DYNAMIC BEHAVIOUR OF T-SHAPED RC WALLS DESIGNED WITH DIRECT DISPLACEMENT-BASED DESIGN

E. Smyrou[1], T.J. Sullivan[2], M.J.N. Priestley[3] and G.M. Calvi[4]

[1] *PhD Candidate, Centre of Research and Graduate Studies in Earthquake Engineering and Engineering Seismology (Rose School), Pavia, Italy*
[2] *Assistant Professor, Dept. of Structural Mechanics, University of Pavia, Pavia, Italy*
[3] *Professor, Centre of Research and Graduate Studies in Earthquake Engineering and Engineering Seismology (Rose School), Pavia, Italy*
[4] *Professor, Centre of Research and Graduate Studies in Earthquake Engineering and Engineering Seismology (Rose School) & Dept. of Structural Mechanics, University of Pavia, Pavia, Italy*
Email: smiroulena@gmail.com

ABSTRACT :

The behaviour of T-section walls in the direction parallel to the web is characterised by different strength and stiffness in the two possible loading directions, with the wall generally being stiffer and stronger when the flange is in tension than when it is in compression. In this paper, a comprehensive set of T-shaped walls is designed according to the Direct Displacement-Based Design principles, utilising the recently developed expressions for limit-state curvatures. The case studies are subjected to extensive nonlinear dynamic time-history analyses. Their performance is examined in terms of displacements, inter-storey drifts, moment and shear force, the envelopes of which are evaluated and compared to the ones obtained after applying the proposed modifications in the design process. Higher mode effects are investigated. Modelling issues related to T-shaped walls are raised, a set of guidelines within the direct displacement-based design frame are proposed and the current capacity design of cantilever walls is revised to include the particularity of T-shaped walls, leading to a more rationalised design of structures containing T-shaped walls.

KEYWORDS: T-shaped walls, capacity design, nonlinear dynamic time-history analyses, higher modes effects, direct displacement-based design

1. INTRODUCTION

In structures, where a permanent and identical or similar subdivision of floor areas in all stories is necessitated, structural walls are typically preferred as the structural system for carrying gravity loads and providing lateral resistance. In hotels or apartments buildings, T-shaped walls are commonly used following a specific repeated pattern that allows space for apartments or rooms between them. Specifically, the flanges of the walls form part of the corridor wall between doorways, and the web divides different hotel rooms or apartments. The structural characterisation is simplified and average values for strength and stiffness for flange in tension and flange in compression can be adopted, if the building layout contains identical, anti-symmetric T-section walls. Otherwise, the need to decide appropriate values for strength and stiffness becomes essential for design. As such, this work aims to develop Direct Displacement-Based Design (DDBD) guidelines that consider the particular characteristics of T-shaped walls. Specifically, a number of case study T-shaped walls will be designed according to the DDBD approach, the performance of which will be assessed by undertaking nonlinear time-history analyses.

2. DESIGN PARAMETERS

2.1. Design Displacement Spectrum and Earthquake Records

The case study T-shaped walls (please see section 3) were subjected to a suite of five earthquake records compatible with the EC8 spectrum for soil type "B" and 0.6g effective spectral acceleration, corresponding to a high seismicity area. The corner period for the design displacement spectrum was extrapolated until 5sec (Figure 1), considering that taller buildings, including most of the case study structures, have design displacements around the displacement corresponding to the corner period, much higher than the value suggested in EC8 (CEN, 2004). Furthermore, the use of an extended corner period accounts for the corner dependence on magnitude (Faccioli et al., 2004) and also accommodates the need for continuing the displacement spectrum to longer periods than the ones commonly plotted in the acceleration spectrum, reflecting thus the fact that the structural period of substitute structure is longer than the elastic period.

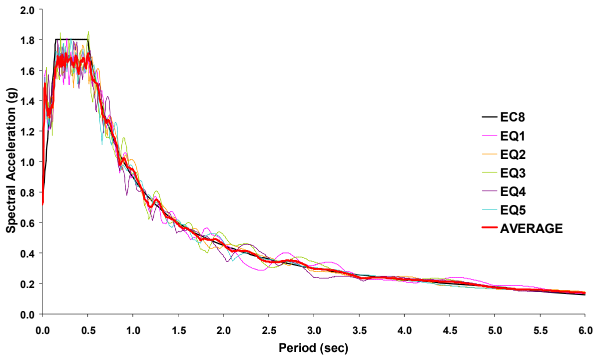

Figure 1 EC8 (CEN, 2004) design spectra and spectra of earthquake ground motions

Artificial records, generated by SIMQKE (Carr, 2008), were utilised because they matched the design spectrum for almost the full period range with comparatively small error. Hence, the use of a lesser number of records in order to obtain meaningful averaged results is allowed, though the code approach commonly prescribes a minimum of seven spectrum-compatible records for design verification, which is the aim of this study. The displacement spectra at levels of damping greater than 5% were obtained using a scaling factor tailored to the artificial records selected.

As far as the results' processing is concerned, averaging positive and negative response displacements seems not appropriate as the structure does not have symmetric stiffness and strength properties in the two directions, especially in the perspective of this study that attempts to investigate the true response characteristics in the two opposite directions of loading. Moreover, the application of the records in one direction may lead to favourable or adverse results, as they will depend on the stiffness and strength properties of the section, as well as the sequence they are mobilised each time. For this reason, additional analyses were conducted reversing the polarity of the accelerograms, providing in the end ten values in total for averaging for each direction of loading.

2.1. Modelling Issues

The nonlinear dynamic analyses were conducted with Ruaumoko (Carr, 2008). P-delta effects were not taken into account and small-displacement analysis was employed. The case study walls were modelled as stick elements with discrete masses at each floor level (Figure 2b). The floor height was considered 3m. One-component member models (Giberson, 1967) were used with plastic hinge lengths defined using the most recent equations proposed by Priestley et al. (2007). The Revised Takeda hysteretic rule (Figure 2c), a recently incorporated rule in Ruaumoko, based on Modified Takeda (Otani and Sözen, 1972) but having a tri-linear backbone, was employed as it offers the option to model the stiffness and strength properties of an asymmetric section distinctively for the two directions.

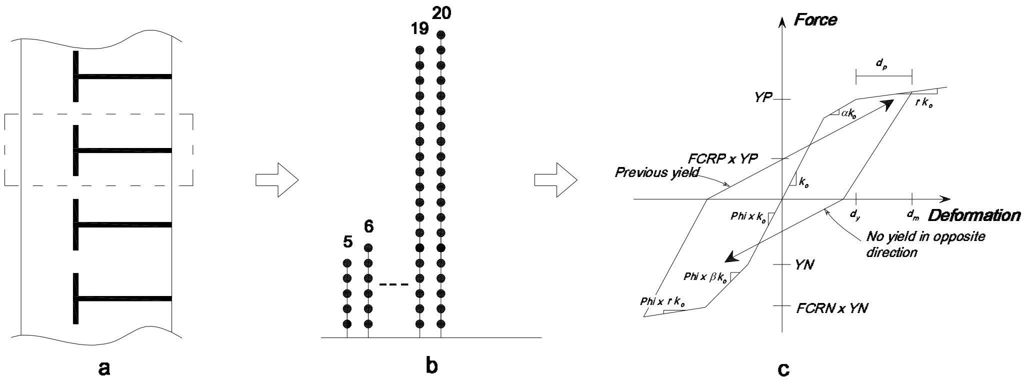

Figure 2 a) Plan view of building with T-shaped walls b) T-shaped walls modelled as stick elements c) Revised Takeda hysteresis rule

As far as the elastic damping is concerned, a user-specified modal damping model was used with a constant value of 5% assigned to all modes, apart from the 1st, where an artificially lower value ξ^* was set (Eqn. 2.2), depending on the expected design ductility of each case study μ and the post-elastic stiffness ratio r (Priestley et al., 2007), accounting thus for the fact that the elastic damping will greatly reduce in the post-yield phase.

$$\xi^* = \xi \cdot \frac{(1 - 0.1 \cdot (\mu - 1)(1 - r)}{\sqrt{\mu / (1 + r\mu - r)}} \tag{2.2}$$

Due to the asymmetry of the section, the order of specification of the values of yield moment plays important role, because it determines the orientation of the section, in conjunction with the sign convention kept in the software and the nodes' coordinates that define the local axes of the elements. Therefore, the definition of the stiffness for each direction of loading should be in agreement with the already set values of yield moment. The moment of inertia, reduced to represent the cracked stiffness, is required for the modal analysis in order to acquire the period of the structure. The moment of inertia inserted in the data defines the stiffness in one direction. Based on the latter and the ratio of stiffnesses required in the Revised Takeda hysteresis rule, the stiffness for opposite direction of loading is also determined (Carr, 2008). Consequently, two different values of fundamental period can be obtained, result of a modelling issue, which does not represent the reality though affects the evaluation of the response of the structure, since the elastic damping is directly affected by the computation of period. So it is important the stiffness used for the estimation of period is associated with the correct level of damping, related to the expected ductility in the direction of loading the stiffness corresponds to.

3. DESIGN OF CASE STUDY T-SHAPED WALLS

The Direct Displacement-Based Design for cantilever wall structures, as developed by Priestley and Kowalsky (2000), determines the equivalent SDOF substitute structure's characteristics (Gülkan and Sözen, 1974), i.e. the equivalent mass, design displacement and effective damping, that are required for the estimation of base shear force and consequently of the base moment demand. However, the current design process does not account for asymmetric structures, such as T-shaped RC walls, that exhibit different strength and stiffness properties for opposite direction of loading. The plausible question that arises is whether the properties for flange in tension (FiT), for flange in compression (FiC) or the average of them are appropriate for the design of T-shaped walls.

In this study, a total number of 48 case study walls were analysed, covering all heights between 5 and 20 storeys with the majority of them being over 11 storeys so that the effects of higher modes are detected. The web-flange ratios of the sections used were equal to 1.0, 1.5, 2.0, 3.0, 4.0 and 6.0. The design drift limit was 3%. The response from nonlinear time-history analyses (THA) was evaluated in terms of displacements, drifts, moments and shears. Due to lack of space no further details on the properties of the case study structures can be given, regarding their design values and capacity, however, the reader is referred to the work of Smyrou (2008).

3.1. Yield Curvature and Moment Capacity

The recently proposed expressions for yield curvature of T-shaped walls (Smyrou et al., 2008) were incorporated in the design so that a more precise estimation of the design displacement is achieved. The yield curvature differs for the two opposite directions of loading, with the one corresponding to flange in compression taking lower values. Being the critical for design, the yield curvature for FiC (Eqn. 3.1) is used for the estimation of the design displacement, the ductility and the equivalent effective damping needed for calculating the base shear force.

$$\phi_y = (1.57 + 0.06A)\cdot\varepsilon_y / l_w \qquad (3.1)$$

where A is the web-flange ratio, ε_y the steel yield strain equal to 0.00225 and l_w the wall web length.

The T-section for each case study was selected so that the moment demand obtained from DDBD using the yield curvature for FiC coincides with the moment capacity of the section for FiC. Assuming the opposite, i.e. designing based on the moment capacity for FiT, is expected to lead to premature failure, since the difference between the moment capacities of sections is significant in many cases. From a comprehensive number of moment-curvature analyses (Smyrou, 2008), it was observed that when a section has moment capacity for FiT similar to the demand obtained after design for FiT, then the moment capacity for FiC falls short considerably of the required DDBD capacity. The moment capacity in both directions of loading, as well as the elastic stiffness of the section, can be estimated by graphs in which the nominal moment and the elastic stiffness are given in dimensionless form as a function of the axial ratio and reinforcement percentage (Smyrou, 2008).

It is noted that the case studies over 15 storeys, i.e. the most flexible structures, exhibited inelastic behaviour, but not at the level of ductility corresponding to their displacement or drift capacity. After the softening of the structure, the effective period for FiC was higher than the displacement corner period, hence the iterative procedure recommended by Priestley et al. (2007) in such cases was followed in order to achieve an expected response displacement compatible with the associated effective damping. The design displacement was limited so that the effective period coincides with the corner period. The base shear and consequently the design base moment for FiC were slightly increased but both constitute an upper limit for the design, since any lower value satisfies the design assumptions.

3.2. Drift Control

Time-history results of the initial designs were checked for maximum drift with respect to the target design drift of 3%. The results consistently indicated that that there was inadequate control of drifts of wall structures taller than 12 storeys, especially in the upper floors. On the contrary, in all cases of shorter structures no exceedance of the drift limit was noticed. This indicates that the higher modes constitute a significant concern for taller buildings that needs to be included in the design procedure. Thereupon, a drift reduction factor was applied to the displacements, which leads to an increased effective stiffness of the structure. The revised base shear force is then computed as the product of the augmented effective stiffness and the initial design displacement computed before the application of the drift reduction, since in reality this is the expected displacement of the structure.

As a consequence of the increased based shear, the base moment demand rises too, leading to stronger sections in terms of flexural capacity. The level of drift reduction was estimated after several analyses of a considerable number of structures of different heights. The values of the drift reduction factor, ω_θ, are presented in Figure 3 as function of the storey number. It is noted that no lowering of the displacement is required for wall structures until 12 storeys high, while the maximum reduction of 13% is noticed as expected for 19- and 20-storey structures. The reduction factor exhibits a fairly constant value for structures with more than 15 storeys, after a gradual transition that takes place between 12- and 16-storey structures.

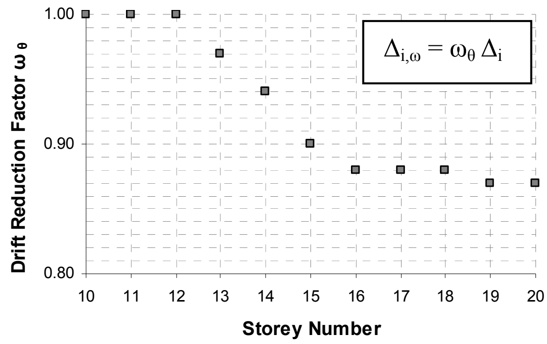

Figure 3 Drift reduction factor ω_θ for different number of storeys

The value of the drift reduction factor was mainly determined by the walls with low web-flange ratio, in which the difference between the two capacities is more pronounced. The lower response drifts observed for walls with higher web-flange ratio could possibly justify slightly higher values for ω_θ. However, the increase in the capacity for FiC may entail a moment demand comparable with the moment capacity of the section for FiT, something that appears in sections with high web-flange ratio, in which the two capacities are comparable. Then, exceedance of the drifts may occur when the flange is in tension in spite of the successfully controlled drifts in the flange-in-compression side.

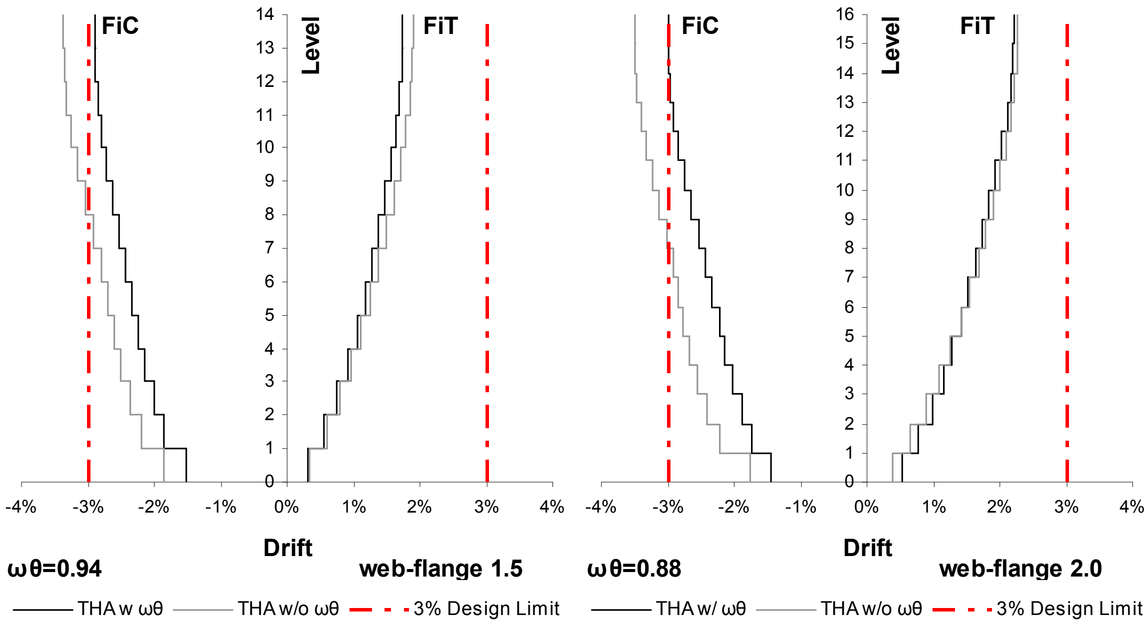

Figure 4 Application of the drift reduction factor ω_θ

The validity of the proposed values for ω_θ was verified with extensive nonlinear time-history analyses on 18 additional T-shaped wall structures of all heights between 12 and 20 storeys, in the design of which the suggested developments were included. Indicatively, the drift profiles, obtained as the average of drift envelopes from time-history analyses, for two of the examined case studies are presented in Figure 4. After the increase of moment capacity due to the application of ω_θ, the drifts of the two structures successfully remain within the design drift limit. The values of ω_θ constitute an upper bound, leading to drift lower than the design drift limit for all the web-flange ratios examined. It is up to the designer to adopt a lower value of ω_θ that will result in more conservative drift profiles, considering the inherent uncertainty in the results due to the number and type of records selected.

4. CAPACITY DESIGN FOR T-SHAPED CANTILEVER WALLS

4.1. Introduction

Wall structures are especially sensitive to higher modes effects due to their stiff elastic properties above the ground storey. This fact was verified by the results obtained after the time-history analyses conducted for this work, according to which the moment profiles deviated from the shape corresponding to the development of the 1st inelastic mode, exhibiting a characteristic bulge at approximately mid-height. Moreover, the values of shears obtained were several times higher than those associated with the design based on 1st mode.

The common approach adopted in codes to account for higher modes is through dynamic amplifications factors, the inherent limitations of which led to multi-modal methods that directly incorporate higher modes (Eibl and Keintzel, 1988; Priestley and Amaris, 2002), attempting to predict the increased moment and shear forces along the height. However, the analytical effort demanded to carry out modal analysis of the designed wall structure to determine the capacity design distribution of moments and shears is unwarranted, so a simpler and more conservative approach was proposed by Priestley et al. (2007), on which the capacity design suggestions for T-shaped cantilever walls are based. Specifically, the proposed capacity design recommendations for T-shaped walls are the outcome of inelastic time-histories on a comprehensive set of case studies consisting of 30 structures designed in accordance with the design procedure developed in the previous section. Due to limited space, the complete set of the results obtained is not presented in this paper but is included in the work of Smyrou (2008). It is finally noted that the calibration of the capacity envelopes was derived for one level of seismic intensity. The dynamic amplification in moment and shear profiles increases for higher intensity. However, the results presented hereinafter can be judged as being on the safe side considering the high ductility demands designed for.

4.2. Moment Capacity

A bilinear envelope is defined for the wall moments in relation to the overstrength base moment capacity $\phi^o M_B$, the mid-height overstrength moment $M^o_{0.5H}$ and zero moment at the wall top. The overstrength factor ϕ^o accounts for strain hardening, the dependable strengths of materials, any reduction factors included in the design and excess in reinforcement content or section size. However, in this work none of the aforementioned reasons occurred as design was undertaken for the expected response deformation and forces, so use of overstrength factor is not justified and hence, ϕ^o takes unit value.

Due to different properties in opposite directions, the capacity envelope for T-shaped walls should be differentiated according to whether the flange in tension or in compression. The mid-height moment is related to the overstrength base moment by a coefficient, $C_{1,T}$, which remains fairly constant for all the case studies examined, regardless of the number of storeys (Figure 5). The recommended value for flange in compression is 0.75, while for flange in tension it is suggested that a value of 0.67 is used, though for shorter structures lower values would be justified.

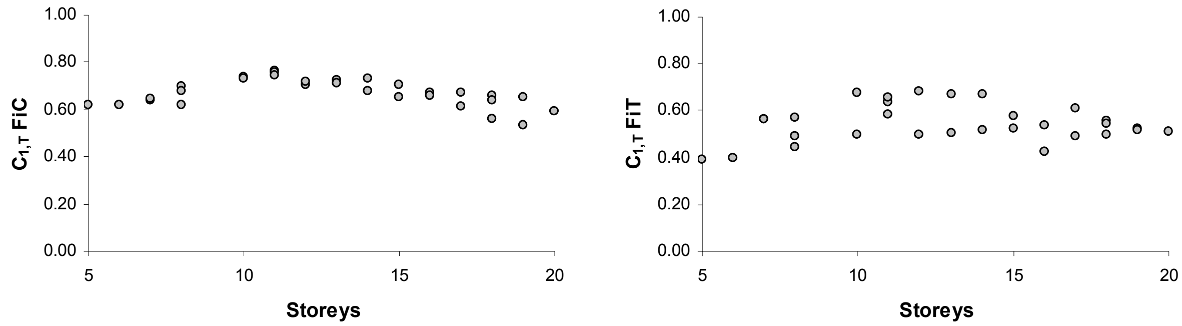

Figure 5 Ratio of mid-height moment over overstrength base moment

4.3. Shear Force Capacity

Time-history analyses indicated that shear force demand up the height of the walls were similar for both directions of loading. As such, a single shear design profile is proposed. The shear force capacity envelope is defined by a straight line between the base and the top shear of the wall. The capacity design base shear is calculated as the product of the DDBD base shear force, the overstrength factor and ω_v, which is equal to:

$$\omega_v = 1 + (\mu / \phi^o)\, C_{2,T} \tag{4.1}$$

where μ/ϕ^o is the effective displacement ductility factor at overstrength and $C_{2,T}$, a coefficient dependent on elastic period T_{el} (Figure 6a), given by the following equation:

$$C_{2,T} = 0.29T_{el} - 0.07 \tag{4.2}$$

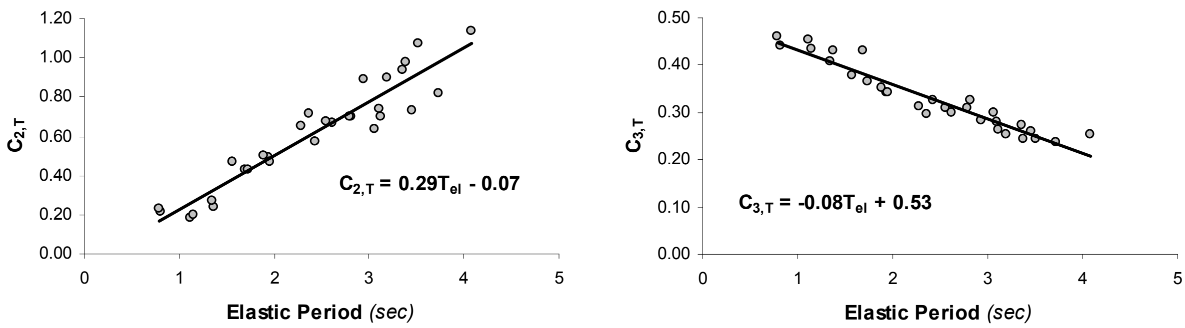

Figure 6 Coefficients for the estimation of shear force capacity envelope

Similarly, the coefficient $C_{3,T}$, that represents the ratio of the design shear force at the top of the wall over the capacity design base shear, is estimated by linear relation with respect to the elastic period (Figure 6b). Specifically:

$$C_{3,T} = -0.08T_{el} + 0.53 \tag{4.3}$$

For the calibration of $C_{2,T}$ and $C_{3,T}$, the elastic period derived from modal analysis was utilised, as it was available, however, the elastic period can be estimated from the effective period T_e, using Eqn. 4.4, in which the ductility μ and the post-yield r ratio participate.

$$T_{el} = T_e \sqrt{\frac{1 + r(\mu - 1)}{\mu}} \tag{4.4}$$

5. CONCLUSIONS

This work modified the established direct displacement-based design methodology in order to be applicable for T-shaped RC cantilever walls, which due to their asymmetry exhibit considerably different strength and stiffness properties for opposite direction of loading. Incorporating in the design process the recently developed equations for yield curvature of T-shaped walls, a large number of case studies were designed in accordance with Direct Displacement-Based Design with the base moment demand for flange in compression governing the design. Despite some excess in capacity, designing for the moment when flange is in tension would lead to inconsistent results. In sequence, the case study walls were subjected to nonlinear time-history analyses, so that the dynamic response in terms of displacements, drifts, moment and shears is examined with special focus on the higher modes effects. A design drift reduction factor was introduced to account for higher-mode drifts in taller structures, as it was found that there was inadequate control of drifts with the current design procedure. A set of values for the drift reduction factor with respect to the number of storeys was provided, validated after numerous verification analyses. Some modelling issues were mentioned briefly. Furthermore, within the concept of a simple and effective capacity design, capacity design envelopes for moment and shear were defined and calibrated based on results from inelastic time-history analyses on T-shaped RC cantilever walls.

ACKNOWLEDGEMENTS
Prof. Athol Carr is gratefully acknowledged for his efforts to facilitate this work. Many thanks also to Ihsan Engin Bal for the useful discussions.

REFERENCES

Carr, A. J. (2007). Ruaumoko2D – A program for inelastic time-history analysis, Department of Civil Engineering, University of Canterbury, New Zealand.
CEN (2004). Eurocode 8-Design of structures for earthquake resistance. Comite Europeen de Normalization, Brussels, Belgium.
Eibl, J. and Keintzel, F. (1988). Seismic shear forces in RC cantilever shear walls. *Proceeding of 9WCEE*, Tokyo/Kyoto, Japan.
Faccioli, E., Paolucci, R. and Rey, J. (2004). Displacement spectra for long periods. *Earthquake Spectra* **20:2**, 347–376.
Giberson, M.F. (1967). The response of nonlinear multi-story structures subjected to earthquake excitation. EERL Report, Institute of Technology, U.S.A
Gülkan, P. and Sözen, M. (1974). Inelastic response of reinforced concrete structures to earthquake motions. *ACI Journal* **71:12**, 604-610.
Otani, S. and Sözen, M.A. (1972). Behaviour of multi-story reinforced concrete frames during earthquakes. Structural Research Series No. 392, University of Illinois at Urbana-Champaign, USA.
Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007). Displacement-based seismic design of structures, IUSS press, Pavia, Italy.
Priestley, M. J. N. and Kowalsky, M. J. (2000). Direct displacement-based seismic design of concrete buildings. *Bulletin of the New Zealand National Society for Earthquake Engineering* **33:4**, 421-444.
Priestley, M.J.N. and Amaris, A.D. (2002). Dynamic amplification of seismic moments and shear forces in cantilever walls. Research Report ROSE - 2002/01, IUSS press, Pavia, Italy.
Smyrou, E., Sullivan, T.J., Priestley, M.J.N. and Calvi, G.M. (2008). Determination of ductility capacity and other section properties of T-shaped RC walls in direct displacement-based design. *Proceedings of 14WCEE*, Beijing, China.
Smyrou, E. (2008). Seismic design of T-shaped RC walls, PhD Thesis, Rose School, IUSS press, Pavia, Italy (in preparation).