New methods for assessment and design of structures in seismic zones: present state and research needs

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ABSTRACT

Significant advances have been accomplished in the last decade in the seismic protection of structures, based on the introduction and refinement of innovative conceptual approaches. The main drawbacks and inconsistencies related to traditional force-based design approach have been recognized as well as the critical role of displacement or deformation demand in characterizing the structural damage and thus the performance of the system when subjected to different level of ground motion intensity. A common generalized trend of developing and adopting displacement/deformation-based approaches within a performance-based design philosophy has resulted.

In this contribution, a state-of-the-art critical review of the current research developments in seismic design and assessment methods based on performance based philosophy, either purely or partially relying on displacement-based or deformation-controlled approaches, will be provided. The influences on recent major code revisions and modifications, as well as on preliminary design guidelines, will be discussed, while reviewing the conceptual fundamentals of the different approaches and underlining main sources of uncertainties and possible future developments.

1 INTRODUCTION

In the last few years a significant shift in the conceptual philosophy of research on seismic design and assessment has been observed, due to the recognition of the primary role of displacements and deformations as reliable and direct indexes of structural (and non-structural) damage. The main drawbacks and possibly un-conservative inconsistencies of the traditional force-based approaches have been recognized and critically discussed.

Within what should be better considered as a rationalization of already known conceptual schemes based on the familiar Limit States approach (traditionally accepted in gravity load design) performance-based approaches have dictated the recent research effort, starting to influence provisions and recommendations in some major codes.

1.1 Current code provisions and trends

The SEAOC Blue Book provides "Preliminary Guidelines for Performance Based Seismic Engineering (PBSE, 1998): a Force-Displacement Approach" (Appendix G – Part 2). These Guidelines are a focused development of the Vision 2000 Conceptual Framework (1996), where the PBSE has
been defined as consisting of a set of engineering procedures for design and construction of structures to achieve predictable levels of performance in response to specified levels of earthquake, within definable levels of reliability. A pure Direct-Displacement Based Design (DBD) approach is presented in addition to the Equal-Displacement Design (EBD) method which is based on the familiar equal displacement approximation developed by Newmark and Hall (1982). The EBD is an adaptation of the traditional Force-Based Design (FBD) code procedure but uses an acceleration displacement (AD) response spectrum or the displacement spectrum rather than the acceleration response spectrum and focuses on displacement rather than forces.

Recognizing that damage control must become a more explicit design concern and that this can be achieved only by introducing some kind of nonlinear analysis into the seismic design methodology, a combination of the nonlinear static (pushover) analysis and the response spectrum approach has been proposed in several methods. In the USA, examples of this approach can be found in the capacity spectrum method, used in ATC 40 and in the nonlinear static procedure, used in FEMA 273. In Japan, the seismic design requirements in the Building Standard Law were revised in June 2000 into a performance-based design format and the capacity spectrum method has been employed. In Europe, important contributions have been made towards development of simplified nonlinear procedures. Unfortunately, the progress of the implementation of Eurocode 8 is slow and most likely it will take a long time before the research results will be codified.

Although a general consensus, after recent earthquake lessons, has been reached that present codes need significant improvements and expansion, a complete acceptance of these concepts in the major code frameworks has been postponed. If on one side this may be partly ascribed to the traditional inertia in incorporating recent research results into codified provisions, as well as to a sort of natural prejudice against any innovation, on the other side it should be recognized and admitted that further developments and completion of validation studies are needed to provide general reliable procedures ready for practical acceptance and use. In some cases the confidence in the deep-rooted forced-based design approach has resulted into the development of hybrid force-displacement approaches.

In this contribution, a state-of-the-art critical review of the current research developments in seismic design and assessment methods based on performance based philosophy, either purely or partially relying on displacement-based or deformation-controlled approaches, will be provided. The conceptual fundamentals of the different approaches are reviewed, underlining the main sources for uncertainties and for possible future developments. The influences on recent major code revisions and modifications, as well as on preliminary design guidelines, will be discussed.

2 DIRECT DISPLACEMENT BASED APPROACH

In the last decade, the deficiencies of traditional force-based approaches have been extensively presented and discussed by several authors (Moehle 1992, 1996; Kowalsky et al. 1995; Bertero 1997; Priestley 1993,1998) and several drafts of “deformation-based” or “displacement-based” procedures have been proposed, with the intent of emphasizing and rationalizing the commonly accepted key-role of displacements in the seismic performance of a structure. However, most of the preliminary proposed procedures retained the trappings of traditional force-based design assumptions, with added effort only in the estimation of the maximum response displacements. A first complete formulation of a pure displacement-based design can be recognized in those proposed by Priestley (1993, 1998) and Kowalsky et al. (1995) for single-degree-of-freedom (SDOF) systems. Subsequently, comprehensive suggestions for extension to multi-degree-of-freedom (MDOF) systems were presented by Calvi and Kingsley (1995). In this simple design procedure, different limit states are defined by limiting displacement or drift (displacement-height ratio) levels, derived converting strain limit states in the material, which appear to be the best indica-
tor of damage potential. In accordance with performance-based design considerations, acceptable limit states and, more directly, corresponding values of global parameters (i.e. displacements/drift) are defined "a priori", depending on the structural typology and on the seismic input level (return period or probability of occurrence of the event). Therefore, in a direct displacement based design approach, a target displacement is directly imposed to be reached by the structure and, by entering displacement response spectra, the structural characteristics (design based shear and effective period) are determined which would guarantee achievement of the target limit state.

As noted by Priestley (1998), a design approach that attempts to design a structure which would achieve, rather than be bounded by, a given limit state under a given seismic input level would essentially result in uniform-risk structures, philosophically compatible with the uniform-risk seismic intensity incorporated in most codes.

2.1 Basic steps of the procedure

The whole procedure, illustrated in Figure 1 as proposed by Priestley (1998), simply uses displacement response spectra, effective stiffness (secant to the target displacement) and equivalent viscous damping. A substitute-structure (Shibata and Sozen, 1976) is adopted to reduce the initial MDOF system to a SDOF system. Entering the displacement response spectrum with the target displacement (function of the assumed deformed shape) of the equivalent SDOF and with an assumed value for the equivalent viscous damping, the effective period of the structure is obtained. Then, by multiplying the obtained effective stiffness and the target displacement, the design base shear for the whole structure is determined.

Figure 1 –Direct Displacement-Based Design procedure (from Priestley, 1998)

It is important to underline that alternative approaches (force-based or displacement-based) practically lead to different procedures to estimate the design base shear, which has to be appropriately
distributed in elevation to determine the required internal forces for the structural members. In both cases capacity design considerations will be adopted while dimensioning the structural members. Furthermore, a FBD approach could lead to a series of iterations since the design base shear (strength) depends on the first estimation of the initial stiffness which, at a section level, is itself a function of the strength (see consideration below in par. 2.1.1).

2.1.1 Transformation to SDOF system: substitute structure

The “substitute structure” analysis procedure proposed by Shibata and Sozen (1976) is adopted to reduce the MDOF system to a SDOF system. Equivalent height and equivalent mass concepts, as well as effective stiffness secant to the target displacement (Fig. 1 a,b), are introduced. The target design displacement ($\Delta_d$) for the SDOF system is a function of the displacement profile ($\Delta_i$) at maximum response assumed for the MDOF system (Fig. 2):

$$\Delta_d = \sum_{i=1}^{n} \frac{m_i \Delta_i^2}{\sum_{i=1}^{n} (m_i \Delta_i)}$$

(1)

being $\Delta_i$ and $m_i$ displacements and masses of the $i^{th}$ degree of freedom (i.e. story level in a building).

![Diagram](image)

Figure 2 – Displacement profile and effective height (from Priestley, 1997)

Although the equations proposed in literature (Paulay and Priestley, 1992; Kowalsky, Priestley and MacRae, 1995) to adequately determine the displacement pattern for multi-degree of freedom systems (i.e. regular frame buildings or wall systems) have been demonstrated to be adequate for design purposes, the assumption on the probable displacement pattern represents a critical step in the whole procedure. The problem is particularly evident in an assessment procedure when dealing with existing under-designed structures/buildings which can be characterized by a non-ductile mechanism of response (soft-story, high damage in the joint regions). Different displacement patterns should be adopted corresponding to increased level of damage into the structure, in order to
take into account the formation of "irregular" plastic mechanisms (due to the lack of a capacity design philosophy).

As anticipated, while a force-based design characterizes a structure in terms of elastic properties (stiffness, damping) appropriate at first yield, direct-displacement based design adopts structural parameters consistent with target maximum displacement: secant stiffness at target displacement and a level of equivalent viscous damping appropriate to the hysteretic energy absorbed during the inelastic response. The problems, intrinsic of a FBD procedure, related to the uncertainties of estimating the initial stiffness, are thus recognized and avoided. In particular, as emphasized by Priestley (1998), an important and invalid assumption is commonly accepted when using a FBD approach: that stiffness is a fundamental property of the section and it is independent of strength, while it should be recognized that the yield curvature is a constant of the section.

2.1.2 Estimation and role of the energy dissipation capacity

The equivalent viscous damping, commonly adopted as a reliable indicator of the energy dissipation capacity of structures, can be expressed as a function of the expected ductility demand (Fig. 1c). In a direct-displacement-based design procedure a reliable level of damping is expected to be estimated a priori depending on the structural typology and target displacement (thus expected ductility). Several curves, for different structural systems and hysteretic rules, have been proposed in literature to relate damping level and ductility, which show, as a common trend, to lead to an asymptotic value for medium-high level of ductility (see Fig.1c).

The first choice of the probable energy dissipation capacity of the system is therefore less critical than what may appear. Furthermore, it should be recognized that the role of energy dissipation in the seismic response of a structure is sometimes over-emphasized. The effects of equivalent viscous damping in reducing the maximum displacement achieved by a S.D.O.F. system decrease significantly when increasing the energy dissipation. This concept can be somehow also derived from the expression adopted for the displacement reduction factor \( \eta \) in Eurocode 8, used to modify the ordinates of an elastic displacement spectrum (5% damped) taking into account the effects of different values of equivalent viscous damping \( \xi \):

\[
\eta = \frac{7}{\sqrt{2 + \xi}}
\]

Further confirmations have been provided and emphasized by recent parametric studies to develop realistic inelastic displacement spectra for a broad range of earthquake database (Borzi et al., 2001): negligible effects on the response seem to result at equivalent damping values larger than 15-20 percent. Therefore once having provided a minimum level of energy dissipation, other structural properties can and should be better maximized, as proposed by Pampanin et al. (2000) referring to "hybrid" systems in which energy dissipation and self-centering capability can be adequately combined.

2.1.3 Seismic demand: displacement spectra

The whole direct DBD procedure is represented in Figure 1. Adequate displacement spectra are entered with the target displacement \( \Delta_p \). The design base shear as well as the structural "effective" period are determined depending on the assumed level of energy dissipation capability (in the form of equivalent viscous damping). Once the seismic demand for the whole structure is determined, the each member is designed according to the internal forces obtained by distributing the base shear along the elevation, as typical of other FBD procedures: particular attention should be given to capacity design consideration and reinforcement detailing.
The definition of reliable displacement spectra, especially for long periods (or "effective" periods), has been recognized as fundamental step for a future generalized adoption of displacement-based approaches. The aforementioned recent studies on inelastic displacement spectra based on a broad database of European recorded strong ground motions (Ambraseys et al. 2000), have also demonstrated the significant inadequacy of displacement spectra directly derived, by integration, from code acceleration spectra, even when a "plateau" displacement limit is adopted for high periods (Bommer and Elashai, 1998; Tolis and Faccioli, 1999).

Furthermore, as implicit of any approximate procedure where a MDOF system is reduced to an equivalent SDOF system, information on the higher mode effects are lost. However, force response is expected to be more affected by higher modes than displacement response, since the higher modes remain elastic and their contribution in the displacement response is limited by their large stiffness, while only the first mode is significantly modified by ductility. Therefore, while the maximum displacement achieved by the structure cannot be strongly modified, adequate amplification factors suggested in literature (i.e. Paulay and Priestley, 1992) should be eventually adopted to take into account high levels of floor accelerations, which may lead to excessive diaphragm forces, as recently confirmed from a pseudo-dynamic large-scale experimental test on a five-story precast concrete building (Priestley et al. 1999).

2.1.4 Considerations for assessment procedure

Concepts involved in direct-displacement based design are also particularly applicable to assessment of existing structures (Priestley and Calvi, 1997). Traditional force-based assessment procedure would result into a "capacity design" evaluation of the whole system in order to estimate the actual hierarchy of strengths and sequence of events (plastic hinges or shear failure in structural elements). In most cases current established procedures tend to be rather rudimentary and based on fast comparison of details with a check list of possible deficiencies, by means of simple screening procedure (JBDPA, 1977, Otani, 2000; ATC, 1982, 1989, 1989).

An attempt to provide a more meaningful "system" approach to the assessment of existing frame buildings was suggested by Priestley and Calvi (1991): a two-levels seismic assessment procedure was outlined to determine the risk associated with both serviceability and ultimate limit states. A significant evolution of this earlier approach, with system response quantified in terms of displacement instead of elastic strength was presented by Priestley (1997) in a comprehensive procedure focused on existing buildings.

The main difficulties in evaluating the seismic response of existing structures, characterized by inadequate detailing as typical of structures designed before the adoption of any seismic codes (i.e. until early 70's in most of the seismic-prone regions) have been recently recognized (Calvi et al., 2002; Cosenza et al., 2002). Furthermore, different approaches should be adopted when a single building is studied in detail or when classes of buildings are considered, for scenario-oriented studies. A comprehensive general displacement-based assessment procedure for classes of buildings has been presented by Calvi (1999).

While extensive analytical and experimental investigations for the design of new earthquake resistant structures have so far characterized the research in Seismic Engineering, the evaluation of the seismic vulnerability of existing structures is a relative recent topic which, only in the last decade, has been subjected to a significant methodological upgrading. A general lack of information, based on experimental tests on the seismic behaviour of under-designed or gravity-load design frame systems or beam-column subassemblies, is observed. Furthermore, simplified assessment procedure are commonly adopted, since the analytical tools available in literature would require to be calibrated on extensive experimental results in order to reproduce specific mechanisms typical of inadequately designed structures.

As part of a co-ordinated national project on the seismic vulnerability of existing reinforced concrete frame buildings designed for gravity loads only, as typical in Italy between the 50's and 70's, quasi-static cyclic experimental tests on 2/3 scaled beam-column joints and a three story frame sys-
tem were performed at the Laboratory of the Department of Structural Mechanics of the University of Pavia (Pampanin et al., 2002; Calvi et al. 2002). The experimental results underlined the significant vulnerability of the joint panel zone region and the critical role of slippage phenomena due to the use of smooth bars and of inadequate anchorage. Particular hybrid brittle failure mechanisms due to interaction of shear damage and stress concentration at the hook anchorage location were observed (as in the case of Tee-joints). In particular the inaccuracy of traditional models (based on nominal shear stress or principal stress considerations) in predicting such damage mechanism, was evident as well as the necessity to develop alternative possible integrations or modifications.

3 COMPREHENSIVE DEFORMATION-CONTROLLED APPROACH

An alternative rational deformation-controlled procedure has been proposed by Fardis and Panagiotakos (1997) as an integrated part of the overall structural design process, which includes Ultimate Limit State (ULS) and Serviceability Limit State (SLS) design for gravity load and limitation of damage under “serviceability” earthquakes. This proposal differs from previous “direct” approaches for displacement based design procedure in many aspects, according to the preliminary concepts presented by Fardis (1995). The seismic design procedure is integrated into the overall structural design, along with the design for non seismic actions and consists of Ultimate Limit States verification against the conventional strain limits for a frequent “serviceability” earthquake and of proportioning the longitudinal and transverse reinforcement of critical regions in the structural members to meet the member inelastic chord rotation demands under the “life-safety” seismic action. In particular, for the scope of this article, the intent of pursuing a genuine deformation-based design approach, in which the individual members are detailed on the basis of inelastic deformation instead of strength demand, appear to be a fundamental concept. While typical end-result of alternative force-based design approaches is the design base shear and, thus, the internal strengths for which sections and members should be proportioned, the proposed procedure is conceived as a first draft of a pure deformation-based design, in which proportioning of the longitudinal and transverse reinforcement for R.C. members is based on required member deformation demand, similarly to what is commonly done today when proportioning the reinforcement for given internal forces.

The proposed method overcomes some present difficulties in proportioning the reinforcement of members for given deformation demands in complex structures. Few contributions have been presented in the literature to relate displacement demand for the whole structure to local member deformation demand: works from Wallace (1994) and Sasani & Anderson (1996) systems are available in literature, limited to wall systems; multistage procedures have also been proposed by Alonso et al. (1996) and Seleviratna and Krawinkler (1996) for multistory buildings.

In this method, which adopts the “chord rotation” (Fig. 3) as the key local displacement/deformation parameter, demand and supply are evaluated, based respectively on appropriate simplified linear-elastic analysis and semi-empirical parametric expressions, in the form of:

a) mean and upper-characteristic peak inelastic chord rotation demands
b) mean and lower characteristic values of member ultimate chord rotations (capacity), as a function of several parameters

Extensive parametric non-linear time-history analyses or available database of experimental test results were used to validate the efficiency of the proposed procedure, showing preliminary promising satisfactory agreements.
3.1 Fundamental steps of the procedure

In this deformation-controlled seismic design procedure for buildings, the proportioning of the structural members is based on the following concepts:

- Beams and walls are proportioned in bending for strength for factored gravity loads and a "serviceability" level seismic action.
- Columns are proportioned in bending for strength for factored gravity loads and capacity-design-moments at beam-column joints.
- Columns, walls and joints are proportioned in shear for capacity design shear forces determined from the flexural capacities of the corresponding critical end sections.
- The final detailed design of all the elements (including not only the detailing but also a possible adjustment of their dimensions and/or longitudinal reinforcement) takes place according to the corresponding peak displacement and deformation demands due to the "life safety" seismic action.

This leads to a feasible overall step-by-step design procedure of a structure, where different performances are adequately guaranteed by a direct control on global (displacement/drift) and local (member chord rotation) deformation demands.

Thus, the critical point of the whole procedure is the accuracy in estimating the inelastic deformation demand and capacity of the structural members, which is the basis of their proportioning, as alternative to a traditional approach (based on internal forces with subsequent control of local and global ductility capacity).

3.1.1 Estimation of deformation demand

The peak deformation demands during the inelastic seismic response are estimated through a simplified linear-elastic analysis. A simple "equal displacement" rule is proposed to be adopted for the
estimation of the peak inelastic displacements and deformations: the interstorey drifts and chord rotations, resulting from nonlinear dynamic response, are assumed to be equal to their elastic counterparts.

The procedure can be summarized as follows:

1) An elastic static analysis with inverted triangular distribution of lateral forces is performed, the magnitude of the forces being arbitrarily defined).
2) The so-computed (elastic) story drifts are used to calculate, through the Rayleigh quotient formula, the first mode period T and the “work-equivalent” peak displacement (u₀) of the equivalent SDOF system (in the form described in Eq 1 for Δ₀).
3) As final step, the elastic displacement spectrum (5% damped) is entered with T and a spectral displacement S₀ of the equivalent SDOF is determined. The structural displacements and deformations computed from the preliminary linear-elastic analysis are scaled by the ratio S₀/u₀ and considered representative of their inelastic counterparts during the nonlinear dynamic analysis.

Extensive nonlinear time-history analyses on RC multistorey frame structures, designed and studied within the PREC8 project (Fardis, 1995) were performed to check the validity of this procedure (Fardis and Panagiotakos, 1997). Satisfactory agreements between elastic predictions and peak inelastic response results were observed when referring to interstorey drift demands, while substantial differences were noted when considering members (beam and columns) chord rotations.

3.1.2 Estimation of deformation capacity

The chord rotation of a member is defined as the angle of rotation of the initial tangent at the end with respect to the chord connecting the two ends of the member, considered at the deformed state (Fig.3). Traditional simplified expressions to estimate the member rotation capacity θₘ have been proposed in the literature (Paulay and Priestley, 1992): in the simple case of uniaxial bending the ultimate end rotation capacity is commonly derived as the product of a plastic hinge length (lₚ) and the ultimate curvature at the end (φₑ).

A purely empirical alternative expression is proposed within this deformation-controlled approach to estimate the ultimate deformation capacity, based on regression analyses on extensive databank of monotonic and cyclic experimental tests on beam specimens. Fundamental specimen variables for the expression are: the shear span ratio, the depth of the member, the concrete strength, the mechanical reinforcement ratios of the tension, compression and confining steel, the ratio of longitudinal bar diameter to the stirrup spacing, the axial load ratio and the type of steel.

4 METHODS FORMULATED IN ACCELERATION - DISPLACEMENT FORMAT

Several recent methods, as the ones included in FEMA 273 (FEMA, 1997), ATC 40 (ATC, 1996), and Japan provisions (Otani et al, 2000), as well as the NZ method (Fajfar, 2000), combine the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. With the exception of FEMA 273, these methods are formulated in the acceleration - displacement (AD) format. In this format, the capacity of a structure is directly compared with the demands of earthquake ground motion on the structure. The graphical presentation allows a visual interpretation of the procedure and of the relations between the basic quantities controlling the seismic response. The capacity of the structure is represented by a force - displacement curve, obtained from a non-linear static (pushover) analysis. The base shear forces and roof displacements are converted into spectral accelerations and spectral
displacements of an equivalent single-degree-of-freedom (SDOF) system, respectively. These spectral values define the capacity diagram. The definition of seismic demand spectrum represents the main difference between different methods. In all cases, the intersection of the capacity diagram and the demand spectrum provides an estimate of the inelastic acceleration (strength) and displacement demand. In this chapter, two methods (capacity spectrum method, used in ATC-40 and in Japanese code, and N2 method) are summarized. The parts of the computational procedure that apply to both methods are described first. Then, the differences between the two methods are presented.

4.1 Elastic spectra in acceleration - displacement (AD) format

For an elastic SDOF system, the following relation applies

\[ S_{de} = \frac{T^2}{4 \pi^2} S_{ae} \]  

(3)

where \( S_{ae} \) and \( S_{de} \) are the values in the elastic acceleration and displacement spectrum, respectively, corresponding to the period \( T \) and a fixed viscous damping ratio. A typical smooth elastic acceleration spectrum for 5\% damping, normalized to a peak ground acceleration of 1.0 g, and the corresponding elastic displacement spectrum are shown in Figure 4a. In the acceleration - displacement (AD) format spectral accelerations are plotted against spectral displacements, with the periods \( T \) represented by radial lines. Consequently, both spectra can be plotted in the same figure (Fig. 4b).

![Graphs showing elastic acceleration and displacement spectra](image)

Figure 4 - Typical elastic acceleration \((S_{ae})\) and displacement spectrum \((S_{de})\) for 5\% damping normalized to 1.0 g peak ground acceleration. a) traditional format, b) AD format.

Note that the spectra in Figure 4 have been plotted only for short- and medium-period range. At longer periods the displacement spectrum is typically constant. Consequently, the acceleration spectrum in the long-period range typically decreases with the square of the period \( T \). In the very-long-period range, spectral displacements decrease to the value of the peak ground displacement. As already mentioned, the actual features of general-purpose elastic spectra in the long- and very-long-period ranges are still under investigation.

4.2 Pushover analysis and transformation to an equivalent SDOF system

Using a pushover analysis, a characteristic nonlinear base shear - top displacement relationship of the MDOF system can be determined. The selection of the appropriate lateral load distribution is an important step within the pushover analysis. A unique solution does not exist. Fortunately, the range of reasonable assumptions is usually relatively narrow and, within this range, different as-
sumptions produce similar results. A common approach (major code provisions) is to apply lateral forces in proportion to a first mode distribution pattern. A practical possibility is also to use two different load patterns and to envelop the results. More sophisticated approaches change the distribution as a function of the displacement pattern of the structure. The limitations of the methods based on pushover analysis should be recognized (see Par. 4.6).

The base shear – top displacement relationship, which applies to the MDOF system, is transformed to the force – displacement relationship of the equivalent SDOF system. The transformation factor(s) depend(s) on the assumed pattern of lateral forces and on the assumed displacement shape. In the N2 method, the assumed lateral forces and displacements are interrelated. As a consequence, the same transformation factor applies to both forces and displacements, and the initial stiffness of the equivalent SDOF system remains the same as that of the MDOF system. In the capacity spectrum method, assumed forces and displacement are not related, thus different transformation factors apply to forces and displacements. Next, the force – displacement relationship of the equivalent SDOF system is idealized into a bilinear form. Here engineering judgement has to be used. In the simple variant of the N2 method, an elastic - perfectly plastic relation is employed. The acceleration - displacement relation (capacity diagram) is obtained if the forces in the force - deformation curve for the equivalent SDOF system are divided by the equivalent mass.

4.3 Seismic demand in the capacity spectrum method

The AD format was proposed in conjunction with the capacity spectrum method (Freeman et al, 1975, Freeman, 1998). In this method the inelastic seismic demand is simulated by a highly damped elastic spectrum. Procedures have been proposed for the determination of equivalent viscous damping, which is intended to take into account the hysteretic energy dissipation. The equivalent (effective) elastic period of the system corresponds to the secant stiffness at the maximum displacement. An iterative procedure is required to determine the seismic demand, which is defined by the intersection of the capacity curve and the demand spectrum. The use of highly damped elastic spectra for the determination of seismic demand is a controversial part of the capacity spectrum method. Moreover, the iterative procedure may not converge as demonstrated by Chopra and Goel (1999). The capacity spectrum method is used in ATC 40 and in the new code in Japan (Otani et al, 2000).

Elastic spectra for high damping values, used in the capacity spectrum method, are plotted in Figure 5. They are based on Newmark – Hall (1982) median values of amplification factors for acceleration and velocity-controlled ranges of spectrum. The capacity diagram and the demand spectra (elastic spectrum for 5% damping and reduced spectrum, which corresponds to a higher value of damping) are shown in Figure 6. The intersection of the radial line, corresponding to the elastic period T* (stiffness) of the idealised bilinear system, and the elastic demand spectrum defines the strength (acceleration) required for elastic behaviour and the corresponding elastic displacement demand. Inelastic displacement demand, denoted as S_d, is determined from the elastic spectrum for equivalent damping. The corresponding ductility demand µ is defined as S_d divided by yield displacement D_0*. The demand diagram corresponds to a specific value of the equivalent damping (25% in Figure 4), which is determined as a function of ductility demand µ. As mentioned, several different ductility – damping relations have been proposed in literature. A partial compilation was made by Freeman (1998). Large ranges of equivalent damping values correspond to a specific ductility value. For a ductility demand equal to 3, for example, values from 16 to 39% have been suggested.

In next steps of analysis, the displacement demand of the SDOF model is transformed to the maximum displacement at the top of the MDOF model, which represents the target displacement for the pushover analysis. Local seismic demands (e.g. storey drifts, joint rotations) are determined by pushing the MDOF model up to the target displacement.
Figure 5 - Demand spectra for different damping coefficients in AD format normalized to 1.0 g peak ground acceleration.

Figure 6 - Elastic and equivalent elastic demand spectra versus capacity diagram.
4.4 Seismic demand in the N2 method

A more straightforward approach for the determination of seismic demand is based on the use of the inelastic strength and displacement spectra, which can be obtained directly by time-history analyses of inelastic SDOF systems. A simpler alternative is to evaluate inelastic demand spectra from smooth elastic spectra by using reduction factors obtained from statistical analyses. Such an approach is used in the latest version of the N2 method (N stands for Nonlinear analysis and 2 for two mathematical models). The N2 method has been developed at the University of Ljubljana during the last 15 years (Fajfar and Gasparsic, 1996). Following the concept of Bertero (1995) and Reinhorn (1997), it has been recently formulated in the AD format (Fajfar, 1999, 2000).

The seismic demand is, like in the case of the capacity spectrum method, determined by an elastic acceleration spectrum $S_a$. In principle, any spectrum can be used. However, the most convenient is a spectrum of the Newmark-Hall type. For an inelastic SDOF system with a bilinear force-deformation relationship, the acceleration spectrum $S_a$ and the displacement spectrum $S_d$ can be calculated as

$$ S_a = \frac{S_{ae}}{R_\mu}, \quad S_d = \frac{\mu}{R_\mu} S_{de} $$

(4),(5)

where $\mu$ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement, and $R_\mu$ is the reduction factor corresponding to ductility and to the hysteretic energy dissipation. Note that $R_\mu$ is not equivalent to the reduction factor $R$ used in seismic codes. The code reduction factor $R$ takes into account both energy dissipation and the so-called overstrength $R_v$. It can be defined as $R = R_v R_{el}$.

Several proposals have been presented for the reduction factor $R_\mu$. An excellent overview has been given by Miranda and Bertero (1994). In the simple version of the N2 method, a bilinear spectrum for the reduction factor $R_\mu$ is used:

$$ R_\mu = (\mu - 1) \frac{T}{T_c} + 1 \quad T < T_c $$

(6)

$$ R_\mu = \mu \quad T \geq T_c $$

(7)

where $T_c$ is the characteristic period of the ground motion (Figure 1a). It is typically defined as the transition period where the constant acceleration segment of the response spectrum (the short-period range) passes to the constant velocity segment of the spectrum (the medium-period range). It roughly corresponds to the period at which the largest energy is imparted to the structure. Equations 5 and 7 suggest that, in the medium- and long-period ranges, the equal displacement rule applies, i.e. the displacement of the inelastic system is equal to the displacement of the corresponding elastic system with the same period.

Starting from the elastic design spectrum, and using equations 5 to 7, the demand spectra for the constant ductility factors $\mu$ in AD format can be obtained (Fig. 7). They represent inelastic demand spectra. It should be noted that the construction of these spectra is in fact not needed in the computational procedure. They just help for the visualisation of the procedure.

The capacity diagram and the demand spectra (elastic spectrum for 5% damping and reduced inelastic spectrum) are shown in Figure 8. Again, the intersection of the radial line, corresponding to the elastic period (stiffness) of the idealised bilinear system, and the elastic demand spectrum defines the strength (acceleration) required for an elastic response and the corresponding elastic displacement demand. The reduction factor $R_\mu$ is equal to the ratio between the elastic acceleration $S_{ae}$ and the yield acceleration $S_{ay}$, representing the acceleration demand of the inelastic system $S_a$.

$$ R_\mu = \frac{S_{ae}}{S_{ay}} $$

(8)
If the elastic period $T^*$ is equal to or larger than $T_C$, the inelastic displacement demand is equal to the elastic one

$$S_d = D^* = S_{d,e} \quad T^* \geq T_C$$

and the ductility demand $\mu$ is equal to the reduction factor $R_\mu$. Otherwise, the inelastic displacement demand is larger than the elastic one. It is determined according to the formula developed from equations 3 and 4

$$S_d = D^* = \frac{S_{d,e}}{R_\mu} \left(1 + (R_\mu - 1) \frac{T_C}{T^*}\right) \quad T^* \leq T_C$$

Graphically, the inelastic seismic demand is defined by the intersection of the capacity diagram and the inelastic demand spectrum for the ductility demand that corresponds to the reduction factor $R_\mu$ (Fig. 8).

At this stage, the displacement demand can be modified if necessary, e.g. to take into account the larger displacements observed for systems with narrow hysteresis loops or negative post-yield stiffness.

The computational procedure which follows is the same as in the case of the capacity spectrum method.

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**Figure 7 - Demand spectra for constant ductilities in AD format normalized to 1.0 g peak ground acceleration.**
4.5 Performance evaluation

The expected performance can be assessed by comparing seismic demands and capacities for the relevant performance level. This evaluation is usually made on global and local levels. In this step, the effect of cumulative damage can be taken into account by reducing the capacity. The global evaluation can be visualized by comparing displacement capacity and demand.

4.6 Approximations and limitations

The described methods are, like any approximate method, subjected to several limitations. Applications of these methods are, for the time being, restricted to the planar analysis of structures. There are two main sources of approximations and corresponding limitations: the pushover analysis and the evaluation of the target displacement. 

**Nonlinear static (pushover) analysis** can provide an insight into the structural aspects that control the performance during severe earthquakes. The analysis provides data on the strength and ductility of the structure which cannot be obtained by elastic analysis. Furthermore, it exposes design weaknesses that may remain hidden in an elastic analysis. On the other hand, the limitations of the approach should be recognized. Pushover analysis is based on a very restrictive assumption, i.e. a time-independent displacement shape. Thus, it is in principle inaccurate for structures where higher mode effects are significant, and it may not detect the structural weaknesses which may be generated when the structure’s dynamic characteristics change after the formation of the first local plastic mechanism. A detailed discussion of pushover analysis can be found in the paper by Krawinkler and Seneviratna (1998). Additional discussion on the relationship between MDOF and SDOF systems is presented in (Gupta and Krawinkler 2000).

One practical possibility to partly overcome the limitations imposed by pushover analysis is to assume two different displacement shapes (load patterns), and to envelope the results. Another possibility, which might be too complicated for practical applications, is to use lateral load distributions that change in each step of the analysis. An “adaptive pushover” analysis has recently been pro-
posed by Satyarno (1998) and implemented in the finite element analysis code Ruaumoko (Carr, 1998).

A second important source of inaccuracy is the determination of target displacement (displacement demand) for the equivalent SDOF system. If inelastic spectra are used, the displacement demand depends on the initial period of the system and on the spectrum. The initial period of the equivalent SDOF system is not uniquely defined, depending on the bilinear idealization of the actual base shear – top displacement curve and being to some extent, based on engineering judgement. Inelastic spectra are based on statistical analyses of structural models and may not apply to structures, whose inelastic behaviour is basically different from that assumed in statistical analyses. The simplest possibility to develop inelastic spectra is to apply, in the medium- and long-period range, the “equal displacement rule.” The equal displacement rule has been used quite successfully for almost 40 years. Many statistical studies (see, e.g., the discussion by Fajfar, 2000) have confirmed that the equal displacement rule is a viable approach for structures on firm sites with the fundamental period in the medium- or long-period range, with relatively stable and full hysteretic loops. A slightly conservative estimate of the mean value of the inelastic displacement may be obtained. The equal displacement rule, however, tends to underestimate inelastic displacements in the case of near-fault ground motions, hysteretic loops with significant pinching or significant stiffness and/or strength deterioration, and for systems with low strength (i.e., with a yield strength to required elastic strength ratio of less than 0.2). Moreover, the equal displacement rule seems to be not satisfactory for soft soil conditions. In these cases, modified inelastic spectra should be used. Alternatively, correction factors for displacement demand may be applied, if available.

In the case of short-period structures, inelastic displacements are larger than the elastic ones. The transition period, below which the inelastic to elastic displacement ratio begins to increase, is roughly equal to the fundamental period of the ground motion $T_c$. In the short-period range, the sensitivity of inelastic displacements to changes of structural parameters is greater than in the medium- and long period ranges. Consequently, estimates of inelastic displacement are less accurate in the short-period range. However, the absolute values of displacements in the short-period region are small and, typically, they do not control the design.

In the case of equivalent elastic spectra, the major source of uncertainty is the determination of equivalent viscous damping. Different relations between maximum ductility and equivalent damping coefficient have been proposed. The recent proposal by Freeman (1998) is based on a calibration aimed at fitting the Newmark – Hall inelastic spectra. So, the question arises, why the detour with equivalent elastic spectra is needed? Why not use inelastic spectra directly? In the short-period range, the equivalent elastic spectra differ a lot from realistic inelastic spectra (see Figs. 6 and 8).

5 SEAOC BLUE-BOOK APPROACH AND FORMULATION

Different design and performance evaluation approaches, including direct displacement-based design, can be applied within the framework of the methods formulated in the AD format. Four quantities define the structural behaviour: strength, displacement, ductility and stiffness. Design and/or performance evaluation begins by fixing one or two of them. The others are determined by calculations. Different approaches differ in the quantities that are chosen at the beginning of the design or evaluation. For example, for a seismic performance evaluation of a newly designed or existing structure, stiffness and strength have to be known, whereas a direct displacement-based design approach starts from a predetermined target displacement.

As already mentioned, the Performance Based Seismic Engineering Preliminary Guidelines contained in the SEAOC Bluebook presents two Displacement Based Design procedures within a general procedure in which seismic hazard levels (EQ-I to EQ-IV), performance levels (structural and non-structural) and enhanced objectives (suggested performances for given level input motion) are
defined. The whole procedure is formulated in AD format by using Acceleration-Displacement Response Spectra (here called ADSR). Alternatively, typical displacement and acceleration spectra can be independently use.

Interstorey drift limits associated with each structural performance level are suggested for different lateral load resisting structural systems (for RC, reinforced masonry, steel and wood structures) for use in preliminary design. Recommendations regarding siting restrictions, yield mechanisms, system damping, ductility limits are also given.

5.1 Direct-Displacement Based Design (DBD) procedure

The Direct Displacement Based Design method contained in the SEAOC Blue Book basically absorbs the aforementioned direct displacement based design procedure proposed by Priestley (1998). As further refinements, Acceleration Displacement Response Spectra can be utilized and the evaluation and check, after member proportioning, of the system initial stiffness and strength at yield points are required (see step 3 below).

5.1.1 Fundamental steps of the procedure

This DBD procedure can be summarized as follow:

Step 1 - Selection of target displacement $\Delta_T$ (similarly as DBD) at a selected performance level, referring to the suggested interstorey drift limits ($\delta_i$) for the structural system and to the expected inelastic mode shape:

$$\Delta_T = \delta_i \cdot h_i \cdot k_2 = \delta_i \cdot h_i \cdot k_1 \cdot k_2$$ (11)

being:

$h_i$ = effective height for the equivalent SDOF system
$h_i$ = height at roof level
$k_1$ = factor to relate the height of the roof to the effective height, according to a “work-based” equivalence.
$k_2$ = factor to relate the expected deformed shape function to a linear displaced shape function

Step 2 - Determine Effective Period at each limit state entering the Displacement Response Spectrum with an appropriate damping value. Alternatively the effective period can be evaluated by entering the Acceleration-Displacement Response Spectrum (ADSR) with $\Delta_T = S_d$, reading the corresponding spectral ordinate $S_a$ and utilizing the following relationship:

$$T^2_{eff} = 4\pi^2 \frac{S_d}{S_a}$$ (12)

The effective stiffness ($K_{eff}$) is then simply determined as a function of the effective mass ($M_{eff}$) and initial period ($T_0$):

$$K_{eff} = 4\pi^2 \frac{M_{eff}}{T^2_{eff}}$$ (13)

Step 3 - The required initial stiffness ($K_{ini}$) at effective yield point can be approximated as (considering an elasto-plastic model):
\[ K_i = \mu K_{\text{eff}} \]  

being \( \mu \) the system ductility.

Step 4 - The required system strength \( V_{\text{base}} \) (design base shear at target drift) is then determined as:

\[ V_{\text{base}} = K_{\text{eff}} \Delta_T \]  

Step 5 - Member strengths are obtained by distributing the base shear along the elevation and the members are finally proportioned, maintaining good agreement with the assumption on initial structural stiffness.

Step 6 - Verification of drift limits and ductility demands at the desired target displacement levels are obtained by performing pushover analyses. If required, design modifications are applied within an iterative design-verification process.

Step 7 - Final design and detailing according to a “capacity design” philosophy

5.2 Equal-Displacement-based (EDB) design procedure

The EDB is based on the “equal displacement” approximation noted by Newmark-Hall (1982) and can be considered an adaptation of the traditional force-based code procedure which uses an acceleration displacement response spectrum (ADRS) or the displacement response spectrum rather than the acceleration response spectrum and focuses on displacements rather than forces.

Since the equal displacement approximation is generally applicable to structures with fundamental period greater than that corresponding to peak elastic response \( T_m \) for the earthquake under consideration and leads to unconservative results for shorter period structure, the EBD procedure can be simply modified by utilizing appropriate relationships based on the “equal energy” approximation to estimate the inelastic displacement.

The EBD utilizes the 5% damped ADRS, the equivalent structure and the expected system ductility factor, \( \mu \), to model the inelastic displacement response. The procedure includes estimation of target drifts, determination of required initial stiffness based on the design ductility limits and the acceleration displacement response spectrum, determination of required strength based on the target displacement, initial stiffness and system ductility. Members are designed based on required stiffness, strength and ductility.

5.2.1 Fundamental steps of the procedure

The whole EBD procedure can be summarized as follow:

Step 1 - Selection of target displacement \( \Delta_T \) (similarly as DBD) at a selected performance level, referring to the suggested interstorey drift limits for the structural system and to the expected inelastic mode shape.
Step 2 - Determine initial period (at yield limit state), entering the 5% damped elastic Acceleration-Displacement Response Spectrum (ADSR) with $\Delta_i = S_i$, reading the corresponding spectral ordinate $S_a$ and utilizing the following relationship:

$$T_i^2 = 4\pi^2 \frac{S_i}{S_a} \quad (16)$$

The initial stiffness at effective yield point ($K_i$) is then simply determined as a function of the effective mass ($M_{eff}$) and initial period ($T_i$):

$$K_i = 4\pi^2 \frac{M_{eff}}{T_i^2} \quad (17)$$

Step 3 - The required system yield strength (design base shear at effective yield) is then determined based on initial stiffness, target displacement and system ductility:

$$V_y = K_i \Delta_y / \mu \quad (18)$$

Step 4 - Member required strengths are obtained by distributing the base shear along the elevation and the members are finally proportioned, maintaining good agreement with the assumption on initial structural stiffness.

Step 5 - Verification of drift limits and ductility demands at the desired target displacement levels are obtained by performing pushover analyses. If required, design modifications are applied within an iterative design-verification process.

Step 6 - Final design and detailing according to a “capacity design” philosophy

Note that the main differences between the DBD and EBD approaches proposed by the SEAOC BlueBook are practically related to the definition of the structural properties of the “equivalent” SDOF system: while the DBD adopts secant stiffness and equivalent viscous damping at target displacement, the EBD uses initial stiffness (at effective yielding point) and a traditional viscous damping value of 5%.

6 RESEARCH NEEDS

The main drawbacks and limits of the methods reviewed above have been discussed. It is worth keeping in mind that the intrinsic nature of any approximate procedure leads to limitations, as a counterpart and compromise of its simplicity. For the time being, some of them are restricted to symmetric structures, may not apply to all types of structural systems and to all types of ground motions. Cumulative damage is not taken into account and higher modes effects should be independently investigated. Although conceptual solutions have been proposed, these problems are far from being thoroughly solved. All methods have been defined in a pure deterministic format. However, due to randomness of the seismic input and of the structural capacities and due to uncertainties inherent to mathematical modelling, probabilistic reliability studies will also be required to estimate appropriate protection factors.
7 CONCLUSIONS

The development and refinement of deformation or displacement based design approach has critically influenced, in the last decade, the research trend in seismic engineering leading to substantial modifications of traditional force-based approaches towards a performance-based design engineering. The fundamental concepts and approaches of recent seismic design and assessment procedures are major trends in research work and the preliminary guidelines have been herein discussed and overviewed, identifying the main limits and necessity for further developments. In a short term period a broad acceptance of these procedure within the major seismic code recommendation has to be encouraged.

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