Recent developments in seismic design of reinforced concrete dual systems

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1. Introduction

The design of frame-wall systems has emerged over the years, as the favoured method of providing effective earthquake resistance in multi-storey buildings. The prescribed force-based approach in the current European seismic code, Eurocode 8 (CEN 2004) for the design of high ductility dual systems typically leads to over-conservatism as it overestimates the design forces in the walls especially in the upper half of the building height. On the other hand, in medium ductility structures, design shear and bending moment envelopes for shear walls according to Eurocode 8 (EC8) do not appropriately account for the overstrength of plastic zones which increases the action effects. Therefore, the resulting envelopes obtained from the code-based approach are not completely satisfactory (Kappos & Antoniadis, 2011).

A modified procedure for developing design envelopes for shear walls in dual systems has been proposed by Kappos and Antoniadis (2006, 2011). These modified envelopes better capture the actual behaviour of the shear walls, accounting for wall overstrength in the critical region (bottom half) and economising the wall design in the upper half of the wall, while they are not more complex than the existing EC8 procedure. This study presents an extension of the proposed flexural and shear design procedures, formulated using 2-D dual frames, to the design of walls in 3D dual building systems. In the range of heights common in practice, 3D buildings are mainly affected by four predominant translational modes (two in each direction) and also a torsional mode, and thus may respond differently from the equivalent planar frames which do not account for torsion effects.

The work presented herein aims at optimising the design of shear walls in dual systems by reducing the cost of reinforcement without jeopardising the seismic safety of the buildings. In the following, a parametric study is presented, concerning a total of four dual systems, all 9-storey high, designed according to the EC8 provisions for two different ductility classes (Medium and High) and two different peak ground acceleration levels (0.16g and 0.24g) that are commonly used in Europe. Results from 3D dynamic inelastic response-history analyses are compared with those from the application of the design procedures of EC8, as well as the Kappos and Antoniadis (2011) methodology adapted here to 3D buildings.
2. Structures Studied and Design Procedures

2.1. Structural Configurations Studied

The basic structure considered for this study is a 9-storey reinforced concrete building comprising of a central core and L-shaped shear walls located in each corner of the building, as shown in Figure 1. The total height of the building above its base is 28.5 m (3 m typical storey, 4.5 m ground storey). The slab thickness is 130 mm, corner walls have a thickness of 250 mm, and the 350 mm thickness of the central walls reduces by 50 mm at every fourth storey.

![Figure 1. Configuration in plan for the 9-Storey dual system](image)

The structure can be classified according to EC8 as a ‘wall-equivalent dual system’ the contribution of walls being greater than 50% but less than 65% of the base seismic shear in the structure in both lateral directions. In prior studies involving 2D models (Kappos & Antoniadis, 2006, 2011), similar configurations were studied, which allows a meaningful comparison of the results in the 2D and 3D cases.

2.2. Design Procedures: General Considerations

The design of the dual structures was carried out according to the provisions of the pertinent Eurocodes (2 and 8), for the medium (DCM) and high (DCH) ductility classes, and for effective peak ground accelerations of 0.16g and 0.24g. The ground conditions were assumed to match those prescribed for subsoil ‘type C’ according to the EC8 classification. Identical member cross sections were used for all ductility classes and ground accelerations levels to ensure that the dynamic characteristics in the elastic range of the structures are the same, thus creating a solid base for a direct comparison of reinforcement requirements. The structure examined in the current study utilises Grade 30/37 concrete, since a lower concrete grade is not permitted if diagonal compression (shear) checks are to be satisfied with the chosen wall dimensions. Grade 500 steel of class C is used for reinforcement, as per usual European practice. The values of the behaviour factor (q) according to EC8 provisions are 5.4 and 3.6 for the DCH and
DCM structures, respectively. Concerning the design of walls in particular, all capacity design requirements of Eurocode 8 regarding design of walls were applied, which included detailed design for flexure and shear (diagonal tension, diagonal compression, and sliding shear failure checks) according to the corresponding shear and flexure envelopes, and proper detailing for local ductility was carried out (confined boundary elements, minimum web reinforcement).

2.3. Flexural Design Procedures for Walls

According to the EC8 capacity design procedure, the bending moment diagram along the height of the wall, irrespective of the ductility class, is given by an envelope of the bending moment diagram from the analysis (static or dynamic), vertically displaced due to the tension shift. Note that the envelope is not defined with respect to the actual strength of the plastic hinge which is not in agreement with usual capacity design principles (Kappos & Antoniadis, 2011).

In the Kappos and Antoniadis method the design of the shear wall at the base is first carried out (using the moments from analysis) to obtain its moment of resistance \( (M_{Rd}) \). The design envelope is then developed with respect to the strength of the potential plastic hinge at the base of the wall. The first step in constructing the flexural design envelope is to multiply the moment of resistance at the base \( (M_{Rd}) \) by an overstrength factor of 1.2 to obtain the magnitude for the design envelope at the base (Equation 1). The remainder of the flexural design envelope requires values from the second translational mode \( (M'_{2}) \) in each direction of the building according to the original methodology developed for 2D structures (Equations 2, 3). It should be noted that the 3D model of the studied building does not have a significant contribution from higher modes, therefore the bending moment magnitudes from the secondary translational modes in each direction are significantly lower than those from the primary modes (approximately one tenth). Hence, the additional criterion introduced in the original methodology governs, according to which the bending moment envelope at any point should not be less than the elastic bending moments from analysis \( (M'_{Ed,el}) \), multiplied by 1.2 times the ratio of flexural strength of wall at base to the corresponding moment from analysis (Equation 4). This criterion was used to develop the remainder of the flexural design envelopes where the values obtained from analysis were multiplied by the factor introduced in Equation 4 below, and values were plotted at each story level. To obtain a linear envelope, the maximum bending moment value corresponding to this criterion was selected above the mid-height of the wall. This magnitude was plotted at the top of the wall and connected linearly to the mid-height point and to the design envelope value at the wall base.

\[
\begin{align*}
M_{Ed, Base} &= 1.2 M_{Rd, Base} \\
M_{Ed, Mid} &= \omega \left( \min \left( 1, \frac{M_{Ed, Base}}{1.2 M_{Rd, Base}} \right) \frac{M_{Ed, Mid}}{M_{Ed, Base}} \right) \\
\text{where, } \omega &= 2.9T - 1 \geq 1
\end{align*}
\]
Lastly, the proposed flexural design envelopes were vertically displaced by the amount of the tension shift at the base (kept constant along the entire height of the wall), as shown in section 4.

2.4. Shear Design Procedures for Walls

The shear design envelopes for the Eurocode 8 method were constructed by increasing the shear forces obtained by analysis by a factor $\varepsilon$ which was taken as 1.5 for DCM structures and from the code-prescribed relationship for DCH structures. Above one-third of the height, the value of the envelope is constant, equal to one-half the value of the base shear envelope.

The shear design envelopes for the modified method were constructed following the procedure outlined in Equations 5-8.

$$V_{wall, top} \geq \begin{cases} \frac{1}{8} V_{wall, base} \\ V_{top, el}^{Base} \\ 1.2 \frac{M_{Base, el}}{M_{Base, el}} \\ M_{Base, el} \end{cases}$$

$$V_{Ed} = V_{Ed, i} + V_i^{h, el} \geq 1.2 M_{Rd}^{h, el} - M_h^{b, el}$$

The wall base shear ($V_{Ed}$) should be first estimated. This is done by considering the initial wall shear force (Equation 5) where $V_i^{h, base}$ 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). In the case that the second term in (5) governs, this initial shear force should just be multiplied with the overstrength factor of 1.2 (Equation 6). Otherwise, the base shear force should be calculated using Equation 7. The shear value for the top of the wall should be calculated using Equation 8 where $V_i^{h, el}$ is the value obtained from elastic analysis for this location. Using this procedure, the magnitudes from both the primary and secondary modes are accounted for in the design of the walls.
3. Modelling and Analysis Issues

3.1. Structural Model

The finite element software packages ETABS (CSI, 2005) and RUAUMOKO (Carr, 2004) were used for the linear and nonlinear analyses, respectively, of the three-dimensional (spatial) structural model. The response spectrum method of analysis was used (as would normally be the case for such a structure). An equivalent frame model was used, wherein all elements, including shear walls, were modelled as line elements with interconnecting rigid beams. Frames and walls are connected together by means of rigid diaphragms in horizontal plane at each floor level. Masses and moments of inertia of each floor are lumped at centres of masses. To directly compare with EC8 design, the elastic flexural and shear stiffness properties are taken to be equal to one-half of the corresponding stiffness of the uncracked elements, while torsional stiffness of the cracked section was set equal to 10% of the torsional stiffness of the uncracked section.

The Kappos and Antoniadis, as well as the Eurocode 8, design procedures were evaluated against the results of inelastic time history analyses, from which the maximum moments and shear forces developed in the walls of the structure during seismic actions of different intensities were found. These analyses were carried out with the aid of the inelastic dynamic analysis code RUAUMOKO 3D (Carr, 2004). The inelastic dynamic response-history analysis using the Newmark constant average acceleration was selected for this study since it is an implicit, unconditionally stable method, which allows longer time steps to be used. A diagonal mass model was selected to represent the weight of the structure. Initial Stiffness Rayleigh damping model was selected, where Rayleigh damping is modelled as proportional to the initial stiffness matrix of the structure. Frame members were chosen to represent the stiffness of the elements in the structure for the current study. Frame members include beam and beam-column members (walls are included in this category, as they are modelled as ‘fat’ columns). The one-component beam member was selected for beams, and reinforced concrete beam-column member was chosen for the modelling of columns and walls. A modified Takeda hysteresis with Drain-2D Unloading was selected to model plastic hinges for all members (Carr, 2004). Strength and stiffness data for members were determined assuming design 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 (Kappos, 1991) were used.

3.2. Input Motions and Scaling Procedure

The study encompassed a variety of both natural records representative of earthquakes that caused serious damage (including collapses and casualties) in Greece from 1975 to 2000 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, whereas records I20_855 and V_C1 are synthetic, derived for typical zones of the cities of Thessaloniki and Volos in the framework of microzonation studies for these cities. The last record used (IRA 14) represents a simulation of a severe seismic excitation caused in the south Aegean Sea at the boundary of lithospheric plates (Kappos & Antoniadis, 2011). As an ensemble, the records used encompass a range of frequency content and duration that is broader than that of a focussed set representing a narrow range of magnitude and epicentral distance; this was done in order to study, in a feasible manner, the effect of the frequency content of the records on the calculated
response. The selected input motions were scaled to the same intensity, i.e. the design one for each structure (0.16g or 0.24g), associated with the ‘damage limitation’ performance level (limit state). This is deemed sufficient for checking the validity of the method when applied to 3D buildings, but if the further goal of assessing the safety of the buildings to lower probability, higher intensity, events is envisaged, additional studies are clearly required.

4. Results

In the following, each of the 9-storey structures is denoted by its ductility class, the acceleration $a_g$ considered for design, and the direction of seismic excitation e.g. DCH 0.24 g – X Direction. It is noted that since most cases followed identical patterns, results for some cases are omitted here for economy of space.

(a) Bending Moment Envelopes due to Y-Direction Earthquake for structure designed to DCH, assuming the design earthquake intensity of 0.16g and cracked members; Central wall

(b) Bending Moment Envelopes due to X-Direction Earthquake for structure designed to DCH, assuming the design earthquake intensity of 0.24g and cracked members; Central wall
(c) Bending Moment Envelopes due to Y-Direction Earthquake for structure designed to DCM, assuming the
design earthquake intensity of 0.24g and cracked members; Central wall

(d) Bending Moment Envelopes due to X-Direction Earthquake for structure designed to DCM, assuming the
design earthquake intensity of 0.24g and cracked members; Corner wall

Figure 2. Bending Moment Envelopes due to X and Y-Direction Earthquake for structure designed to DCH and
DCM, assuming the design earthquake intensities of 0.16g and 0.24g and cracked members; Central and Corner
walls.
4.1 Comparison of EC8 and proposed methods

Main results for bending moment patterns are given in Figure 2, while corresponding results for shear forces are given in Figure 3. It has to be clarified that nonlinear response-history analysis using member-type models can only capture quantities like bending moments and shear forces in the elements, but cannot possibly capture effects associated with tension shift, i.e. the increase in the tensile force in the reinforcement due to diagonal shear cracking. Hence, caution is required in comparing design envelopes with tension shift (see Figure 2) with moment envelopes from nonlinear analysis. One possible interpretation is that the latter should be compared with the design envelopes without tension shift, on the understanding that the reported moments from analysis do not account for tension shift. The Code procedure for tension shift is an approximate one, even prior to the yielding of reinforcement, and moreover it is not correct to assume that in the nonlinear range the increase in the tensile force will be according to the shift rule adopted by the Code, which does not account for post-yield effects.

It is noted from Figure 2 that the proposed design envelopes exceed the Eurocode design envelopes at the base and then decrease with increasing wall height. This is due to the inclusion of overstrength of the critical region of the wall in the proposed methodology, which is not considered in the Eurocode 8 design envelopes, inappropriately in the authors’ opinion and in contrast to what is done in other prestigious codes, such as the New Zealand 3101 (Standards New Zealand 2006). It is further observed that the modified flexural design envelopes are significantly lower than the EC8 design envelopes in the upper half of the wall. Minimum flexural reinforcement requirements prevail for the upper half of the wall height for the walls designed in accordance with the modified method, whereas this is not the case for the Eurocode 8 methodology.

The Eurocode 8 shear design envelopes are generally observed to exceed the proposed method for both central and corner walls; the latter decrease in the upper one-third of the wall. Minimum requirements generally govern the shear design according to both design methodologies. The shear design obtained using both methodologies is similar, however the proposed methodology leads to a decrease in shear reinforcement in the upper one-third of the walls.
(b) Shear Force Envelopes due to the Y-Direction Earthquake for structure designed to DCM, assuming the design earthquake intensity of 0.24g and cracked members; Central walls.

(c) Shear Force Envelopes due to the Y-Direction Earthquake for structure designed to DCH and DCM, assuming the design earthquake intensity of 0.16g and cracked members; Corner walls.

Figure 3. Shear Force Envelopes due to the X and Y-Direction Earthquake for structure designed to DCH and DCM, assuming the design earthquake intensities of 0.16g and 0.24g and cracked members; Central and Corner walls.

The Eurocode 8 method and the proposed method results are observed to be similar when the seismic demand is lower as in the case of the walls of the 0.16g DCH structure, where minimum reinforcement governs along the entire height of wall, hence concealing any differences that might result from the different approaches. However, as the seismic demand increases, the EC8 methodology leads to an increase in design conservatism along the total wall height; this is attributed to the fact that overstrength is ignored in flexural design but is taken into account in shear design.
4.2 Verification against nonlinear analysis results

A non-linear response-history analysis was conducted in order to assess the alternative shear wall design methodologies. It is evident from Figures 2 and 3 that the nonlinear dynamic analysis resulted in lower values for both flexure and shear than both the proposed design envelopes and the EC8 ones. It can be noted that the proposed design method adequately envelopes the non-linear flexural response of the wall. Ignoring (on the safe side) the tension shift, it is observed that EC8 under-predicts the actual flexural response of the shear walls in the lower part of the building; clearly this is due to ignoring the overstrength of the base section. The EC8 flexural design envelope remains linear above the vertical tension shift with a constant slope along the entire height of the wall. The proposed flexural design envelope is generally consistent with the bending moments resulting from nonlinear analysis, with small overshooting around mid-height, which can be easily accommodated when the tension shift effect is taken into account in determining the reinforcement.

In the upper one-third of the shear wall height the EC8 shear design envelope has much higher values than the actual (inelastic) response. On the contrary, the proposed shear design envelopes are in general fully consistent with the inelastic analysis predictions.

5. Conclusions and recommendations

This study explored the feasibility as well as the suitability of the method proposed by Kappos and Antoniadis (2011) for the design of walls in 3D buildings with dual structural systems. The procedure was applied to a number of 9-storey dual systems, designed to the provisions of Eurocode 8 for two different ductility classes (M and H) and two different design acceleration levels (0.16 and 0.24g); verification of the alternative design procedures was carried out using non-linear response-history analysis for a number of input motions.

Regarding the feasibility, it was found that with some straightforward adjustment, the procedure can indeed be applied to 3D structures; nevertheless, the governing design criteria are not always the same as in the case of 2D systems for which the method was initially developed. Regarding the suitability, the results presented herein indicate that the current EC8 procedure does not adequately capture the seismic response of the buildings; a key weakness is that it does not account for flexural overstrength of the walls and another issue is that higher mode effects in the inelastic range are overestimated in the upper part of walls. Both of these weaknesses are tackled by the proposed procedure. The resulting economies in reinforcement in the upper half of the structure are about 30%, compared to that resulting according the EC8 design envelopes. Inelastic response-history analysis validates the proposed methodology, since it indicates that it adequately captures the actual response of the shear walls when subjected to ground motions consistent with their design seismic action.

It is noted that the present study considered only one building height (9-storey). Hence it is recommended that additional studies be carried out involving three-dimensional structures of different heights. Moreover, verification against higher levels of seismic action is worth carrying out, with a view to validating the reliability of the proposed design procedure. These additional parametric studies would lead to the collection of sufficient background information, which would eventually allow a revision in
current European seismic code procedures for the design of shear walls in dual structures; this would also be a timely effort, since the new Eurocode 8, currently at the evolution phase, is expected to enter into force by the year 2020 or 2021.

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