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Motives for and impediments facing Direct Displacement-Based Seismic Design

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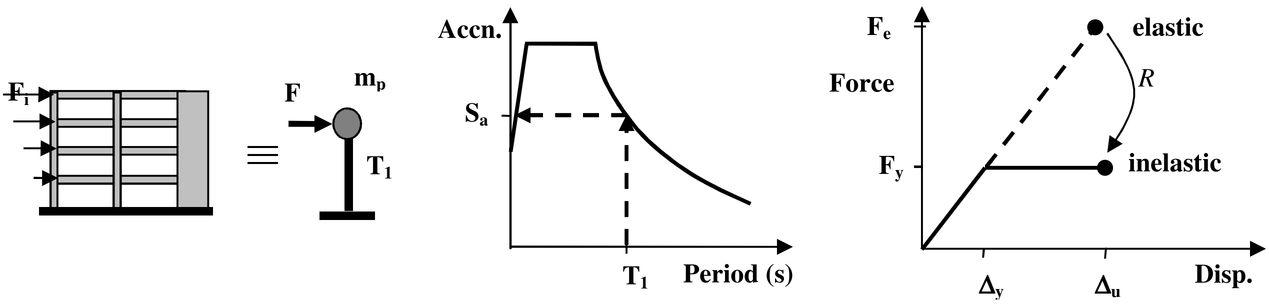
ABSTRACT: In his 1993 paper entitled “Myths and Fallacies in Earthquake Engineering”, Nigel Priestley identified some fundamental shortcomings with existing force-based seismic design methods and in the years that followed, proposed and developed the Direct displacement-based design (DBD) procedure. The Direct DBD approach is now well developed and tested, with numerous scientific publications, a text and a model code offering guidance on its application for a wide range of structural typologies and technologies. This paper reviews the background and motivations for Direct DBD, highlights the performance of the method, identifies current impediments to its use and suggests and discusses possibilities for its further development.

1 INTRODUCTION

1.1 Background and motives for displacement-based design

Today, and since the late 1970s in some countries, building codes allow engineers to use one of two seismic design methods: (i) the equivalent lateral force (ELF) method and (ii) the modal response spectrum (MRS) analysis method, with the former method being applicable to regular buildings of limited height. Figure 1 provides a conceptual overview of the ELF method, with Figure 1(a) illustrating how one first identifies the period, T_1, and participating mass, m_p, associated with the fundamental mode of vibration of the building. An elastic spectral acceleration demand can then be read (Fig.1b) from the design acceleration response spectrum and multiplied by the participating mass to obtain an elastic equivalent lateral force (F_e in Fig.1c). To allow for structural ductility capacity, μ, the elastic force is divided by a factor, R, (sometimes referred to as a behaviour factor, q), set as a function of the structural typology, to obtain the final seismic design forces. The peak displacement demands are estimated using so-called R-μ-T relationships, with the equal-displacement rule often adopted in which the inelastic and elastic spectral displacement demands are assumed equal. The MRS method also adopts the same procedure, but includes additional modes of vibration in the process.



(a) Equivalent SDOF system with initial period T_1

(b) Use of elastic acceleration response spectrum

(c) Reduced design forces to allow for inelastic response

Figure 1: Conceptual overview of the ELF design procedure (from Sullivan, 2013).

In his 1993 paper entitled “Myths and Fallacies in Earthquake Engineering”, Priestley (1993) identified a number of fundamental shortcomings with such force-based seismic design (FBD) methods (as well as other myths and fallacies in earthquake engineering). Perhaps the two most important shortcomings, in the author's opinion, with the FBD procedure are:

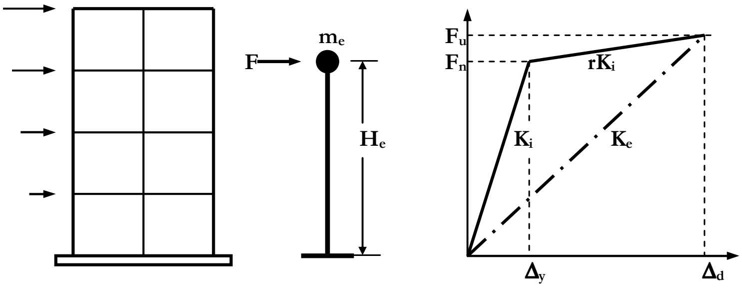
- *The geometrical proportions of structures do not affect their allowable ductility demand.* The FBD procedures recognise that certain structural systems tend to possess greater ductility capacity than others. For example, a well detailed RC wall structure can rightly be expected to possess a greater ductility capacity than a masonry block wall building without reinforcement. However, what the FBD approach overlooks is that large variations in ductility capacity can occur even within the same structural configuration. This is essentially because even if code detailing for ductility were successful in ensuring certain levels of ductility capacity at a section level, this does not necessarily ensure uniform levels of system ductility capacity. Consider, for example, the variation in displacement ductility capacity obtained for two RC cantilever piers with the same section diameter of 1.0m, detailed to possess the same curvature ductility capacity of 18.0, but with pier A possessing a height of 3.0m and pier B possessing a height of 8.0m. Even though the only difference between the two piers are their geometric proportions (or height, in this case), it can be shown (according to Direct DBD provisions detailed in Priestley et al. 2007) that the displacement ductility capacity of pier A is around 9.4 whereas the ductility capacity of pier B is closer to 6.7. This underlines the fact that large variations in ductility capacity can exist between structures that belong to the same structural typology; a fact that appears to be ignored in FBD.
- *Higher mode actions are treated in the same way as fundamental mode actions.* In FBD the force demands obtained from elastic analyses are reduced by a factor R to account for the system ductility capacity, as shown in Figure 1c for the ELF method. It was also stated that the same approach is followed for the MRS analysis approach, such that the elastic force demands associated with each mode of vibration are reduced by the same reduction (or behaviour) factor. However, the development of non-linear response in a MDOF system does not affect higher mode actions in the same way as fundamental mode actions (Priestley and Amaris, 2002; Sullivan et al. 2008) and this should be accounted for during capacity design. The same observation can be made for deformation demands; the fraction of total drift caused by higher mode excitations will tend to change as non-linear response develops (and may become either more or less significant). These points underline the fact that higher mode actions should not be treated in the same way as fundamental mode actions and given this point, Priestley et al. (2007) argue that modal analyses can provide a false impression of increased accuracy.

There are also other issues with FBD methods, such as insufficient consideration of hysteretic type when estimating the inelastic displacement demands from elastic response spectra or the incorrect use of cracked stiffness estimates in RC structures that are independent of the section strengths, and for more detail readers should refer to Priestley et al. (2007). However, the two bulleted points above are considered particularly important because it does not appear that simple changes can be made to the FBD method to address them. In light of such issues, Priestley et al. (2007) and others have developed the so-called "Direct" DBD procedure as a rational alternative to FBD.

1.2 Overview of the Direct displacement-based design procedure

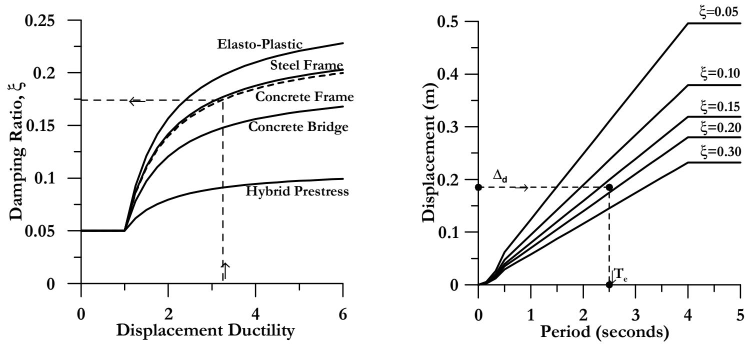
The fundamentals of Direct DBD can be reviewed with reference to Figure 2, recognizing that there are four components to the procedure:

- Representation of the MDOF structure (shown in Fig.2a as a frame building, although the procedure is the same for all structures) as an equivalent SDOF system, characterised by an effective mass and a target design displacement.
- Consideration of the linear secant stiffness to maximum design displacement response, rather than the pre-yield elastic stiffness, as illustrated for the equivalent SDOF structure non-linear force-displacement response shown in Figure 2b.
- Use of relationships, such as those shown in Figure 2c, between displacement ductility demand and equivalent viscous damping (EVD). Such relationships appear to be a function of structural typology but to be precise, they are set considering the structural hysteretic properties, and are calibrated to the results of NLTH analyses (see Pennucci et al. 2011 for an up to date discussion).
- Use of design displacement spectra for different levels of equivalent viscous damping (Fig.2d).



(a) SDOF Simulation

(b) Effective Stiffness Kₑ



(c) Equivalent damping vs. ductility

(d) Design Displacement Spectra

Figure. 2 Fundamentals of Direct Displacement Based Design (Priestley et al, 2007).

For MDOF systems the approach also requires definition of a design displaced shape, expected at development of the limit state value of deformation in the building (considering structural and non-structural elements). To this extent, the design displacement profile will be that which satisfies all relevant deformation limits (e.g. strain, curvature or drift limits) contemporarily. The displaced shapes can be either established using simplified mechanics-based expressions (as in the case of RC wall structures) or empirical expressions obtained from NLTH analyses (as in the case of RC frame structures). The text by Priestley et al. (2007) and the model code for Direct DBD (Sullivan et al. 2012a) provide guidance on the definition of the design displaced shape for a wide variety of structural systems. Allowance for higher mode effects on deformation demands is also made for taller buildings, typically through the use of simple empirical expressions (again see Sullivan et al. 2012a). With the displaced shape known, the following equations can be used (together with EVD expressions) to identify the equivalent SDOF characteristics and arrive at the system design base shear demand:

$$\text{Design displacement: } \Delta_d = \sum_{i=1}^{n}\left(m_i\Delta_i^2\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right) \tag{1}$$

$$\text{Effective mass: } m_e = \sum_{i=1}^{n}\left(m_i\Delta_i\right)^2 / \sum_{i=1}^{n}\left(m_i\Delta_i^2\right) \tag{2}$$

$$\text{Effective height (buildings): } H_e = \sum_{i=1}^{n}\left(m_i\Delta_i H_i\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right) \tag{3}$$

$$\text{Effective stiffness: } K_e = 4\pi^2 m_e / T_e^2 \tag{4}$$

$$\text{Design base shear: } F = V_B = K_e\Delta_d \tag{5}$$

In Eqs.(1) to (5), Δ_i, m_i, and H_i are, respectively, the design displacement, the mass and height (for buildings) of the i mass locations of the structure. The ductility demand is required for estimation of EVD/inelastic displacement spectra and for this purpose the yield displacement, Δ_y, is calculated by the designer (guidance on this is provided for a large range of structural systems in Priestley et al.

(2007) and the model code for Direct DBD (Sullivan et al. 2012a)) and can typically be computed as a function of only the structural geometry and material properties, without knowledge of strength. The effective period, T_e, is found from displacement spectra scaled to the system damping level (or ductility demand) as indicated in Figure 2d. Once the design base shear for the equivalent SDOF system is obtained, it can be redistributed as a set of equivalent lateral forces to the MDOF system, which can then be analysed in order to define the required strength of plastic hinge actions. Capacity design procedures should then be followed, with account for higher mode effects, to compute design forces for all actions that do not form part of the expected plastic mechanism. The approach has also been developed to account for a range of complex phenomena, such as P-delta effects, torsion and soil-structure interaction. For further details see Priestley *et al* (2007) and Sullivan et al. (2012a).

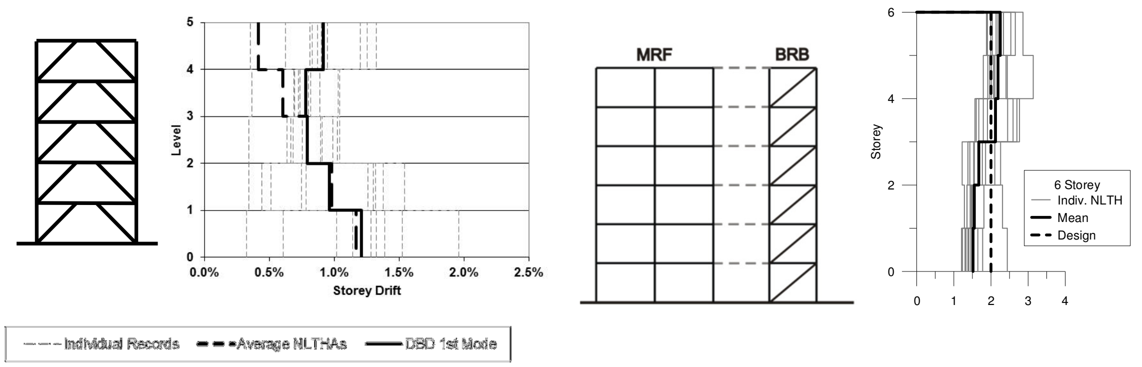
Once the procedure is appreciated in full, it emerges that all the issues with force-based design mentioned earlier are overcome. Furthermore, the approach provides engineers with a better sense of the role that structural proportions, material properties, member detailing, and capacity design concepts all play in the apparent seismic risk. Additionally, by focussing on the fundamental mode response and accounting for higher mode effects at the capacity design stage (and in a simplified manner for the control of deformation demands) the methodology recognizes that higher mode actions should not be treated in the same way as fundamental mode actions. Because of these differences with FBD, results, such as those presented in Sullivan (2013a), indicate that DDBD may lead to design base shear forces as much as four times greater or four times less than code FBD approaches, suggesting that endeavours to find a "quick-fix" to FBD methods are likely to be futile.

2 PERFORMANCE OF THE DIRECT DBD APPROACH

The Direct DBD procedure has now been successfully tested on a wide range of structural typologies. The typical verification approach adopted first sees the design of a set of structures to pre-defined performance objectives that should be satisfied for a specific intensity level. Subsequently, non-linear numerical models of the as-designed structures are developed and subject to non-linear time-history (NLTH) analyses using sets of ground motions selected to be compatible with the design intensity level. The Direct DBD method is then considered to be verified if the performance objectives are satisfied at the design intensity level.

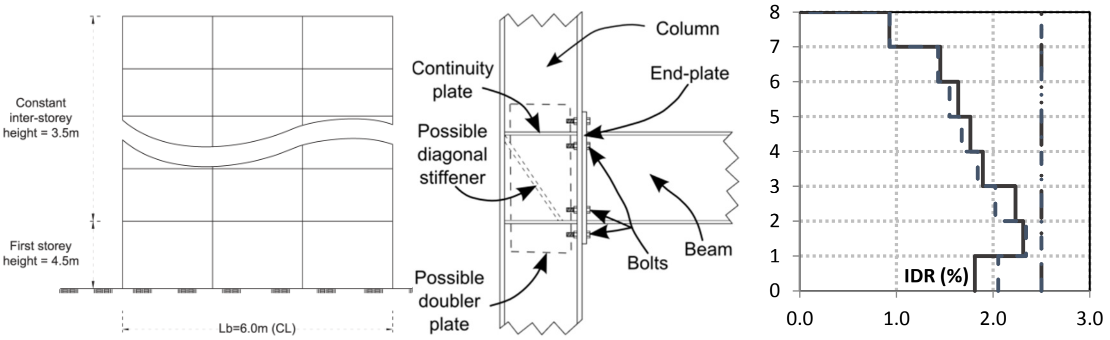
To illustrate the type of results that have been obtained from this process and thereby provide further motivation for its use, Figures 3 to 7 compare the peak storey drift demands obtained from NLTH analyses with the design drift limit for a range of structural typologies. While storey drift has been selected as the performance measure for this paper, the Direct DBD procedure can, and is, also used to control other types of deformation demands, such as curvature demands in RC wall structures, chord rotation demands in EBF links (Sullivan, 2013) or joint rotation demands in steel MRFs with partial strength joints (Roldan et al. 2015). The design storey drift limit differs between figures (ranging from 1.0% to 2.5%), since the results are taken from various studies in the literature which adopted different drift limits depending on the building code in use.

As shown in Figures 5, 6 and 7, the Direct DBD procedure has also been tested successfully for some types of structures that might have been expected to be outside its capabilities; Figure 5 illustrates results obtained for a vertically irregular RC frame-wall system, while Figure 6 (left) shows results for a 20-storey building with complex geometry and Figure 7 shows results for a 40-storey coupled wall building. It can be seen that despite the apparent complexity of such buildings, the method works well (and this can be somewhat attributed in these cases to the role that RC walls play in controlling the displaced shapes). The approach has also been recently tested, with reasonable success but also with areas for improvement, for steel moment resisting frames (MRFs) with set-backs (Nievas and Sullivan 2015). The right side of Figure 6 presents verification results for a seven-span RC deck-arch bridge in the transverse direction of response, presented by Khan et al. (2014), demonstrating that the approach works well also for bridge structures. In fact, the Direct DBD methodology has been extensively developed and tested on a wide range of bridge structures (see Calvi and Kingsley 1995 and Kowalsky 2002 for early examples), and complex bridges such as cable-stayed bridges with viscous dampers (Calvi et al. 2010).



(a) Steel EBF system (Sullivan, 2013)

(b) Dual MRF BRB system (Maley et al. 2010)



(c) Steel MRF system with (partial strength) bolted extended end-plate joints (Roldan et al. 2015)

Figure. 3 Illustration of verification results (in terms of storey drift) for different types of steel structures.

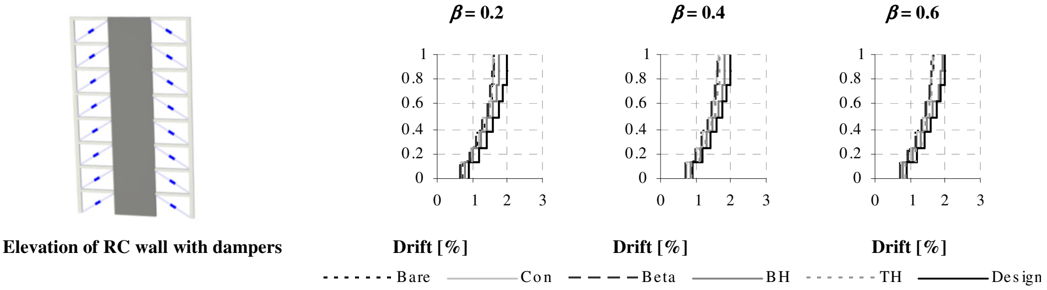


Figure. 4 Illustration of verification results (in terms of storey drift) for RC walls with different arrangements of non-linear viscous dampers (Lago, 2011).

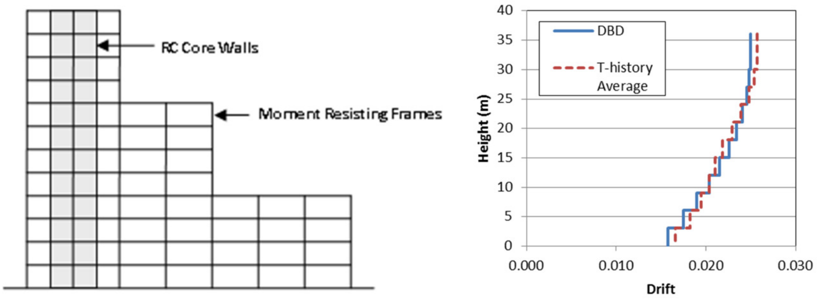


Figure. 5 Illustration of NLTH analysis verification results (in terms of storey drift) for a vertically irregular 12 storey RC dual-system building (after Salawdeh 2011).

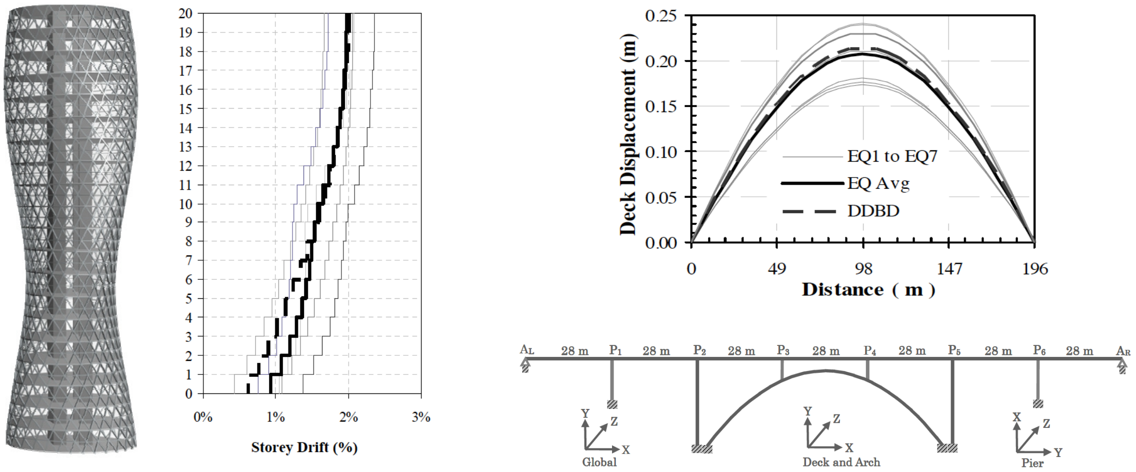


Figure. 6 Illustration of NLTH analysis verification results (in terms of storey drift) for (left) a 20-storey building possessing complex geometry (Lago et al. 2011) and (right) a RC arch-deck bridge (Khan et al. 2014).

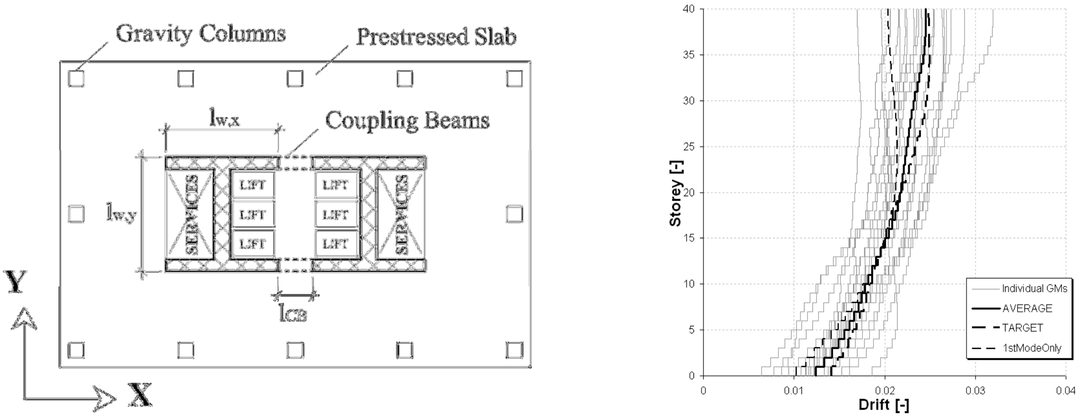


Figure. 7 Illustration of NLTH analysis verification results for a 40-storey RC coupled wall building in which the Direct DBD method included an analytical estimation of higher mode drifts (Pennucci et al. 2011a).

For taller buildings, the Direct DBD procedure is usually applied with a simplified empirical allowance for higher mode effects on deformation demands in which the allowable peak 1st mode drift is set lower than the design limit in order to allow for an increase in drifts due to higher modes (see Sullivan et al. 2012a for details). However, Figure 7 illustrates results obtained for a 40-storey coupled RC wall building by Pennucci et al. (2011a) for which effective modal superposition considerations were used to compute higher mode drift demands during the Direct DBD phase. As can be seen, the results suggest that effective modal superposition approaches may well prove useful for cases where higher modes are expected to contribute significantly to deformation demands.

3 A VISION FOR THE FUTURE

Despite the significant developments made to the Direct DBD procedure and the fairly extensive testing that has been done (noting that the results presented in the previous section are just a small snapshot of the verification studies that have been carried out), the approach is not yet widely used in practice. While there may be various reasons for this, thoughts on the main issues are provided below:

- *Displacement-based design is not a code method.* In most parts of the world Direct DBD is not codified and because of this, practitioners may face difficulty getting designs approved by local authorities (despite the improved accuracy it provides over FBD methods) and are unlikely to be able to benefit significantly by applying it, unless the project is sufficient to justify advanced NLTH verification analyses at the detailed design stage. A draft model code for Direct DBD exists (Sullivan et al. 2012a) and there are some countries in which Direct DBD is codified. However, code implementation will clearly be an important task for the future.

- *The approach is perceived to be difficult to apply*. Most practicing engineers will be familiar with the MRS analysis method which can literally be applied to any type of structure using a range of commercial software packages. In contrast, a good understanding of the structural system and loadpaths is required to apply Direct DBD to a complex structure, and calculations are usually done by hand/spreadsheet. A software has recently been developed for Direct DBD of RC buildings (Sullivan et al. 2012) that will make the approach easier to apply but will also, unfortunately, make "black-box" DBD more possible. Nevertheless, it is clear that further efforts should be made to develop software to assist engineers with Direct DBD.
- *Lack of financial incentive*. Practitioners applying Direct DBD currently will inevitably compare their results with those from FBD; if FBD is non-conservative they will likely modify their design whereas if Direct DBD indicates that savings are possible, they may not pursue these since it could be difficult getting the design approved by local authorities. As such, given the current lack of financial incentive for Direct DBD (also considering the additional time required to apply the method), there are few practitioners actually using it. This result could also be partly due to the seismic performance objectives of current international codes; engineers are prompted to check that, according to code analysis methods, certain limit states are not exceeded for certain design intensity levels with little scope given to engineers to instead satisfy broader performance objectives, which could permit accurate design to be rewarded.

In relation to this last point, an important change taking place in the US via the PEER performance-based earthquake engineering (PBEE) approach (FEMA, 2012), is the promotion of the probabilistic assessment of time-based performance measures, such as the annual expected fatalities (or injuries), monetary losses and repair time. An interesting prospect then emerges for seismic design; if building codes were to require the evaluation of such performance measures, with the definition of a maximum allowable annual probability of collapse, and if the insurance industry were to set insurance premiums as a function of loss assessment results, new challenges would emerge for seismic design methods, including Direct DBD. To this extent, the Direct DBD procedure is currently being developed to enable simplified application of the PEER PBEE procedure (Welch et al. 2014, Sullivan et al. 2014). Despite the challenges this entails, it is proposed that the Direct DBD approach be developed further in this direction because seismic risk can be better quantified and communicated via time-based performance measures, instead of current code pass-fail assessments for specific intensity levels.

4 CONCLUSIONS

The Direct DBD method is now well developed and tested, with numerous scientific publications, a text and a model code offering guidance on its application for a wide range of structural typologies and technologies. This paper has briefly examined the background and motivations for Direct DBD, demonstrating that the method has been successfully verified for a range of structural typologies. Despite this, the approach is not yet widely used in practice and this appears to be principally because it is not a codified procedure, is not implemented in commercial software and does not appear worthwhile financially to consultants. The basic solution to these points would be to introduce Direct DBD into building codes. Furthermore, however, it has been suggested that future building codes themselves may benefit from the introduction of time-based performance measures and if this were to occur, then new challenges for seismic design methods, including Direct DBD, would emerge.

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