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Evaluation and suggestions for improvement of seismic design procedures for R/C walls in dual systems

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SUMMARY

This paper aims to shed some further light on the seismic behaviour and design of reinforced concrete (R/C) walls which form part of dual (frame + wall) structures. The significance of post-elastic dynamic effects is recognized by most seismic codes in the definition of the design action effects on walls, i.e. bending moments and shear forces. However the resulting envelopes are not always fully satisfactory, particularly in the case of medium-to-high-rise buildings. Relevant provisions of modern seismic codes are first summarized and their limitations discussed. Then an extensive parametric study is presented which involves typical multi-storey dual systems that include walls with unequal lengths, designed according to the provisions of Eurocode 8 for two different ductility classes (M and H) and two effective peak ground acceleration levels (0.16g and 0.24g). The walls of these structures are also designed to other methods, such as those used in New Zealand and Greece. The resulting different designs are then assessed by subjecting the structures to a suite of records from strong ground motions, carrying out inelastic time history analysis, and comparing the results with the design action effects. It is found that for (at least) the design earthquake intensity, the first two modes of vibration suffice for describing the seismic response of the walls. The bending moment envelope, as well as the base shear of each wall, are found to be strongly dependent on the second mode effect. As far as the code-prescribed design action effects are concerned, only the NZ Code was found to be consistently conservative, while this was not always the case with EC8. A new method is then proposed which focusses on quantifying in a simple way the second mode effects in the inelastic response of the walls. This procedure seems to work better than the others evaluated herein.

Keywords: dual systems, reinforced concrete walls, Eurocode 8, New Zealand Code, seismic shear, higher mode effects

1. INTRODUCTION

Over the last 15 years or so, dual structural systems, i.e. systems combining frames and walls that jointly carry seismic loading have been promoted by most seismic codes which specify for them force reduction factors (behaviour factors, q, in Eurocode terminology, see CEN 2004 [1]) that are the same as those of ductile frames and generally higher than those used for (uncoupled) wall systems. This is a notable change compared to what happened up to the 1980's when higher q-factors were specified for (ductile) moment resisting frames than for dual structures. Despite some limitations that walls impose with regard to the architectural design, dual systems are currently a common choice for multi-storey R/C buildings [2] (in Greece, since the 1980's, they are the only system used for buildings with four or more storeys), since they present a number of practical advantages, notably the excellent control of horizontal displacements both in the lower and in the upper part of the building (the latter is not the case in systems comprising solely walls).

In modern capacity design procedures for dual systems the bending moments along the height of a wall, as well as the shear forces, are magnified in an empirical way, in order to exclude yielding outside the critical, plastic hinge, region (namely the lower part of the wall) and the formation of non-ductile failure mechanisms, respectively. It is important to note that these action effects depend not only on possible overstrength of the plastic hinge regions (after yielding) but also on inelastic dynamic effects. Reducing the elastic force in a wall (through the behaviour factor, q) increases the possibility of yielding under the response of a higher mode of vibration, since plastic hinging at the wall base does not significantly affect response in the higher modes [2]. It is also clear that this effect should be more pronounced as the importance of a higher mode of vibration increases (this is generally the case as the height of the structure increases). The problem was addressed by Paulay and Priestley [2] and simplifying procedures to deal with it were presented; the dynamic magnification factors (ω) remained unchanged for almost two decades and constitute the backbone of the capacity design method even for the recently released (2006) NZS3101 [3]. Meanwhile the proposal of Keintzel [4] was adopted by Eurocode 8 for high-ductility structures (DC H). Other researchers have addressed issues not properly covered by codes, such as the co-existence of walls of substantially different length [5, 6, 7] or the (direct) displacement-based design of dual systems [8, 9].

The work presented here aims to shed some additional light on the case of post-yield bending moment and shear force amplification in planar walls of medium-to-high-rise dual systems. A parametric study is carried out, concerning a total of eight dual systems, 9-storey and 15-storey high, designed to the provisions of EN 1998-1 [1] for two different ductility classes (M and H) and two different effective peak ground acceleration levels (0.16g and 0.24g). Hence the influence of both the height and the design ductility class of the structure on the design action effects of the walls could be studied for a range of practical interest. Results from 2D dynamic inelastic time-history analyses are compared with those from the application of the most sophisticated available procedures, i.e. those of the 2004 Eurocode 8 [1] and the 2006 NZS3101 [3] (which directly refers to [2]), as well as the somewhat simpler procedures adopted by the current Greek Seismic Code EAK2003 [10]. Based on the findings of this parametric study, a simple proposal is introduced for defining both bending and shear design envelopes, which applies (with slight modifications) to walls of different configurations, i.e. those which can be classified as vertical cantilevers and those which exhibit a mixed action, namely a combination of column (in double curvature) and cantilever action.

2. STRUCTURES STUDIED AND DESIGN PROCEDURES

2.1 Structural configurations considered

Figure 1 shows the configuration in plan, as well as in elevation, of the 15-storey building with dual system studied here. In a previous study by the authors [7] two similar configurations of medium-rise (9-storey) dual systems were studied. In the first configuration only two central (6m) walls were present, while the second one includes six wall elements of different length (top of Fig. 1). The two centrally located walls are 6m long, while the four corner ones are 2m long. Note that only the x-direction was considered in the analysis performed here; clearly, in an actual building of a similar configuration some walls should normally be present in the other horizontal direction as well. Due to the fact that these large walls were found to behave the same, irrespective of whether some small walls were also present in the dual system, it was decided to address only the “multiple wall length” configuration for the 15-storey building (Fig. 1), which is the focus of the present study. Note that this configuration (that represents a rather broad class of dual systems) was selected with a view to studying the interaction between walls of unequal lengths, both carrying shear forces due to strong ground motion; the choice of walls with significantly different length was deliberate, since this configuration, albeit commonly encountered in practice (mainly for architectural reasons), is also the one less covered by existing codes, as will be made clear subsequently.

A key parameter in dual systems is the relative stiffness of the walls and the frames, which is not straightforward to quantify. The wall-to-floor ratio (r_w) is an easy to define (albeit incomplete) descriptor in this respect. In the structures studied, r_w is equal to 0.15 and 0.23% (for two large walls, and two large plus four small walls, respectively) in the case of the 9-storey building, and to 0.11 and 0.17% for the 15-storey building. These are values of practical relevance, representative of actual practice in seismic-prone areas in Europe (and elsewhere). Another important parameter affecting the relative stiffness of walls and frames is the depth of the beams; the relatively deep beams shown in Fig. 1 (500 to 700mm) are typical of usual practice in southern European, but not necessarily of that in other areas of the world. Parametric studies involving the beam depths (which were gradually reduced up to the point of reducing to a flat slab system) have shown that while the fraction of base shear carried by the walls is hardly affected by reductions in beam depth, the influence on the fraction of the base overturning moment is substantial. Due to space limitations, the remainder of the paper focuses on the basic configurations (shown in Fig. 1); nevertheless, it should be kept in mind that the response of these dual systems is representative of those with relatively strong/stiff frames, and quite different from those with weak frames and, especially, of dual systems with flat slabs. Of course, it should be recalled here that coupling of frames through flat slabs is not a recommended practice, and indeed Eurocode 8 does not allow this type of coupling in high ductility (DC H) structures.

2.2 Design procedures: General considerations

The design of these structures was carried out according to the provisions of the final versions of the pertinent Eurocodes (2 and 8), i.e. EN1992-1-1 [11] and EN1998-1 [1], for the medium (DC M) and high (DC H) ductility classes specified in EC8, and for effective peak ground accelerations (a_g) of 0.16g and 0.24g (typical values in Southern Europe). The ground conditions were assumed to match those prescribed for subsoil 'type C' according to the EC8 classification. Note that the same member cross-sections were used for all DC's and a_g's; this assures that the dynamic characteristics of all structures (in the elastic range) are the same, thus creating a solid base for a direct comparison of their responses. In the following, each of the 15-storey structures is denoted by its ductility class and the a_g considered for design, e.g. DCH0.16g. Details for the 9-storey dual systems are omitted here for economy of space, but are given in full in [7]; it is noted that all four 9-storey structures were designed for a_g=0.16g.

The materials of the structural members were assumed to be the same in all 15-storey structures; more specifically, C30/37 concrete (characteristic cylinder strength of 30 MPa) and S500 steel (characteristic yield strength of 500 MPa) were used. It is pointed out here that in the 9-storey buildings lower concrete and steel grades were used (C20/25 and S400); the use of higher concrete strength in the 15-storey buildings was necessary due to shear requirements, as explained later. The values of the behaviour factor (q), selected according to EC8 provisions, were 5.4 and 3.6 for the DC H and DC M structures, respectively. The only difference for a frame structure would have been that the overstrength-dependent component of q would have been 1.3 instead of 1.2 that is taken for the ('wall-equivalent') dual structure; the resulting q is 8% lower in the latter case.

The Response Spectrum method of analysis was used (as would normally be the case for such high-rise structures), although the equivalent lateral force method is not prohibited from the EC8 provisions (the fundamental period of the structures is less than 2sec and there are no geometric irregularities). As far as the design of walls is concerned, which was the focal point of this investigation, all capacity design requirements of Eurocode 8 regarding slender walls (aspect ratio $h_w/l_w>2$) were applied: design for flexure and shear (diagonal tension, diagonal compression, and possible sliding shear failure were checked) according to the corresponding linear envelope of the calculated moment and shear diagrams, and proper detailing for local ductility was carried out (confined boundary elements, minimum web reinforcement). Table 1 summarises the reinforcement used in the various walls.

It has to be emphasized that for the section thickness (b_w) of the high-ductility wall members inside the plastic hinge region, the critical step was the diagonal compression check of the web. This resulted in substantial overstrength of the large walls due to minimum reinforcement requirements. By maintaining the same wall cross-section in all different structures, the overstrength issue was also encountered in the DCM cases even if smaller section thickness could have been used there. An effort to minimise the "overstrength factor" was made, by reducing the wall thickness as much as possible, not taking into account the limits induced by the diagonal compression checks or minimum values for the confined boundary elements. The reduction in overstrength values was so small that a new round of analysis for the reduced thickness was deemed futile. It was clear that if the overstrength was to be minimised, the only feasible measure would have been a different configuration, i.e. more walls and/or reduced length of wall sections, and/or smaller beam depths that would increase the moment demand in the wall (see discussion of effect of relative wall-frame stiffness in section 2.1).

2.3 Flexural design procedures for walls

According to the EC8 [1] capacity design procedure, the design bending moment diagram along the height of the wall, irrespective of ductility class, should be given by an envelope of the bending moment diagram from the analysis (static or dynamic), vertically displaced due to the tension shift (Fig. 2a). Note that the envelope is *not* defined with respect to the actual strength of the plastic hinge; this is not in agreement with usual capacity design principles [2, 3] and will subsequently be shown to be a handicap, especially in cases where the moment of resistance (the flexural strength) is substantially higher than the moment derived from analysis.

In contrast to EC8, the actual strength is taken in the NZS3101 [3] procedure that refers to the Paulay and Priestley [2] methodology for slender walls of dual systems. The bending moment envelope defined there has the same shape as the EC8 one, but uses the moment of resistance for the wall base (plastic hinge region) rather than the value obtained from analysis.

Finally, the Greek Seismic Code EAK [10] adopts a procedure for wall bending moments, intended to result into a simpler and less conservative design than the aforementioned EC8 and NZS3101 procedures. According to the Greek Code, the envelope of bending moments calculated from analysis is multiplied by a constant capacity design factor equal to 1.3 times the ratio of the flexural strength of the wall at its base to the corresponding moment from analysis. Moreover, the wall design bending moment at any location should not be taken less than 1/3 the value at the base; care is also taken for tension shift, as in the other codes.

2.4 Shear design procedures for walls

The EC8 capacity design procedure [1] for shear is defined by the envelope shown in Fig. 2b, which is deemed to adequately account for higher mode effects. The design shear at the wall base is taken as 1.5 times the value calculated from analysis in DC M structures, while in DC H structures, by multiplying the analytically derived value with the magnification factor ε, initially suggested by Keintzel [4], accounting for both overstrength due to the development of a single plastic hinge at the base and higher modes effects (through the fundamental period of the building) [12].

According to the New Zealand [2, 3] procedure, the design shear force at any level above the critical section of the primary plastic region in a structural wall, V_o^*, should be at least equal to the corresponding shear force (V_E) found from the analysis multiplied by an overstrength factor, ϕ_o, and a dynamic magnification factor, ω^*_v, i.e.

$$V_0^* = \omega_V^* \varphi_0 V_E \tag{1}$$

The dynamic magnification factor is given by the following expression

$$\omega_V^* = 1 + (\omega_V - 1) \cdot n_V \tag{2}$$

where

$$\omega_V = 1.3 + \frac{n_t}{30} \leq 1.8 \tag{3}$$

for buildings with number of storeys n_t>6 (as herein). The overstrength factor ϕ_o is the ratio of the flexural overstrength of the wall at its base to the corresponding moment from the analysis, while η_v describes the fraction of the total base shear carried by the walls. To account for higher mode effects in the upper storeys Paulay and Priestley also suggest a distribution of the wall shear along the height similar to that shown in Fig. 2b.

A modified version of this procedure was proposed by Kappos and Antoniadis [7] in order to account for the relative contribution of walls with different length to the total base shear; for this purpose the n_V factor was redefined as the ratio of the base shear carried by each group of walls with essentially the same length to the total base shear. A slightly modified shear force envelope was also proposed; the difference lied in the upper third of the height, resulting in less conservative values [7].

Finally, according to the Greek Code, the envelope of shear forces calculated by analysis is multiplied by the same constant capacity design factor used for the definition of the bending moment design envelope. Moreover, the wall design shear at any location should not be taken less than 1/3 the value at the base.

3. ANALYSIS AND MODELLING ISSUES

The alternative procedures described in sections 2.3 and 2.4 were evaluated against the results of inelastic time history analyses, from which the maximum moments and shear forces developed in the walls of the structures of Fig. 1 during seismic actions of different intensity were found. These analyses were carried out with the aid of the inelastic dynamic analysis code RUAUMOKO 2D [13]. Damping was modelled as mass and initial stiffness proportional Rayleigh damping, with ξ=5% for the first two modes, given that the initial stiffness of the members was based on the secant stiffness at yield, i.e. no energy dissipation occurred after cracking but prior to yielding.

The program uses point-hinge (lumped plasticity) modelling, with phenomenological hysteresis laws governing the behaviour of each hinge (typically the ends of an R/C member) allowing a member-by-member modelling approach to be adopted. The one-component beam model was used for all members, while the hysteresis rule used to represent the inelastic behaviour of the hinges was the modified Takeda one, with elastic unloading. Clearly, more sophisticated models for walls are available in the literature (accounting for features such as the elongation of the tension side of the wall and the related shift of the neutral axis), but it was beyond the scope of this paper to identify the most appropriate model; rather, the choice made reflected practicality, as well as compatibility with other studies on the same issue (design of walls), which used similar, or even simpler, models. Strength and stiffness data for R/C members were determined assuming mean values for the material properties and using a fibre model for the analysis of critical sections, wherein appropriate constitutive laws for steel and for unconfined and confined concrete [12, 14] were used.

As far as the stiffness of R/C members is concerned, two different models of each structure were examined:

i. One based on the stiffness assumptions corresponding to moderate levels of inelasticity, i.e. $0.5EI_g$ for all members (EI_g is the stiffness calculated on the basis of gross section properties), as prescribed by EC8 [1]. The first three natural periods of these models of the 15-storey building (which accounted for 96% of the total mass) were 1.10, 0.32, and 0.16 sec.
ii. A second model, wherein the secant stiffness of the section at yield, $EI_{ef}=M_y/\varphi_y$, was used for all R/C members (M_y and φ_y are the yield moment and curvature, respectively). For beams, EI_{ef} was taken as the average of the values obtained for positive and negative bending, while for columns it corresponded to the axial loading (N) due to service loads (this is also the mean value of N during the time-history analysis). The fundamental natural period of these models varied from 1.28 to 1.34 sec.

As the spectral peaks of most records used here typically appear at period values significantly shorter than the above fundamental period values, it is expected that the response of the second, more flexible, model will be more favourable than that of the first (stiffer) model, with the possible exception of the displacements.

Due to the high slenderness ratio of the walls in the structures studied (height to wall ratio equal to 7.7 or more) and the fact that no shear failures preceded flexural ones, the effect of shear deformations on the displacements of the structures was not significant. In all analyses an effective shear stiffness equal to 50% the gross section value (GA) was assumed; a parametric study with values equal to 25% and 100%GA showed that the effect of this assumption on the results was very small.

4. INPUT MOTIONS AND SCALING PROCEDURE

A number of records representative of earthquakes that caused serious damage (including collapses and casualties) in Greece during the period 1975-2000 were used; they included both natural and synthetic records. The three natural records (AegX, A399_L and LefL) are from the 1995 Aegion, the 1999 Athens and the 2003 Lefkada earthquakes, respectively, while records I20_855 and A4 are synthetic ones, derived for typical zones of the cities of Thessaloniki and Volos in the framework of microzonation studies (carried out by the staff of the Geotechnical Section of the University of Thessaloniki) for these cities. The last record used (Ira14) represents a simulation of a severe seismic excitation (having a magnitude of 7.8) caused in the south Aegean Sea at the boundary of lithospheric plates [15]. It should be clear that, as an ensemble, the records used encompass a number of characteristics that are broader than those of a focussed set representing a narrow range of magnitude and epicentral distance; this was done in order to study in a feasible way the effect of frequency content of the records on the calculated response.

Given that the effect of higher modes in the inelastic range changes with the level of inelasticity induced in a structure, the foregoing input motions were scaled to various intensities, which are typically associated with distinct performance levels (limit states), such as 'damage limitation' (associated with a_g=0.16g or 0.24g, i.e. the design earthquake intensity), and 'survival' (non-collapse). For the survival earthquake, an intensity equal to 2 times the design one was selected; these fractions are broadly consistent with hazard curves derived for Greece, for probabilities of exceedance of 10% and 2% in 50 years, respectively. The scaling method used herein is a modified Housner technique suggested by Kappos [14], wherein the scaling factors are derived by considering the areas under the pseudovelocity response spectra of the records used and of the design earthquake (EC8 spectrum), between appropriately defined bounds of the natural period, corresponding to the fundamental periods of the structures under consideration. It is pointed out that, since the fundamental period of a structure changes for different assumptions regarding the effective stiffness (see section 3), slightly different scaling factors were used for each model. It is also noted that scaling of the records to match the design spectrum in the long-period range leads to exaggerated values in the short period range, a feature typical not only to greek records but also to practically all records from shallow earthquakes. This is expected to trigger the second mode of the structures studied (the third one was found to have a negligible contribution, as will be seen in the next section). To evaluate the effect of scaling of the natural records on the results, a second set of artificial, spectrum-compatible, records, was produced using the program ASINC [16], which matched well the entire EC8 spectrum. This second set can be considered as representative of ground motions that are expected to be less critical with respect to the higher mode contributions; however, such records are rather difficult to find, at least in European databases.

5. RESULTS OF PARAMETRIC STUDY

5.1 Seismic behaviour of walls

Prior to presenting response envelopes for the studied structures, it is attempted to gain some insight into their shape and identify their key features; this is essential not only for a proper evaluation of the procedures addressed herein, but also for obtaining a basis for proposing an improved design methodology (see section 6).

Fig. 3 shows selected moment diagrams (snapshots from time-history analysis for records scaled to the design earthquake intensity) for the 15-storey DCH0.24g structure; the superposition of these diagrams results in the envelopes of bending moments and shear forces derived from the inelastic dynamic analysis presented (after proper statistical treatment, namely the calculation of mean values and mean ± one standard deviation values) in Fig. 4 to 9, both for the central (6m) and the corner (2m) walls of the studied structures. The part of these envelopes that is quite different from the envelope determined by elastic modal analysis is clearly the one around the middle of the central walls. The difference should be attributed to plastic hinging and post–elastic dynamic effects. Also noticeable (but anticipated [7]) is that the two types of walls (long and short) do not exhibit the same seismic behaviour; the central (long) one shows a cantilever-like response, while the corner (short) wall responds in a mixed mode, lying in-between that of a cantilever (single curvature) and of a column (double curvature with contraflexure point close to mid-height). Whether a wall responds essentially as a vertical cantilever or not, depends not so much on the aspect ratio of its section but primarily on how stiff and strong the wall is compared with the beams that frame into it at storey levels (see Fig. 1).

Bending moment diagram 1 (M_1, Fig.3) is the one that dominates the envelope only for the base of the walls, namely the yield moment plus the possible overstrength if the wall yields (central wall) or the maximum elastic value if the wall remains within the elastic range. By studying typical moment diagrams for the first two or three modes of vibration of a simple cantilever coupled with frames, one recognises that diagram M_1 indicates a prevalent fundamental mode response. Apart from possible plastic hinging at the base of the walls, M_1 diagrams also reflect inelastic behaviour of beams belonging to the first 3-4 storeys; the remaining members behave elastically. Diagram M_2 is the one that provides the envelope values in the middle part of the walls; a second mode effect can be clearly detected here. This diagram also accounts for the unusual curvature that the actual envelope exhibits (with respect to an elastically derived one i.e. the continuous red, saw-like, diagram of Fig. 4) around the middle part of the central wall; this effect is not so marked in the corner wall. Little or no plastic hinging accompanies this response, mainly as a result of the capacity design of the walls. Finally, the M_3 moment diagram contributes the lower third of the envelopes (except for the base area) and also the envelope for the top three storeys of the long wall. It can not be attributed to a single mode response; it rather seems to be a blend of first and second mode responses. Diagrams similar to those in Fig. 4 to 9 for the 9-storey structures are given in [7] ; they are compared with those of the 15-storey structures (that are the main focus of the present paper) in the following.

Another issue deserving some attention is the effect of the frequency content of the ground motion on the calculated M and V envelopes. In Figs. 4 and 5 these envelopes are provided separately for the first (blue line) and the second (dark green line) set of records (the other figures show results for the first set only). As anticipated, the effect of the second mode is less significant for the second set, especially in the middle part of the wall. However, the M and V envelopes are still quite different from those based solely on the first mode, as can be clearly seen from comparing the envelope from the second set to those from first mode patterns (e.g. the light green curve in Fig. 4, corresponding to EAK2003 design, or the M_1 patterns in Fig. 3).

Figures 10a and 11a compare the normalised moment envelopes (mean values) for the central and the corner walls of the 9-storey and the 15-storey dual systems, respectively (DCH0.16g case). It is clear that the alteration of the analysis-derived moment envelope due to higher mode effects (basically the second one) is clear only in the high-rise case. It is evident then, that for a possible prediction of the actual moment envelopes, one should quantify (in a logical way) the extent to which the second mode of vibration affects the response of the wall.

Figure 12 shows the distribution with height of the $V_{in}/V_{dyn,el}$ ratio (normalised to the respective value of this ratio at the base of each wall), where V_{in} denotes the maximum (in absolute terms) shear forces derived from inelastic dynamic analysis and $V_{dyn,el}$ the shear forces derived from the response spectrum method (design response spectrum). The differences between the two types of walls (central/long and corner/short) are clear. The first one exhibits almost no increase in the value calculated from modal analysis (if the base shear is predicted accurately) up to the mid-height of the wall, and then the $V_{in}/V_{dyn,el}$ ratio can be approximated as linearly increasing up to the top of the wall where the actual ratio is almost double the value at the base. The trend is different for the corner walls, wherein, apart from the wall base, the $V_{in}/V_{dyn,el}$ ratio is almost constant and equal to half the value at the base. By following these two patterns, one for the central (cantilever) wall and one for the corner (mixed cantilever and column action) wall, one can define the design shear force envelope on condition that the wall base shear is adequately predicted (see section 6). Figure 8 shows, inter alia, the possible timing of the peak bending moments and shear forces ('abs M simultaneous to max[(absV)]'). Since the bending moments that develop simultaneously with the peak shear forces are *not* the same as the peak bending moments (see snapshots in Figure 3), it is clear that the peaks of the two response quantities (M and V) are out-of-phase. Maximum shear forces correspond to maximum bending moments only at the (first-mode dominated) base of the wall.

As far as the maximum wall base shear is concerned, one should bear in mind the following observation (refer again to Fig. 8): for the wall base shear to peak, wall base bending moment should be maximized, while the moment at the top of the ground floor segment of the wall should be as low as feasible (this condition maximizes the moment gradient which results in the shear force). This is equivalent to lowering as much as the situation permits the first point of contraflexure, while the base moment remains close to its maximum value. It is noted that for cantilever behaviour, the higher the vibration mode, the lower the first point of contraflexure is located. This can be used in conjunction with the observation that, at least in the 2D case considered herein, the first two modes of vibration typically suffice (as seen previously) for the seismic response of a wall to be described, in order to establish a basis for the estimation of the wall base shear.

The shear behaviour of the shorter walls of the 9-storey dual systems [7] resembles that of the taller ones; the wall base shear depends on the second mode effect which is reduced in comparison to the high-rise (15-storey) case.

The second mode effect was also found to be sensitive to the frequency content of the records used. Wherever the second mode affects the response (middle part of the wall for bending moments, and the base section for shear forces) the dispersion in the calculated response quantities is maximized (see curves for $\pm\sigma$ in Figures 4 to 9).

Finally it has to be mentioned that the maximum wall base shear calculated for the corner walls, is in general more than q times the value calculated from dynamic or equivalent static analysis of the dual structure for the design response spectrum. This is attributed to the post-elastic dynamic (higher mode) effects, as well as to the substantial overstrength due to minimum reinforcement and other design requirements. The Eurocode (as well as the Greek and other national codes) consider the unfactored (no reduction by q) value from elastic analysis as an upper bound to the response (cf. Equation 1); however it is shown here that in cases of substantial overstrength and/or significant higher mode effects (i.e. tall buildings), this is not a proper upper bound.

5.2 Evaluation of the wall design procedures

The most critical issue with regard to the evaluation of the flexural design procedures, is the failure of the EC8-prescribed bending moment envelope to adequately "envelope" the inelastic response of the central walls; this was found to be the case irrespective of the ductility class (M or H) or the level of seismic action (0.16g or 0.24g) considered in design (Figs. 4, 6, 8). This was anticipated (to some extent) since this design envelope is not defined, as it ought to, with respect to the moment of resistance (i.e. the flexural strength) of the potential plastic hinge region. When the overstrength of the plastic hinge is substantial (due to design requirements such as minimum reinforcement ratios and/or the thickness required to satisfy shear checks), the discrepancy between the design values and the calculated inelastic response actions is even greater. As a result, yielding is expected in other than the critical (base) regions. This point also applies for the 9-storey central (6m) walls; however it is less significant there, due to the predominantly first mode response (Fig. 10). The rationale behind such a design provision, could be that EC8 anticipates the wall section dimensions to be carefully chosen with a view to excluding the possibility that minimum requirements prevail. Of course, the general case can not be tackled in this way, and it was shown in Section 2 that there are good reasons (such as the need to satisfy requirements other than flexure) why significant overstrength may result in the wall critical region. Design bending moments calculated from the EC8 formula seem to be insufficient also for short walls, but to a lesser extent (Fig. 6), compared to the long ones. This is a general remark for almost all the design procedures evaluated herein , and is attributed to differences in the seismic behaviour of these walls.

For the earthquake associated with the no-collapse requirement (twice the design intensity), yielding of the wall elements (in other than the base sections) was even more pronounced; in this case yielding was also detected along the height of the corner walls. Nevertheless, the calculated interstorey drifts nowhere exceeded the 3% limit [14], while no sign of a possible formation of a "sidesway" mechanism was detected. The latter is notable in the sense that no care for avoiding plastic hinging in columns was taken (it is not required in wall-equivalent dual systems), while the elements which were supposed to provide safety against possible soft-storey mechanism formation (i.e. the walls) yielded even for the design earthquake. In fact, only a minimal fraction of the columns did actually yield. This was mainly attributed to code requirements with regard to the normalized axial forces allowed in columns, which led to column section dimensions quite larger that those actually needed for flexural strength; thus, minimum reinforcement requirements also governed the design of columns. Therefore, the possibility can be envisaged to relax the ductility requirements for columns that form part of a dual system, when the walls carry more than (about) 80% of the total base shear (used as a convenient substitute for total strength, as also done in EC8). A more 'liberal' distribution of lateral forces among columns and walls can also be seen as a possibility, i.e. dropping the strict rule of assigning storey shears on the basis of fixed percentages of the member rigidities and distributing freely (and more or less arbitrarily) the base shear between the frames and the walls, a concept put forward by Paulay [17], and subsequently used in the displacement-based design of dual systems [8, 9]. It is beyond the scope of this paper to evaluate the feasibility of this approach, which, in the authors' opinion, has both advantages and disadvantages.

The Greek Seismic Code procedure for flexural design, although accounting for possible plastic hinge formation in the critical region of the wall, does not hint to significant post-elastic dynamic effects; the second-mode-affected part of the calculated envelope for the central walls (the central part of the wall) is by no means covered by this envelope (Fig. 4). Also noticeable is that the design action effects in the upper third of the central wall are much higher than the calculated ones. In contrast, this first-mode-dominated envelope, was found [7] to perform well for the central walls of the 9-storey structures where the higher mode effects are limited with regard to the flexural response.

Of the bending moment envelopes examined here, that of Paulay and Priestley [2] was found to be the most efficient one in terms of tackling the higher mode effect. However, for this to actually envelope the moments calculated from inelastic time-history analysis, a high overstrength factor has to be used (1.4), and the additional resistance due to tension shift should be taken into account (Figs. 4, 6, 8); the latter is not strictly legitimate, as the tension shift is intended for accommodating tensile forces caused by diagonal shear cracks, not by flexure. On the other hand, the New Zealand method was found to be conservative for the corner walls, and also the central walls of the 9-storey systems where higher mode effects are less pronounced.

Therefore, as far as the bending moment envelopes are concerned, none of the examined design procedures seems to adequately handle the general case, i.e. walls with different seismic behaviour (cantilever vs. mixture of column and cantilever) and/or exhibiting different higher mode effects (15-storey vs. 9-storey).

Shear force envelopes (Figs. 5, 9) depend on the wall base shear, since they are defined on the basis of this value. EC8 adopts a different calculation method for medium and high ductility walls; the Keintzel [4] formula, adopted only for DC H structures, seems to work well (it envelopes the actual shears without being over-conservative), at least for the central walls (Fig. 5). It is also the more transparent relationship, since it incorporates separately the factors related to overstrength and to higher mode effects. The EAK2003 procedure for shear design is unconservative for most of the height of the central walls, despite the relatively high overstrength factor (1.3) used (in lieu of a sophisticated rule for accounting for higher mode effects). The effect of overstrength factor is even more obvious in the Paulay and Priestley methodology; the factor used (1.4), as well as the dynamic magnification (ω_V), render this wall base shear calculation as the most conservative in all cases. It should be mentioned, though, that the strain-hardening assumed for steel in the present analysis was only 15%, which is consistent with tempcore steel currently used in most European countries, but lower than that for steel used elsewhere, e.g. in New Zealand.

The modified formula by Kappos and Antoniadis [7], which accounts for the presence of walls of unequal length (modified n_V factor), does not change the previous observations, since, in the rather extreme case studied here (large difference in wall lengths), the central walls carry the main share of the total base shear (80%, while corner walls carry only 12%). Distribution of design shears along the height appears to be markedly different in each approach; these differences would have been minimized if the same wall base shear was used in all procedures. On the basis of the inelastic analysis results, it appears that a slight modification of existing code procedures is probably needed for the upper third of the wall shear design envelope; this is a feature captured by the modified envelope proposed in [7]. It is also clear that the 50% increase in the calculated shear forces, prescribed for the medium ductility case in EC8 (Fig. 9), is insufficient and leads to unsafe solutions, and so does the EAK2003 procedure. Similar differences regarding the adequacy of available design procedures for the long walls were also spotted out in the case of the 9-storey systems [7].

Applying the same shear design procedures to the corner walls (Fig. 7), leads to essentially the same conclusions with regard to the EAK2003 and EC8 procedures. Paulay and Priestley's method exhibits a much higher degree of conservatism; this is mainly due to the dynamic magnification factor, which is held constant irrespective of the type of wall considered (long or short). To remedy this situation (which was also clearly detected in the shorter, 9-storey, walls) the n_V factor of the Paulay and Priestley method was modified [7], for the relative contribution of walls with different length to be considered. This modified version leads to less conservative design shear forces for the corner walls both for the 15-storey and the 9-storey structures .

6. A TENTATIVE PROPOSAL FOR THE CAPACITY DESIGN OF WALLS

The results of the foregoing parametric study point to the relevance of attempting to develop a method (retaining the simplicity which characterizes those in existing codes) for defining design envelopes of wall bending moments and shear forces in dual structures (axial forces can be taken as those from the seismic load combination, as suggested by EC8, or other code procedures can be used, e.g. [3]), taking due account of some clear trends identified in this (admittedly, limited) study. The method presented herein takes into account the finding that the first two modes of vibration (in the direction considered) suffice for describing the in-plane seismic response of medium-rise and medium-to-high-rise walls, in a practical context. Clearly, at this stage of development wherein an effort was made to concentrate on the essential issues and arrive at some usable conclusions that could benefit design practice, only 2D configurations were studied; therefore, issues such as coupled transverse-torsional response (in 3D buildings) cannot be directly tackled by the approach used herein, and require further study.

From the analysis point of view, only an elastic modal analysis using the design response spectrum is required, but the results for each mode should be stored in a way that they can be processed in two phases, as described in the following. First, the bending moments calculated by combining modal responses in the standard way (i.e. accounting for modes that contribute 90% of the total mass) and dividing by the force reduction factor (q). Then all potential plastic hinge regions should be designed, including the bases of the walls.

For the final design envelopes of the walls to be constructed, it is proposed to use the elastic (unfactored) bending moments and shear forces for the first two modes of vibration. As discussed in Section 5, the second mode affects the moment envelope mainly around the middle of the building height (the lower part is affected mainly by the first mode). Hence, the *mid-height point* is introduced in the proposed envelope, in a similar fashion as in the

New Zealand procedure for walls (see §CD4.2 of [3]). The design bending moment of this point of the wall is first estimated via the following expression:

$$M_{Ed}^{m} = \omega \cdot M_{2}^{m} \cdot \min\{1, 1.2M_{Rd}^{b} / M_{2}^{b}\} \quad (4)$$

where M_{Ed}^m is the design bending moment of the mid-height point of the wall, M_2^m and M_2^b are the bending moments at the middle and the base point of the wall corresponding to the second mode of vibration calculated from the dynamic analysis for the elastic response spectrum, M_{Rd}^b is the moment of resistance of the wall base section (plastic hinge section) and ω is a dynamic magnification factor which can be expressed, performing regression on the results of dynamic analysis, as a function of the fundamental period of the structure:

$$\omega = 2.9T - 1.0 \geq 1.0 \quad (5)$$

If a coefficient 3.0, instead of 2.9 (the value from regression), is used in (5), the difference in the calculated moments is less than 5%, hence acceptable for design purposes. It is noted that regression was based on the results from the first set of records, since a design proposal should be based on the 'unfavourable scenario', particularly so since this also happens to be the more plausible one (the records in the set are typical of conditions in S. Europe and other parts of the world where shallow earthquakes are the rule). The dual criterion in equation (4) (i.e. use of the minimum of two values) is meant to account for the possibilities that yielding at the base can be dominated either by the first mode (most common case) or by the second one. For the base moment the value $1.2M_{Rd}^b$ is proposed; note that the overstrength factor of 1.2 is identical to the EC8 prescribed one (hence, it is consistent with steel properties currently used in Europe) and that vertical web reinforcement should be taken into account in the calculation of the flexural resistance M_{Rd}^b. The previously defined mid-height point is connected with the base moment and with the design moment (see below) at the top, with straight lines, to complete the design envelope. To make the method applicable also in the case of short-to-medium-rise walls, where minor or no higher mode effects are expected, an additional feature is necessary: The design bending moment at any location should not be taken less than the values obtained by multiplying the envelope of bending moments calculated by standard analysis, with a constant capacity design factor equal to 1.2 times the ratio of the flexural strength of the wall at its base (M_{Rd}^b) to the corresponding moment from analysis (M_{Ed}^b).

Results from the application of the preceding method are presented in Figures 4, 6, and 8, for both central and corner walls; note that the diagrams include the aforementioned $1.2M_{Rd}^b/M_{Ed}^b$ magnification. It is clear from the comparisons with the actual (dynamic inelastic) moment envelopes that the middle point of the proposed envelope, offers flexibility to the method, rendering it capable of describing the response of walls with different geometries.

For a design *shear force envelope* to be defined, the wall base shear should be first estimated. The condition under which the wall base shear is maximized was explored in the previous section; the key idea is that for a given moment at the base (resulting from the design for flexure) maximum shears result from the dynamic force pattern whose resultant is closest to the base (this is the pattern wherein the first point of contraflexure is located closest to the base). This is implemented by first considering the following (initial) wall shear force:

$$V_{Ed,i}^{b} = V_{2}^{b} \cdot \min\{1, M_{Rd}^{b} / M_{2}^{b}\} \quad (6)$$

where V_2^b is the shear force at the base of the wall corresponding to the second mode of vibration calculated from dynamic analysis for the elastic response spectrum (the other notations remain unchanged). Given that the first two modes were found to describe well the response of the walls, this term ensures that the second mode response is maximized, which is equivalent to accounting for the point of contraflexure that is closest to the base.

In the case that the second term in brackets governs, this initial shear force should just be multiplied with the overstrength factor (1.2). Otherwise, the initial shear force should be increased by adding the term:

$$V_1^b \cdot \frac{1.2 \cdot M_{Rd}^b - M_2^b}{M_1^b} \tag{7}$$

where V_1^b and M_1^b are the bending moment and shear force at the base point of the wall corresponding to the first mode of vibration (for the elastic response spectrum). This term ensures that the base moment retains its maximum value.

Variation with height follows one of the two patterns already described in section 5.1; one for walls which exhibit a cantilever behaviour (such as the central walls here) and one for walls with a 'mixed' behaviour (Fig. 12). Figures 5, 7, 9 present the application of the previously described procedure to both central and corner walls. For the distinction between a pure cantilever behaviour and a mixed one with column features, a rather simple recommendation can be given: If the first point of contraflexure for the first mode response is located lower than 1/3 of the wall height, then a mixed behaviour should be normally expected. In all cases the shape of moment diagrams can be used as an effective predictor of the anticipated response.

7. CONCLUSIONS AND RECOMMENDATIONS

A review of leading current code procedures for capacity design of walls in dual structures has shown that most codes address (but do not necessarily tackle adequately) the issue of post-elastic dynamic effects. Bending moment envelopes are defined either 'graphically' (Eurocode 8 [1], and the New Zealand 3101[3], by a broad reference to Paulay and Priestley's [2] proposed envelope) or 'analytically', via magnification factors and minimum values for the upper part of the walls (Greek Code). Magnification factors for shear action effects account in some cases for both flexural overstrength (at the wall base) and higher mode effects subsequent to the formation of plastic hinges (EC8 and NZS3101), and in other cases only for the overstrength factor (Greek Code). Higher mode effects in the upper part of the walls are addressed either by specifying minimum values for wall shears (Greek Code), or by a more sophisticated design envelope (EC8, and NZS3101).

From the application of the aforementioned procedures to 9-storey and 15-storey dual systems, having walls of unequal length, designed to the provisions of Eurocode 8 for two different ductility classes (M and H) and two different design acceleration levels (0.16g and 0.24g), that were then subjected to a number of input motions and analysed in the inelastic range, it was possible to evaluate the code-specified design action effects against values calculated from inelastic time-history analysis. It is found that the formation of plastic hinges at the base of the walls and post-elastic dynamic effects lead to a greater variation between the design action effects calculated directly through an elastic modal (or equivalent static) analysis and those calculated from time-history analysis.

On the basis of the results presented here, it is clear that code procedures do not adequately cover the general case, i.e. walls of different seismic behaviour (cantilever vs. mixture of column and cantilever) and different height (15-storey vs. 9-storey), which exhibit different higher mode effects. It is clear, though, that code procedures should account for flexural overstrength, inelastic higher mode effects both in the lower and in the upper part of walls, and relative strength of each wall with respect to horizontal loading. Whether this should be the case for high ductility structures only, or for 'limited ductility'

(DC M) ones as well, is probably more of a 'political' than a technical question, in the sense that the obvious answer would be affirmative, unless other considerations such as ease of application and/or economy of the resulting design (regardless of seismic performance) enter the picture.

A new method, retaining the simplicity of the existing ones, was tentatively proposed here. The hallmark of this procedure is that it quantifies the second mode effect for both bending moments and shear forces. The resulting envelopes, although based on the limited number of 2D configurations studied, are deemed to apply to a rather broad class of medium-rise and medium-to-high-rise walls, forming part of dual structures, and being characterised by different seismic behaviour and different post-elastic dynamic effects.

It has to be emphasized that due to minimum reinforcement requirements and/or due to the structural configurations considered, the effect of the q-factor on the design action effects of the walls was not fully explored. Hence additional parametric studies are recommended, involving more structural configurations (2D as well as 3D, different number of storeys and different ratios of wall to total floor area), to provide a sufficient background that would then permit revising existing code procedures.

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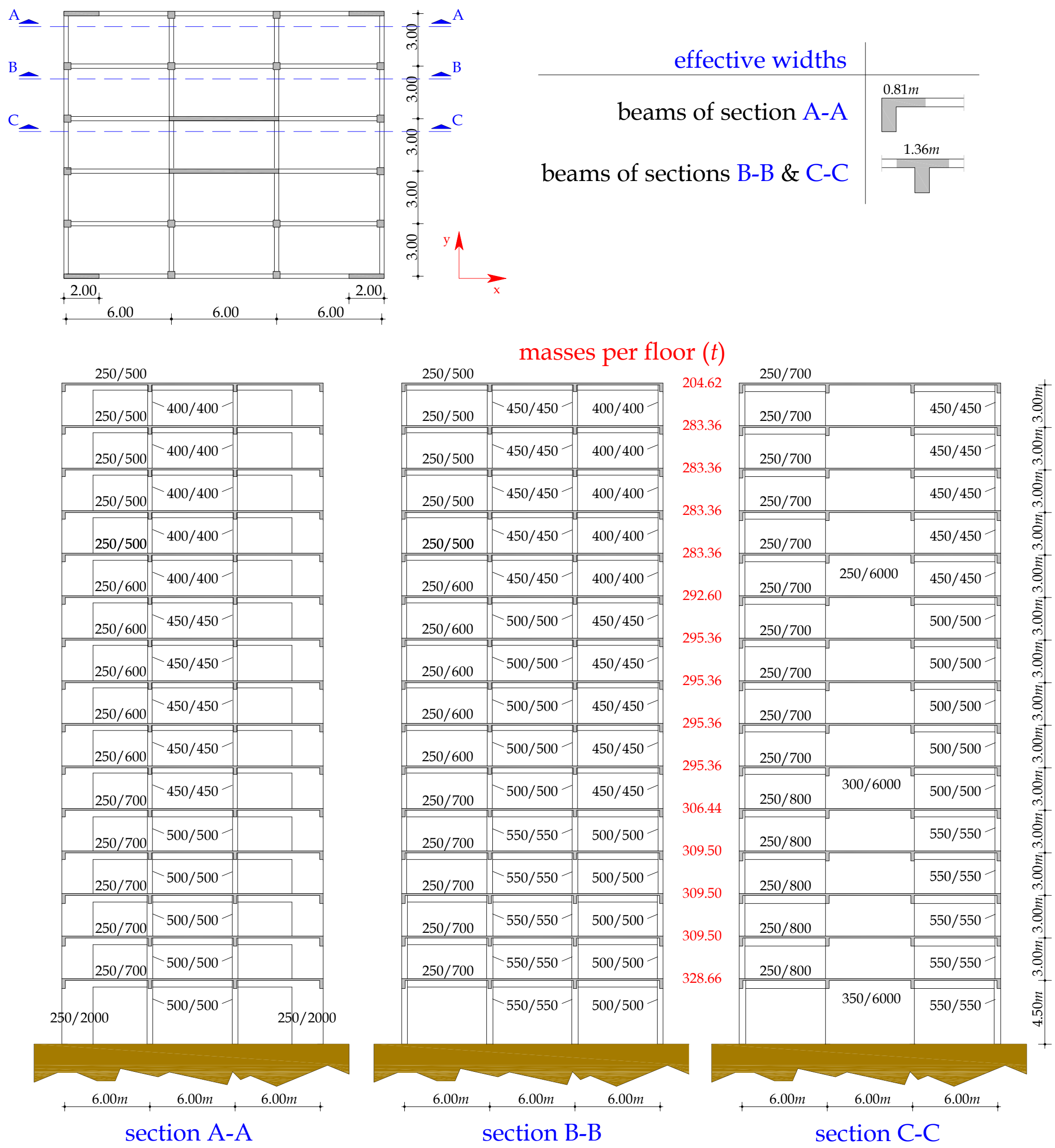

Fig. 1: Configurations in plan and in elevation and geometric data for the 15-storey building (same plan configuration applies to the 9-storey building, see [7]).

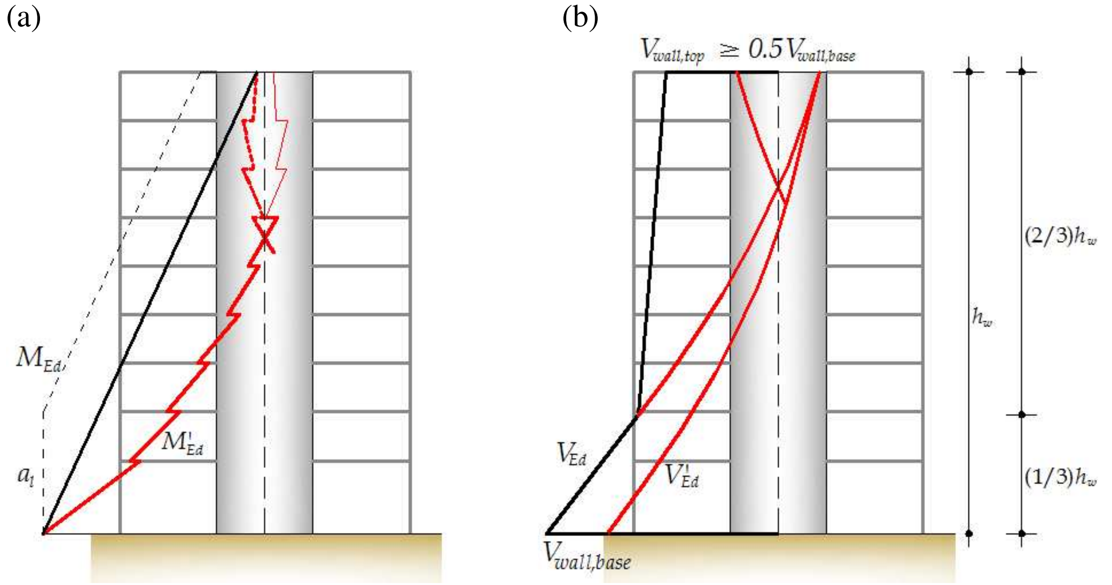

Fig. 2 Design envelopes for walls of a dual system (Eurocode 8 procedures):
(a) Bending moment envelope M_{Ed}, moment from analysis M'_{Ed} and tension shift a_l;
(b) Shear force envelope V_{Ed}, shear diagram from analysis V'_{Ed}.

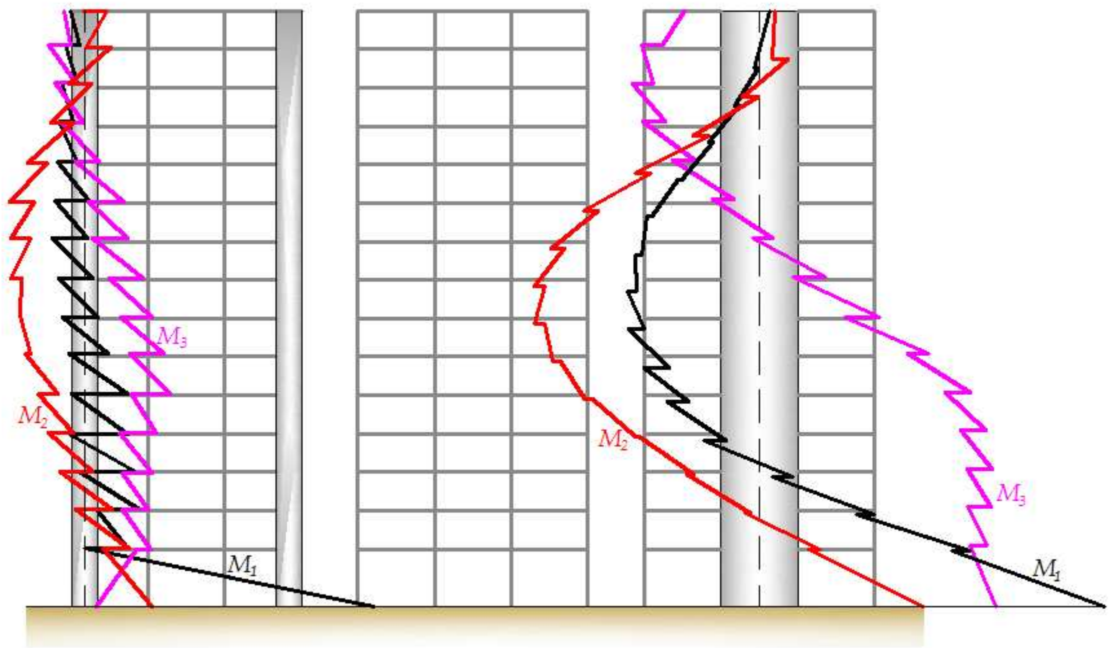

Fig. 3 Bending moment diagrams (snap shots from time-history) for Thessaloniki I20_855 record, DCH 0.24g 15-storey structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions).

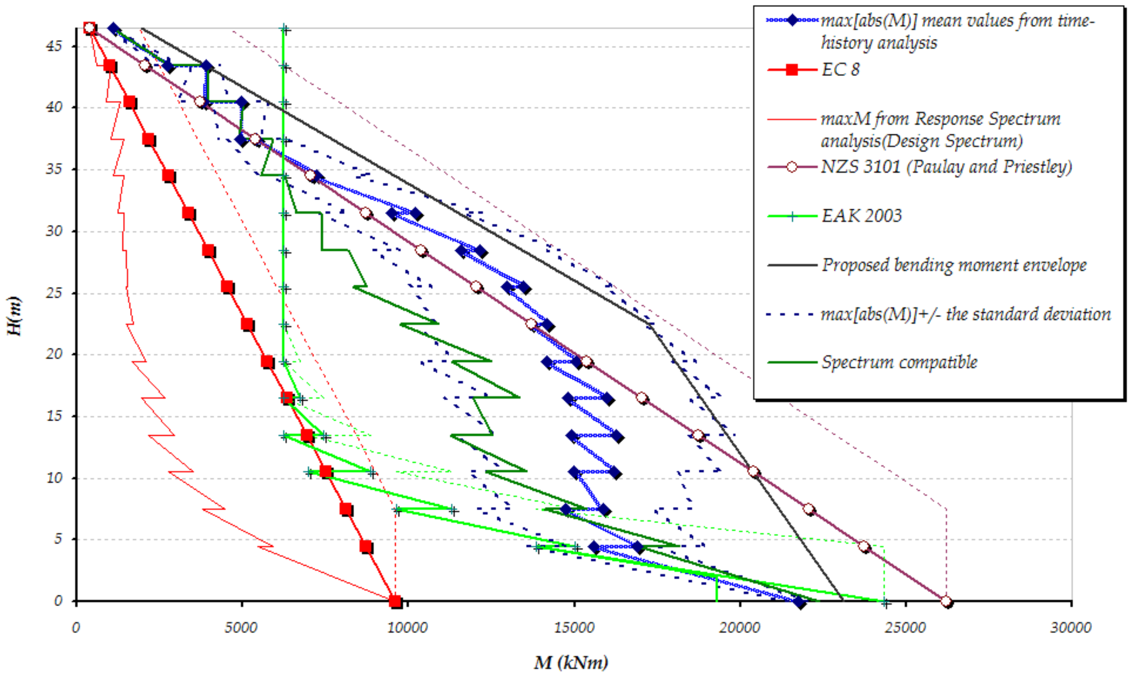

Fig. 4 Envelopes of bending moment diagrams, DCH 0.24g 15-structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Central wall.

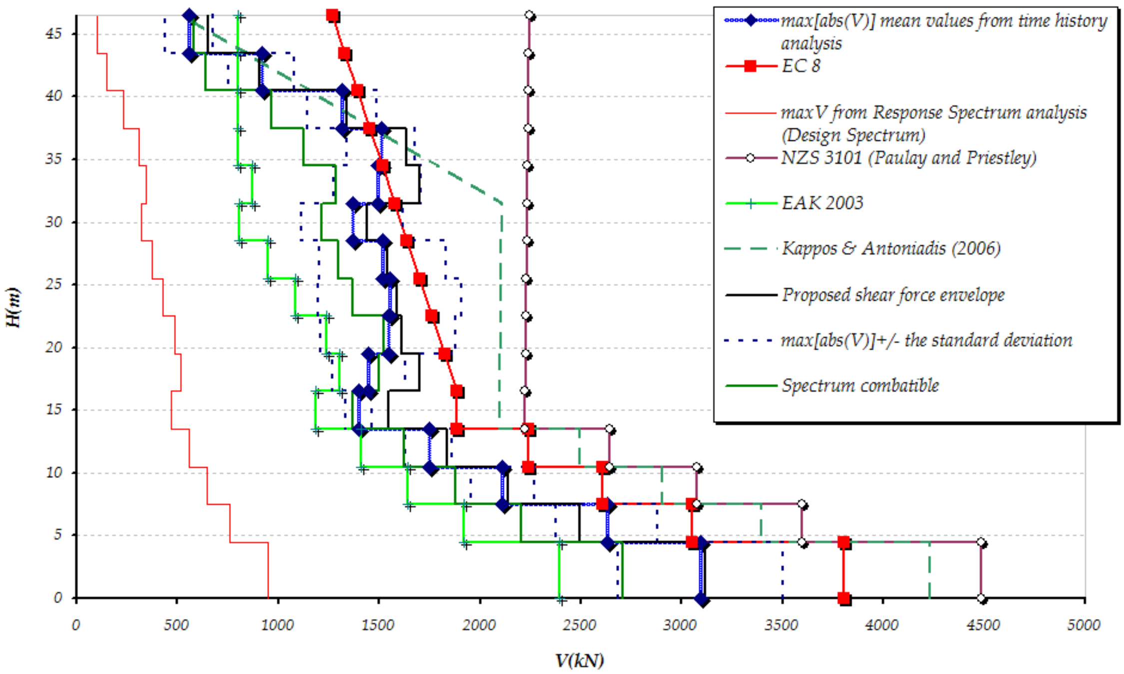

Fig. 5 Envelopes of shear force diagrams, DCH 0.24g 15-structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Central wall.

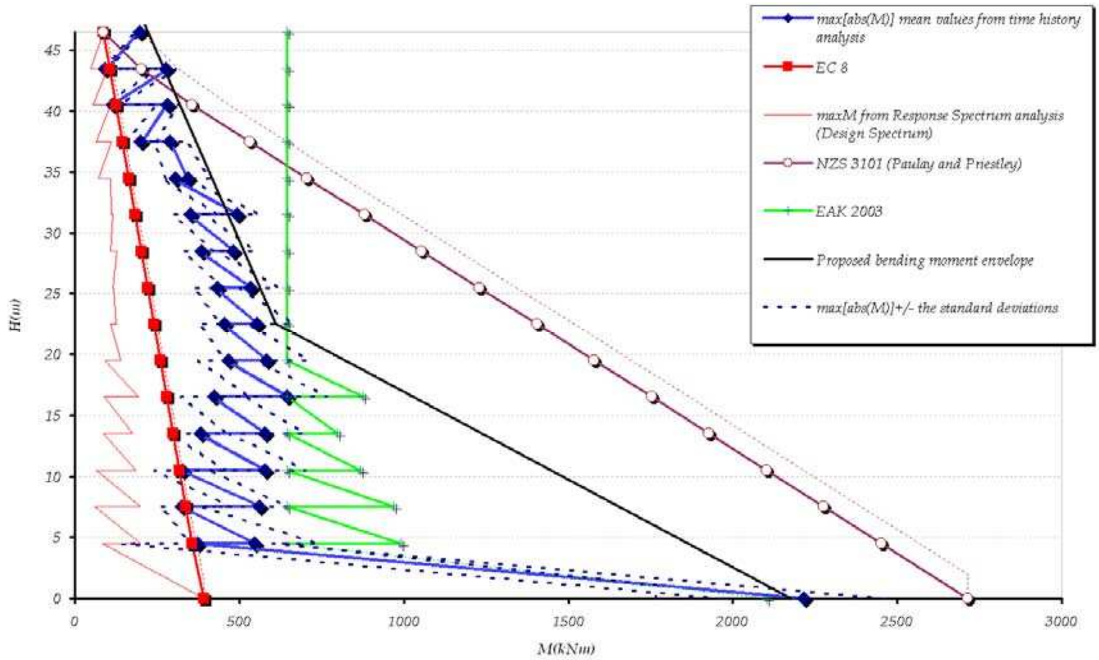

Fig. 6 Envelopes of bending moment diagrams, DCH 0.24g structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Corner wall.

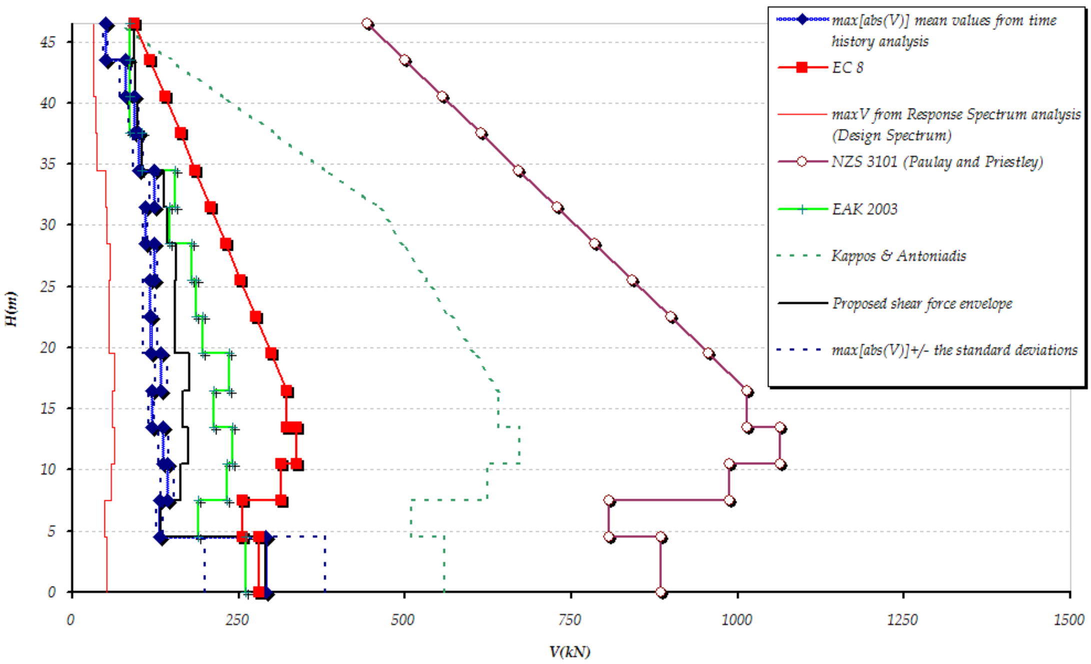

Fig. 7 Envelopes of shear force diagrams, DCH 0.16g structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Corner wall.

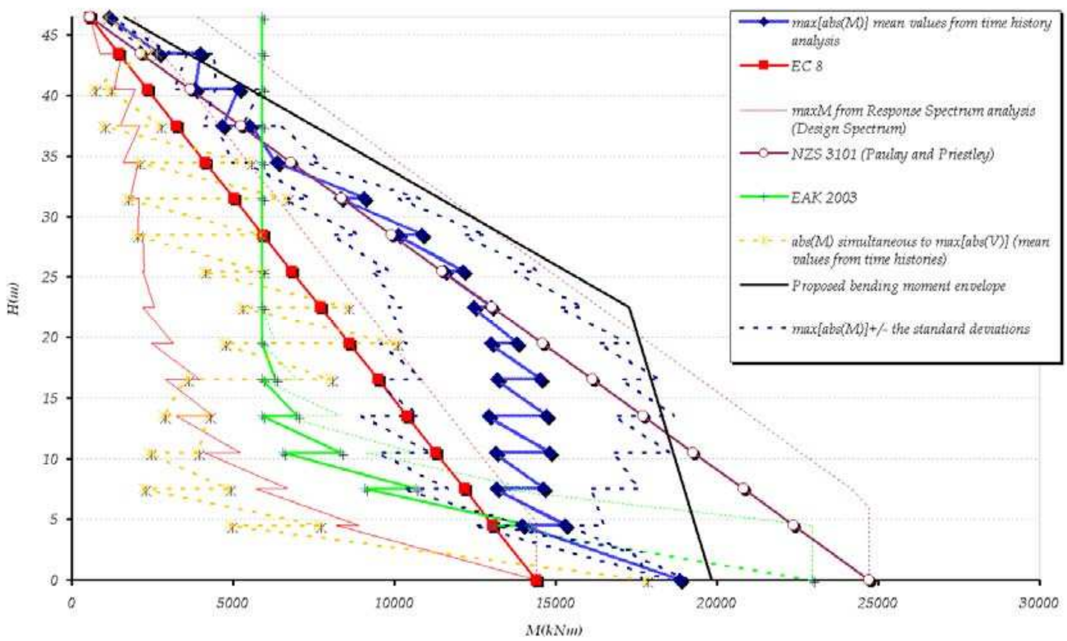

Fig. 8 Envelopes of bending moment diagrams, DCM 0.24g structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Central wall.

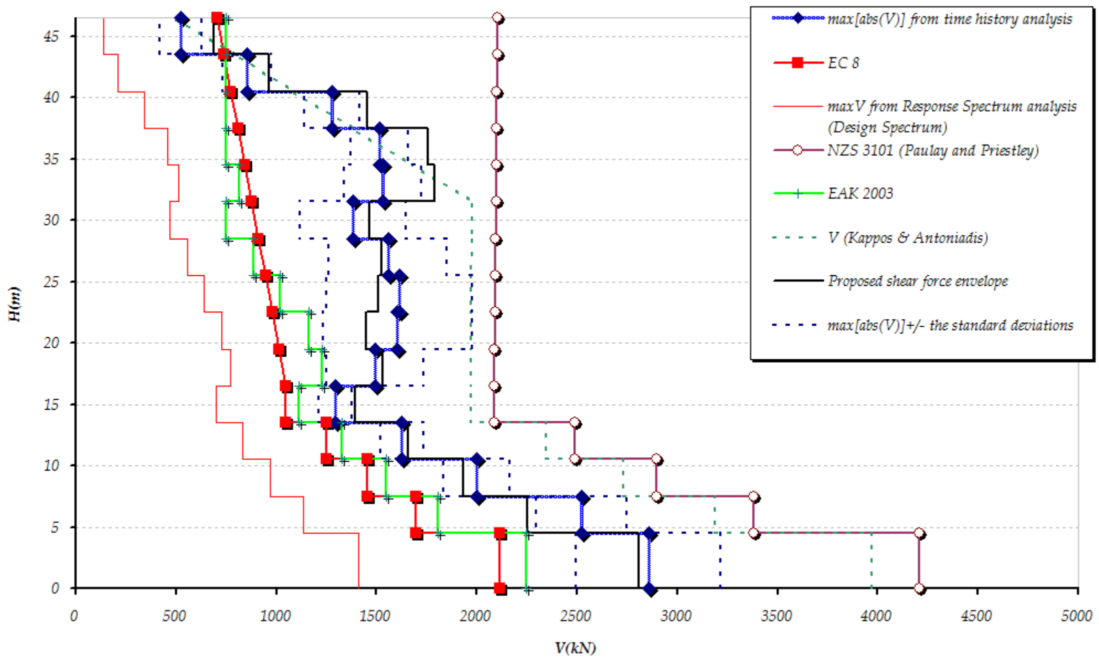

Fig. 9 Envelopes of shear force diagrams, DCM 0.16g structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Corner wall.

(a) (b)

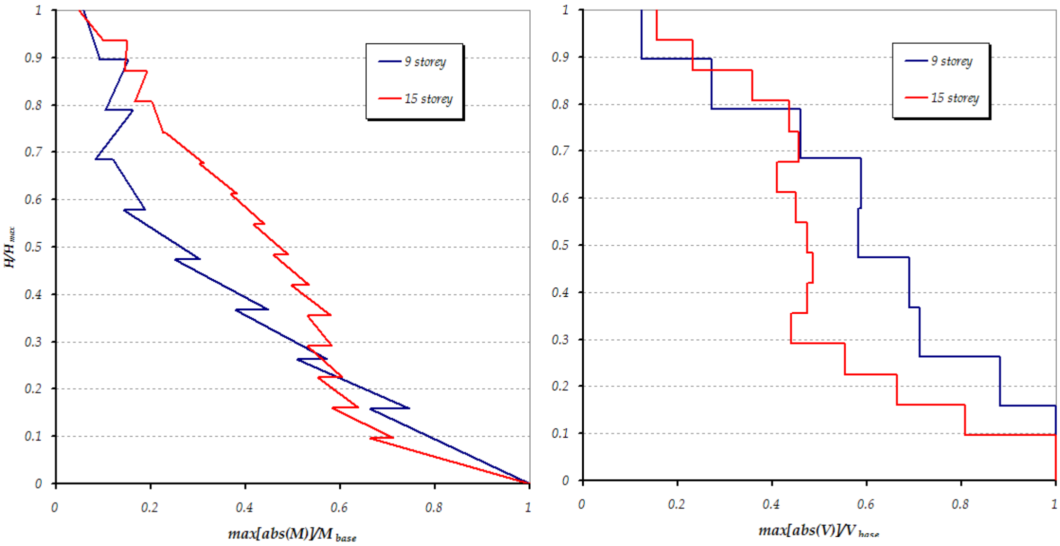

Fig. 10 Normalized envelopes of (a) bending moments and (b) shear forces, for DCH 0.16g structures (both 9 and 15 storey cases), assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Central walls.

(a) (b)

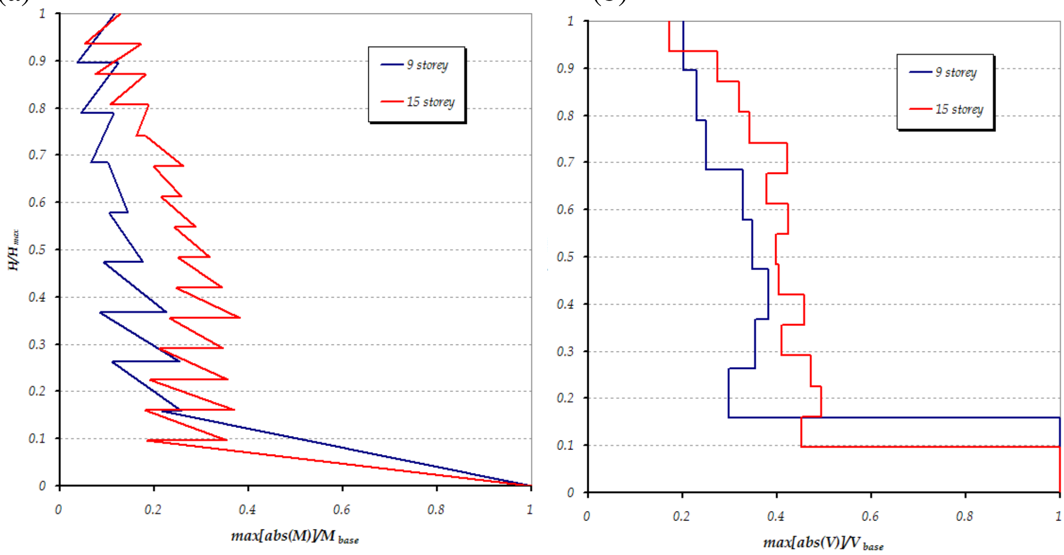

Fig. 11 Normalized envelopes of (a) bending moments and (b) shear forces, for DCH 0.16g structures (both 9 and 15 storey cases), assuming the design earthquake intensity and moderately cracked members (EC 8 provisions). Corner walls.

(a) (b)

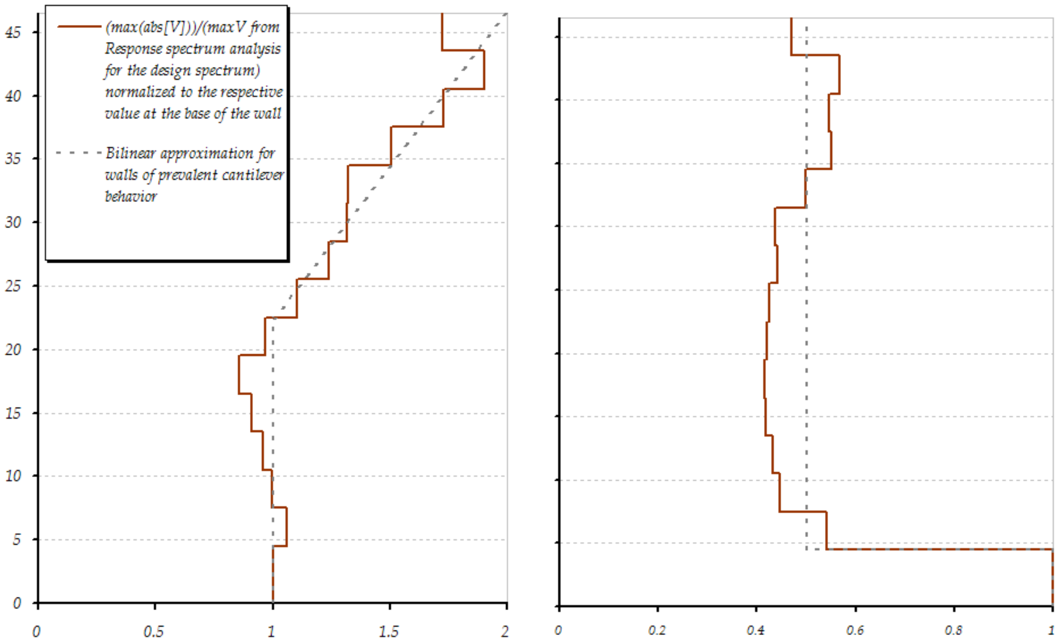

Fig. 12 Normalized envelopes of shear forces, for (a) the central wall of DCH 0.24g 15-storey structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions) and (b) the corner wall of DCM 0.24g 15-storey structure, assuming the design earthquake intensity and moderately cracked members (EC 8 provisions)

Table 1. Basic data for the design of the walls studied

	Reinforcement type			
	(1)	*(2)*	*(3)*	*(4)*
Central walls of DCH structures				
Ground storey – 1st (critical region)	20∅12	∅6/70	2×∅10/200	2×∅10/200
2nd(first storey above critical region)	20∅12	∅6/90	2×∅10/200	2×∅10/200
3rd – 4th	18∅10	∅6/200	2×∅10/200	2×∅10/200
5th- 14th	18∅10	∅6/200	2×∅10/250	2×∅10/250
Corner walls of DCH structures				
Ground storey (critical region)	12∅10	∅6/60	2×∅8/200	2×∅8/150
1st (first storey above critical region)	12∅10	∅6/110	2×∅8/200	2×∅8/200
2nd – 14th	6∅10	∅6/200	2×∅8/200	2×∅8/200
Central walls of DCM structures				
Ground storey – 1st (cr. region)	10∅12+4∅14	∅6/90	2×∅8/400	2×∅8/250
2nd – 4th	8∅12+4∅14	∅6/200	2×∅8/400	2×∅8/250
5th – 9th	8∅12+4∅14	∅6/200	2×∅8/400	2×∅8/300
10th- 14th	8∅12+4∅14	∅6/200	2×∅8/400	2×∅8/400
Corner walls of DCM structures				
Ground storey (critical region)	4∅10+4∅12	∅6/80	2×∅8/400	2×∅8/400
1st – 14th	6∅10	∅6/200	2×∅8/400	2×∅8/400

(1) Longitudinal reinforcement in the boundary elements
(2) Transverse reinforcement in the boundary elements
(3) Vertical web reinforcement
(4) Horizontal web reinforcement