

Chapter 13

Pushover Analysis for Plan Irregular Building Structures

Mario De Stefano and Valentina Mariani

Abstract Nonlinear static procedures (NSPs), also known as “pushover methods”, represent the most used tool in the professional practice for assessment of seismic performance of building structures. Most of the methods subscribed by major seismic codes for seismic analysis of new or existing buildings have been originally defined for simple regular structures.

Nevertheless, perfect regularity is an idealization that very rarely occurs and, in principle, the concept of irregularity itself is a *fuzzy* one. Most codes attempt to give a definition to the concept of “regularity”, considering issues related to the distribution of mass, stiffness and strength in the building, both in plan and in elevation. Real buildings rarely comply with these regularity requirements, resulting in a barely reliable application of the basic NSPs. Code specifications concerning irregular structures are in need of improvement and they do not provide for clear and specific guidelines for the seismic analysis of such structures. Therefore the problem of the seismic evaluation of irregular structures is still an open one and basic issues need to be further explored.

The present paper aims at providing a wide outlook on the problem of the seismic assessment of plan irregular building structures. Firstly, a brief review of the elastic and inelastic methods for the assessment of the torsional effects induced by in-plan irregularity is presented, mainly aimed at the definition of the variables governing the problem. Then, the basic features of the most important NSPs are discussed, followed by the description of the recent improvements developed for irregular structures. Since there is not yet a fully satisfactory solution, pros and cons of the various approaches are outlined, highlighting the most promising methods and the issues that are yet to be investigated. Finally, recommendations for code improvement are suggested.

M. De Stefano (✉) • V. Mariani
Department of Architecture, University of Florence, Piazza Brunelleschi 6, 50121 Firenze,
Italy
e-mail: mario.destefano@unifi.it; valentina.mariani@unifi.it

13.1 Introduction

Structural irregularities are one of the major causes of damage amplification under seismic action. Past earthquakes, indeed, have shown that buildings with irregular configuration or asymmetrical distribution of structural properties are subjected to an increase in seismic demand, causing greater damages. The sources of irregularity in a building configuration can be multiple and of different kinds and are usually classified in two major categories: irregularities in plan and in elevation. The first type is related to in plan asymmetrical mass, stiffness and/or strength distributions, causing a substantial increase of the torsional effects when the structure is subjected to lateral forces. The second one involves variation of geometrical and/or structural properties along the height of the building, generally leading to an increase of the seismic demand in specific storeys. Both these types of irregularity often entail the development of brittle collapse mechanisms due to a local increase of the seismic demand in specific elements that are not always provided with sufficient strength and ductility.

Most seismic codes, including EC8-1 (2004), provide empirical criteria for the classification of buildings into regular and irregular categories with reference to: mass and lateral stiffness variations in plan and in elevation (and related eccentricities), shape of the plan configuration, presence of set-backs, in-plan stiffness of the floors (rigid diaphragm condition), continuity of the structural system from the foundations to the top of the building. This list is not comprehensive of all the possible causes of irregularity and there is no definition for the degree of irregularity of the overall three-dimensional system. Code definitions fail to capture some irregularities, especially those resulting from the combination of both plan and vertical irregularities. Moreover, system irregularity does not solely depend on geometrical and structural properties of the building, but can also be induced by the features of the earthquake excitation and increased by the progressive damage of the structure.

Considering this scenario, there is an urgent need to define and measure structural irregularity with a more rational approach, to deeply understand its effect on the seismic behavior and consequently upgrade seismic codes with specific and effective prescriptions for irregular buildings.

Among the two aforementioned types of structural irregularity, in-plan irregularity appears to have the most adverse effects on the applicability of the classical nonlinear static procedures (NSPs), precisely because such methods have been developed for the seismic assessment of structures whose behavior is primarily translational. This is the reason why, in recent years, the extension of NSPs to plan irregular building structures has been widely investigated by specialists in this field.

13.2 Brief Review of the Assessment Methods of Induced Torsional Effects in Plan Irregular Structures

The dawn of the studies concerning the torsional effects featuring irregular buildings dates back to the 30s of the last century (Ayre 1938), due to an increasing awareness of the complexity of the response of non-symmetric buildings to seismic actions, that is not purely translational, but involves torsional deformations that in most cases adversely affect their seismic behavior.

In the early studies (Housner and Outinen 1958; Bustamante and Rosenblueth 1960; Kan and Chopra 1977; Reinhorn et al. 1977) the problem has been faced in the elastic range, referring to simplified one-storey or multistorey models. Some research is still under development in this field (for state-of-the-art reports, refer to Anagnostopoulos et al. 2013 and to previous reviews by Rutenberg 1992, 2002; Rutenberg and De Stefano 1997; De Stefano and Pintucchi 2008), even if the assumptions made for formulating such models involve many simplifications.

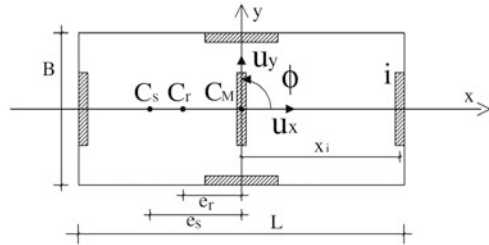
Nevertheless, these studies mainly succeeded in underlining the parametric nature of the problem. The main identified parameters that play a crucial role in the definition of the torsional behavior of irregular structures are the uncoupled natural periods, the stiffness eccentricity and the stiffness radius of gyration (non-dimensionalized with respect to the mass radius of gyration). These parameters, for a one-storey building, are defined as follows, with reference to the x direction (Fig. 13.1). Similar equations apply to the y direction.

- Uncoupled natural period $T_x = 2\pi\sqrt{\frac{m}{K}}$
where m and K are the total mass and stiffness in x direction respectively;
- Stiffness eccentricity $e_{sx} = \frac{1}{L} \frac{\sum_{i=1}^N k_{yi}x_i}{K}$
i.e. the distance (along x direction) between the stiffness centre C_S and the mass centre C_M ;
- Torsional stiffness $I_{p,k} = \sum_{i=1}^N [k_{yi}(x_{i,C_s})^2 + k_{xi}(y_{i,C_s})^2]$
i.e. the polar moment of inertia of system stiffness computed with respect to the axes parallel to the z direction and passing through C_S ;
- Stiffness radius of gyration $d_s = \frac{1}{\rho L} \sqrt{\frac{I_{p,k}}{K}}$
non-dimensionalized with respect to the mass radius of gyration ρ .

Lately, the problem has been widely faced even in the inelastic range, introducing parameters related to resistance, i.e. strength eccentricity and strength radius of gyration. These parameters, for a one-storey building, are defined as follows, with reference to the x direction (Fig. 13.1). Similar equations apply to the y direction.

- Strength eccentricity $e_{rx} = \frac{1}{L} \frac{\sum_{i=1}^N F_{yi}x_i}{F}$
i.e. the distance (along x direction) between the strength centre C_R and the mass centre C_M ;

Fig. 13.1 Simplified scheme of a one-storey building, for the identification of the key parameters characterizing the torsional behavior of plan irregular structures



- Torsional strength $I_{p,f} = \sum_{i=1}^N [F_{yi}(x_{i,C_r})^2 + F_{xi}(y_{i,C_r})^2]$
 as defined by De Stefano and Pintucchi (2010), i.e. the polar moment of inertia of system strength computed with respect to the axes parallel to the z direction and passing through C_R ;
- Strength radius of gyration $d_r = \frac{1}{\rho L} \sqrt{\frac{I_{p,f}}{F}}$
 non-dimensionalized with respect to the mass radius of gyration ρ .

The studies in the inelastic range have been conducted by analyzing both the one-storey and the multistorey models. In the former case, methods considering uni-directional eccentricity, strength and ground motion were developed, subsequently improved considering these parameters in both principal directions. Concerning the multistorey models, some simplified shear-type models have been developed as well as detailed plastic hinge type models (see reviews by Rutenberg 1992, 2002; Rutenberg and De Stefano 1997; De Stefano and Pintucchi 2008; Anagnostopoulos et al. 2013).

Shifting from elastic to inelastic range, the parametric dependence of the problem become more complex and less analytically determined. One key-aspect is for example the assumption of a proportional relationship between stiffness and strength, that can be considered valid for pre-normative existing structures not designed for torsional effects, but not for more recent buildings designed according to modern seismic codes. Other issues are related to the evaluation of the effect of level of ductility of the structure, assumption of different nonlinear constitutive laws etc.

This large amount of studies has not yet led to general conclusions. Indeed, since many parameters affect the problem, different combinations of assumptions have often led to conflicting conclusions. Moreover, both one-storey and multi-storey models still suffer from several shortcomings related to many simplifying assumptions, that often make very difficult the generalization of obtained results.

13.3 Fundamentals of Classical Nonlinear Static Procedures

The formulation of the nonlinear static analysis, often defined as “pushover analysis”, dates back to the 70s of the last century. Although it has only recently been included in seismic code provisions, the procedure itself has been already largely applied in the past, in research and design applications. With the coming of the performance-based (PB) design philosophy, pushover analysis turned to be the most used approach for the seismic assessment and design of structures, and became the starting point of all the so-called nonlinear static procedures (NSPs). PB design focuses on the actual performance of the structure under earthquake conditions, defining multiple performance objectives related to multiple seismic action levels. The modern PB design/assessment methods generally refer to displacements and deformations as performance targets.

The best way to evaluate the seismic performance of a structure is the nonlinear dynamic analysis (NLDA) that represents the most rigorous and accurate approach, as it directly provides the behavior of the structure under a series of seismic records. Nevertheless it should be kept in mind that the response is sensitive to the input ground motion, therefore several analyses are required with increased complexity, computational costs and time consumption. This is the reason why NLDA is still far from an extensive application in common practice.

Given the aforementioned limitations in the use of NLDA, in the last decades the NSPs have been brought to the forefront of seismic design/assessment of structures. Basically, the methods are based on the evaluation of three key quantities: seismic capacity, seismic demand and performance. In all the NSPs, the seismic capacity is evaluated through pushover analysis, that consists of “pushing” the structure with an increasing lateral load pattern, in combination with gravity loads, until the attainment of the structure collapse. As the load increases, the structure shifts from elastic to inelastic field and the overall behavior can be expressed in terms of global significant quantities, e.g. base shear and displacement of a control point (generally the top of the structure). The plot of the top displacement versus the total base shear is currently known as “capacity curve”.

The seismic demand is a representation of the expected earthquake action through acceleration and displacement spectra. Generally in the NSPs the seismic demand is expressed in terms of “target displacement”, that represents the maximum inelastic displacement that the structure should be able to undergo.

Finally, the performance, very clearly defined in ATC-40 (1996), “is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demand of the earthquake such that the performance of the structure is compatible with the objectives of the design”. This definition represents the core meaning of PB design/assessment methods.

The various NSPs mainly differ in the evaluation of the seismic demand, that represents a key aspect, because of the need to account for the inelastic response of the structure. Several approaches are available. The most well-known NSPs,

suggested also by the most important worldwide seismic codes, are briefly described in the following.

13.3.1 Capacity Spectrum Method

The Capacity Spectrum Method (CSM) has been firstly proposed by Freeman et al. (1975) and Freeman (1998, 2004) as a rapid seismic evaluation procedure and then developed into a seismic design/assessment method adopted by the California Seismic Safety Commission through the ATC-40 (1996) guidelines, lately improved considering innovative features suggested in the FEMA-440 (2005) report. The CSM is a graphical procedure that compares the capacity of the structure, in terms of capacity (pushover) curve of an equivalent Single-Degree-Of-Freedom (SDOF) system, with the seismic demand, in the form of a response spectrum. Both capacity and demand are expressed in the Acceleration-Displacement Response Spectrum (ADRS) format.

The pushover curve of the MDOF system is converted in the equivalent push-over curve of a SDOF system and then bi-linearized according to the equal energy or equal displacement rules. Finally it is expressed in terms of spectral acceleration S_a and spectral displacement S_d obtaining the capacity spectrum. The seismic demand is represented by several spectra with different values of equivalent viscous damping ratio ξ . The graphical verification consists in checking if the capacity spectrum can extend through the envelope of the demand spectrum. If yes, the building is able to undergo the seismic demand action. Otherwise, if the capacity spectrum has no intersection with the demand spectrum, the structure does not resist the design earthquake. The intersection between capacity and demand spectra represents a performance point in terms of maximum acceleration and displacement for the SDOF system.

Once defined a certain performance point on the capacity curve, in order to quantify the deficiency (or the exceedance) of the capacity with respect to demand, the elastic spectrum has to be iteratively scaled until it intersects the capacity curve in correspondence of the assumed capacity (performance) point. The scaling procedure is done through spectral reduction factors related to equivalent viscous damping values, that account for the inherent viscous damping of the structure (generally assumed as 5 %) and hysteretic damping. Therefore the seismic capacity evaluation is done through damped elastic spectra.

The main limitation of the CSM is that the inelastic response of the structure is represented with over-damped elastic spectra, characterized by modified values of damping. This issue will be lately overcome with the development of the N2 method by Fajfar and Fischinger (1988), which considers the use of constant-ductility inelastic spectra, rather than over-damped elastic spectra.

13.3.2 N2 Method

The N2 method, firstly proposed by Fajfar and Fischinger (1988) and then developed in Fajfar and Gašperšič (1996), Fajfar (1999, 2000), is the NSP adopted by the Eurocode 8 (EC8-1 2004) and represents a modified version of the CSM. In the N2 method indeed the evaluation of the seismic demand is based on the use of inelastic spectra, instead of highly damped elastic spectra, as done through the CSM.

Therefore this method maintains the clarity of a visual graphical representation of the capacity-demand comparison, in combination with a more consistent approach related to the use of inelastic demand spectra as an alternative to highly damped elastic spectra, that indeed, have no physical basis. The inelastic spectra are derived reducing the elastic spectrum by a reduction factor R_μ , directly related to the hysteretic dissipative capacity of the structure, expressed by the ductility factor μ , i.e. the ratio between the maximum displacement and the yield displacement of the SDOF bilinear capacity curve.

The target displacement is determined referring to the equal displacement rule for medium and long period range, while for short period range, the target displacement is larger than the one associated to the corresponding equivalent elastic system (Fig. 13.3). More in details, the method assumes that in the medium/long period range ($T^* \geq T_C$) the equal displacement rule applies, i.e. the displacement of the inelastic system S_d is equal to the displacement of the associated elastic system S_{de} characterized by the same period T^* (Fig. 13.2a). This means that in the above mentioned range of periods $R_\mu = \mu$. Therefore the seismic demand in terms of inelastic displacement, can be obtained by intersecting the radial line corresponding to the period of the SDOF system with the elastic demand spectrum.

On the other hand, in the case of short-period structures ($T^* < T_C$) the inelastic displacement is larger than the elastic one and the equal displacement rule does not apply anymore (Fig. 13.2b). Consequently $R_\mu < \mu$ and it can be determined as the ratio between the elastic acceleration demand S_{ae} and the inelastic acceleration capacity S_{ay} . The inelastic displacement demand is, in this case, equal to $S_d = \mu \cdot D_y^*$, being D_y^* the yielding displacement of the SDOF system. The ductility factor can be derived from the reduction factor by the relation:

$$\mu = (R_\mu - 1) \frac{T_C}{T^*} + 1$$

In both cases ($T^* \geq T_C$ and $T^* < T_C$) the inelastic acceleration demand S_a is equal to the elastic one S_{ae} and it can be determined at the intersection of the radial line corresponding to the period of the SDOF system with the elastic demand spectrum.

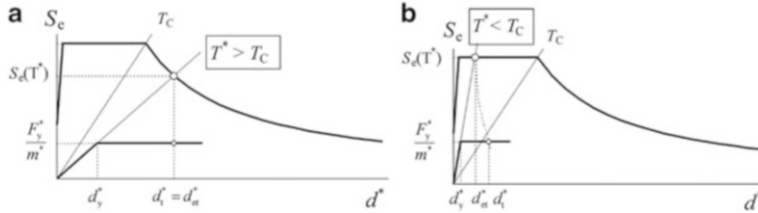


Fig. 13.2 Evaluation of the inelastic displacement demand S_d for (a) short-period structures ($T^* < T_C$) and (b) medium/long period structures ($T^* \geq T_C$), according to EC8-1 Annex C “Determination of the target displacement of the equivalent SDOF system”

13.3.3 Displacement Coefficient Method

The Displacement Coefficient Method (DCM), adopted by FEMA 356 (2000), is a simplified procedure for the estimation of the seismic demand, that applies a series of corrective coefficients to the elastic spectral displacement demand so as to obtain a target displacement, i.e. the maximum inelastic displacement demand. The following relation applies for the determination of the target displacement δ_t :

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

The four modification coefficients (C_0, C_1, C_2, C_3) have been evaluated through a statistical approach based on time history analyses of SDOF models of different types. They account for: the difference between the roof displacement of a MDOF building and the displacement of the equivalent SDOF system, i.e. the amplification of displacement with respect to the spectral one; observed difference in peak displacement response amplitude for nonlinear response as compared with linear response, as observed for buildings with relatively short initial vibration periods (validity limits of the equal-displacement approximation); the effect of hysteresis type on the maximum displacement response; second order effects.

13.4 Extension of NSPs to Plan-Irregular Buildings

The current trends in research concerning the improvement of the NSPs are primarily focused on two main issues: (i) the effects of stiffness degradation and changes in dynamic properties related to progressive damage with the need for an update of inertial forces to be applied as a function of the level of inelasticity; (ii) the contribution of higher modes of vibration, intended to account for the effects of vertical and in-plan irregularity.

Within the first issue a lot of research contributions have been produced in recent years, introducing the adaptive pushover methods (APM). The first procedures, initially applied to concrete frames, have been developed by Reinhorn (1997) and Bracci et al. (1997) that used inelastic storey forces of the previously equilibrated load step to update the lateral load pattern. Afterwards Gupta and Kunnath (2000) proposed a constantly updated load pattern depending on the results of an eigenvalue analysis performed each step, assuming the tangent or secant stiffness related to the deformations of the previous load step.

Concerning the second issue, a large research effort has been devoted to the improvement of the pushover methods so as to consider the contribution of higher modes. This aspect is strictly related to structural irregularity, because irregular structures are generally characterized by significant participating mass ratio of higher modes. The basic NSPs indeed, relate the dynamic behavior of the structure, assumed as a multi-degree of freedom (MDOF) system to an equivalent one-degree of freedom (SDOF) system, which considers the contribution of the main translational mode only.

From the dynamic point of view, a plan irregular building is that for which one or more rotational modes have a significant participating mass ratio. Therefore the dynamic behavior of the structure cannot be defined referring to one translational mode only. The basic NSP approach is not reliable for plan irregular buildings, for which the first translational mode is not representative of a more complex dynamic behavior, that involves both translational and rotational components.

Among the many proposed methods developed in this research field, two main approaches can be recognized: the first one aims to take into account the contribution of more eigenmodes. One of the first attempts has been done by Paret et al. (1996) and it is known as multi-modal pushover (MMP) procedure. Structure's capacity for each mode is then compared with earthquake demand using CSM. Chopra and Goel (2002) developed a similar approach known as modal pushover analysis (MPA), in which several independent pushover analyses are carried out, considering different load patterns associated to different modal shapes. Specifically, in the case of plane irregular structures, the method involves the application of both lateral forces and torque at each level of the building. The results are finally combined by the square-root-of-sum-of-squares (SRSS) rule or the complete-quadratic-combination (CQC) rules. Afterwards, Chopra et al. (2004) proposed the modified modal pushover analysis (MMPA) in which the inelastic response associated to the first mode is combined with the elastic contribution of higher modes. Extensions of this approach with the adaptive load formulation have also been proposed in Shakeri et al. (2012) and Tabatabaei and Saffari (2011).

These methods involve the running of several analyses, one for each modal shape considered and the results are then combined with SRSS or CQC. Moreover the use of quadratic combination rules to sum up the effects of the different modes, like in the linear range, is not strictly correct. Therefore Elnashai (2001) proposed an adaptive pushover procedure able to include, in a single analysis run rather than combining results from more analyses, all features mentioned above. The method uses the combination rules to update the force distribution each step, rather than

combining the effects. However this approach has the disadvantage that the use of quadratic combination rules of modal contributions for the definition of load pattern at each step leads inevitably to positive increments, and hence to a monotonic increase in the load vector.

The inability to reproduce sign change in the applied load patterns has been overcome by the definition of adaptive procedures where the load patterns are based on displacements. This approach, namely displacement-based adaptive pushover (DAP), has been firstly proposed by Antoniou and Pinho (2004) and is based on prescribed adaptive displacement patterns from which the lateral loads are derived. In this way it is possible to capture changes in the sign of lateral loads, even if the displacement increment remains always positive. This approach has been also adopted within the Adaptive Capacity Spectrum Method (ACSM) by Casarotti and Pinho (2007).

On the other hand, the second approach is still based on the first modal shape, but with the awareness that a single target displacement is no longer sufficient to describe the overall dynamic behavior of irregular buildings, because torsional effects entail amplifications and reductions of the displacement demand at the two opposite ends of the storey. In this framework, Tso and Moghadam (1997) and Moghadam and Tso (2000a, b) defined a procedure for monosymmetric structures subjected to one component excitation. The method consists in the evaluation of target displacements in the different resisting elements through elastic response spectrum analysis; consequently the load patterns are determined and several 2D pushover analyses are performed for the different resisting elements. The method has been applied for the evaluation of the seismic progressive collapse of 3-storey RC moment resisting buildings with different levels of plan eccentricity (Karimiyan et al. 2013).

With a similar approach, an extended version of the N2 method has been proposed by Fajfar et al. (2005a, b) for the application to plan irregular building structures. In the extended N2 method, linear elastic analysis is used to define the torsional amplification of lateral displacements to account for the torsional response, on the assumption that the elastic envelope is conservative with respect to the inelastic one.

Another method has been proposed by Bosco et al. (2012), on the bases of previous studies by Ghersi and Rossi (2000), Calderoni et al. (2002) and Ghersi et al. (2007), who introduced the use of “corrective eccentricities” related to the elastic and inelastic parameters that define the torsional behavior of the building. These eccentricities are then used to define the application points of the load vectors, on either sides of the CM so as to obtain an envelope of plan distribution of maximum displacements.

In the following sections, the basic features of the methods addressing to the two main approaches for the seismic assessment of plan irregular building structures will be described, outlining the advantages and drawbacks of each single approach and trying to identify the most promising methods and the issues that are yet to be more deeply investigated.

13.4.1 *Modal Pushover Analysis*

One of the main approaches in the developing of NSPs for the analysis of irregular building structures involves the evaluation of the contribution of more eigenmodes in the analysis. Within this approach, the major contribution has been given by Chopra and Goel (2004) who extended the previously defined MPA to unsymmetric-plan buildings. The fundamentals of the method remained the same of the original version of MPA (Chopra and Goel 2002), based on structural dynamics theory, in which the seismic demand due to individual terms in the modal expansion of the effective earthquake forces is determined by a pushover analysis using the inertia force distribution associated to each single mode. The total seismic demand of the inelastic system is then determined combining the modal demands associated to multiple modes with the SRSS or the CQC rules. Actually this superposition of effects is valid in the linear range, therefore the use of these combination rules represents the first approximation of the method. The second one is the neglecting of coupling among modal coordinates associated with the modes of the corresponding linear system arising from yielding of the system. The original method has been then improved in Goel and Chopra (2004) with three major enhancements: inclusion of P- Δ effects due to gravity loads for all modes (initially it was included only for the first mode); computation of plastic rotations of elements from the total storey drift and not through combination rules; idealization of the pushover curve of n^{th} mode at the peak roof displacement obtained from inelastic SDOF system for the selected ground motion, leading to a reduction of the dependence on the ground motion.

The application of the method to unsymmetrical-plane building structures involves no particular changes in the general approach, except that two lateral forces and a torque are applied at each floor level. The CQC rule is suggested in this case, more suitable for unsymmetric-plan buildings, which may have closely-spaced frequencies of vibration.

Further developments are provided by Reyes and Chopra (2011a, b) who extended the method to 3D eccentric buildings subjected to two components motion and defined the practical modal pushover analysis (PMPA), introducing another simplification: the seismic demands are estimated directly from the elastic design spectrum without performing any NLDA of the modal SDOF systems for each ground motion, thus avoiding the complications of selecting and scaling ground motions.

All the improved versions of the MPA appear to perform rather well, the adopted approximations does not overly affect the results, with respect to those obtained by NLDA, with the exception of cases in which the analyzed structure has close modal periods and strong coupling of the lateral and torsional motions. In this case the individual modal responses attain their peaks almost simultaneously and consequently the CQC modal combination rule become not valid anymore, especially for lightly damped systems. Significant discrepancies with NLDA are also found as the structure experiences high levels of inelasticity with significant degradation in lateral capacity.

13.4.2 *Extended N2 Method*

In recent years, an important step toward the inclusion of torsional effects into pushover analysis has been done by Fajfar et al. (2005a, b) with the definition of an extended version of the N2 method, based on a combination of results of a pushover analysis performed on a 3D model of the structure, that controls the target displacement distribution at the center of mass along the height of the building, with a dynamic modal analysis which controls lateral displacement distribution due to torsional effects. Therefore, modal analysis is used to estimate the displacement amplification due to torsional behavior, that cannot be captured with the standard NSPs.

The displacements obtained by pushover analysis are amplified through a corrective factor, given by the ratio of the normalized displacement obtained by modal analysis and that coming from pushover analysis. The normalized displacement is the displacement in a specific point of the horizontal plane divided by the displacement in the center of mass. Only amplifications of target displacement are considered, whilst reductions in lateral displacements, typical at the stiff edge of the structure, are neglected, with the assumption of a “no-reduction rule”. In this way, it is assumed that the elastic envelope of lateral displacements is conservative with respect to the inelastic one and therefore dynamic modal analysis provides an upper bound of the torsional amplification. Such assumption is supported by findings from several studies demonstrating that displacement amplifications decrease at the flexible side as the structure experiences larger inelasticity, i.e. torsional effects decrease in the inelastic range. This behavior has been observed both for torsionally flexible structures (Fig. 13.3a), i.e. structures characterized by a ratio between the uncoupled torsional frequency and the uncoupled lateral frequency lower than 1, and torsionally stiff structures (Fig. 13.3b), i.e. structures for which the same ratio is larger than 1. On the other hand, the behavior at the stiff side resulted less predictable, influenced by several modes of vibration and by the ground motion in the transverse direction. For torsionally flexible structures, displacement amplification can be found also at the stiff side, although decreasing with plastic deformation. In extreme cases the behavior becomes similar to that of torsionally stiff structures (de-amplification at the stiff side). Typical qualitative behavior of torsionally stiff and flexible structures is represented in Fig. 13.4 which shows the variations of lateral displacement demands at both flexible and stiff sides, with respect to a torsionally balanced structure.

The extended N2 method appears to be a very promising approach aimed at the application of pushover analysis to irregular building structures, because it combines conceptual clarity with simplicity of application. Nevertheless, the basic assumption of the conservativeness of the elastic envelope of lateral displacements with respect to the inelastic one surely needs to be further investigated. De Stefano and Pintucchi (2010) performed a wide parametric analysis on one-storey models and found that the method lose its conservativeness for very torsionally stiff structures, such as shear-walled buildings, for which the inelastic response almost

Fig. 13.3 (a) Torsionally flexible structure; (b) torsionally stiff structure (FEMA 274 (1997))

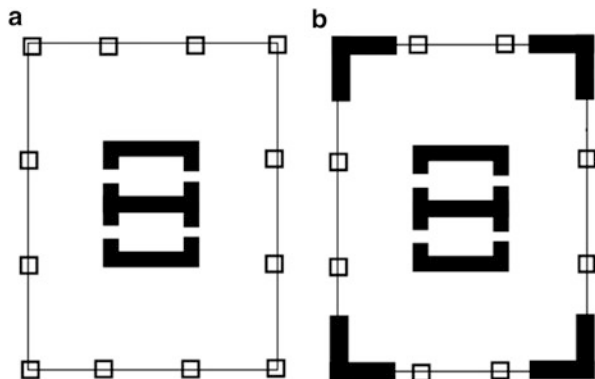
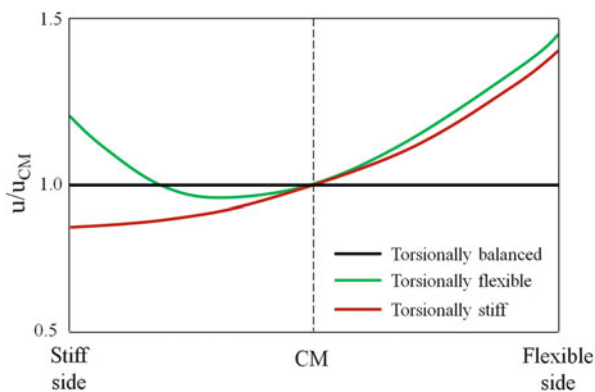


Fig. 13.4 Qualitative comparison of plan-wise shapes of lateral displacement envelopes



always exceeds the elastic one. This is mainly due to the development of a strength eccentricity related to the failure of one or more components of the structural system, leading to a significant increase of the inelastic torsional effects.

Some preliminary parametric boundaries to applicability of the extended N2 method have been defined in terms of stiffness and strength radii of gyration and of behaviour factor q . The procedure resulted effective for values of d_s and d_r lower than 1.3, characterizing most building framed structures, and q values higher than 2.

Other authors tested the procedure on sample multi-storey buildings structures. Bhatt and Bento (2012) applied the extended N2 procedure, together with the CSM, the MPA and the ACSM to two case studies of real existing plan irregular structures. They found that the extended N2 method was the most suitable method, among all the evaluated NSPs, because it was the only one to present always conservative results with respect to average time-history analysis results, both at the flexible edge (S1 in Fig. 13.5) and stiff edge (S23 in Fig. 13.5). Bosco et al. (2013) made a comparative evaluation of the N2, the extended N2 and the corrective eccentricities methods on a set of asymmetric single-storey systems and a set of 12 multi-storey buildings. Authors defined the extended N2 method as the easier to apply and the one always giving conservative results, though sometimes overly conservative.

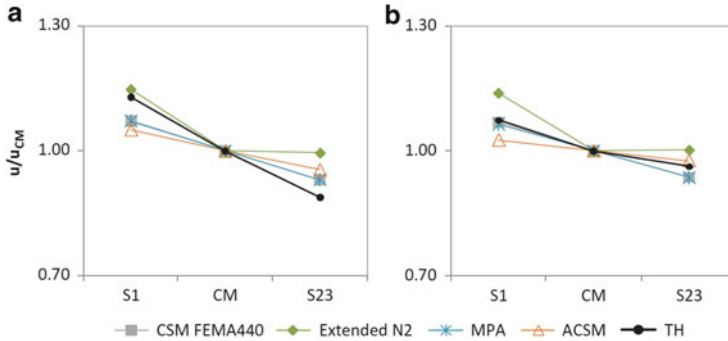


Fig. 13.5 Normalized top displacements obtained with several NSPs and time-history analysis (TH): (a) seismic intensity of 0.2 g; (b) seismic intensity of 0.4 g (Bhatt and Bento 2012)

Recently the extended N2 method has been improved to take into account higher modes effects both in plan and elevation (Kreslin and Fajfar 2011, 2012) and has been applied, as an alternative to incremental dynamic analysis, to determine the relationship between seismic demand and seismic intensity for different values of the seismic intensity measure. In this case the method has been called Incremental N2 method (Dolsek and Fajfar 2004, 2007).

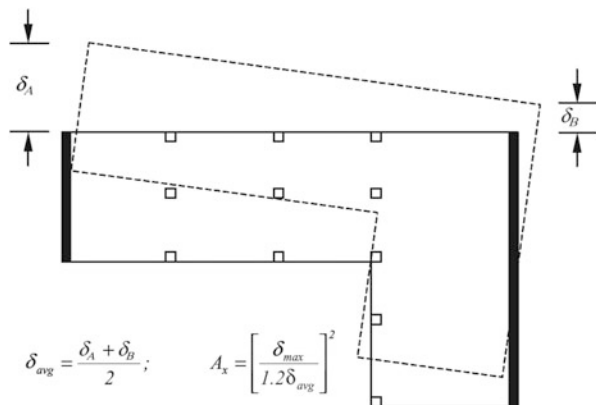
13.4.3 Specifications of Major Seismic Codes on Applicability of NSPs to Irregular Buildings

Despite the large efforts of researchers aimed at a better understanding of the seismic behavior of irregular building structures and at the enhancement of the current NSPs, most regulatory bodies appear to have not yet translated the achieved research developments into seismic codes.

Even the criteria for the definition of plan and vertical irregularity are still not exhaustive, as underlined by a statement in EC8-1 (2004), where it is asserted that “it shall be verified that the assumed regularity of the building structure is not impaired by other characteristics, not included in these criteria”. However, American codes provides for a more accurate and analytical definition of torsional irregularity, based on results of numerical analysis and not only on geometrical and qualitative evaluations on the structural features of the building. ASCE7-10 (2010), indeed, defines that a torsional irregularity exists when the ratio of the maximum storey drift at one end of the structure δ_{max} ($\delta_{max} = \max\{\delta_A, \delta_B\}$) and the average of the two storey drifts at the two ends A and B of the structure δ_{avg} is larger than 1.2 (Fig. 13.6).

For the purpose of this paper, in the following only specifications related to applicability of NSPs to irregular buildings are summarized, based on the current major seismic American and European codes. The basic American reference codes

Fig. 13.6 Definition of torsional irregularity according to ASCE7-10 (2010)



are International Building Code (IBC 2012), ASCE 7-10 (2010) for general building structures and International Existing Building Code (IEBC 2012), ASCE 31-03 (2003) (Seismic Evaluation of Existing Buildings), ASCE 41-06 (2006) (Seismic Rehabilitation of Existing Buildings), recently joined and implemented in the ASCE 41-13 (2013) (Evaluation and Retrofit of Existing Buildings). Basically ASCE 41-13 (2013) retains the three-tiered approach found in ASCE 31-03 (2003), while relying on the technical provisions in ASCE 41-06 (2006) as the basis for all the analytical procedures.

Concerning European codes, the Eurocode 8 part 1 (EC8-1 2004), containing general rules for seismic design of buildings and Eurocode 8 part 3 (EC8-3 2005), concerning seismic assessment and retrofit of buildings, are considered.

The IBC mostly recalls ASCE7-10 (2010) for earthquake design. ASCE 7-10 (2010) does not require any form of nonlinear analysis for traditional buildings that do not incorporate seismic isolation or passive energy systems. The permitted analytical procedures are: equivalent lateral force analysis, modal response spectrum analysis and seismic response history procedures. Therefore it does not contain specific prescriptions on the use of NSPs. The only limitation on the choice of the analysis type with reference to torsional irregularity, is that equivalent lateral force analysis is not allowed for torsionally irregular structures.

American seismic codes for existing buildings (IEBC, ASCE 31-03 (2003), ASCE 41-06 (2006) and ASCE 41-13 (2013)) also define limitations at the use of linear analyses based on the existence of structural irregularities and to excessive values of DCR (Demand-Capacity Ratio) evaluated through linear static or dynamic analysis. If one or more structural components are characterized by DCR higher than 2 and any kind of structural irregularity exists (in-plane and out-of-plane discontinuities, weak storey, torsional strength/stiffness irregularity), then linear procedures are not applicable and shall not be used. More restrictive criteria are also defined for the application of linear static analysis. According to IEBC, NSPs are the fundamental tool to perform a Tier 3 analysis, i.e. the most

advanced phase of an existing building evaluation process ASCE 31-03 (2003), necessary when the previously performed two phases (Tier 1 and 2) have evidenced potential deficiencies of the building.

NSPs are considered acceptable in most cases, but should be used in conjunction with a linear dynamic procedure, if higher modes effects are significant. This condition should be verified performing two modal response spectrum analyses: one considering sufficient modes to produce 90 % mass participation, another considering only the first mode participation. If the shear in any storey resulting from the first analysis exceeds 130 % of the corresponding storey shear considering only the first mode response, higher modes effects have to be considered significant.

The combined use of pushover analysis and response spectrum analysis appears as a precursor to the basic idea in the development of the extended N2 method. Moreover FEMA 273 (1997) prescribes that the effects of torsion shall not be used to reduce force and deformation demands on components and elements, somehow recalling the no-reduction rule of the extended N2 method. Notwithstanding the conceptual connections with the N2 method, most of the current American seismic codes and guidelines (IBC 2012, ASCE 41-13 (2013), ATC 40 (1996) and FEMA 440 (2005)) refer to the CSM and to the DCM as analysis procedures for the evaluation of seismic capacity of building structures.

Even in EC8-3 (2005) the prescription to take into account higher modes effects is defined for buildings with a fundamental period higher than $2s$ or $4T_c$. In this case the code requires to perform a NLDA or “special versions” of NSPs. Nevertheless the code does not provide any suggestion concerning which kind of upgraded NSPs should be used and refer to national codes for more specific provisions.

EC8-1 (2004) provides for the application of the N2 method, although it recognizes the absence of a full suitability for irregular building structures. Nevertheless, no restriction to the use of this method for irregular structures is defined. EC8-1 (2004) declares that the conventional procedure “may significantly underestimate deformations at the stiff/strong side of a torsionally flexible structure, For such structures, displacements at the stiff/strong side shall be increased, compared to those in the corresponding torsionally balanced structure”. To do that, EC8-1 (2004) implicitly refers to the extended N2 method, as it prescribes to evaluate the amplification factor to be applied to the displacements of the stiff/strong side through an elastic modal analysis of the spatial model. Nevertheless, no specific prescriptions are provided to account for displacement amplifications at the flexible side, observed for both torsionally stiff and flexible structures. Therefore, the extended N2 method is only partially adopted, highlighting how EC8 provisions for the application of pushover analysis to irregular building structures are still lacking and not satisfactory.

Another weak point of the code is that it allows the use of two separate planar models even for plan irregular structures that comply with some other prescriptions: well-distribution and sufficient rigidity of cladding and partitions, building height lower than 10 m, diaphragm behavior of the floors, centres of lateral stiffness and mass approximately on a vertical line and adequate torsional stiffness. The assumption of this simplification has been questioned by Athanatopoulou and Avrimidis

(2008) that demonstrated how the use of two planar models for the nonlinear static analysis of a sample plan irregular building complying the aforementioned conditions, led to unconservative results with respect to NLDA.

Even ASCE 41-13 (2013) in some cases allows for the use of NSPs on two-dimensional models, when the building has rigid diaphragms and the displacement multiplier η , i.e. the ratio of the maximum displacement at any point on the floor diaphragm to the average displacement, does not exceeds 1.5. When NSP is applied to two-dimensional models, the target displacement shall be amplified by the maximum value of η calculated for the building.

13.5 Conclusions

Classical NSPs for the evaluation of seismic vulnerability of buildings have been originally defined for symmetric, regular structures and it is demonstrated that torsional behavior calls into question their validity for the seismic evaluation of torsionally sensitive structures. Therefore there is the urgent need for an update of such methods aimed at a reliable application to irregular building structures. Two major approaches have been identified concerning the improvement of NSPs: the first one is based on the inclusion of the contribution of higher modes in the analysis and has led, among others, to the development of MMP and MPA procedures; the second one focuses on the need to account for amplification of displacement demand, through corrective factors to be applied to the target displacement. Under this perspective, the most promising developed procedure is the extended N2 method, that combines in a synergic way the results coming from pushover analysis and response spectrum modal analysis. The procedure appears to be the most effective in the evaluation of displacement amplification due to torsional effects while maintaining simplicity and clarity for practical applications. The main assumption is that the elastic displacement pattern is conservative with respect to the inelastic one and this aspect need further investigations, because it cannot be the case for very torsionally stiff structures and for low ductility values.

Despite the large efforts of researchers aimed at the improvement of the classical NSPs for a reliable application to irregular building structures, most regulatory bodies appear to have not yet transposed the achieved developments into major seismic codes. Both European and American codes are still in need of improvement regarding specific prescriptions concerning the seismic analysis of irregular structures. There is the awareness of a partial unsuitability of classical NSPs, some improved solutions have been proposed by researchers, but a comprehensive and always suitable set of rules to extend NSPs to plan irregular buildings has not yet been established.

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