COMPDYN 2011
III ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

SEISMIC SHEAR DEMAND ON RC STRUCTURAL WALLS: REVIEW AND BIBLIOGRAPHY

Avigdor Rutenberg

Technion – Israel Institute of Technology
Haifa 32000 Israel
avrut@technion.ac.il

Keywords: Earthquake engineering, structural walls & wall frames, higher modes, shear demand.

Abstract: Effect of higher vibration modes on the seismic shear demand of reinforced concrete cantilever walls has been studied since the 1970's. The shear amplification becomes more important with increasing fundamental period (tall buildings) and increasing ductility demand (R or q factors). Yet, studying the relevant recommendations of structural engineering researchers and provisions of various seismic codes reveals that there is no consensus regarding the extent of shear amplification and of the inter-wall distribution of shear demand in structural systems comprising walls of different lengths.

Paper presents the available formulas for predicting shear amplification in ductile walls and dual systems (wall-frames), as well as techniques for estimating the shear distribution among interconnected unequal walls. The consideration, or absence, of these effects by leading seismic code provisions is also noted.

One effect that impacts the shear amplification is shear cracking mainly in the plastic hinge zone of the wall near the base resulting in very low effective shear stiffness. Also, deliberately allowing development of plastic hinging along the wall height (contrary to standard code requirements) has been suggested. These appear to lead to appreciably lower shear amplification than previously predicted.

Finally, an extensive bibliography (circa 100 refs.) is provided.

1 INTRODUCTION

This paper reviews the literature on the seismic shear demand on RC cantilever walls. It is mainly concerned with shear amplification due to higher vibration modes, and it also reports on the post-yield redistribution of shear demand among the walls in systems comprising several walls of different lengths. Amplification of wall shear due to higher vibration modes in the *elastic range* was not widely recognized by structural engineers until the early 1970's, although it was, or at least should have been, a known phenomenon to users of modal analysis for the seismic design of RC walls in tall buildings. However, the larger shear magnification in *yielding* flexural walls due to higher modes was first observed by New Zealand consulting engineers, which led to the 1975 pioneering paper by Blakeley et al [9]. The problem was also studied by researchers at the Portland Cement Association, and these resulted in a series of PCA reports [21, 23] and papers [22, 24] by Derecho and coworkers. The amplification tables proposed by Blakeley et al [9], the charts by Derecho et al [23], as well as later amplification expressions or charts (e.g. [29, 36, 37, 79, 81, 88, 95, 96]; and for wall-frames: [4, 38, 39, 43, 56, 81, 89]) were mainly based on results of nonlinear response history parametric studies using suites of accelerograms. Obviously, the result depended on the choice of these suites. Theoretically oriented general formulation for the shear amplification was given by Keintzel [27, 50, 51]. Extensions were proposed by Priestley [79, 80, 81], and for wall-frames by Sullivan et al [100]. Very recently Fischinger et al [32] presented a correction to Keintzel's formula, and Pennucci et al [76] modification thereof.

Shear amplification becomes more important with increasing fundamental period (tall buildings) and increasing ductility demand (R or q factors). Yet, studying the relevant recommendations of structural engineering researchers and provisions of various seismic codes reveals that there is no consensus regarding the extent of shear amplification, its height-wise variations, and the inter-wall shear demand distribution when the structural system consists of interconnected wall of different lengths.

This paper is concerned only with *shear demand* amplification on structural walls due to the effects of higher modes, but not with moment magnification along the height, which derives from the same source. Issues related to the shear capacity and post-yield shear flexure interaction are beyond the scope of the present study.

The phenomenon is described first, then available procedures to predict the amplification are presented, and relevant code provisions are briefly discussed. Shear amplification in dual or wall-frame structures is also included in the review, and post-elastic shear demand redistribution among the walls in systems with walls of unequal length is considered. The paper closes with a discussion including some unresolved issues.

The structural engineering communities in North America and in many other countries have been slow in recognizing the need to amplify shear demand, if at all. The objection of many researchers and engineers to the rather large amplification in the nonlinear range obtained from numerical simulations may perhaps explain the plethora of publications as essays in persuasion, manifested by the quite long bibliography on this rather limited problem at the end of this paper.

2 THE SHEAR AMPLIFICATION PHENOMENON

In the *linear range* the amplification of shear demand in flexural walls due to higher vibration modes has been routinely accounted for by means of response spectrum analysis. This matter is well covered in standard textbooks. Figure 1, taken from Chopra [18], shows the base shear V_b normalized by the total weight W in buildings analyzed by the modal response spectrum technique for a typical design spectrum. It can be seen that for a flexural cantilever wall ($\rho = 0$, ρ = beam to column stiffness ratio) the base shear increases appreciably with increasing natural period due the effect of higher modes compared with the design spectrum. This fact explains the requirement in many seismic codes (e.g. [16]) to permit use of the equivalent lateral force procedure only for buildings with $T_1 < \max(4T_c, 2.0 \text{ sec})$, T_c = corner period to the velocity related spectral acceleration region. As can be seen, in frame structures ($\rho >> 0$) the shear amplification due to higher modes is appreciably much smaller. The main reasons for this difference are: (1) the combined effective mass of the higher modes in flexural walls is circa 35% of the total mass vs. circa 25% in frames ; (2) the period ratios of the 1st mode to the higher ones is much larger for flexural walls (e.g. $1^{st}/2^{nd} \approx 6$ vs. ≈ 3 for ideal frames). Hence, when T_1 of a flexural wall is in the velocity spectral region, the higher modes - the spectral accelerations of which are usually located at the high acceleration plateau – have a larger effect on the shear than in frames. This effect is even more pronounced when T_1 is longer so that $S_a(T_2)$ and even $S_a(T_3)$ are in the velocity spectral region. Indeed, the main effect of the higher modes is due to the 2nd mode (e.g. [50, 66, 68]) and, to a much lesser extent, to the 3rd. mode. Evidently, the shape of the spectrum also strongly affects the extent of amplification. The reader is referred to Chopra [18] and Humar & Mahgoub [41] for detailed consideration of the phenomenon. The way seismic codes consider this effect is described subsequently.

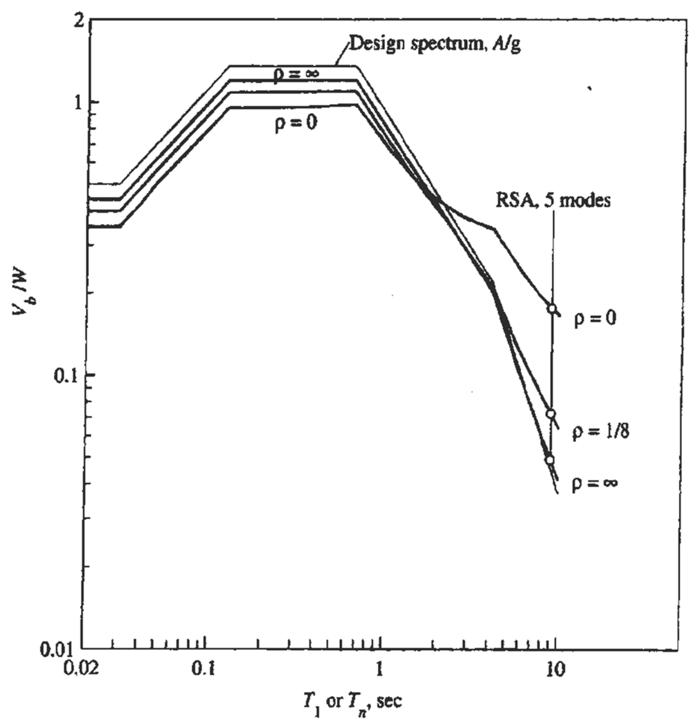

Fig. 1: Normalized base shear V_b/W for 3 values of ρ; for walls $\rho = 0$ (Chopra 2001).

As already noted, the shear amplification due to higher modes in the *nonlinear range* has attracted considerable interest since the Blakeley et al 1975 paper [9]. They showed, by means of nonlinear response history analysis (NLRHA), that shear demand increases with natural period and falling damping ratio. Similar results were obtained by Derecho et al [21], Keintzel [49] and Kabeyasawa [43, 44, 45], who also demonstrated that shear demand increases with ductility demand. The following explanation for the latter effect was given by Paulay & Priestley [75]. At some instants during the response the higher modes combine with the 1st mode in a height-wise force distribution similar to the 2nd mode. The higher mode shapes of hinged and fixed base cantilever are quite similar, as shown in Fig. 2, suggesting that the formation of a plastic hinge at the wall base does not significantly affect the higher modes. Hence, while the plastic hinge reduces the 1st mode shear response to the level commensurate with its flexural capacity, the response of higher modes remains circa their elastic level. In other words, the shear response can be approximated as a superposition of the 1st mode shear reduced by the force reduction factor R (q in Europe), while the higher modes' response remains elastic, thereby amplifying their shear demand relative to that due to the first mode. Note that due to the plastic hinge formation and unloading/reloading softening the periods of higher modes are also elongated, hence amplification is likely to be somewhat lowered. Similar conclusions were drawn from the response of a flexural wall with substantially reduced stiffness at its base – the plastic hinge [66]. This approach forms the basis for several analysis procedures developed to predict shear amplification, as described subsequently. As in the linear range, the base shear amplification increases with increasing natural period since higher modes become more important, and as a result the shear resultant moves downwards. Also, as in the linear range, the spectral shape plays an important role.

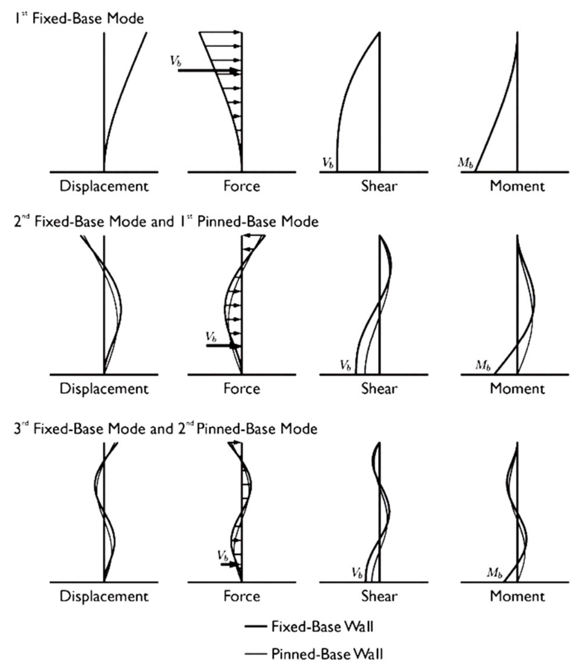

Fig. 2: Fixed base and hinged cantilever mode shapes (Wiebe & Christopoulos 2009).

It is interesting to note that base shear amplification in dual wall frame systems (WFs) is lower than in structural walls. As already observed the period ratios in frames are much lower, and their 1st mode mass participation is higher. Also, the shear demand in frames depends mainly on moment demand. Indeed, shear amplification in WFs strongly depends on the wall-to-frame shear ratio. More details on WFs are given in Section 5.

3 PROCEDURES TO PREDICT HIGHER MODES SHEAR AMPLIFICATION

The work of Blakeley et al [9] brought about changes in the provisions of the New Zealand seismic code [63] and in European model codes [13, 14]. However, the latter provisions explicitly restricted the magnifications to the equivalent lateral force (or static) procedure. This turned out to be unfortunate since the corollary was that amplification was understood not to be required when using modal analysis (as, e.g. the Israeli seismic code [97] without Amendment 3, is interpreted). It also led engineers to overlook the fact that a major part of the shear amplification due to higher modes takes place in the post-elastic range. This attitude still persists also in the National Building Code of Canada NBCC-2005 [61], which only accounts for linear multi-mode shear amplification. However, the present Canadian Concrete code CSA-A23.3-04 [19] requires considering nonlinear shear amplification, yet gives no guidance to that effect (see also [10]), although the commentary to the 1994 version of that code suggested adopting the New Zealand shear amplification formulas. Note that in the provisions and the proposals described subsequently it was tacitly assumed that the plastic hinge could form only at the wall base, and that the prescribed moment envelope took care of that.

The amplified shear V_a is given in in terms of the base shear V_d from analysis, as follows:

$$V_a = \omega_v V_d \qquad (1)$$

In the New Zealand code:

$$\omega_v = \begin{cases} 0.9 + \dfrac{n}{10} & n \le 6 \\ 1.3 + \dfrac{n}{30} \le 1.8 & n > 6 \end{cases}$$

. And in CEB:

$$\omega_v = \begin{cases} 0.9 + \dfrac{n}{10} & n \le 5 \\ 1.2 + \dfrac{n}{25} \le 1.8 & n > 5 \end{cases}$$

As can be seen the NZ code is somewhat more conservative than CEB for $5< n< 15$.

In spite of their widespread use there are several problems with these formulas. First, the only governing parameter is the number of storeys, which is a crude proxy for the natural period. Also, the upper bound amplification is arbitrarily set at 1.8 for a 15 storey building. However, perhaps a more serious limitation is their independence of the excitation level, which affects the shear demand appreciably, and of the shape of the applicable response spectrum. Note also that, by today's standards, the Blakeley et al study [9] was based on a very limited data set: only 5 accelerograms, two of which were artificial. Also, bilinear moment-curvature hysteresis was assumed, while many modern studies use the modified Takeda model, considered to be more realistic. On the other hand, the bilinear model is somewhat more conservative [62].

Shear amplification design charts were presented by Derecho et al, e.g., [22, 24]. These are based on a parametric study on structural walls, 10 to 40 storey high, using the modified Takeda model for the NLRHA carried out for 6 earthquake records. They presented shear amplification relative to the base shear as evaluated by the then governing UBC provisions vs. fundamental natural period for several values of rotational ductility demand μ_r, as shown in Fig.3. Since the set of records was normalized for a given Housner spectral intensity (SI= 1.5) a correction factor (practically proportional) was provided for different SIs. One difficulty in applying this approach is its dependence on μ_r rather than on the strength reduction factor R, requiring making assumptions regarding the relation between these two parameters.

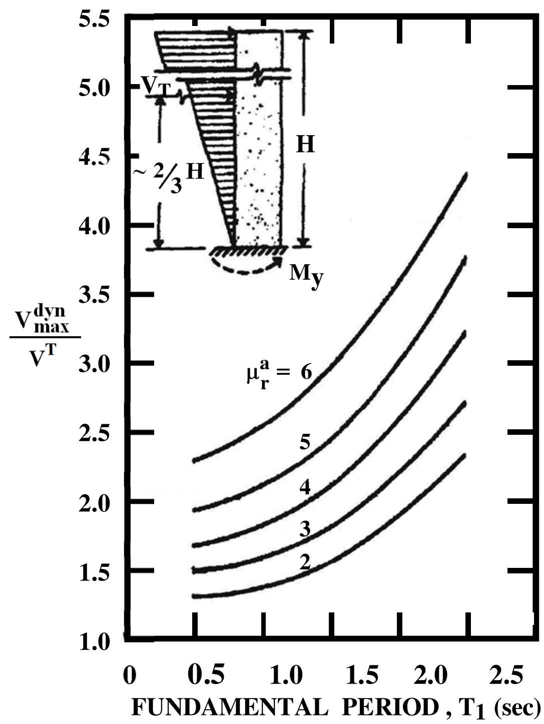

Fig. 3: Base shear demand amplification for several values of rotational ductility (Derecho et al 1981)

Based on parametric NLRHA several investigators proposed simple expressions to estimate the maximum base shear V_a by decomposing it into the 1st mode and the higher modes responses. Kabeyasawa (e.g. [43], see also [4]) gave the following expression for wall frames:

$$V_a = V_d + D_m W A_e \tag{2}$$

in which V_d = base shear of an inverted triangular loading, W = total weight, A_e = peak ground acceleration and D_m is a given factor increasing with the number of storeys (i.e., period). Based on shake-table tests Eberhard & Sozen [25] proposed $D_m = 0.3$ for 9 and 10 storey wall frames, in agreement with Kabeyasawa's results. The actual amplification in wall frames depends to a large extent on the base shear share carried by the wall. Hence the predictive ability of formulas such as Eqn. 2 is rather limited.

A similar formula for isolated walls was presented by Ghosh et al [36, 37] based on a parametric study similar to that of Derecho et al [23]:

$$V_a = M_y/0.67H + D_m W A_e \tag{3}$$

in which M_y = the yield moment at base and H = wall height. The advantage of this form is the explicit statement that the base shear is that at moment capacity level rather than at the design shear obtained from analysis, which is usually lower. For isolated walls the study by Seneviratna [95, 96], described subsequently, confirmed the equation *format* of Kabeyasawa and of Ghosh except for the short period range and for low values of R.

In 1988 Keintzel [27] proposed a simple expression for shear amplification by explicitly assuming that the 1st mode shear is limited by the flexural capacity of the wall at the base, and that the higher modes respond linearly. This formula was eventually incorporated into Eurocode 8 [15, 16]:

$$\omega_v = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q \tag{4}$$

ω_v = 1st mode shear amplification factor – but see next paragraph
q = R = strength reduction (behaviour) factor
M_{Ed} = design bending moment at wall base
M_{Rd} = design flexural resistance at wall base
γ_{Rd} = overstrength factor due to steel strain-hardening
T_1 = fundamental period
T_c = higher period corner of the constant acceleration plateau (Fig. 2)
S_e (T) = Spectral ordinate (Fig. 2).

This is a very useful formula since it considers most of the parameters governing shear amplification in structural walls. It does not require exact evaluation of the 2nd mode natural period, and is practically spectrum-shape independent, in contradistinction to parametrically evaluated amplification formulas or plots, which are not (e.g. [79, 88, 95, 96]). It also accounts for the two sources of overstrength. However, it is too conservative for very tall buildings, i.e., those with T_2 in the velocity spectral region, and it is sensitive to the calculated value of the 1st mode spectral acceleration S_a, which for RC walls depends on design assumptions. It is believed that S_a at the natural period based on yield displacement should be the appropriate one (e.g. [68]). In view of the extensive use of this formula (EC8 is now applied in circa 20 countries) some background information may be in order. Based on parametric studies Keintzel assumed that the SRSS modal combination can also be applied in the post-elastic range, and also that the contributions of the first two modes are important, hence:

$$V_a = \sqrt{\left(V_{Ed,1}\right)^2 + \left(qV_{Ed,2}\right)^2} \tag{5}$$

in which $V_{Ed,i}$ are the 1st and 2nd mode shear forces from analysis. He also assumed that the contribution of the 2nd mode is 0.1 S_e (T_1)/ S_e (T_2), in which $0.1 \approx 0.32^2 \approx$ the 2nd to 1st mode mass contribution ratio squared. Considering that design flexural overstrength (M_{Rd}/M_{Ed}) affects only the 1st mode, and dividing by $V_{Ed,1}$, Eqn.4 is obtained. This development shows, as made clear by Fischinger et al [31, 32], that the amplification ω_v should factor only the 1st mode shear $V_{Ed,1}$. However, in EC8 ω_v amplifies V_{Ed} as evaluated either by the equivalent lateral force procedure (usually inversely triangular loading based on the total building weight) or by modal analysis, and, without making the distinction, evidently leads to different results. Keintzel concluded that ω_v should be bounded by q (R), but this is too low considering that the elastic bound should be based on the SRSS shear - not the 1st mode one. Note also that the present EC8 restricts the application of Eqn. 4 to walls designed for high ductility (DC-H), while for medium ductility walls (DC-M) ω_v = 1.5. This has been found to be too low [31, 32, 88], suggesting that Eqn.4 should be applied also to DC-M walls, as indeed was the case in past versions of EC8.

Keintzel [50] already observed that Eqn. 4 is a simplification of an expression he considered to be more accurate. A different correction, which is based on an extensive parametric study using the Modified Takeda model and assuming hinge formation only at wall base, was proposed by Fischinger et al [32]. This correction affects the 2nd term in Eqn.4 when M_{Rd}/M_{Ed} is large, hence is of limited practical use.

A natural extension of Keintzel's formula proposed by Priestley [79], which can also predict the shear demand along the height of the wall and not only at base, is the following:

$$V_a = \sqrt{\left(V^*_{Ed,1}\right)^2 + \mu^2\left\{\left(V^*_{Ed,2}\right)^2 + \left(V^*_{Ed,3}\right)^2 + \cdots\right\}} \tag{6}$$

in which$V^*_{Ed,1}$ is the overstrength factored 1st mode design shear and μ is the displacement ductility demand. Priestley [79] reported satisfactory agreement with NLRHA results for a wide range of wall heights.

Another modification of Keintzel's formula was recently proposed by Pennucci et al [76]. The basic assumption therein is that the total response can be obtained as the absolute sum of the 1st mode shear reduced by q (denoted R_m), and of the higher modes SRSS response, which is reduced by another factor (R_p). It has similar format as Eqns. 2 and 3, and is given by:

$$V_a = \frac{V_{Ed,1}}{R_m} + \frac{\sqrt{\left(V_{pin,2}\right)^2 + \left(V_{pin,3}\right)^2 \ldots..}}{R_p} \tag{7}$$

in which $V_{Ed,1}$ and $V_{pin,i}$ are the modal shear forces at their *elastic level*, and the subscript *pin* denotes pin-based modal shear response. Based on parametric studies Pennucci [76] found that, as expected, the higher modes reduction factor R_p is related to R_m: large when R_m is low, and becoming smaller with increasing R_m, asymptotically reaching 1.0, i.e. linear behaviour. A considerable improvement over Eqn. 6, particularly for long periods and large strength reduction factors, and very satisfactory agreement with NLRHA results along the building height were reported. Hence, it appears that the approach can make better allowance for the period elongation due to base hinging and for the lowered stiffness of the unloading/reloading hysteresis loop (Modified Takeda). Yet, since R_p is evaluated parametrically, it is to some extnt spectrum shape dependent.

A simple formula also in the format of Eqns. 2 and 3 was proposed by Panagiotou & Restrepo [66]. It is based on the observation that the amplification can practically be accounted for by adding a factored 2nd mode shear envelope to that of the 1st mode shear at base yield. The shear demand at storey i can then be approximated as:

$$V_a = V^*_{Ed,1} + q\rho_{12}\left(\sum_{j=i}^{n}\Gamma_2\Phi_2^j\right)mSa_2 \tag{8}$$

in which ρ_{12} is a factor to be determined, and $\Gamma_2\Phi_2^i$ = min[4.28h_i/H, 0.6; -2.5h_i/H + 1.85], Γ_2 = 2nd mode participation factor, Φ_2^i = mode shape ordinate at floor i , h_i = floor height from base, H = total wall height, Sa_2 = 2nd mode spectral acceleration. Good agreement with NLRHA results for near-fault records was reported. Note, however, that the application of the formula to other cases depends on ρ_{12}, which was provided only for a small number of pulse-type near-fault ground motions.

The amplification formulas or charts given subsequently are parametrically based; hence they are only applicable to spectra having shapes that are similar to the mean spectra of the suite of records forming the data base for the NLRHA. Also, they are sensitive to the chosen damping ratios.

Base shear demand amplification charts based on 15 records for wall structures having elastic–plastic moment-rotation hysteresis designed using SRSS load pattern were provided by Seneviratna [96]. Figure 4 presents the mean amplification, defined as the ratio of the maximum base shear of the multi-degree of freedom V_b(MDOF) to the single degree of freedom (SDOF) base shear $F_y(\mu)$, vs. T_1. These are displayed for several values of μ(SDOF) - the ductility ratio of the SDOF system. It can be seen the amplification increases with T_1 and with μ(SDOF). As already observed, the amplification w.r.t the SRSS loading would be smaller since higher modes also amplify the shear in the linear range.

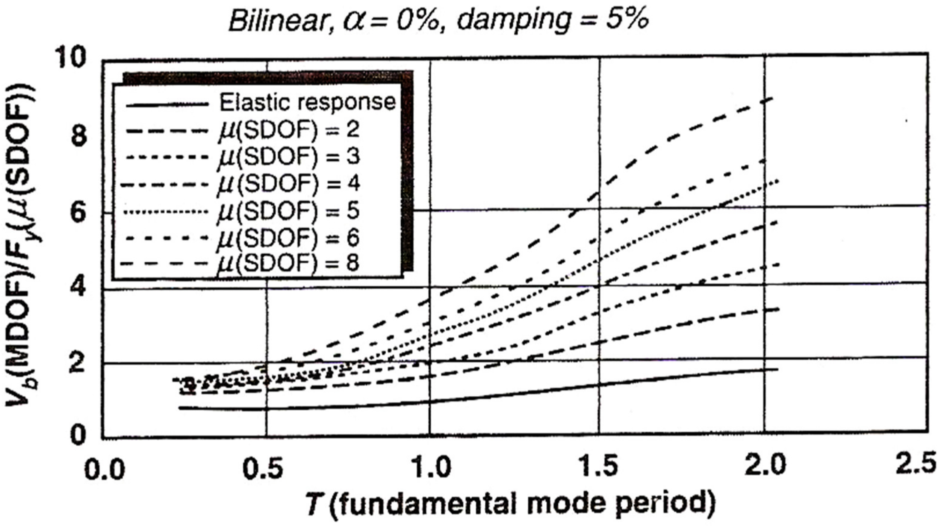

Fig.4: Base shear demand amplification for several values of displacement ductility (Seneviratna 1995)

The amplification formula provided by Priestley, [79] was based on NLRHA results for walls assuming Modified Takeda hysteresis. It has a very simple form, and is given in terms of the displacement ductility demand μ, and T_1:

$$\omega_v = 1 + \mu B_{(T)}/\Phi; \qquad B_{(T)} = 0.067 + 0.4(T_1 - 0.5) \leq 1.15. \qquad (9)$$

in which Φ is the flexural overstrength. Evidently, in routine analysis, μ needs to be evaluated, unless the equal displacement approximation or another conversion formula is invoked.

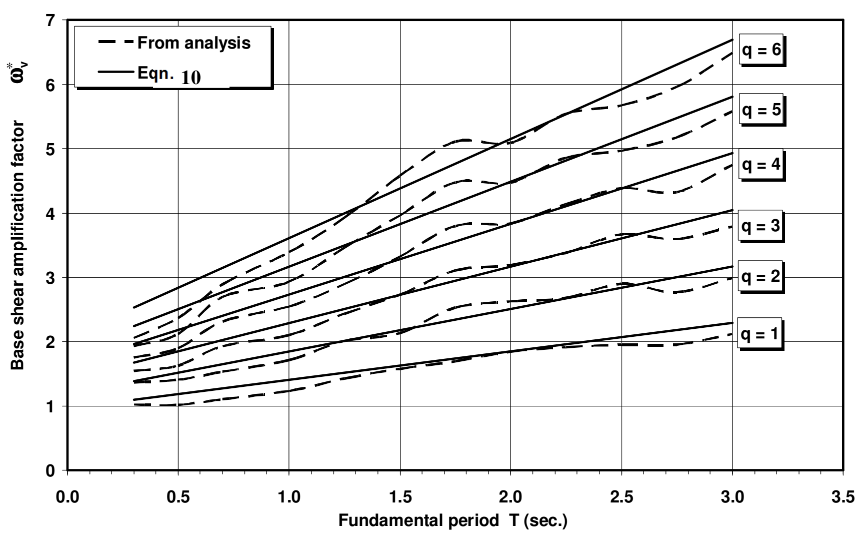

Fig. 5: Eqn. 10 - Base shear demand amplification for several values of strength reduction factor q (=R) (Rutenberg & Nsieri 2006)

A somewhat longer expression given in terms of q (or R), not incorporating flexural overstrength, was presented by Rutenberg & Nsieri [88]. It is given by:

$V_a = [0.75 + 0.22(T_1 + q + T_1 q] V_d$ (10)

in which V_d is the base shear due to a triangularly distributed base shear. Eqn. 10 is shown in Fig. 5 by the straight lines. Elastic-plastic response and 5% tangent stiffness damping in the 1st and 5th modes were assumed. Note that 5% damping is now believed to be too high for very tall buildings (e.g. [92]), for which the major sources of damping have a lower effect.

4 STOREY SHEAR DEMAND

Whereas, as noted, Eqns. 6, 7 and 8 are able to predict the shear demand for the full wall height, this is not the case with the other design equations listed above, that apply only to the base shear. This is because in many cases the ω_v normalized SRSS storey shears overestimate the expected shear values for large wall portions of long period structures, as shown in Fig. 6.

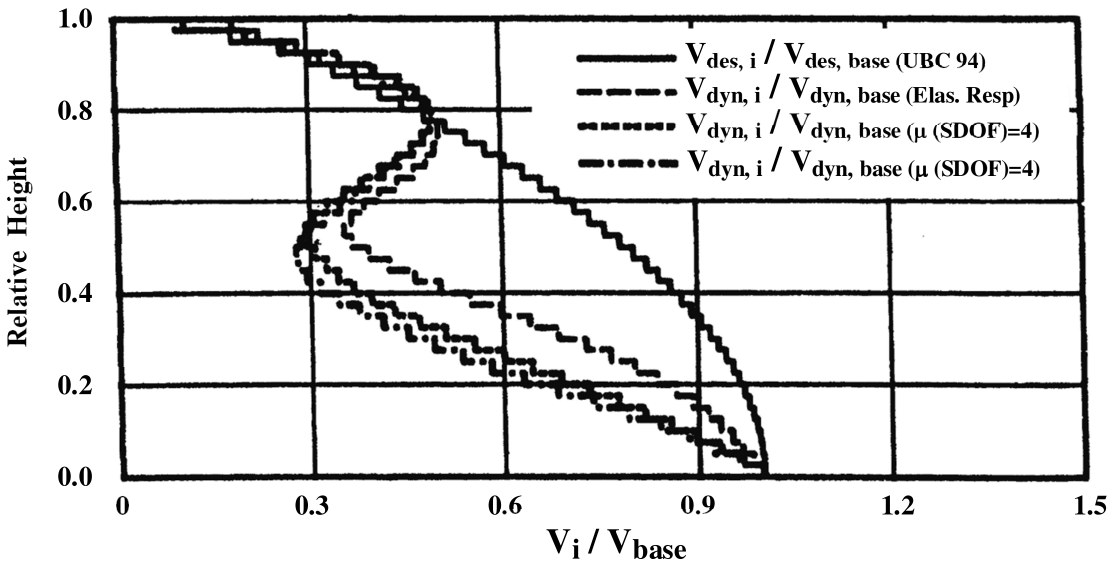

Fig.6: 40-storey wall shear envelopes: code loading design pattern, T = 2.05s: mean for $S_{1,a}$ records, bilinear, $\alpha = 0\%$, damping 5% (Seneviratna & Krawinkler 1997)

Apparently, this problem was first addressed by Iqbal & Derecho [42] who provided a wall height shear demand envelope that has not changed appreciably since. A design envelope as function of the fundamental period T_1 is shown in Fig. 7, in which ξ is given by:

$\xi = 1 - 0.3T_1 \geq 0.5$ (11)

Note that Eqn.11 resembles Fig. 5.4 in EC8 [16], but therein it is confined to walls in dual WF systems. A similar but straight-line envelope with the wall-top shear of:

$(0.9 - 0.3T_1)\ V_a \geq 0.3\ V_a$, (12)

was given by Priestley et al [81] and Model Code [56].

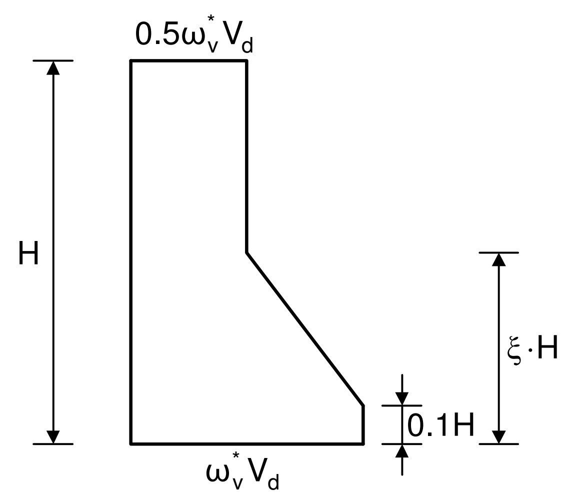

Fig.7: Shear force design envelope (Rutenberg & Nsieri 2006)

5 MORE ON DUAL WALL FRAME (WF) SYSTEMS

Some studies on dynamic shear demand amplification in WFs followed naturally from those on flexural walls, but not all. As already noted wall shear amplification in WFs was studied by Kabeyasawa (e.g. [44]) and Aoyama [4], see Eqn. 2. Paulay and his co-workers [38, 39] observed that, as in flexural walls, higher vibration modes amplified the shear force demand on the *walls* of WFs, but to a lesser extent, and proposed a simple prediction formula accounting for this response reduction. This extension of the shear amplification formula for flexural walls based on Blakeley et al [9] to walls of dual systems is also given by Paulay & Priestley [75]. This WF shear amplification factor ω_v^* is given by:

$$\omega_v^* = 1 + (\omega_v - 1)\,\eta \tag{13}$$

in which ω_v is the free standing wall shear amplification factor and η is the fraction of the base shear carried by the walls. Although the format of this formula is appealing its predictive capability appears to be limited [89].

Alwely [3] made ω_v^* dependent on the non-dimensional frame-to-wall stiffness ratio αH, as evaluated from the linear continuum approximation (e.g. [99]). Kappos & Antoniadis [46] proposed a procedure to estimate more realistically the shear demand in the upper storeys and also a modified version of the Goodsir et al [38, 39] formula (Eqn. 13) to account for shear amplification in WFs with unequal walls. Very recently Kappos & Antoniadis [47, 48] provided formulas to improve the shear distribution match with NLRHA results all along the wall height.

The codified guidance regarding the amplification factor ω_v^* is rather limited. The New Zealand seismic code NZS 3101- 2006 [63] refers only to Paulay & Priestley [75] where Eqn. 13 is presented. Note that therein ω_v is that given by Eqn. 1. It is clearly seen that when $\eta \leq 1.0$ then $1 \leq \omega_v^* \leq \omega_v$. Eqn. 13 assumes that ω_v^*, as ω_v, is independent of the expected ductility demand or q (R). Note also that Eqn. 1 depends indirectly on T_1 through its dependence on the number of storeys n.

EC8 [16] considers WFs in which $\eta > 50\%$ as “wall–equivalent”, hence in such cases wall shear in high ductility class (DC-H) systems should be amplified per Eqn. 4.

Applicable US codes do not include dynamic shear amplification requirements even for flexural walls (except the Commentary to the 1999 SEAOC code [93]). In practice, however, amplification is considered in tall buildings design, as described subsequently. Quite recently Priestley et al [81], in the draft displacement-based seismic design code at the end of their book (also [56]), proposed a dynamic base shear amplification expression for WF walls, which requires predicting the displacement ductility demand μ_{sys} of the dual WF system. When $0.4 \leq \eta \leq 0.8$ the wall base shear amplification is given by:

$$\omega_V^* = 1 + \frac{\mu}{\Phi} C \tag{14}$$

$$C = 0.4 + 0.2(T_1 - 0.5) \leq 1.15$$

in which Φ is an overstrength factor (usually taken as 1.25). Note that ω_V^* is assumed independent of η, hence cannot be in agreement with Eq. 13.

Based on the deflected shapes observed in several time windows during NLRHA, Sullivan et al [100] recently proposed a variant of Keintzel's formula using so called "transitory inelastic modes". They concluded that the higher modes' periods elongated, and suggested that good agreement with NLRHA of the base shear per Keintzel could be obtained when the higher modes' periods were evaluated based on low post-yield secondary stiffness at wall base. This basically amounts to replacing the μ^2 factored higher modes fixed base shears (at design level) in Eqn. 6 by their thus computed counterparts.

A NLRHA based formula was very recently proposed by Rutenberg & Nsieri [89]:

$$V_a = 0.4 + 0.2(T_1 + q + \eta) + 0.13 T_1 q \eta \tag{15}$$

in which η is the elastic wall base shear to total base shear ratio. As all formulas based on NLRHA parametric studies it is applicable to design spectra similar to the mean spectrum of the suite of records used for their derivation. It was also found that the results are quite sensitive to the design yield levels of the girders over the frame height

6 EFFECTS ON SHEAR AMPLIFICATION: SEISMIC ISOLATION, MULTIPLE HINGES, ROCKING, AND SHEAR DEFORMATION

6.1 Seismic isolation, multiple hinges along height and rocking

It is often necessary to increase the strength over significant portions of the wall height if yield is to be confined - per capacity deign - to the plastic hinge zone at the base; whereas allowing yield to propagate upwards (with base level flexural strength) leads to some reduction in the shear demand, e.g., as shown in Fig. 8. Adding another plastic hinge at wall mid-height, as proposed by Panagiotou & Restrepo [67], leads to more pronounced lowering of shear demand envelope. Furthermore, when the wall is designed such that plastic hinges can develop at several locations along the height as done by Rad [82, 84], the shear demand falls appreciably, as illustrated in Figs. 9 & 10. Evidently, the last approach requires special reinforcement detailing along all the wall height.

Wiebe & Christopoulos [103] proposed a design to allow rocking of several wall segments with respect to each other in order to reduce the contribution of higher modes, using unbonded post-tensioning. Appreciable reductions in shear and bending moment were reported.

Twenty storey seismically isolated cantilever wall buildings designed with lead-plug rubber bearings were studied by Calugaru & Panagiotou [12]. Three variants were considered: single isolator at base, isolators at base and at mid height, and isolators at base and at 0.7 of height. As expected, these buildings demonstrated significant reduction in the important response parameters compared with standard fixed base, i.e. with a single plastic hinge, buildings. The buildings with dual isolation led to a further reduction in shear demand on the upper 60% of the height.

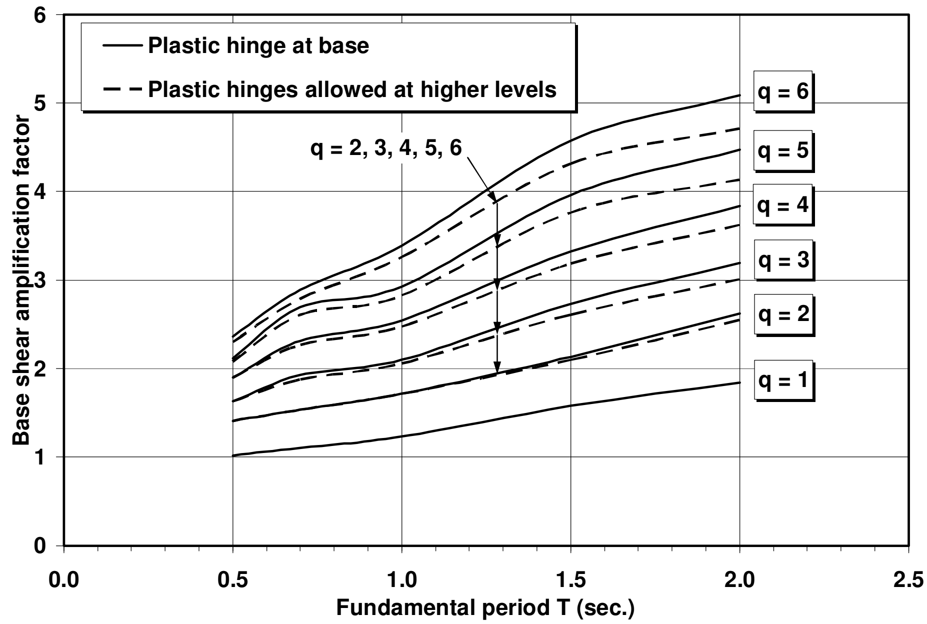

Fig.8: Plastic hinge at base only vs. plastification at higher levels –base shear flexural strength assumed over building height (Rutenberg & Nsieri 2006)

6.2 Shear deformation

Already in 1988 Eibl & Keintzel [27] observed that smaller shear rigidity lowers the higher modes dynamic amplification. In their study on the shear distribution among wall of different lengths Rutenberg & Nsieri [88] noted a substantial lowering in shear demand in cracked sections when the effective shear rigidity GA_{ve} was evaluated based on the analogous truss model [72], which is appreciably smaller than the recommended values based on the gross section (GA_{vg}) for both cracked and uncracked walls by ASCE 41-06 [6], or 50% thereof for cracked walls by EC8 [16]. Gerin & Adebar [33] provided expressions to predict the shear deformation, including shear cracking and yielding, observing that a typical effective shear rigidity of diagonally cracked concrete could approximately be taken as 10% GA_{vg}, somewhat lower than the GA_{ve} assumed by [88]. Rad & Adebar [82, 84] used effective shear stiffness as a simple way to demonstrate the significant influence of cracking on shear amplification, as shown in Fig 11. Wallace [102] suggested modelling the post-cracking shear stiffness in the plastic hinge zone as circa 2.5% GA_{vg}.

7 NOTES ON PRACTICE IN THE USA

Seismic design practice in the US, particularly in California, is highly regarded worldwide and often adopted; hence it may be useful to comment on the situation there with respect to dynamic shear amplification.

It is interesting that although researchers, code-writers and practicing engineers were exposed to the many publications on the post-yield shear demand magnification of higher modes, this has not been manifested in US seismic codes, e.g., ASCE 7-05 [5], as would be expected, particularly, since other major codes such as NZ3101 [63] and EC8 [16] included such provisions for many years. As noted, only the Commentary to the 1999 SEAOC [93] considered this issue, referring the reader to the New Zealand code. Also, shear amplification is recognized, albeit to a limited extent, by ASCE 41-06 [6] code for upgrading of existing buildings:

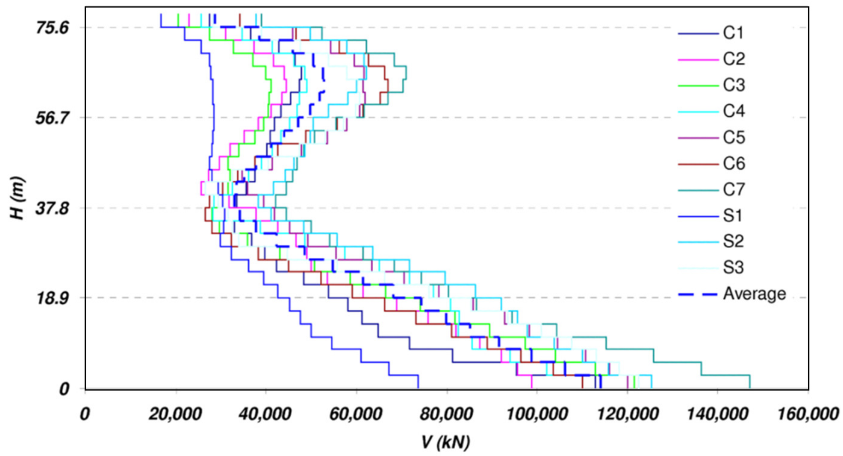

Fig. 9: Shear force envelopes with hinging permitted only at wall base, R =5 (Rad & Adebar 2008)

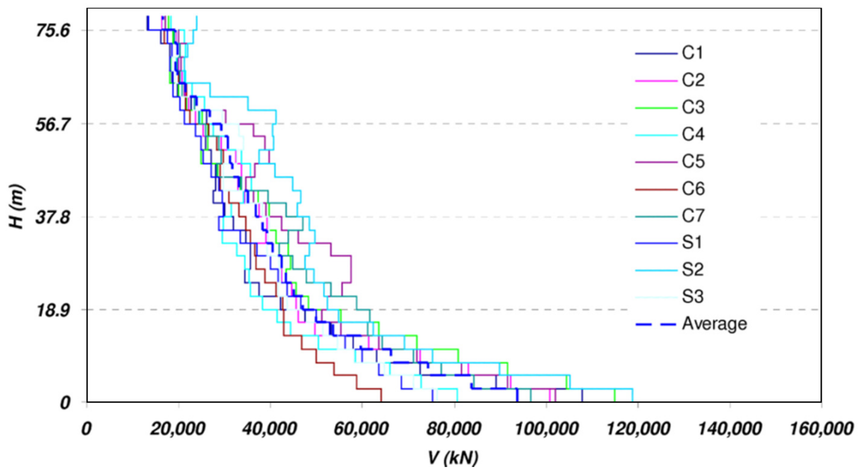

Fig. 10: Shear force envelopes, flexural hinging at many locations over height, R =5 (Rad & Adebar 2008)

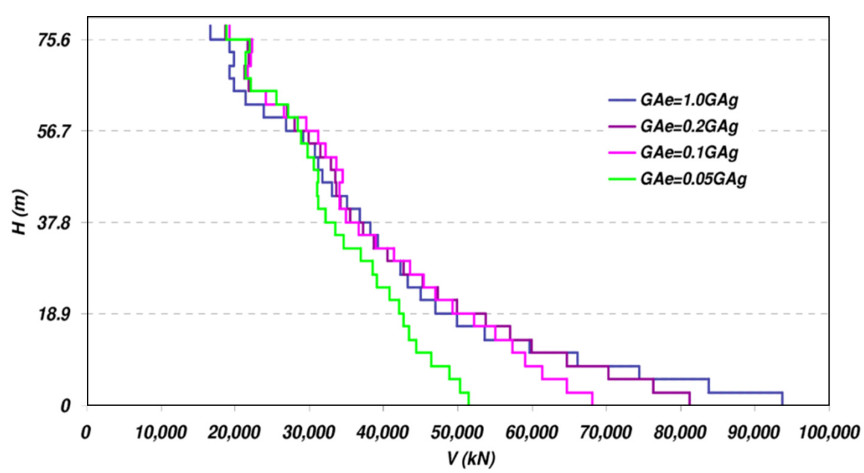

Fig. 11: Influence of effective shear stiffness on average shear force envelopes, flexural hinging at many locations along wall height, R =5 (Rad & Adebar 2008)

the shear force at the wall base is calculated assuming uniform lateral force distribution over the wall height, i.e., the shear resultant is located at wall mid-height (clause 6.7.2.4 therein) rather than at approximately its 2/3rd, thereby increasing the base shear by circa 35%. It is only when NLRHA - one of the recommended procedures by US codes - is carried out that dynamic shear amplification is accounted for. However, code based analysis per this procedure appears to be the exception rather than the rule.

The resurgence in high-rise construction in the west coast of the United States during the early and mid-2000's has led to the application of performance-based non-prescriptive design procedures ([20, 54, 94]. Such design requires using a suite of at least seven pairs of appropriate ground motion histories; hence, dynamic amplification is thereby considered. Indeed, researchers and practitioners in California were soon reporting on large higher modes magnifications [53, 57, 58, 59, 105]. For very tall buildings, circa 40 storeys, these studies predicted shear amplifications on the order of 4 relative to code. It is interesting to note that one designer [52] observed, after designing over two dozen tall buildings, that "*design which follows the prescriptive provisions of the building code will likely result in a shear capacity that falls well short of the likely demands the structure will experience during a significant seismic event. As shear failure of a structural wall is typically viewed as non-ductile, undesirable response, this outcome raises serious concerns….*"

8 POST-ELASTIC REDISTRIBUTION OF SHEAR DEMAND AMONG WALLS IN SYSTEMS WITH INTERCONNECTED WALLS OF UNEQUAL LENGTH.

The shear force distribution among the walls in regular multistorey structures laterally supported by a number of reinforced concrete cantilever walls with markedly different lengths is strongly affected by the different yield displacement levels of the walls. In the post-elastic range the share of the shear force carried by the shorter walls is larger than their share in the bending moment.

In a single storey structure the shear distribution among the walls within the elastic range is proportional to their stiffness. Following yielding of the longer wall additional loads will be carried by the shorter ones. It is evident that design for shear on the basis of relative stiffness underestimates the forces on the shorter walls since shear capacity should be made proportional to flexural strength. However, the situation in a multistory building is different as shown in Fig.12, showing a simplistic model of a two storey system supported by two walls. To fix ideas, assume Wall 2 is very much stiffer than Wall 1; hence, the horizontal force on Wall 1 is negligible so long as Wall 2 has not yielded. With Wall 2 yielding at base the force ΔH is distributed as shown in Fig.12b, where for simplicity it was assumed that the floor diaphragms are pin-connected to the walls and are infinitely rigid in plane. It can be seen that displacement compatibility requires a shear force redistribution resulting in a very large shear force acting on Wall 1.

Rutenberg and coworkers (e.g. [85, 86, 88]) demonstrated that the shear demand on the flexible walls could be very much larger than commensurate with their relative stiffness or even flexural strength. These studies were mainly carried out by means of cyclic pushover analysis on bilinear force-displacement hysteretic models. However, since redistribution is mainly due to compatibility enforcement rather than equilibrium it is sensitive to modelling assumptions. Hence further studies replaced the routine modelling (G= 0.4E, in-plane floor rigidity, bilinear hysteresis) with more realistic models. Indeed, the earlier results were shown [7, 88] to overestimate the base shear demand on the shorter wall. Beyer [7] also observed that the greatest effect was due to "locking in" of compatibility forces transmitted by the floor diaphragms. Yet, it appears that the effect of this relaxation depends on the extent of plasticity

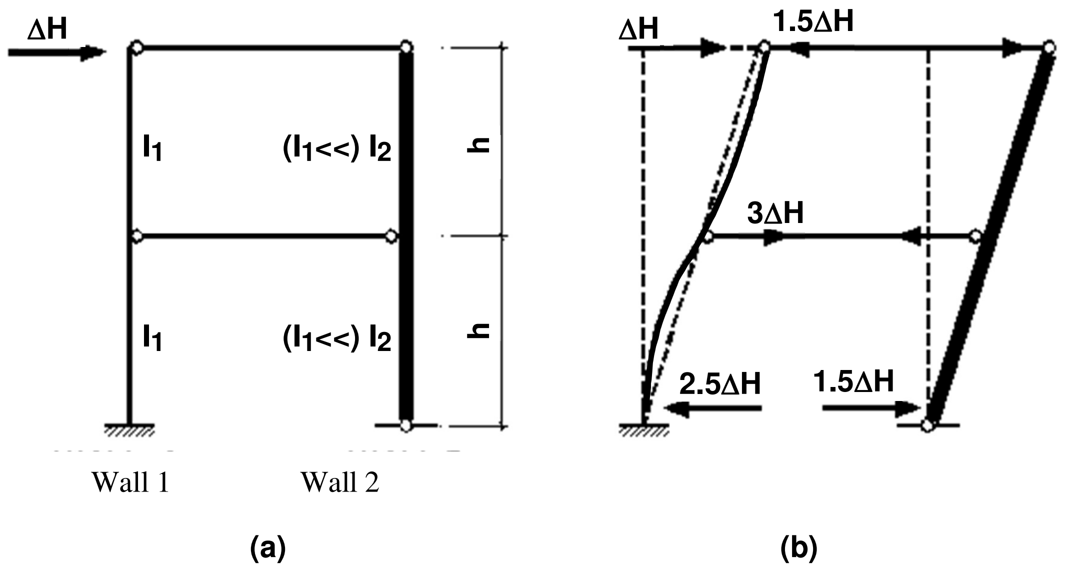

Fig. 12: Two-storey wall system: (a) properties and loading, (b) floor forces and deflected shapes (Rutenberg 2004)

propagation along the wall's cross-section, which in turn depends on whether the longitudinal reinforcement is mainly concentrated at the wall ends (flanges) or is uniformly distributed, which was not considered. Also, Beyer's conclusions were obtained from a two-wall model with modest lengths ratio (1.5: 6.0m & 4.0m respectively); more pronounced stiffness differences are likely to increase the share of the short wall shear force. However, Priestley et al [81] concluded, based on Beyer's work [7], that these compatibility induced forces might normally be ignored in design. Further study on redistribution, including these effects, is now under way [8].

Beyer [7] proposed using the pushover analysis [85] to test the propensity of the shorter wall for excessive shear demand. In that procedure the resultant lateral force is located at the minimum dynamic effective height $h_{eq,\ dyn}$ given by Eqn.16:

$$h_{eq,\ dyn} = h_{eq,\ static} / \omega_v \tag{16}$$

In which $h_{eq,\ static}$ is the height of the statically computed shear resultant, and ω_v is the dynamic amplification factor. Alternatively, $h_{eq,\ dyn}$ may be taken for the applicable spectrum from plots such as Fig. 2 in [91], if available. If the base shear on the short wall sharply increases while the base shear of the long wall drops off after formation of the plastic hinge at the base of the long wall, coupling effects are expected to be significant. Note that ω_v in Eqn. 16 is predicated on the assumption that the total base shear on a group of walls is equal to that on an isolated wall whose strength and stiffness equal the respective sums of these properties of the individual walls. This was found to be justified for the record suites used by Rutenberg & Nsieri [88].

Adebar & Rad [1], using pushover analysis, studied a two-wall system with markedly different properties (a long flanged wall and a short rectangular one). They were mainly interested in the effect of shear deformation on the response, modelling both flexure and shear with tri-linear force displacement relations, i.e., a kink at cracking and a yielding plateau (very short for shear). As expected, cracking and yielding were found to affect the shear force redistribution along the height, in some cases predicting shear failure. In particular, diagonal shear cracking led to significant shear force increase in the short wall at the 2nd floor level. Note, however, that concentrated plasticity was not accounted for.

9 DISCUSSION

In many countries dynamic shear amplification is now routinely accounted for in wall design, yet, as shown, there are large differences in the extent of amplification - from relatively low ω_v in the New Zealand code to quite high ones in EC8. As already noted, the Canadian code requires considering amplification only when the equivalent lateral force procedure is carried out, and then only the linear multi-modal effect. Interestingly, the Romanian seismic code P100-1/2006 [64], while generally adopting the EC8 provisions for RC structures, excluded Eqn. 4, based mainly on the peculiarities of the 1977 Vrancea earthquake record. Instead, it specifies $\omega_v = 1.5$ for statically computed base shear or 1.2Φ (Φ = flexural overstrength), whichever is larger [77, 104].

The very large amplifications predicted by NLRHA were met in many instances with some skepticism. For a widely expressed attitude it is perhaps best to quote Paulay [74], who wrote w. r. t. computed magnifications much larger than given in the New Zealand provisions: *"I found at that time* (early 1980's, AR) *that the duration of predicted extremely high shear demands was typically 0.02 to 0.03 seconds. So we pacified our conscience with the unanswered question: 'is this time interval long enough to enable a wall region to fail in shear?'"*. On the other hand, ignoring shear peaks with durations less than 0.03 seconds do not appear to lead to substantial reduction in shear demand. A preliminary study by Panagiotou & Calugaru [65] on records of three earthquakes showed that demand was lowered only by 6%-15%.

It has often been observed that the maximum shear may not occur simultaneously with maximum moment over the wall height; hence the large amplification factors could be questioned (e.g. [55]). Yet, simultaneous peaks at wall base level were observed in analyses by Rad [82] and Tremblay et al [101] for several historical records; and the large amplifications were evaluated for the wall base. No doubt these two issues deserve study.

The main reason for the skepticism appears to be the feeling that standard nonlinear modelling of RC walls, particularly in the plastic hinge zone, does not capture faithfully the complex response. As noted above, using the Takeda rather than the bilinear hysteresis lowers computed shear demand somewhat, and more importantly, realistic shear behaviour modelling results in substantial fall in shear demand (Fig. 11); considering shear-flexure interaction might lower it further. On the other hand, it has been demonstrated that the 5% damping routinely assumed in NLRHA is quite unconservative for tall buildings design, and damping ratios lower than 2% were reported (e.g. [92]).

Another important issue is the experimental and historical evidence for nonlinear dynamic shear amplification The study by Eberhard [25, 26, 98] on wall frames suggests that the formula proposed by Kabeyasawa (e.g. [43]) and Aoyama [4] is in approximate agreement with the experimental results. The experimental study by Panagiotou et al [66, 68, 69] did not focus on shear amplification per se, yet this phenomenon has been observed in their shake table tests. Shear amplification was also observed by Tremblay, Leger and their co-workers in an on-going shake-table testing program [55]. The large dynamic amplifications predicted by the many parametric studies listed in the bibliography and by Keintzel's formula, or variations thereof, were not observed in these studies.

On the whole it appears that the seismic performance of well-designed and constructed cantilever wall buildings has been satisfactory. Yet, damage to shear walls was reported after several earthquakes. Blakeley et al [9] referred to reports on two cases of extensive diagonal tension cracking; however, it is not clear whether those or other such failures can be attributed to dynamic shear amplification. Shear failures in walls were observed after the February 2010

Chile earthquake, but these were attributed mainly to discontinuities and similar effects such as stress concentrations near openings. Targeted studies are now under way [11].

The reader who chooses not to perform NLRHA with refined modelling may well ask which of the several formulas reviewed here he/she should adopt. In order to answer this question it is necessary to perform extensive parametric studies. On the other hand it does not appear that the differences among the several variants of Keintzel's formula are excessive. Strictly speaking, parametrically derived formulas and design charts are applicable only to the mean spectrum of their data base.

These design procedures do not account for multiple hinges when the vertical wall reinforcement follows approximately (in several steps) modal analysis moment envelope. They also do not consider wall shear cracking, which has significant effect on shear amplification, and on the response of interconnected walls. The actual impact of shear cracking on amplification depends on cyclic shear-flexure coupling; but such a model is not yet available [102].

In view of the above, it is believed that judiciously conceived NLRHA-based design can better account for the many variables affecting dynamic shear amplification.

10 ACKNOWLEDGEMENTS

The author is indebted to T. Postelnicu and D. Zamfirescu for explaining the relevant provisions in the Romanian code P100-1/2006, to M. Panagiotou for providing peak shear plots for several US records and to K. Beyer for her comments on interconnected walls. The help of R. Adoni, A. Aines and I. Elkayam is gratefully acknowledged.

REFERENCES AND BIBLIOGRAPHY

The list below, although quite extensive, is by no means exhaustive, as can be seen from relevant publications referred to in some of the listed publications. Research reports subsequently published in conference proceedings or journals are usually not included.

[1] Adebar P & Rad BR (2007). Seismic shear force distribution in high-rise concrete walls. *9th Canadian Conference on Earthquake engineering*, Ottawa, Canada.

[2] Adebar P, Mutrie J & DeVall R (2005). Ductility of concrete walls: the Canadian seismic design provisions 1984 to 2004. *Canadian Journal of Civil Engineering*, **32**, 6, 1124-1137.

[3] Alwely F (2004). Seismic behavior of ductile wall-frame structural systems. M. Sc. Thesis, Faculty of Civil & Environmental Engineering, Technion, Haifa, Israel (in Hebrew, English summary & captions).

[4] Aoyama H (1987). Earthquake resistant design of reinforced concrete frame buildings with "flexural" walls. *Journal Faculty of Engineering, University of Tokyo*, **XXXIX**, 2, 87-109. Also in: *Earthquake resistance of reinforced concrete structures*, a volume honoring Hiroyuki Aoyama, University of Tokyo, 1993.

[5] ASCE (2006) Minimum design loads for buildings and other structures. ASCE Standard ASCE/SEI 7-05. American Society of Civil Engineers, Reston, VA.

[6] ASCE (2007). Seismic rehabilitation of existing buildings. ASCE Standard ASCE/SEI 41-06. American Society of Civil Engineers, Reston, VA.

[7] Beyer K (2005). Design and analysis of walls coupled by floor diaphragms. M.Sc. thesis, Rose School, University of Pavia, Italy.

[8] Beyer K (2011). Personal communication, February.

[9] Blakeley RWG, Cooney RC & Megget LM (1975). Seismic shear loading at flexural capacity in cantilever wall structures. *Bulletin New Zealand National Society Earthquake Engineering*, **8**, 278-290.

[10] Boivin Y & Paultre P (2010). Seismic performance of a 12-storey ductile concrete shear wall system designed according to the 2005 National building code of Canada and the 2004 Canadian Standard Association standard A23.3. *Canadian Journal of Civil Engineering*, **37**, 1-16.

[11] Boroschek (2011). Personal communication, January.

[12] Calugaru V. & Panagiotou M (2010). Seismic Isolation Using Single and Dual Shear Hinging of Tall Cantilever Wall Buildings Subjected to Near Fault Ground Motions. *9th US National & 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

[13] CEB (1980, 1983, 1985). Model Code for Seismic Design of Concrete Structures, Bulletin d'Information, Comite Europeen du Beton, No.133, 160, 165 (respectively), Lausanne, Switzerland.

[14] CEN (1988). Eurocode No 8 – Structures in seismic regions –Design. Part 1 General and building". Brussels, Belgium.

[15] CEN (1993). Eurocode 8 - Earthquake resistant design of structures. Pt. 1.3 General rules – seismic actions and general requirements for structures. Final Draft PrENV, Brussels, Belgium.

[16] CEN (2004). Eurocode 8 - Earthquake resistant design of structures. Pt. 1.3 General rules – specific rules for various materials and elements", Brussels, Belgium.

[17] Chaallal O & Gauthier D (2000). Seismic shear demand on wall segments of ductile coupled shear walls. *Canadian Journal of Civil Engineering*, **27**, 506-522.

[18] Chopra, AK (2001). *Dynamics of Structures,* 2nd edn. Prentice Hall, Upper Saddle River, NJ.

[19] CSA (1994, 2004). Design of concrete structures. Standard CAN/CSA-A23.3-04, Canadian Standards Association, Rexdale, Canada.

[20] CTBUH (2008). Recommendations for the seismic design of high-rise buildings. Council on Tall Buildings and Urban Habitat, Chicago, IL.

[21] Derecho AT et al (1978). Structural walls in earthquake-resistant buildings – Analytical investigations, dynamic analysis of isolated structural walls – Parametric Studies. Final Report to the National Science Foundation RANN, Portland Cement Association, Skokie, IL.

[22] Derecho AT, Iqbal M & Corley WG (1980). Determining design force levels for earthquake-resistant reinforced concrete structural walls. *7th World Conference on Earthquake Engineering*, Istanbul, Turkey, Vol. 5.

[23] Derecho AT et al. (1981). Structural walls in earthquake resistant buildings, dynamic analysis of isolated structural walls - development of design procedure, design force level. Final Report to the National Science Foundation ASRA, Construction Technology Laboratories, PCA, Skokie, IL.

[24] Derecho AT & Corley, WG (1984). Design requirements for structural walls in multistory buildings. *8th World Conference on Earthquake Engineering*, San Francisco, CA, Vol. 5.

[25] Eberhard MO & Sozen MA (1989). Experiments and analyses to study the seismic response of reinforced concrete frame-wall structures with yielding columns, Structural Research Series 548, Civ. Engrg. Studies, University of Illinois, Urbana, IL.

[26] Eberhard MO & Sozen MA (1993). Behavior-based method to determine design shear in earthquake-resistant walls. *Journal of Structural Engineering ASCE*, **119**, 2, 619-640.

[27] Eibl J & Keinzel E (1988). Seismic shear forces in RC cantilever shear walls. *9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, Vol. VI.

[28] ***fib*** (2010). Model Code 2010, First complete draft, Vol. 2. International Federation for Structural Concrete, Lausanne, Switzerland.

[29] Filiatrault A, Anderson DL & DeVall R (1992). The effect of weak foundation on the seismic response of core wall type buildings. *Canadian Journal of Civil Engineering*, **19**, 530-539.

[30] Filiatrault A, D'Aronco D & Tinawi R (1994). Seismic shear demand of ductile cantilever walls: a Canadian perspective. *Canadian Journal of Civil Engineering*, **21**, 363-376.

[31] Fischinger M , Isakovic T & Rejec K (2010a). Seismic Behavior of RC Structural Walls and Eurocode 8 Provisions. *9th US National & 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

[32] Fischinger M, Rejec K & Isakovic T (2010b). Shear magnification factors for RC Structural Walls in Eurocode 8. *14th European Conference on Earthquake Engineering*, Ohrid, Macedonia.

[33] Gérin M & Adebar P (2004). Accounting for shear in seismic analysis of concrete structures, *13th World Conference on Earthquake Engineering*, Vancouver, Canada.

[34] Ghorbanirenani, I et al (2009a). Distribution of inelastic demand in slender R/C shear walls subjected to Eastern North America ground motion. *ATC-SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, San Francisco, CA.

[35] Ghorbanirenani I et al (2009b). Shake table tests and repair of ductile slender reinforced concrete shear walls. *9th US National & 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

[36] Ghosh SK (1992). Required shear strength of earthquake-resistant reinforced concrete shear walls, in: *Nonlinear seismic analysis and design of reinforced concrete buildings*, P. Fajfar & H. Krawinkler eds., Elsevier, London.

[37] Ghosh SK & Markevicius VP (1990). Design of earthquake resistant shearwalls to prevent shear failure. *4th US National Conference on Earthquake Engineering*, Palm Springs, CA, Vol. 2.

[38] Goodsir WJ, Paulay T & Carr AJ (1983). A study of the inelastic seismic response of reinforced concrete coupled frame-shear wall structures. *Bulletin New Zealand National Society for Earthquake Engineering*, **16**, 3, 185-200.

[39] Goodsir WJ, Paulay T & Carr AJ (1984). A design procedure for interacting wall-frame structures under seismic actions. *8th World Conference on Earthquake Engineering*, San Francisco, CA, Vol. 5.

[40] Humar JL & Rahgozar MA (2000). Application of uniform hazard spectra in seismic design of multistorey buildings. *Canadian Journal of Civil Engineering*, **27**, 563-580.

[41] Humar J & Mahgoub MA (2003). Determination of seismic design forces by equivalent static load method. *Canadian Journal of Civil Engineering*, **30**, 287-303.

[42] Iqbal M & Derecho AT (1980). Inertia forces over height of reinforced concrete structural walls during earthquakes, in: Reinforced concrete structures subjected to wind and earthquake forces. ACI Publication SP-63, American Concrete Institute, Detroit, MI.

[43] Kabeyasawa T & Ogata K (1984). Ultimate-state design of RC wall-frame structures, *Transaction of the Japan Concrete Institute*, 629- 636.

[44] Kabeyasawa T (1987). Ultimate-state design of RC wall-frame structures, *Pacific Conference Earthquake Engineering*, Auckland, New Zealand, Vol. 1. Also in: *Earthquake resistance of reinforced concrete structures*, a volume honoring Hiroyuki Aoyama, University of Tokyo, Tokyo, 431- 440, (1993).

[45] Kabeyasawa T (1988). Evaluation of column and wall actions in the ultimate state design of reinforced concrete structures. *9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, Vol. VIII.

[46] Kappos AJ & Antoniadis, P (2007). A contribution to seismic shear design of R/C walls in dual systems. *Bulletin of Earthquake Engineering*, **5**, 3, 443-466.

[47] Kappos AJ & Antoniadis PS (2010). An improved procedure for the seismic design of reinforced concrete walls in dual systems. *14th European Conference on Earthquake Engineering*, Ohrid, Macedonia.

[48] Kappos AJ & Antoniadis PS (2011). Evaluation and suggestions for improvement in of seismic design procedures for RC walls. *Earthquake Engineering and Structural Dynamics*, **40**, 35-53.

[49] Keintzel E (1984). Ductility requirements for shear wall structures in seismic areas. 8th World Conference on Earthquake Engineering, San Francisco, CA, Vol. 5.

[50] Keintzel E (1990) Seismic design shear forces in reinforced concrete cantilever shear wall structures. *European Journal of Earthquake Engineering*, **3**, 7-16.

[51] Keintzel E (1992). Advances in the design for shear of RC structural walls under seismic loading, in: *Nonlinear seismic analysis and design of reinforced concrete buildings,* P. Fajfar & H. Krawinkler eds., Elsevier, London.

[52] Klemencic R (2008). Performance based seismic design – rising. *Structure- Magazine*, June, 10-13.

[53] Klemencic R et al (2007). Performance-based design of ductile concrete core wall buildings – issues to consider before detailed analysis. *The Structural Design of Tall and Special Buildings*, **16**, 599-714.

[54] LATBSDC (2005, 2008). An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region, Los Angeles Tall Buildings Structural Design Council, CA.

[55] Leger P (2010). Personal communication, March.

[56] Model Code for Displacement-Based Design (2009). GM Calvi & TJ Sullivan, eds. DBD09, draft subject to public scrutiny, IUSS Press, Pavia, Italy.

[57] Moehle JP (2006) Seismic analysis design and review for tall buildings. *The Structural Design for Tall and Special Buildings*, **15**, 5, 495-513.

[58] Moehle JP (2008). Performance-based seismic design of tall buildings in the U.S. *14th World Conference on Earthquake Engineering*, Beijing, China.

[59] Moehle J, Bozorgnia Y & Yang TY (2007). The Tall Buildings Initiative. *SEAOC Convention*, Squaw Creek, CA, 315-324.

[60] Naeim F ed. (2001). *The Seismic Design Handbook*, 2nd edn. Kluwer Academic, Boston, MS.

[61] NBCC (1995, 2005). National Building Code of Canada. Associate Committee on the National Building Code. National Research Council of Canada, Ottawa.

[62] Nsieri E (2004). The seismic nonlinear behaviour of ductile reinforced concrete cantilever wall systems. M.Sc. thesis, Technion, Haifa, Israel (in Hebrew, English summary & captions).

[63] NZS (1982, 1995, 2006) "NZS 3101: Part1, Concrete Structures Standard; Part 2, Commentary on the design of concrete structures, New Zealand Standards, Wellington.

[64] P100-1/2006 (2006). Seismic design code P100 Part 1- Design provisions for buildings. Technical University of Construction, Bucharest, September, (in Romanian).

[65] Panagiotou M (2011). Personal Communication, January.

[66] Panagiotou M & Restrepo JI (2007). Lessons learnt from the UCSD full-scale shake table testing on a 7-story residential building. *SEAOC Convention*, Squaw Creek, CA.

[67] Panagiotou M & Restrepo JI (2009). Dual-plastic hinge design concept for reducing higher-mode effects on high-rise cantilever wall building. *Earthquake Engineering and Structural Dynamics*, **38**, 12, 1359-1380.

[68] Panagiotou M & Restrepo JI (2010). A displacement-based method of analysis for regular reinforced concrete wall buildings: Application to a full-scale 7-story building slice tested at UC San Diego. *Journal of Structural Engineering ASCE*, Posted ahead of print, October.

[69] Panagiotou M, Restrepo JI & Conte JP (2010). Shake table test of a 7-story full scale reinforced concrete wall building slice, Phase I: Rectangular wall. *Journal of Structural Engineering ASCE*, Posted ahead of print, October.

[70] Panagiotou M, Calugaru V & Visnjic T (2009). Higher mode effects on the seismic response of tall cantilever wall buildings subjected to near fault ground motions. *78th SEAOC Annual Convention*, San Diego, CA.

[71] Panneton P, Leger P & Tremblay R (2006). Inelastic analysis of a concrete shear wall building according to the National Building Code of Canada 2005. *Canadian Journal of Civil Engineering*, **33**, 854-871.

[72] Park R & Paulay T (1975). *Reinforced concrete structures.* Wiley, New York.

[73] Paulay T (1986). Design of R/C structural walls. *Earthquake Spectra*, **2**, 4, 810-814.

[74] Paulay T (2007). Personal communication, January.

[75] Paulay T & Priestley, MJN (1992). *Seismic design of reinforced concrete and masonry buildings*. Wiley, New York.

[76] Pennucci D, Sullivan, TJ & Calvi GM (2010). Evaluation of higher mode effects in tall building response. *14th European Conference Earthquake Engineering*, Ohrid, Macedonia.

[77] Postelnicu T (2011). Personal communication, January.

[78] Priestley MJN (2003a). *Myths and fallacies in earthquake engineering, revisited*, The Mallet Milne Lecture, 2003, IUSS Press, Pavia, Italy.

[79] Priestley MJN (2003b). Does capacity design do the job? An investigation of higher modes effects in cantilever walls. *Bulletin New Zealand Society for Earthquake Engineering*, **36**, 4.

[80] Priestley MJN & Amaris A (2003). Dynamic amplification of seismic moments and shear forces in cantilever walls. ***fib*** *Symposium, Concrete structures in Seismic Regions*, Athens, Greece.

[81] Priestley MJN, Calvi GM & Kowalsky MJ (2007). *Displacement-Based Seismic Design of Structures*. IUSS Press, Pavia, Italy.

[82] Rad RB (2009). Seismic shear demand in high-rise concrete walls. Ph.D. thesis, University of British Columbia, Vancouver, Canada.

[83] Rad RB & Adebar P (2006). Shear demand on high-rise concrete walls: Influence of diaphragms below grade. *8th U.S. National Conference on Earthquake Engineering*, EERI, San Francisco, CA.

[84] Rad BR & Adebar P (2008). Dynamic shear amplification in high-rise concrete walls: effect of multiple flexural hinges and shear cracking. *14th World Conference on Earthquake Engineering*, Beijing, China.

[85] Rutenberg A (2004). The seismic shear of ductile cantilever wall systems in multistorey structures. *Earthquake Engineering & Structural Dynamics*, **33**, 881-896.

[86] Rutenberg A & Leibovich E (2002a). On the lateral force distribution among structural walls in multistorey buildings. *Bulletin New Zealand Society for Earthquake Engineering*, **35**, 231-242.

[87] Rutenberg A & Leibovich E (2002b). The irregular post-yield behaviour of regular multistorey wall structures. *3rd European Workshop on Seismic Behaviour of Irregular and Complex Structures*, Florence, Italy.

[88] Rutenberg A & Nsieri E (2006). The seismic shear demand in ductile cantilever wall systems and the EC8 provisions. *Bulletin of Earthquake Engineering*, **4**, 1-21.

[89] Rutenberg A & Nsieri E (2010). On the seismic shear demand in ductile RC dual systems. *9th US National & 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

[90] Rutenberg A, Nsieri E & Levy, R (2003). Seismic Shear Forces in Cantilever Wall Systems. *Budownictwo*, Centre of Structural Integrity, Opole Technical University, Poland, 294/2003, 43-57.

[91] Rutenberg A, Nsieri E & Levy R (2004). Seismic Shear demand on ductile systems in multi-storey structures. *13th World Conference on Earthquake Engineering*, Vancouver, Canada.

[92] Satake N et al (2003). Damping evaluation using full-scale data of buildings in Japan. *Journal of Structural Engineering ASCE*, **129**, 4, 470-477.

[93] SEAOC (1999). Recommended lateral force requirements and commentary. Structural Engineers Association of California, Sacramento, CA.

[94] SEAONC (2007). SEAONC Recommended Administrative Bulletin for San Francisco. Structural Engineers Association of Northern California, San Francisco, CA.

[95] Seneviratna GDPK & Krawinkler H (1994). Strength and displacement demands for seismic design of structural walls. *5th US National Conference on Earthquake Engineering*, Chicago, IL, Vol. 2.

[96] Seneviratna GDPK (1995). Evaluation of inelastic MDOF effects for seismic design. Ph.D. Dissertation. Stanford University. Also: Seneviratna GDPK & Krawinkler H (1997). Report 120, John A. Blume Earthquake Engineering Center. Stanford University, CA.

[97] SI 413 (1998) and Amendment No.3 (2009). Design provisions for earthquake resistance of structures. Standards Institution of Israel, Tel-Aviv.

[98] Sozen MA & Eberhard MO (1992). Design shear for earthquake-resistant walls, in: Nonlinear seismic analysis and design of reinforced concrete buildings, Report 103, John A. Blume Earthquake Engineering Research Center, Stanford University, CA.

[99] Stafford Smith B & Coull A (1991). *Tall Building Structures: Analysis and Design*. Wiley, New York.

[100] Sullivan TJ, Priestley MJN & Calvi GM (2008). Estimating the higher mode response of ductile structures. *Journal of Earthquake Engineering*, **12**, 3, 456-472.

[101] Tremblay R et al (2008). Seismic response of multi-storey reinforced concrete walls subjected to Eastern North America high frequency ground motions. *14th World Conference on Earthquake Engineering*, Beijing, China.

[102] Wallace JW (2010). Performance-based design of tall reinforced concrete core wall buildings, in: *Earthquake Engineering in Europe*, M Garevski & A Ansal eds., Springer, Utrecht, Germany.

[103] Wiebe L & Christopoulos C (2009). Mitigation of higher mode effects in base-rocking systems by using multiple rocking sections. *Journal of Earthquake Engineering*, **13**, 1, 83–108.

[104] Zamfirescu D (2011). Personal Communication, January.

[105] Zekioglu A et al (2007). Case study using the Los Angeles Tall Buildings Structural Design Council guidelines: 40-storey concrete core wall building. *The Structural Design for Tall and Special Buildings*, **16**, 583-597.