a relatively even distribution of shears between the frames and walls can be expected. After reaching yield, the moments at the base of the wall cannot be increased. However, above the first floor level, the shears and moments in the wall can continue to increase (from fundamental mode actions if a full plastic mechanism has not formed or due to higher mode actions). As such, a wall bending moment profile of the form shown on the left side of Fig. 7(c) can develop. The tendency for this moment profile to develop during non-linear dynamic response has been observed by Sullivan *et al.* (2006) and Sullivan *et al.* (2008), examining the results of several NLTH analysis studies of frame-wall structures. One recognizes that as the base strength in the wall is limited, an increase in the wall moment demand at the first floor actually causes a reduction of the wall shear at the ground storey (since the wall shear is the slope of the bending moment diagram). Consequently, the frames must absorb the difference between the wall base shears (which are reducing) and the total shears (which are increasing), and this inevitably means a spike forms in the frame shear profile at the ground storey, as is evident if Fig. 7(a). A spike in frame shear is also evident in the results presented earlier in Fig. 2(b), where it can be seen that when the first floor displacement is a maximum, at 4.95s, both the wall shears and frame shears are high. Then, as the wall shears reduce over a short period of time due to a second mode of vibration, the frame shears remain fairly constant (and tend not to reduce until the first floor displacements reduce).

This tendency to form a spike in the frame shear at the ground storey was observed by Goodsir (1985) and Sullivan *et al.* (2006) using lumped-plasticity analytical models. Interestingly, as was observed recently by Ciaponi (2010), if a fibre-element representation of the RC wall is used, the wall yielding can extend up above the first floor level and consequently, the spike in frame shear is registered not at the ground storey level, but instead at the first floor level. Such an observation clearly is significant for capacity design as it suggests that the amplification proposed by the Paulay and Goodsir (1986) approach may significantly underestimate column shears in the first storey (or possibly higher storeys if very long walls are present in the building). While this does highlight an area for future research, note that an upper bound to the frame shear can be approximated as the shear that causes a column sway mechanism to form in the storey, and the column shear reinforcement necessary to sustain these forces should not be excessive owing to the slender form of columns used in typical frames.

## 4. Alternative capacity design methods

### 4.1 Existing capacity design alternatives in the literature

There are a number of capacity design approaches proposed in the literature, with the aim of better quantifying wall shears and moments. A particularly innovative proposal was made by Kabeyasawa (1987) in which fundamental mode shear forces in the walls are limited to those associated with a plastic mechanism and are then combined with a higher mode shear component. This approach is very similar to the approach developed by Priestley and Amaris (2002) for wall structures and Rodriguez *et al.* (2002) for floor diaphragms some 15 years later. In the modified-modal superposition approach proposed by Priestley and Amaris (2002), the wall shears are predicted by using response spectrum analyses to identify elastic (unreduced) higher mode shear components in the walls which are then combined through an SRSS approach with the inelastic (reduced) 1st mode wall shears. Unfortunately, while this approach appears to work well for medium-rise cantilever wall structures, Sullivan *et al.* (2006) showed that the approach grossly overestimates the shears and moments in walls of frame-wall structures. As such, an alternative modal superposition method was proposed by Sullivan *et al.* (2006, 2008) that accounts for higher mode period lengthening by using modal characteristics obtained from eigenvalue analyses of models that incorporate the post-yield tangent stiffness at plastic hinge zones (or more specifically, the tangent-stiffness of plastic hinge zones at deformations associated with the 1st mode peak response). While initial applications of this approach suggest that it may be able to predict wall shears well, a further variation to the modal superposition method has recently been recommended by Priestley *et al.* (2007) in which the effective (secant) stiffness of the structure at the 1st mode peak response is proposed instead of the tangent stiffness at plastic hinge zones.

While modified modal superposition methods are certainly an interesting possibility for capacity design that could merit further development in the future, it is recognized that such approaches are considerably more time-consuming than traditional capacity design methods of the type proposed by Paulay and Goodsir (1986). In recognition of this, Priestley *et al.* (2007) also propose a simplified capacity design procedure for frame-wall structures that specifies shear and moment envelopes for the walls in a similar manner to the Paulay and Goodsir (1986) approach. The envelopes proposed by Priestley *et al.* (2007) for the wall shears and moments are shown in Fig. 8.

The capacity design envelopes shown in Fig. 8 aim to include the effects of overstrength, through the factor $\phi^o$, and for higher modes through the use of higher mode magnification factors. The mid-height moment in the wall, $M^o_{0.5H}$, is set as a function of the wall base moment, $M_b$, according to Eq. (5) (from Priestley *et al.* 2007).

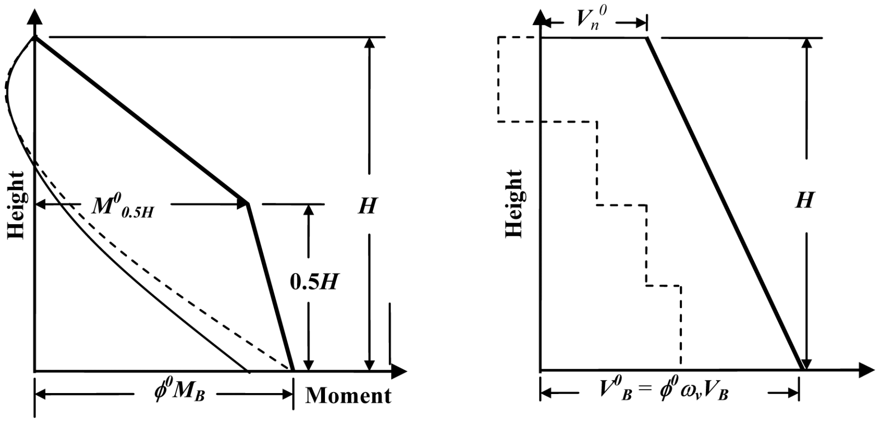



Fig. 8 Simplified capacity design envelopes for frame-wall structures proposed by Priestley *et al.* (2007)

$$M^0_{0.5H} = C_{1,T} \cdot \phi^o M_b \tag{5}$$

Where $C_{1,T}$ is given by Eq. (6)

$$C_{1,T} = 0.4 + 0.075T_i \cdot \left(\frac{\mu}{\phi^o} - 1\right) \geq 0.4 \tag{6}$$

In which $T_i$ is the initial period (in seconds) of the structure and $\mu$ is the ductility demand on the frame-wall system.

In a similar fashion, Priestley *et al.* (2007) propose that the base wall shear is established using Eq. (7).

$$V^0_{Base} = \phi^o \omega_V V_{base} \tag{7}$$

Where the dynamic magnification factor, $\omega_V$, is given by Eq. (8)

$$\omega_V = 1 + \frac{\mu}{\phi^o} C_{2,T} \tag{8}$$

And where $C_{2,T}$ is given by Eq. (9)

$$C_{2,T} = 0.4 + 0.2(T_i - 0.5) \leq 1.15 \tag{9}$$

Finally, Priestley *et al.* (2007) propose that the roof level wall shear, $V_{n0}$, is simply set as 40% the base shear of Eq. (7).

The expressions proposed in Eq. (5) to Eq. (9) are empirical expressions, based on the results of non-linear time-history analyses. As was shown earlier with reference to Fig. 6, application of this method to predict the wall shears and moments in two different 8 storey frame-wall structures suggest that it may work well, and therefore it is worth reviewing the conceptual basis of the approach.

As indicated by Eqs. (6) and (9), the simplified methodology quantifies higher mode effects in proportion to the initial period and the expected ductility demand. Intuitively, the proposal that higher mode actions increase with period appears rational because one can appreciate that higher-modes will not be significant for short buildings, which are characterized by low periods of vibration, whereas higher modes can be expected to be much more significant for taller buildings. Initially, it may be less evident why the higher mode actions should also be a function of the ductility. However, an important point made by Priestley and Amaris (2002) and Priestley *et al.* (2007) is that the relative magnitude of higher mode forces in RC wall structures should be intensity dependent and therefore, by incorporating the ductility demand into Eqs. (5) and (9) the expressions are rendered intensity-dependent. To visualize the basis for this, consider the acceleration spectra indicated in Fig. 9 for three levels of seismic intensity corresponding to peak ground accelerations (PGAs) of 0.2 g, 0.4 g and 0.6 g.

The figure identifies relative levels of design forces as a function of spectral accelerations for both the fundamental period of vibration, $T_1$, and the second mode of vibration, $T_2$. As expected, the spectral accelerations and therefore design forces of the second mode of vibration increase with an increase in seismic intensity. However, for the fundamental mode of vibration, the figure proposes

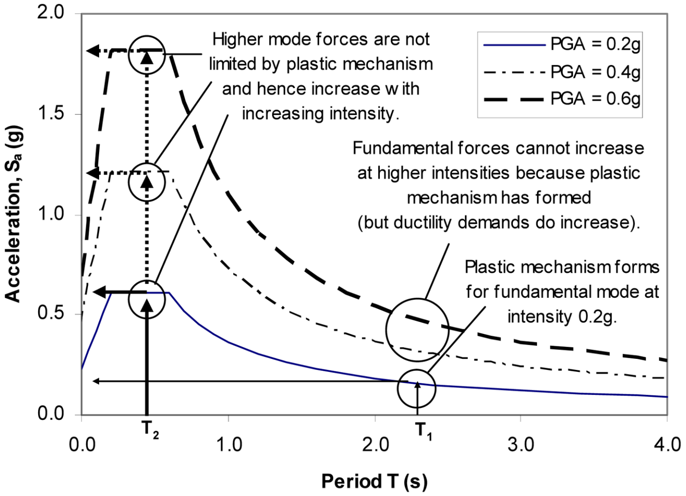
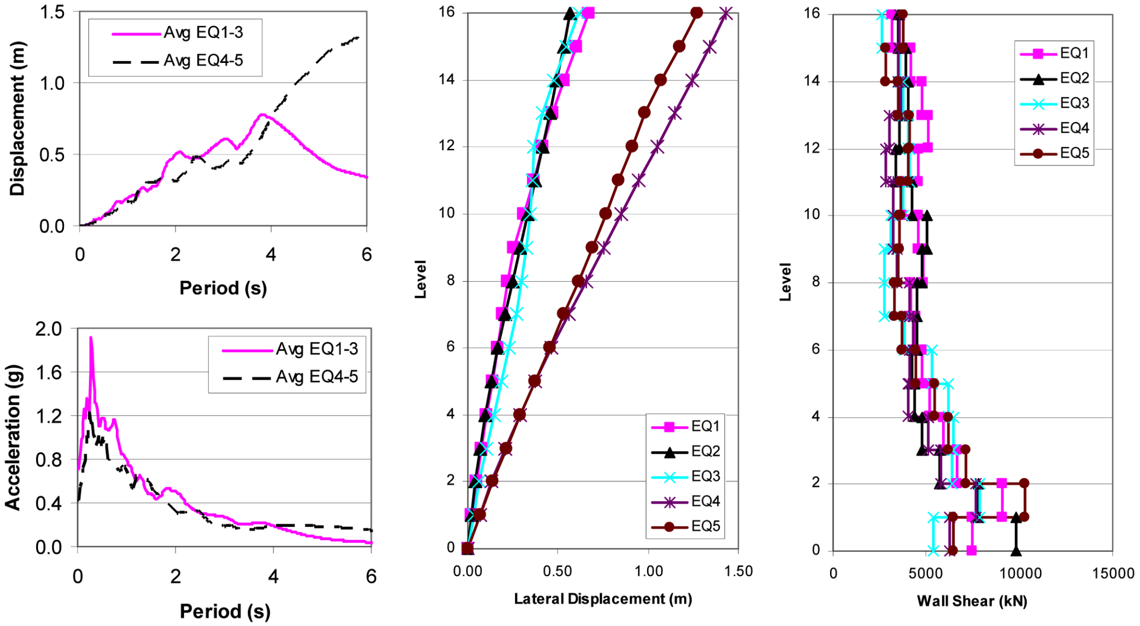



Fig. 9 Influence of increased seismic intensity on relative magnitude of higher mode forces in relation to fundamental mode forces in RC walls

that if the intensity at a PGA = 0.2 g is sufficient to form a plastic mechanism, then an increase in intensity does not correspond to an increase in fundamental mode design forces, since the flexural capacity at the base has developed and this prevents the further development of inertia forces associated with the first mode but not those of higher modes. Therefore more intense seismic excitation only causes larger ductility demands on the first mode configuration but may induce larger inertia forces on higher mode configurations. Taking these points into account, it is therefore clear why Priestley *et al.* (2007) propose that the capacity design forces be dependent on ductility.

Despite these motivations to include the ductility demand within capacity design procedures, it is argued that there are important cases in which the ductility demand cannot properly reflect the seismic intensity effect. Fig. 10 presents the maximum displacements and wall shears recorded through non-linear time-history analyses of a 16-storey frame-wall structure subject to five different ground motions. For details of the non-linear time-history modelling and analysis approach, readers should again refer to Sullivan *et al.* (2006). From the displacement profiles of Fig. 10(b), it is clear that the ductility demands on the system for ground motion records EQ1 to EQ3 are half those imposed by records EQ4 and EQ5. However, despite the large differences in ductility demands, the wall shears recorded for the five records are all of similar magnitude, and in fact, records EQ1 and EQ2 register the largest base shear in the walls. As such, one sees that the capacity design methods should not be ductility dependent.

The reason for this behaviour becomes clear if one considers the average spectra for the grounds motions, shown in Fig. 10(a). The spectral displacement demands of records EQ1 to EQ3 are much lower than those of records EQ4 and EQ5 at long periods (in the fundamental mode effective period range) thereby explaining the large differences in ductility demands for the different records. On the other hand, at short and medium periods it can be seen that the spectral accelerations of the two sets of records are of similar intensity, with records EQ1 to EQ3 being characterised, on average, by slightly larger spectral accelerations than records EQ4 and EQ5. Recalling the considerations made with reference to Fig. 9, it therefore becomes clear how the wall shears can be of similar magnitude despite the differences in ductility demand. Considering this, a new, alternative simplified capacity design possibility that is intensity dependent, but does not depend on the ductility demand, will be




(a) (b) (c)

Fig. 10 (a) Average spectra (acceleration spectra top and displacement spectra bottom) of five scaled records used in NLTH analyses of a 16 storey frame-wall structure, (b) maximum displacements, (c) maximum wall shears recorded in the NLTH analyses

outlined in Section 4.2.

Before discussing new possibilities for simplified capacity design, it is important to point out that another valid capacity design approach is in fact to undertake non-linear time-history analyses. To use NLTH analyses (and this option may become more feasible as computing power increases in the future), a computer model of the structure is built in which plastic hinge strengths correspond to overstrength material properties (e.g. using Eqs. (1) and (2)). Only the plastic hinge locations are modeled as being non-linear, and those elements and actions that are not part of the intended plastic mechanism are modeled with elastic stiffness properties. The non-linear time-history analyses are then conducted and since the analyses are fully dynamic and include overstrength effects, the maximum elastic forces recorded throughout the structure are taken as the capacity design forces. Such an approach is, in fact, that commonly used to establish empirical dynamic amplification factors of the type proposed by Paulay and Goodsir (1986) and Priestley *et al.* (2007) described earlier. Clearly, one concern with amplification factors, obtained empirically from NLTH analyses, relates to the uncertainty associated with the NLTH analyses themselves. Modeling and analysis techniques for NLTH analysis of structures are still evolving, and considering the differences in NLTH analysis predictions observed in typical blind-prediction competitions (see, for example, Restrepo *et al.* 2010, E-defense 2009), one can conclude that the uncertainties of NLTH analyses can be large.

One current uncertainty for the NLTH analysis of RC walls relates to the effects that shear deformations have on the seismic response. The shear stiffness of RC walls is recognised to decrease as inelastic flexural deformations develop (Dazio 2000, Beyer *et al.* 2008), and this is not captured by common NLTH analysis approaches in which a linear elastic shear stiffness is assumed.

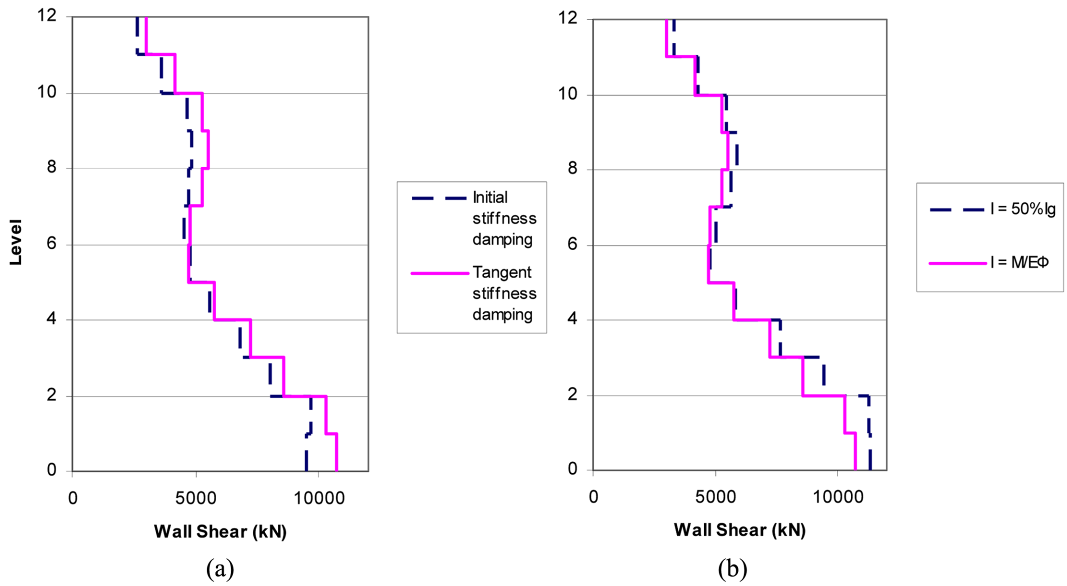



Fig. 11 Change in wall maximum shears due to (a) use of initial stiffness Rayleigh damping model versus tangent stiffness Rayleigh damping model, (b) use of $I_{cr}$ = 50% $I_{gr}$ versus $I_{cr}$ = M/$\phi_y$ (adapted from Sullivan *et al*. 2006)

It is anticipated that changes in shear stiffness during the seismic response could affect the manner in which shears are distributed to frames and walls of dual systems, as well as higher mode response. Future research should therefore investigate how significant this uncertainty is for the capacity design forces of frame-wall structures.

In order to briefly illustrate how some other modeling and analysis decisions can affect wall shears forces in frame-wall structures, Fig. 11 presents the average maximum shears recorded by Sullivan *et al*. (2006) in a 12 storey frame-wall structure. The effect of two variables is highlighted in the figure: (i) the effect of selecting a traditional initial-stiffness based Rayleigh elastic damping model versus an arguably (refer Priestley *et al*. 2007) improved tangent-stiffness based Rayleigh model, and (ii) the effect of modeling walls with cracked stiffness approximated by M/E$\phi_y$ (an approach recommended by Priestley *et al*. 2007) versus 50% the gross section stiffness (the approach recommended by the EC8). The figure illustrates that changing either variable can alter the predicted wall shears by between 10% to 15%. This highlights the fact that the performance of empirical capacity design approaches will depend greatly on the quality of the modeling and analysis decisions used in their development. To overcome this issue it is recommended that, in addition to improving NLTH modeling and analysis techniques, capacity design methods be developed wherever possible using rational mechanics based concepts instead of being purely empirical in nature.

### *4.2 A new possibility and areas for future research*

Considering the limitations of current capacity design methods, a new possibility for the simplified prediction of capacity design wall forces of frame-wall structures is presented in this section. Similar to the simplified procedures proposed by Paulay and Goodsir (1986) and Priestley *et al.* (2007), the new approach, illustrated in Fig. 12, specifies bi-linear shear and moment envelopes for the walls. The wall base shear is obtained from the sum of two components; the wall base shear associated with the overstrength first mode forces, $\phi^o V_{b1}$, plus the wall base shear due to higher mode actions, $V_{bh}$, as shown in Eq. (10)

$$V_{b,wall} = \phi^o V_{b1} + V_{bh} \tag{10}$$

Where it is proposed that $V_{bh}$ be approximated as

$$V_{bh} = PGA \times (M_{total} - M_{eff}) \times \frac{3.26}{1+\beta_{fr}} e^{-0.04f} \leq 2.5PGA \times (M_{total} - M_{eff}) \tag{11}$$

Where $PGA$ is the peak ground acceleration, $M_{total}$ is the total seismic mass and $M_{eff}$ the effective (participating) mass of the fundamental mode for the excitation direction under consideration, $\beta_{fr}$ is the proportion of total overturning resisted by the frames in the frame-wall structure, and $f$ is a period factor (with units of s) obtained from Eq. (12).

$$f = H_n^2 \sqrt{\frac{M_{total}}{H_n \sum EI_{vert}}} \tag{12}$$

Where $H_n$ is the total height of the building, $E$ is the concrete modulus of elasticity and $I_{vert}$ is the second moment of inertia of the walls and columns, such that the sum of the $EI_{vert}$ is the sum of the flexural stiffness of the vertical structural elements. Readers will note the similarity between Eq. (12) and period expressions for uniform cantilever beams (see, for example, Chopra 2000).

As such, the base shear expression aims to include the seismic intensity through the $PGA$ term, the effect of different frame-wall strength proportions through the $\beta_{fr}$ term, and the effect of higher modes using mass and period terms. Note that the limit of $2.5PGA(M_{total} - M_{eff})$ in Eq. (11) is considered a conservative but easily computed upper bound for the higher mode shears and future research could provide better indications for this limit. The constants of 3.26 and 0.04 were

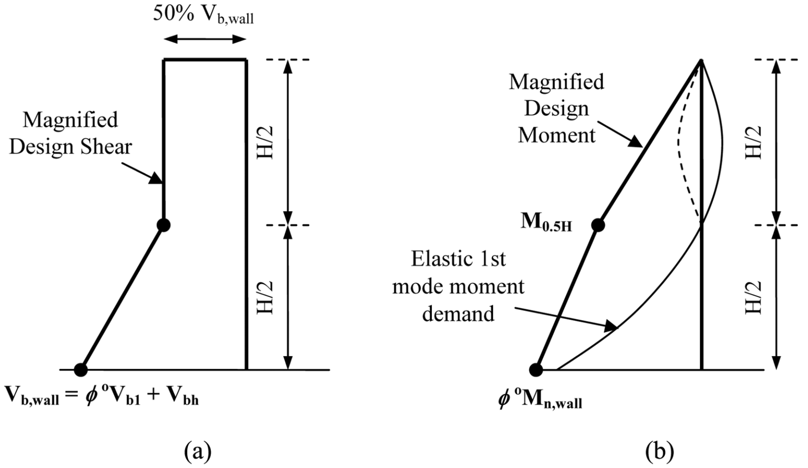



Fig. 12 Alternative simplified capacity design profiles for wall actions in RC frame-wall structures

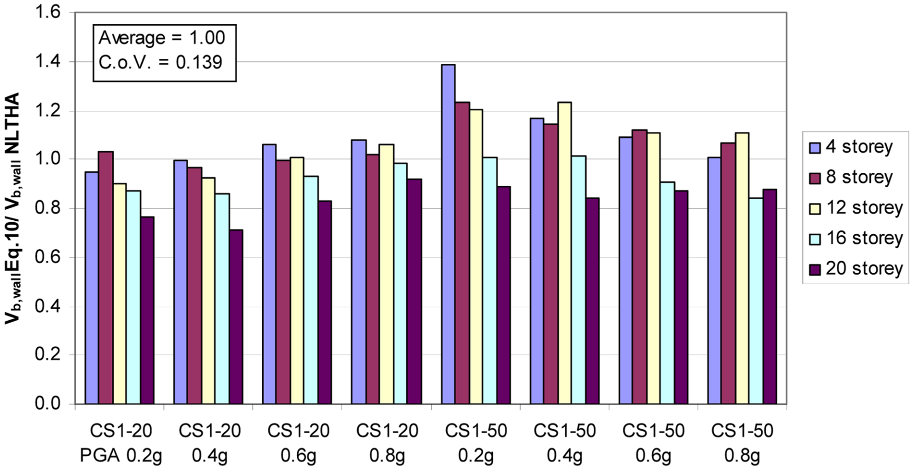



Fig. 13 Ratio of wall base shear approximated by Eq. (10) to average peak values obtained from non-linear time-history analyses at different seismic intensities

obtained by calibrating the expression with the results of 200 NLTH analyses (5 accelerograms, 10 buildings and 4 different levels of PGA) conducted by Sullivan *et al*. (2006) on frame-wall buildings ranging between 4 and 20 storeys in height. The ratio of the wall base shear predicted from Eq. (10) to the NLTH analysis base shear (averaged for the five ground motions) for each of the case study buildings and ground motion intensity levels is presented in Fig. 13.

The results of Fig. 13 indicate that that the approach looks to be very promising as a coefficient of variation of only 0.139 is obtained for the ratio of the predicted to recorded wall base shears. However, while the constant of 3.26 in Eq. (11) has led to an average prediction ratio of 1.0, one can note that a larger constant could be appropriate to ensure that the average prediction ratio is greater than 1.0 for the majority of the cases studied. In addition, as stated in the previous section, NLTH analyses are uncertain and therefore future research may indicate that the constants of 3.26 and 0.04 need to be modified to match the results of more accurate analyses.

From the results of the same set of NLTH analyses run by Sullivan *et al*. (2006) it was found that the mid-height wall shear to the base shear had an average ratio of 0.52 and a coefficient of variation of 0.22. Consequently, the wall shears over the top half of the building may be approximated as being 50%$V_{b,wall}$, as shown in Fig. 12(a).

An estimate of the mid-height moment, $M_{0.5H}$, in the walls of frame-wall structures may be obtained through use of Eq. (13).

$$M_{0.5H} = \phi^o M_{0.5H,1} + \frac{V_{bh} H_n \beta_{fr}}{2} \tag{13}$$

Where $V_{bh}$ is the higher mode base shear component from Eq. (11), $H_n$ is the total building height and $\beta_{fr}$ is again the proportion of total overturning resisted by the frames in the frame-wall structure. $\phi^o M_{0.5H,1}$ is the moment at mid-height of the wall associated with peak response of the fundamental mode. The moment should therefore be evaluated for a plastic mechanism that forms under a 1st mode distribution of equivalent lateral forces, with plastic hinge strengths corresponding to overstrength values. The moment can be obtained either from pushover analyses or through simplified hand calculations using the recommendations of Sullivan *et al*. (2006). Note that Eqs. (11) and (13) have been developed and calibrated for $\beta_{fr}$ values in the range of 0.20 to 0.70, and it is expected that alternative capacity design expressions would be required for $\beta_{fr}$ values outside of this range.

The ratio of the mid-height wall moment predicted from Eq. (13) to the NLTH analysis mid-height wall moment (averaged for the five ground motions) for each of the case study buildings and ground motion intensity levels examined by Sullivan *et al*. (2006) is presented in Fig. 14. It is apparent that the average ratio is 0.79, suggesting that the expression is generally non-conservative. However, this was deemed acceptable since some flexural yielding over the upper levels of walls is not considered detrimental to the seismic response. The concept of mid-height wall yielding might at first appear to go against a traditional capacity design philosophy, which aims to ensure that plastic hinges only form in specified zones associated with a plastic mechanism associated with 1st mode actions. However, the idea of permitting the development of ductile inelastic mechanisms for other modes of vibration is not completely at odds with the capacity design advocated by Park and Paulay (1975) (described at the start of the paper), provided that the designer suitably details the structure with the higher mode response in mind. In fact, research by Sullivan *et al*. (2006) for RC frame-wall structures, Panagiotou and Restrepo (2009) for RC wall structures, and Weibe and Christopolous (2009) for rocking pre-cast RC wall structures, suggests that permitting the development of limited curvature ductility demands at the mid-height of walls may not be detrimental to the response and may significantly ease capacity design requirements. However, future research should look to better establish how much mid-height moment reduction is acceptable and thereby confirm whether the average ratio of 0.79 ratio obtained from Eq. (13) is appropriate. Note that the approach offered by Eq. (13) looks to be promising as a coefficient of variation (CoV) in the base shear ratio of 0.173 was obtained, but future research could also identify simple ways of reducing the CoV. Furthermore, as stated in the previous section, NLTH analyses are uncertain and therefore future research may certainly indicate that the higher mode moment term in Eq. (13) should be modified.

Overall, therefore, this alternative capacity design approach is relatively simple and appears to be

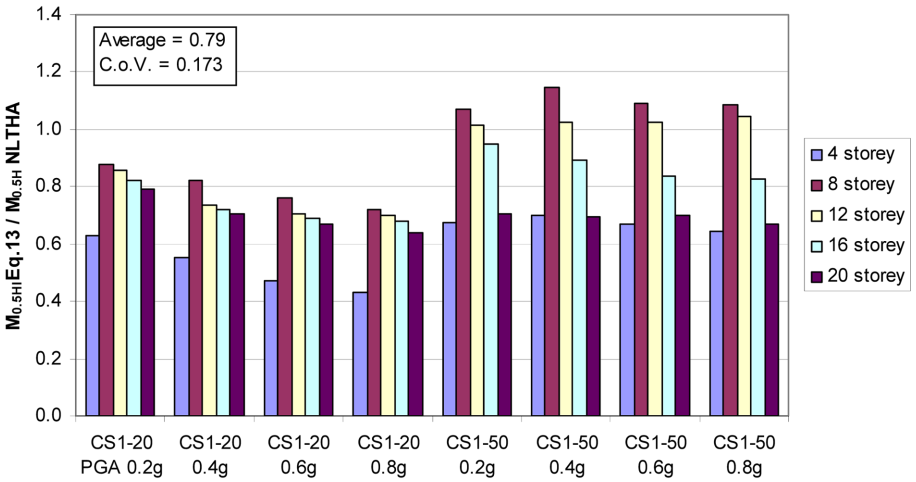



Fig. 14 Ratio of mid-height wall moment approximated by Eq. (13) to average peak mid-height moment values obtained from NLTH analyses at different seismic intensities

effective based on the results of NLTH analyses conducted by Sullivan *et al*. (2006). However, it is emphasised that the approach should be considered as being preliminary and future research should verify and refine the approach, ideally examining a larger database of buildings and making reference to experimental evidence. One particular aspect for future research, for instance, should investigate whether the shape of the acceleration spectrum should be included in the capacity design formulations. The equations presented in this work include constants that have been calibrated from the results of NLTH analyses for ground motions spectrum compatible with the EC8 spectrum for soil type C. As the ratio of the PGA to the spectral acceleration plateau (and the width of the spectral acceleration plateau) is typically dependent on the soil type, it is apparent that capacity design procedures should also take this into account.

## 5. Conclusions

This paper has highlighted and reviewed some important considerations for the capacity design of RC frame-wall structures. The work has emphasized the fact that capacity design forces will be affected by material overstrength, higher mode effects and secondary loadpaths associated with the 3-dimensional structural response. In order to ensure that RC frame-wall structures perform well it has been shown that the prediction of the peak shears and moments that develop in the walls is particularly important and unfortunately very challenging. Through a review of NLTH analysis results, it has been shown that there are a number of serious limitations with capacity design procedures included in current codes. The basis and potential of alternative capacity design procedures available in the literature has been reviewed, and a new simplified capacity design possibility has been proposed. The new procedure ensures that the capacity design forces are related to the seismic intensity, higher mode periods and frame-wall strength proportions. Comparison with the results of 200 NLTH analyses on frame-wall structures ranging from 4 to 20 storeys suggest that the new method is able to predict wall base shears and mid-height wall moments reliably. Efforts have been made to highlight the uncertainty with capacity design procedures and emphasise the need for future research on the subject.

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