Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1470

POTENTIAL BENEFITS OF AN ENERGY FACTOR DISPLACEMENT-BASED DESIGN APPROACH

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ABSTRACT

A number of different displacement-based design (DBD) methods have been put forward over the past decade in order to address shortcomings with current force-based design approaches. Considerable progress has been made in developing DBD procedures for a wide variety of structural typologies. However, compared with quick force-based design methods such as the equivalent lateral force method, one could argue that the new displacement based design approaches are too complex and time-consuming for simple low and medium-rise structures. In this work a new simplified displacement-based design approach, referred to as the energy-factor method, is described and applied to a number of case-study wall structures. The results of the method are compared with those obtained from the equivalent force-based design approach and the Direct DBD method. The performance of the method is then gauged through a series of non-linear time-history analyses using spectrum-compatible accelerograms. The results indicate that the simple energy-factor method could offer excellent possibilities for the performance-based design of regular low and medium-rise structures.

Introduction

Current code design methods, such as the equivalent lateral force method and the modal response spectrum method, have been shown (Priestley, 1993) to be based on a number of flawed concepts and as such, the methods are ineffective in controlling the damage developed in a structure by an earthquake. In order to obtain more effective seismic design methods, a large number of displacement-based design (DBD) methods have been proposed and as reported by Sullivan et al. (2003) the level of complexity and effectiveness of the different displacement-based methods varies significantly. Some methods, such as the Direct DBD method (Priestley et al., 2007) or the yield-point spectrum method (Asccheim and Black, 2000; Tjhin et al., 2007), have been developed further than others and enable application to various structural typologies. An advantage of both of the aforementioned methods is that they only require routine calculations and are relatively fast to apply as analytical modeling should not be required. However, they are still considerably more time-consuming than the equivalent lateral force method incorporated in current codes, and require engineers to possess a good understanding of

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an arguably complex subject. This, plus the fact that advanced non-linear analysis methods are currently required to demonstrate that DBD solutions conform with building standards, could explain why practitioners appear to be relatively reluctant to start using the new methods.

Recognising the need for simple, fast and effective seismic design methods, this paper outlines a displacement-based design approach that is of similar complexity to the equivalent-lateral force (ELF) method currently included in seismic codes. In contrast to the ELF method, however, it will be argued that this new "energy-factor" method can provide effective control of the damage that can be expected in an earthquake. To indicate the potential of the method, the new procedure will be used to design a variety of case study wall structures and results compared with other established design procedures. The performance of the methods will then be gauged through non-linear time-history analyses using a suite of spectrum-compatible accelerograms.

The Energy-Factor Method for Displacement-Based Design

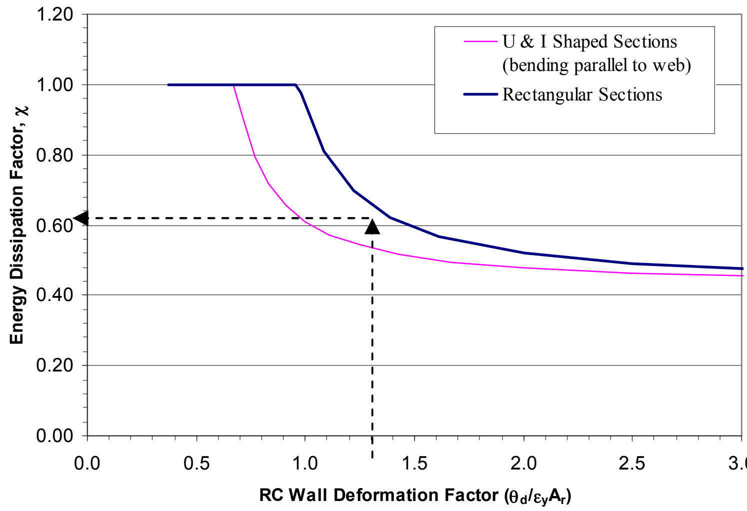
Energy-based seismic design methods are not new, with Housner (1959) considering the possibility in the 1950s, as well as many others such as Zahrah and Hall (1984), Fajfar and Fischinger (1990), and Uang and Bertero (1990), with Japanese standards including energy-based approaches as per Akiyama (1985). However, as concepts of performance-based design evolve, energy methods have received less attention, possibly because they tend to focus on structural damage with little consideration of the potential damage to non-structural elements. Furthermore, they typically strive to control damage by limiting the amount of energy that is dissipated. In this work, energy demands are considered but rather than design to energy dissipation limits, the storey drift is selected as a more tangible design parameter that lends itself more easily to performance-based design in line with other displacement-based design approaches.

The energy-factor method aims to control the peak inelastic deformations by equating the external energy that an earthquake imposes on a structure in its peak response cycle with the internal work done by the structure to reach that deformation state. The basis of the design approach is presented in Fig.1. For simplicity, the work imposed on the structure is taken as the mass of an equivalent SDOF representation of the structure multiplied by half of its spectral velocity squared. The internal work done is considered as the sum of the stored energy and the energy dissipated through hysteretic response. As shown in Fig.1b, the design procedure is based on a linear elastic analysis of a SDOF model with an effective stiffness corresponding to the secant stiffness of the actual non-linear system at the design deformation point. The basis of the deign procedure is to determine the design resistance by estimating the proportion of external work that remains stored in the linear system at peak response, through the use of an energy dissipation factor. This paper will illustrate how this energy-factor method can be applied to RC wall structures. For a detailed description of the method, readers should refer to Sullivan (2010).

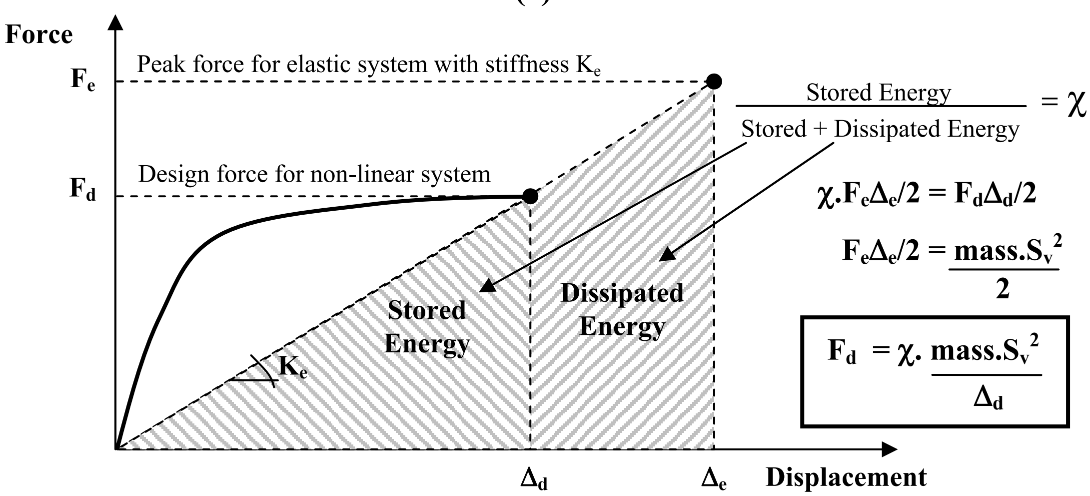
The energy-factor method for RC wall structures

The first step in the procedure is to set a deformation factor, D_f, for the building. Physically, the D_f parameter is a non-dimensional measure of the total deformation imposed on the structure at the design displacement level. As explained in Sullivan (2010), it is proportional to the ductility demand on RC wall structures. For RC wall structures this deformation factor is simply:

$$D_f = \frac{\theta_d}{\varepsilon_y A_r} \tag{1}$$



(a)



(b)

Figure 1. Overview of the Energy-Factor Method: (a) Energy Dissipation Factors for RC Wall Structures, (b) Effective Stiffness concept and energy relations used to identify required design strength (adapted from Sullivan, 2010).

Where θ_d, is the design storey drift, ε_y is the yield strain of the longitudinal reinforcement, and A_r is the aspect ratio of the walls.

The next step is to determine the energy-factor for the system. This can be done using the deformation factor to enter Fig. 1(a) and read off the energy-factor, χ, for the relevant type of RC wall structure. The curves shown have been developed by first relating deformation factors to ductility demands and then considering effective period inelastic spectra data stemming from the work of Grant et al. (2005). Note that the work by Grant et al. (2005) only considered a limited number of accelerograms and future research may use a larger database of records to refine the curves. Different curves are presented for the energy factor as a function of the wall section shape as the section shape affects ductility demands and therefore energy dissipation.

The energy factor, χ, is used to calculate the design base shear, V_{base}, for the building using Eq.2.

$$V_{base} = \chi . f_s . \frac{Mass}{\theta_d} . PSV^2 \leq 2\pi\sqrt{\chi} . m_e . \frac{PSV}{T_c} \quad (2)$$

Where *Mass* is the total mass of the building, θ_d is the design drift limit, *PSV* is the peak spectral velocity, and f_s is a substitute structure factor that considers the effective (participating) mass, m_e, as well as the characteristic displacement, Δ_d, of the substitute structure, related to the design drift and building height, as shown by Eq.3.

$$f_s = \frac{m_e}{Mass} \frac{H\theta_d}{\Delta_d} \quad (3)$$

Where *H* is the total structural height and the other symbols are as described above. The factor f_s can be obtained with knowledge of the displaced shape at peak response together with the mass distribution (see Sullivan (2010) for further details). A good approximation for wall structures is:

$$f_{s,walls} = \frac{(4n-1)}{3n} \quad (4)$$

Where *n* is the number of storeys. As such, the design base shear for an RC wall building can be calculated through the Energy-Factor method with knowledge of only the structural geometry, material characteristics, the design deformation, and the spectral velocity. Note that the peak spectral velocity, PSV, of the response spectrum is included in Eq.2 since this is equal to the design spectral velocity at the effective period of most structural systems, as can be appreciated when one considers the typical form of velocity spectra shown in Fig. 2. For structures of very short period it is clear that the PSV overestimates the demands, and as such, a limit on the design base shear is shown on the right of Eq.2 that considers the fact that the spectral velocity is lower than the PSV for periods shorter than T_C. It is also apparent that the PSV would be inappropriate for long period structures, but note that the Energy-Factor method is intended only for low to medium rise structures, which will typically have periods shorter than T_D.

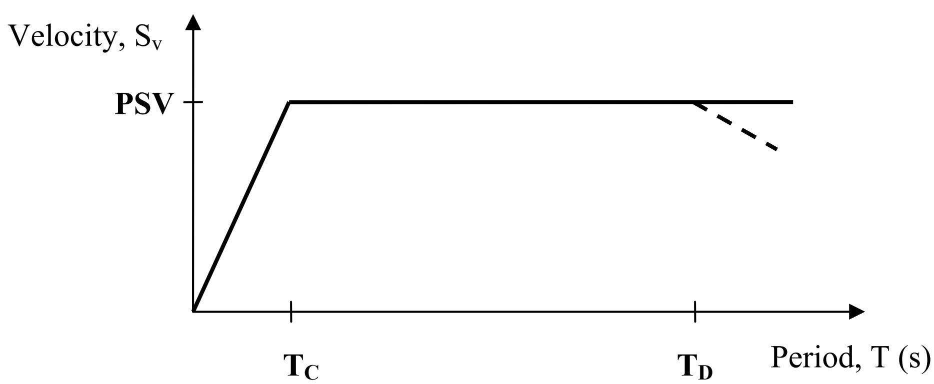


Figure 2. Form of velocity response spectra typically included in seismic design codes (Sullivan, 2010).

With the design base shear of the substitute structure known, the required overturning resistance can be calculated from:

$$M_{O/T} = f_h.H.V_{base} \quad (5)$$

Where the height factor, f_h, is the ratio of the effective height, H_e, to the total height, H.

$$f_h = \frac{H_e}{H} \quad (6)$$

For RC wall systems, the factor can be reasonably approximated by:

$$f_{h,walls} = \frac{3n}{(4n-1)} \quad (7)$$

The energy-factor method is therefore very simple and fast, and this has been the key objective in its development. However, the procedure has some limits, since the energy factors, the substitute structure factor and the height factor expressions have empirical components. The range of design parameters for which the expressions have been developed are:

- Wall aspect ratio from 2.5 to 15.0. Walls are cantilever type walls without coupling beams and capacity design is required to ensure a flexural mechanism develops.
- Design storey drifts between 1.5% and 2.5%.
- Reinforcement strengths in the range of 300-600MPa.

Furthermore, the design steps just presented consider a basic design scenario in which foundation flexibility, torsion and p-delta effects are negligible. The means of dealing with such phenomena within the energy-factor method should be addressed as part of future research.

Investigating the Performance of the Method

Case study structures and design inputs

The proposed design method has been tested for RC wall structures through the design and analysis of two sets of 4 and 12-storey RC wall structures, illustrated in Fig. 3. The first set possess walls with an aspect ratio of 3 and the second possess walls with and aspect ratio of 6. In addition, design solutions are developed for two seismic regions: (i) a region of moderate seismicity with a design PGA of 0.25g, and (ii) a region of high seismicity with a design PGA of 0.4g. A design drift limit of 2.0% was selected to control damage to non-structural elements at the damage-control limit state. The structures have a regular layout with rigid foundations. It is assumed that the floor slabs will act as rigid-diaphragms in plane, fully flexible out of plane.

The concrete and reinforcement material properties adopted for the seismic design are values that could typically be found in building practice. Values for the concrete include: (i) f'_c = 30.0 MPa and (ii) E_c = 25740 MPa. The expected strengths adopted for the reinforcing steel include: (i) f_y = 500 MPa and (ii) E_s=205000 MPa. Material strengths are not factored to dependable strength levels and instead these values have been taken as the expected strength and stiffness characteristics, as is appropriate for seismic design.

Design has been performed using the design spectral shape shown in Fig. 4, constructed in accordance with the Eurocode 8 (CEN-EC8 1998). The spectrum corresponds to EC8 type 1 spectrum with soil type C. The displacement spectrum from EC8 has been extrapolated beyond the 2.0s cut-off period proposed in the EC8 because the two second period is much lower than that used by other codes where it is envisaged the methodology could also be applied. Fig.4 also includes the average response spectra of five accelerograms used to undertake non-linear time-history verification studies, as explained later in the paper.

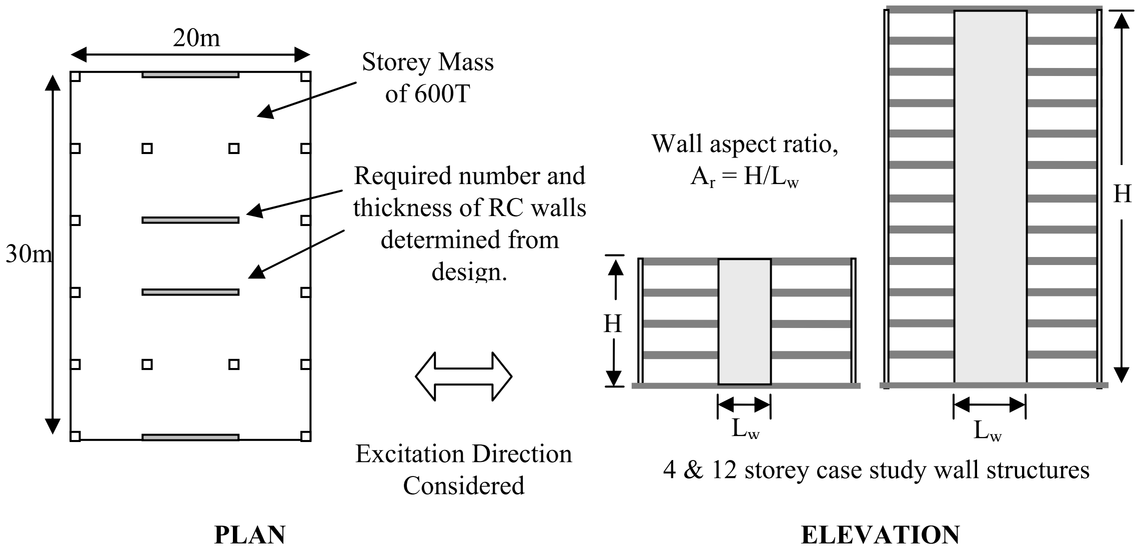


Figure 3. Plan and elevation of case study wall systems examined in this work (Sullivan, 2010).

In order to compare the required design strengths with those required by more well-known approaches, the case study structures have also been designed using the equivalent lateral force (ELF) method presented in EC8 and the Direct DBD method of Priestley et al. (2007). For the ELF method the building period was obtained using a simple height-dependent expression from the EC8 and a behaviour factor of 4.0 was adopted. For the Direct DBD approach the guidelines of Priestley et al. (2007) were followed for the damage control limit state, with a wall curvature limit of 0.072 and a design drift limit of 2.0%.

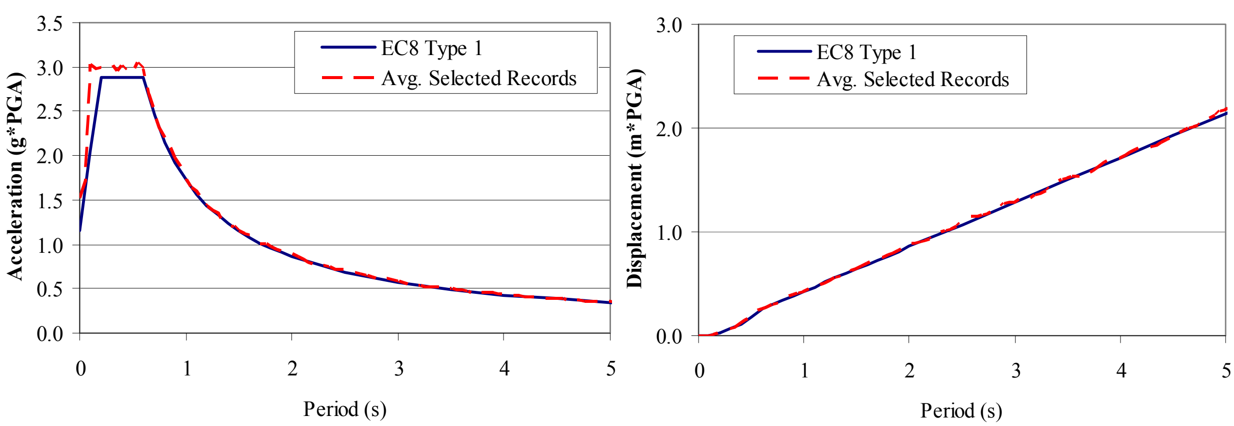
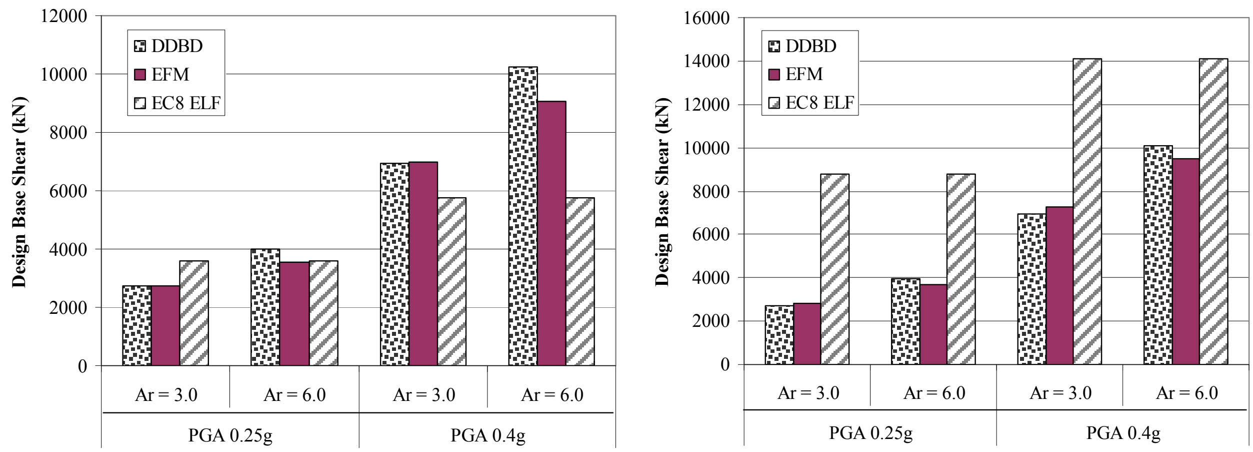


Figure 4. Comparison of normalized design acceleration spectra (left) and displacement spectra (right) with average spectra of spectrum compatible accelerograms.

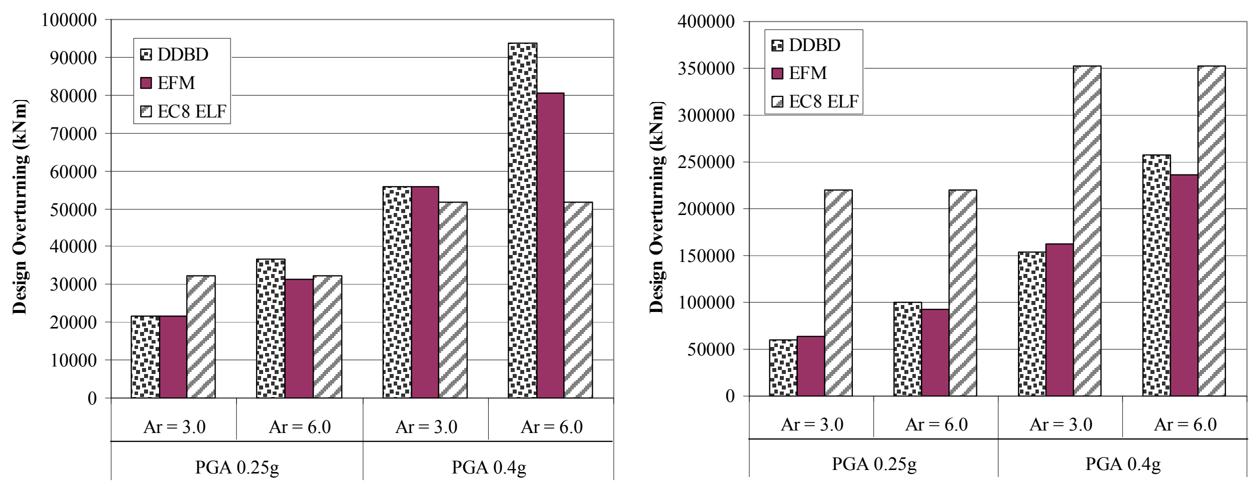
Design strengths

Design was undertaken only to the point that gave the required flexural strengths of the base of the walls. Capacity design shears and bending moments were not calculated as only plastic hinge strength values are required to gauge the performance of the methods through non-linear time-history analyses, as will become clear in subsequent sections. Fig.s 5 and 6 show the design base shear and design overturning respectively for the 4 and 12-storey case study structures.



(a) 4-Storey Buildings **(b) 12-Storey Buildings**

Figure 5. Design base shear for (a) the 4 storey case study buildings and (b) the 12 storey buildings, as per the Direct DBD Method (DDBD), Energy Factor Method (EFM), and the Equivalent Lateral Force Method of Eurocode 8 (EC8 ELF) (Sullivan, 2010).



(a) 4-Storey Buildings **(b) 12-Storey Buildings**

Figure 6. Design overturning for (a) the 4 storey case study buildings and (b) the 12 storey buildings, as per the Direct DBD Method (DDBD), Energy Factor Method (EFM), and the Equivalent Lateral Force Method of Eurocode 8 (EC8 ELF) (Sullivan, 2010).

Performance evaluation through non-linear time-history analyses

Non-linear time-history analyses were carried out using Ruaumoko (Carr, 2007) to gauge the performance of the trial methodology. Lumped plasticity models of the case studies were constructed in which the strengths of the walls were set to exactly match the design values obtained using the relevant design procedure. Elastic properties were assigned to elements that are not intended to yield, inferring that appropriate capacity design would have ensured that inelasticity is concentrated only in regions associated with the collapse mechanism. The RC wall base elements were represented using the Takeda hysteretic model, with 5% post-yield displacement stiffness and the unloading model of Emori and Schonbrich with a force-displacement unloading stiffness factor of 0.5, reloading stiffness factor of 0.0 and reloading power factor of 1.0. Refer to the Ruaumoko manual (Carr, 2007) for further details. Plastic hinge lengths were calculated using the recommendations from Priestley et al. (2007).

The analytical models use effective section properties up until yield, obtained by taking the design strength and dividing by the yield curvature. Approximations for yield curvature were obtained from expressions provided by Priestley et al. (2007). The cracked elastic periods of the structures ranged from 0.7s to 3.0s. Elastic damping was incorporated using a Wilson-Penzien (1972) damping model with a reduced first mode damping component in line with recommendations of Priestley et al. (2007). The floors were modelled as rigid diaphragms fully flexible out of plane. P-delta effects were not considered. Each model was subject to a suite of five real accelerograms modified by Pennucci (2010) to be compatible with the design spectrum, as shown earlier in Fig. 4.

Fig. 7 presents the average of the peak storey drifts obtained from the non-linear time-history analyses. As the designs were governed by non-structural drift requirements, the method performance can be gauged by comparing the average of the maximum recorded drifts with the design drift value of 2.0% (shown as a bold dashed line in Fig.7).

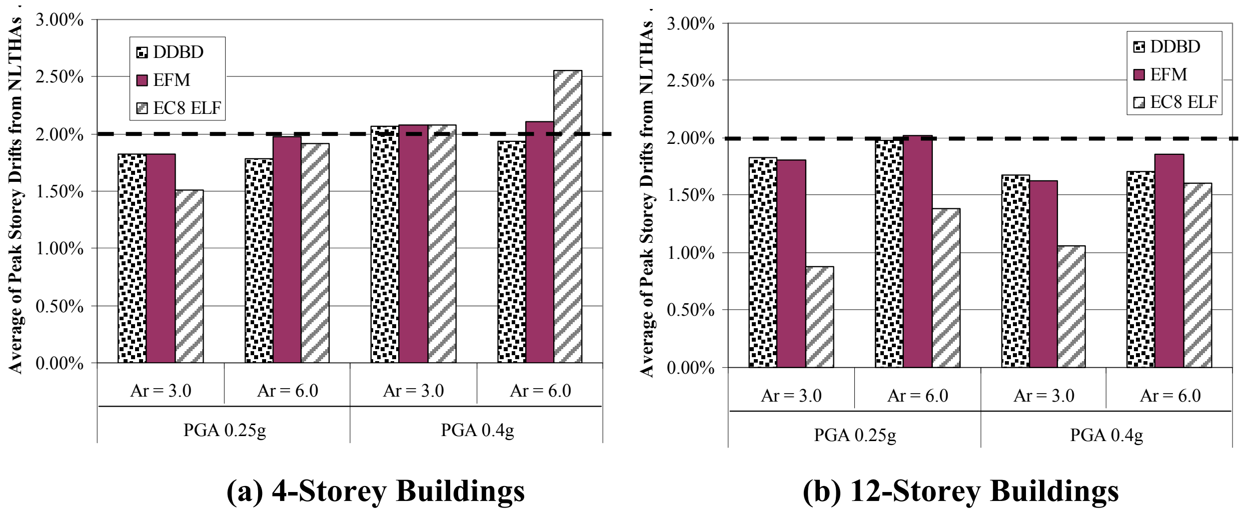


Figure 7. Average of peak storey drifts from NLTH analyses of (a) 4 storey and (b) 12 storey case study buildings, for the Direct DBD Method (DDBD), Energy Factor Method (EFM), and the Equivalent Lateral Force Method of the Eurocode 8 (EC8 ELF).

Considering Fig. 7, one observes that the equivalent lateral force (ELF) method is unable to provide consistent control of the response, with drifts ranging from 0.88% to 2.55%. The method appears to be non-conservative for low-rise walls with high aspect ratio in regions of high seismicity and is very conservative for medium rise wall structures with low aspect ratio in regions of medium seismicity. The ELF results indicate an average maximum drift of 1.62% and a large standard deviation of 0.55%. In contrast, the Direct DBD approach achieved an average maximum drift of 1.85% and a standard deviation of 0.14%, while the Energy-Factor method recorded an average maximum drift of 1.70% and a standard deviation of 0.14%.

As structural damage is also clearly important for the seismic performance, Table 1 presents the maximum curvature ductility demands which range from 2.0 to 11.5. Note that well detailed RC walls can sustain curvature ductility demands of around 18.0 (Sullivan et al. 2006) thus confirming that the structural deformation limits were not critical for the design of the walls.

Table 1. Average of maximum curvature ductility demands recorded from NLTH analyses.

	4 Storey Buildings				12 Storey Buildings			
	PGA 0.25g		PGA 0.4g		PGA 0.25g		PGA 0.4g	
	Ar = 3.0	Ar = 6.0	Ar = 3.0	Ar = 6.0	Ar = 3.0	Ar = 6.0	Ar = 3.0	Ar = 6.0
DDBD	9.3	3.3	11.2	4.1	11.5	5.0	10.7	4.0
EFM	9.3	3.8	11.2	4.4	11.1	5.5	10.2	4.6
EC8 ELF	7.0	3.7	11.2	6.4	3.5	2.0	5.2	3.4

Discussion

The results of this study permit many interesting points to be made. A particularly interesting observation regarding the proposed design method is that the fundamental mode design base shear expression is independent of the building height and therefore is independent of the building period. Traditional force-based design techniques would tell us that this is inappropriate. However, there have already been indications that this should be the case with Priestley (2000), Sullivan et al. (2006) and Priestley et al. (2007) identifying that the design base shear obtained from Direct DBD is independent of the building height. The case studies examined in this paper support the notion that height-independent design base shear expressions provide more consistent protection against damage than the height and period dependent expressions on which force-based design approaches are based.

Another interesting observation to be made about the energy-factor method is that the required seismic resistance is very dependent on structural proportions. The importance of structural proportions has also been recognised by Browning (2001) and Kowalsky (2001) amongst others. In this work, the effect of structural proportions has been emphasised by examining wall buildings possessing walls of different aspect ratio. The results of time-history analyses confirm that, in contrast to the philosophy behind current force-based design methods, the design strength should vary as a function of geometric proportions. This reflects the influence that structural proportions have on ductility demands and energy dissipation.

The ability of seismic design methods to provide consistent control of the response in different regions of seismic intensity can also be reviewed. It is evident that the ELF method does not properly account for seismic intensity, providing generally more conservative results in regions of medium seismicity. For further discussion of this see Priestley et al. (2007).

Ultimately, the aim of seismic design should be to control the risk posed by earthquakes. This small study has demonstrated that the ELF method does not permit the control of risk for RC wall structures, with a standard deviation in maximum drifts of 0.56%. In contrast, for the range of case study structures considered, both the Direct DBD method and the proposed Energy-Factor method recorded a standard deviation in drift of only 0.14%, indicating much improved control of seismic risk. As the energy-factor method is simpler than the Direct DBD method, and provides much improved control with respect to the ELF method, future research should look to develop it as a potential substitute of the ELF method in current codes.

Conclusions

In this work a new simplified displacement-based design approach, referred to as the Energy-Factor method, has been proposed. Interesting features of the method are that geometrical proportions of the structure are used as a key design input, and the fundamental mode design base shear is independent of building height. The method has been outlined for cantilever RC walls with either rectangular or U or I-sections, designed to the damage control limit state. The Energy-Factor method has been applied to a number of case-study wall structures and the results compared with those obtained from the EC8 equivalent force-based design approach and the Direct DBD method. The method exhibits similar trends to the Direct DBD approach but is computationally more efficient. The performance of the methods has been gauged through a series of non-linear time-history analyses. The results indicate that the simple energy-factor method could offer excellent possibilities for the performance-based design of regular low and medium-rise structures. Future work is required to illustrate how the method can consider coupled wall structures and walls with different length. In addition, energy factor curves for other structural typologies such as RC frame structures are required, and the possibility of adapting such an approach to seismic assessment should be explored.

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