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Multilevel performance-based procedure applied to moderate seismic zones in Europe

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Abstract. The Performance-based Earthquake Engineering (PBEE) concept implies the definition of multiple target performance levels of damage which are expected to be achieved (or not exceeded), when the structure is subjected to earthquake ground motion of specified intensity. These levels are associates to different return period (RP) of earthquakes and structural behaviors quantified with adopted factors or indexes of control. In this work an 8-level PBEE study is carried out, finding different curves for control index or Engineering Demand Parameters (EDP) of levels that assess the structural behavior. The results and the curves for each index of control allow to deduce the structural behavior at an a priori unspecified RP. A general methodology is proposed that takes into account a possible optimization process in the PBEE field. Finally, an application to 8-level seismic performance assessment to structure in a Spanish seismic zone permits deducing that its behavior is deficient for high seismic levels (RP > 475 years). The application of the methodology to a low-to-moderate seismic zone case proves to be a good tool of structural seismic design, applying a more sophisticated although simple PBEE formulation.

Keywords: PBEE, seismic assessment, damage index, reinforced concrete structures, non-linear analysis, low-to-moderate seismic zone

1. Introduction

In the structural design it is always important to take into account the structural redundancy and the continuity (and minimum values) in terms of stiffness, strength and ductility or ultimate non-linear dissipation.

Each one of these 3 factors presents problems for a complete structural evaluation. The stiffness is very influenced by the type and behavior of the non-structural elements. At low earthquake levels it is possible to suppose that the non-structural elements condition the structural stiffness. The strength greatly depends on the geometry, the shape sectional characterization and the material 'really' utilized. The ductile capacity is represented by an ultimate displacement ductility ratio (μ) according to its relative capacity to dissipate energy after yielding at a high earthquake level (Qiang X *et al*. 2008). However, the assessment of μ is complicated for multiple

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degree of freedom (MDOF) structures. Limiting of building height, irregularities, damping, non-structural elements, and other sources, are causes of bias. Nowadays, the correct assessment of the energy dissipation for MDOF structures is becoming more and more important.

If an actual Performance-Based Earthquake Engineering (PBEE) calculation is applied, a more exact study must be carried out on this triad of implicated factors: stiffness, strength and inelastic dissipation (ductility). The acknowledged PBEE method establishes some levels of seismic study from the adopted return period (*RP*) or return times of earthquakes. Then, it is necessary to find a seismic representative load for every level. Next, by means of some parameters of analysis (control index) the structural behavior is verified against adopted maximum thresholds. These control indexes are appropriately termed Engineering Demand Parameters (*EDP*) and they have relation to damage measure (*DM*).

This method is starting to be well-known in high-seismicity zones, such as USA and Japan, but it is not yet appropriately established in low-to-moderate seismic zones. Yet it is important to bear in mind that the structural behavior and the damage levels exhibited by buildings located in these zones will be similar to major seismicity zones. It must be had into account that a PBEE application in low-to-moderate and high seismicity zones must leads to same safety levels. This point is sometimes forgotten, which results in a minimization of risk assessment in low-to-moderate seismicity zones.

Commonly, the design procedure starts with the specification of some desired performance objectives for the structural system, given the hazard environment in which it is to be built , and then provides a direct rational path by which the structure may be designed to attain these goals (Bozorgnia and Bertero 2004). When two levels are used, the two explicit performance objectives have to guarantee: (i) to protect life under a rare or exceptional seismic action, by preventing the collapse of the structure or parts thereof and maintaining structural integrity and residual load capacity, and (ii) to reduce property loss due to a frequent event, by limiting structural and non-structural damage. The no-local-collapse performance objective is achieved by dimensioning and detailing structural elements for a combination of strength and ductility that provides a safety factor against a loss of gravity load capacity and lateral load resistance (Fardis 2002). On the other hand, damage limitation performance objective is achieved by limiting deformations (lateral displacements) of the system to levels acceptable for the integrity of all its parts including non-structural ones (Bozorgnia and Bertero 2004).

Advances correlated to the PBEE method are indisputable. The fact of merely applying it in a simplified form, i.e. only with two levels, will always be much better than a method where only one level of study is considered (habitually with *RP* = 475-500 years). On the other hand, the most common type of structural analysis (spectral modal analysis) utilized in the majority of standards, is beginning to be questioned due to the means and knowledge that it is available at present. It seems logical to apply a more realistic method such as non-linear dynamic analysis (*NLD*) and besides, if a large amount of records is applied, it is necessary to complete a statistical application to results. From now on, this appropriate global method of analysis will be named Performance-Based Earthquake Engineering Statistical Non-linear Dynamic (*PBEE-SNLD*).

From FEMA (FEMA 451-B; NEHRP 2007; FEMA 273. NEHRP 1997; FEMA 356-2000), the PBEE basically considers four levels of study to different *RP*. The proposed levels are: Operational or Serviceability (O), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). These levels can be defined as: 50/50/72, 20/50/225, 10/50/475 and 2/50/2475. Here xx/xx/xx denotes percentage exceedance/years/return period, respectively.

In Europe, EC8 (Eurocode 8. EN 1998-1:2004) establishes two levels: collapse design level and damage limitation level. EC8 specifies spectral shapes, but does not specify hazard levels for the two performance objectives, because the choice of the level of safety and serviceability is left to the discretion of member countries. The recommendation is to use a 10/50/475 hazard for collapse prevention and a 50/50/72 hazard for damage limitation (Bozorgnia and Bertero 2004). However, it is known that the European zone does not have a uniform seismicity. Although, an important challenge is to define a design earthquake that results in a uniform seismic safety margin for these very different seismic zones.

In Europe, actually, the performance level is not new since some authors suppose it assimilable to Limit State design, largely utilized for structural calculations over the last 25 years. The Limit State Method (LS) defines an Ultimate Limit State (ULS) that assesses the safety provided for people and/or structure, and a Serviceability Limit State (SLS), on use or operation level that assesses the damage to property. Then, the seismic tendency is to follow the inertia to consider two levels (something very debatable). It is necessary to bear in mind that conceptually a great probabilistic difference can exist for the types of possible loads, for instance: wind, snow and seismic loads in a defined site with different *RP*. The combination of the applied loads should be coherent in each *RP* or level. On the other hand, the risk that represents an unpredictable dynamic seismic load may be greater than the others loads and a static approximation cannot be correct.

Some authors (Bommer and Pinho 2005) consider it insufficient the study of the two levels established by EC8. The observations in recent earthquakes (in Spain and Italy) seem to clearly justify this opinion. Among two extreme levels generally accepted, 50 and 2475 years of return period; several levels would mean that the structure would be examined with different results in relation to the control index utilized in each one. Here, a return period of 2475 years may be interpreted as Maximum Credible Earthquake (MCE) or collapse ground motion. It would be interesting to examine if the best-suited additional levels are on short *RP* or long *RP*, according to if they belong to low or high seismicity or if this is indifferent. Each examined level will depend, in a different way, on the characteristics of structural stiffness, strength and ductility (or non-linear dissipation). It is possible to propose very many levels and find a sort of ‘continuity’ in the results of the control index adopted. This would permit having a complete field of results/structural responses for all the possible levels that the structure may have, some found directly and others found by interpolation between acknowledged levels.

All codes have in common the description of earthquake intensity in terms of discrete hazard levels associated with satisfactory performance at the performance levels defined in the guidelines. Besides, the return period dependence of the earthquake intensity is quantified through a probabilistic hazard analysis, but in general, the performance assessment is assumed as deterministic. Work must be carried out in this way.

The addition of more seismic design levels is equivalent to put more constraints on the structural performance. The selection of these additional design earthquakes and corresponding performance limits, however, needs to be carefully done to ensure internal consistency. Also, since the probability is prescribed on the seismic hazard uncertainties in the structural capacity and demand has not been considered, the reliability of the structure to withstand a specific limit state is still unknown. In a reliability-based design, the limit-state reliability analysis is reversed. In other words, the problem lies in the determination of the required structural capacity for given target reliabilities to withstand a set of structural limit states. Such design procedures have been recently developed that represent a large step forward in accounting for uncertainty in demand and capacity in codes and standards.

In doing so, the authors recognize that a more reliable procedure has to be developed according to the present state of the art of seismic engineering, but that they should be comprehensible and simple. Therefore, as Bertero and Bertero (2002) correctly point out, "With the amount of specific software, spreadsheets, and mathematical packages available today, simplicity should be redefined. A numerical procedure is not simpler because an equation has fewer terms or some important parameter is ignored. A numerical procedure is simpler when it is easily understood and when the designer can go from the performance objectives to the design values in an explicit and transparent way".

In this work a study of PBEE analysis with many levels, a statistical evaluation of results, and finally a possible coherent optimization procedure are proposed. A multilevel seismic performance assessment is applied (on a complete multilevel seismic performance design see the recent papers by Palermo *et al*. (2014)).

The methodology is applied in a simple and transparent way and here the multilevel analysis is understood as more complete than the frequently adopted 4-levels. From a calculated structure with Spanish seismic code, an example with 8-levels seismic performance assessment is carried out. Then, a graphic description for each particular control index adopted will permit to show it for the complete behavior of a structure in a wider range of return periods and possible earthquakes. Besides, it is important to point out that this procedure would permit to identify the most restrictive level and to carry out a structural optimization from the stiffness, strength and non-linear dissipation values adopted for the structure.

2. Analysis of multilevel PBEE

2.1 General methodology

A complete and general PBEE methodology would always be composed of three parts:

1. Preliminary design/analysis
2. Dynamics analyses (PBEE-SNLD)
3. Structural performance optimization

The first block is the start of the calculation that permits the designer to adopt a structure and sectional dimensions with certain reliability. The calculation is carried out following a static analysis. Prescriptions of seismic codes are used. The second one is the PBEE-SNLD calculation as was defined before. A set of real and possible records must be adopted and adjusted to representative design spectra for each seismic level. Finally, the third block is the process to optimize the previous results in an iterative way.

To a structural SNLD calculation type, it is important to establish the levels of interest for a structure. Evidently, a larger amount of levels on disposal will increase the complexity of the calculus. However it will assess the structural behavior in a more complete way. As previously discussed, conceptually the minimum number of levels to be considered in PBEE application is two and at present, the consideration of four levels has become popular. The optimal quantity is the one, which describes the structural behavior with enough reliability for all the possible earthquakes in a determined site. Basically, the complete development is established identifying three parts: first, from a determined/adopted return period of earthquake to find the representative value of the seismic action, second, the characterization, modeling and the complete process of calculation of the structure, and third, the description of analysis and evaluation of results.

The proposed PBEE-SNLD methodology is summarized in the flowchart of Fig. 1.

The enumerated topics in Fig. 1 will now be discussed:

1. Preliminary analysis. This is a static calculus and performance evaluation for a combination of gravity and live loads, and seismic static loads. From here, the initial structural and sectional design is done. It is more useful for guiding the designer in relation to structural dimensions, sectional sizes, etc. All is obtained from a static calculation point of view such as that recommended in current codes. The preliminary seismic action is taken into account from a spectral modal analysis for one or two seismic referential levels, i.e. 50/50/72 and 10/50/475. The first one corresponds to a damage limit level and the second one to a life safety level. Here, two referential response spectra must be used. Currently some codes still give a unique designing level that is frequently 10/50/475.

2. Hazard Level. At this stage multiple seismic levels are adopted in dependence of the importance of the structure, which could demand a more precise study and with more levels. In the majority of codes two or three structural importance types are adopted. The adopted seismic levels can be from a minimum of two to a prefixed maximum. To adopt a maximum of eight levels should be considered enough. The levels must include earthquakes with *RP* from 50 years to 2475 years.

3. Characterization level. This is the characterization of one level of analysis. The level must be characterized from an acknowledged and tested attenuation regression equation in the region and site of analysis. From the preliminary analysis, the dynamic elastic characterization of the structure is found. Then, from a complete database of seismic records, all the records with a criterion of spectral adjust may be selected. This criterion is a matching of frequencies distribution in a predefined range respect to the reference spectrum. Also, another criterion that demonstrates being sufficient, is the adjustment in a value larger than S_a (T_1) which takes into account the stiffness degradation at high levels. Here, it is important that a suitable and optimal intensity measure (*IM*) be adopted. Good referential works exist in this subject as: Giovenale *et al*. (2004); Iervolino *et al*. (2005); Trombetti *et al*. (2008) and Silvestri *et al*. (2009).

4. Seismic analysis. Analysis is done with a non-linear dynamic analysis. All chosen records are used. At this stage, the structural modelling has great importance and specially the characterization of plastic hinges. In determined situations, the structural modelling has demonstrated that it can be more important than the type of chosen method for calculation. Ideally, the properties of plastic hinges should be obtained from a reliable laboratory test. Then, the results must be treated in a statistical way.

5. Seismic assessment. At this stage the values obtained from the statistical results are verified to the values of control (*EDP*) adopted for the seismic level of study. The control indexes are more appropriate in one or another level, according to the structural properties of stiffness, strength or ductility (non-linear dissipation) to be assessed. The most used control indexes are:

- interstory drift ratio (Δ_{di}),
- roof drift ratio (Δ_{roof}),
- local or global ductility (μ),
- damage index (*DI*),
- normalized hysteretic dissipated energy (*NHE*),
- equivalent hysteretic velocity (V_H),

- local maximum plastic hinge rotation (θ_{pmax}),
- local cumulative plastic hinge rotation (θ_{pcum}), among others.

The most used displacement damage indexes are Δ_{di} and Δ_{roof}, but it is important to take into account that high damage with low displacement and not much damage with high displacement may exist, according to the structural stiffness and strength disposes. Therefore, these displacement indexes must be combined with others that take into account a non-linear dissipation, basically like hysteretic energy. Moreover, Δ_{di} can also be a good damage indicator on high levels. In this levels can put in evidence a dangerous seismic behavior of soft storey when an interstory drift reaches values much larger than the others.

These parameters at each hazard level are then checked against their respective defined threshold values that result from a combination of seismic codes criteria, experimental study and finally, engineering judgment with the designer criterion.

6. Seismic re-design. From the evaluation at each level of study, it can be seen if the seismic design is adequate or not. If it does not verify the results, a re-design is necessary and therefore a complete new calculation (including the preliminary analysis with static loads) must be performed. After an iterative process of checking of the studied level, all previous levels must be re-calculated with the last sectional values of verification found.

7. Finally, an optimization process must be carried out. The process consists in detecting the most restrictive level of analysis and to adjust the dimensions of the section that verify this level. Next, proceeding to the re-calculation of all the other levels, to check that no other level that the most restrictive does happen All real structural loads must be considered (it is necessary to re-adapt the dead loads). Note that only an optimized seismic design for future seismic risk is obtained. Other important optimization sources such as capital investment, structural material, complexity of degree of design that influences the set-up of the structure, evaluation of the costs of damage, retrofitting, etc., are not considered here.

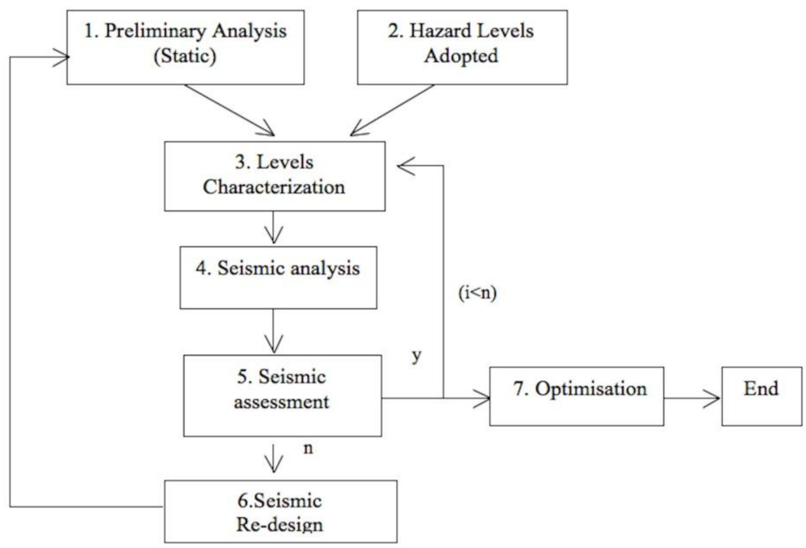

Fig. 1 Framework for multilevel performance-based earthquake engineering design

2.2 Levels, selection and number of records

Two aspects that need special attention and reasoned study in PBEE-SNLD analysis are the adopted levels and the selection of records for them. The different levels that are adopted will be based on *RP* terms in order to be able to establish continuity in all possible earthquakes for a structure. However, each level is associated to the definition of its seismic action and the evaluation of the possible structural behavior. Some levels are imposed by codes but other ones can be defined by the designer and the owner. The selected levels at different *RP* must be represented, in general, by sufficiently reliable spectra for the zone and site (including the important local effects of the ground). Using a Poisson distribution, a proposal for eight levels is indicated in Table 1.

In Table 1 it is possible to see that two groups of levels in global way are defined: 'service' levels and 'ultimate' levels. In a more simplified manner, the first group act on the service conditions of the structure and the second one bring the structural damage into play. The equivalent FEMA levels as reference are indicated in Table 1: Operational (OP), Life Safety (LS) and Collapse Prevention (CP). Also, the *RP* and exceedance period of 50 years are indicated. These eight levels give a complete range of the structural behavior for any expected earthquake.

For the spectral level definition, several models adapted to different parts of the world exist (Boore *et al*. 1997; Abrahamson and Silva 1997; Ambraseys *et al*. 1996; Campbell and Bozorgnia 2007; Gasparini *et al*. 2011) in terms of magnitude (M), epicentral (or hypocentral) distance (Δ) and soil conditions (s). In Europe a classical model is the one proposed by Ambraseys *et al*. (1996). More recently several models for different conditions have been developed (Douglas *et al*. 2010). However, the differences between them (in one site) cannot be so decisive as a good posterior criterion of selection method of records. Once the spectral referential curves have been found, from the attenuation function, seismic records must be searched for a database which will be compatible with spectra in the periods of interest.

From ASCE-07 (ASCE 7-05 2006) and EC8, it is necessary that records are compatible in a range of established periods and the medium spectrum of all the records used is compared to the referential. Complementary investigations have been done in this issue (Trombetti *et al*. 2007; Trombetti *et al*. 2011). This process can be complicate and an approximate way can be adopted. Records can be scaled in S_a (T) in T_1 value (T_1 is fundamental period) or better still, in a bigger value than T_1, e.g. $1.05T_1$ - $1.20T_1$ (Catalan *et al*. 2010). This simplification could be accepted if many records are utilized. On the other hand, uncertainty from dynamic structural characterization should be carefully assessed (Lepidi *et al*. 2009; Foti *et al*. 2011; Foti *et al*. 2012a, b, c; Foti 2013; Foti *et al*. 2014; Foti 2014; Foti 2015a,b; Diaferio *et al*. 2007; Diaferio *et al*. 2014; Diaferio *et al*. 2015).

Table 1 Proposed definition of multilevel PBEE-SNLD analysis

Level	Level name	Key	RP(years)	Exceed./years	Typical behavior
1	Service 1	S_1	50	63/50	Service
2	Service 2 [OP]	S_2	72	50/50	Service
3	Medium Service 1	MS_1	175	25/50	Service
4	Medium Service 2	MS_2	308	15/50	Service
5	Life Safety 1 [LS]	LS_1	475	10/50	Ultimate
6	Life Safety 2	LS_2	744	6.50/50	Ultimate
7	Collapse 1	C_1	975	5/50	Ultimate
8	Collapse 2 [CP]	C_2	2475	2/50	Ultimate

Records are chosen with low values of ε (Baker and Cornell 2005) in the period of scaling in order to avoid important spectral dispersions when scaling is done. Here ε is defined as dispersion value of S_a on T of reference between spectrum of adopted record and referential attenuation function.

Another important aspect to take into account is the amount of records to choose. Evidently, the greater the quantity of possible records is, the more exact the procedure is from a statistical point of view. However, if the selection of records process is not reliable, even with a great statistical precision, the results will not be better. Being that from EC8 the minimal recommended value would be 7 pairs records (for plane analysis) in order that the media may be considered, the number of events that permits minimizing the media error on normal and lognormal distributions must be greater than 30 (Benjamin and Cornell 1970). A common problem in low-to-moderate seismic zones is the reduced amount of records available for high levels of earthquake magnitude. These levels can define the capability of the structure to avoid risks of human losses and structural collapse (from LS to CP), therefore for these levels, the minimum number of records must be respected, perhaps more than in other levels.

3. Application to a low-to-moderate seismic zone

3.1 Spanish seismic zone case study. Structural characterization

As a case study, an application to the procedure which follows an 8-levels seismic performance assessment will be carried out. One RC frame structure of eight stories was designed according to the current Spanish codes (EHE 2008; NCSE-02 2002). The structure was assumed to be built in the southern part of Spain, which is a global low-to-moderate seismicity zone. One numerical model that represented this frame was developed using IDARC-2D code (Reinhorn *et al*. 2006). The frame analyzed is representative of the structures in the long (1.2-1.5 s) period range. The frame belongs to a building that is regular and symmetric in plan. A modal analysis has been carried out and a very small difference between the structural period for 3D and 2D configuration was observed. Hence, a 2D frame type model was utilized. The columns of each story have the same reinforcement along their height, while in the beams it is different for each end. The structure is shown in Fig. 2.

The properties of each member ends are summarized in Table 2. In this table, h is the cross-sectional height; b is the cross-sectional width; ρ_{tot} is the total longitudinal reinforcement ratio in the columns; φ_s is the diameter of the stirrups; ρ and ρ' are the reinforcement ratios of the longitudinal reinforcement in the bottom and upper sides, respectively, of the beam; and s_s is the spacing of the stirrups.

Table 3 show the dynamic 2D properties of the frame.

The building is assumed to be located in Granada (Spain) on a soil between type II and type III (medium to soft soil). The structure was through calculated with the Spanish concrete code EHE and Spanish seismic code NCSE-02. In this code a Uniform Hazard Spectrum as elastic spectrum with 5% of damping is applied. Only one level (as the code indicates) has been used for the analysis. This design spectrum is defined for 10% in 50 years —10/50/500 ≅ 10/50/475— which corresponds to a Life Safety level of the seismic demand in the framework of the PBEE —i.e. PBEE LS. The Spanish seismic code allows ductility-based reduction factors ranging from 1 (no ductility) to 4 (very high ductility); in this study a realistic value of 3 was adopted.

Table 2 Sectional characteristics of 8-storey frame. a) column properties; b) beam properties

a) Column properties

Type	h (mm)	B (mm)	ρ_{tot}	φ_s (mm)	s_s (mm)
c01	450	450	0.015	8	150
c02	450	450	0.0165	8	150
c03	400	400	0.019	8	150
c04	400	400	0.020	8	150
c05	350	350	0.025	8	150
c06	350	350	0.014	8	150
c07	300	300	0.029	8	150
c08	300	300	0.015	8	150
c09	250	250	0.027	8	150
c10	250	250	0.022	8	150

a) Beam properties

Type	h (mm)	b (mm)	ρ	ρ'	φ_s (mm)	s_s (mm)
b01	400	300	0.005	0.014	8	80
b02	400	300	0.005	0.0136	8	80
b03	400	300	0.005	0.015	8	80
b04	400	300	0.005	0.014	8	80
b05	400	300	0.005	0.014	8	80
b06	400	300	0.005	0.014	8	80
b07	400	300	0.005	0.014	8	80
b08	400	300	0.005	0.0128	8	80
b09	400	300	0.005	0.0117	8	80
b10	400	300	0.005	0.0128	8	80
b11	400	300	0.005	0.010	8	80
b12	400	300	0.005	0.0128	8	80
b13	400	300	0.005	0.005	8	80
b14	400	300	0.005	0.0128	8	80

Table 3 Dynamic properties of the frame

Mode	T(s)	Relative modal weight (%)
1	1.34	76.56
2	0.476	14.85
3	0.287	3.92
4	0.195	2.15
5	0.126	1.29
6	0.099	1.01

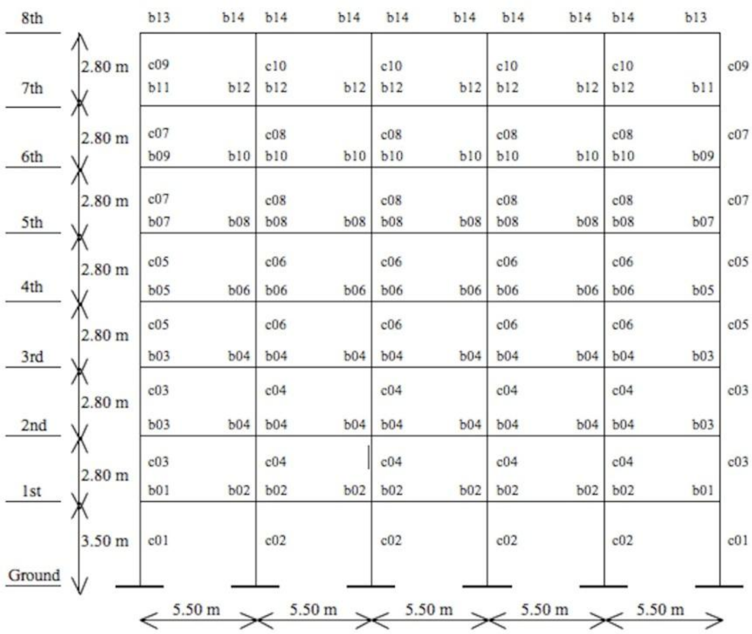

Fig. 2 Elevation of the eight-story frame

The stresses induced by the seismic actions were determined by a spectral modal analysis, and combined with those caused by the gravity loads and wind, as established by the Spanish concrete code. In determining the dimensions and reinforcement of the members, a concrete compressive strength f_{ck} = 30MPa and yield strength for the reinforcing steel f_{yk} = 400MPa were used. According to the Spanish concrete code, f_{ck} and f_{yk} were reduced by 1.5 and 1.15, respectively. As for the modeling of the cyclic behavior of the members, the hysteretic rule that controlled the degradation of the plastic hinges followed the Kunnath-Reinhord model (Reinhorn *et al*. 2006). In this preliminary design, the fundamental elastic period T_1 is 1.34 s. It is known that it should be adopted one bin of records and matching its medium spectrum to referential on in a range of T. Here, it will be applied a great quantity of records scaling in an only period, and although the lengthening of T produces by damage is different for each levels (higher in more demanding levels), a mean value of 10% is adopted for all of them. Then, a referential scaling period with 10% of lengthening will be considered (T=1.474 s).

3.2 Characterization of the 8-levels PBEE

The design acceleration that NCSE-02 gives for Granada (Granada), with C=1.40 (soil coefficient between type II and type III) is a_c=0.245 g, with g the acceleration of gravity. This value corresponds to 10/50/475 level or PBEE LS level. This PGA's value is the greatest in Spain. Then, different relations of NCSE-02 may be used to find: $a_{ci} = f(RP_i)$, $I_i = f(a_{ci})$ and $M_i = f(I_i)$. Here, I is the intensity and M is the magnitude. The subscript i denotes the correspondent level studied. In a simplified way, the type of magnitude and intensity are not

Table 4 Characterization of the eight levels analyzed for the site under study (Granada)

Level	A_c(g)	I	M	Δ (km)	Soil Type
S_1	0.10	7.45	5.45	15	Medium/soft
S_2	0.14	7.82	5.66	15	Medium/soft
MS_1	0.17	8.12	5.82	15	Medium/soft
MS_2	0.20	8.41	5.98	15	Medium/soft
LS_1	0.245*	8.68	6.13	15	Medium/soft
LS_2	0.28	8.89	6.25	15	Medium/soft
C_1	0.31	9.03	6.33	15	Medium/soft
C_2	0.44	9.53	6.70	15	Medium/soft

* NCSE-02 value

specified in formulas. Due to the coherence of formulas, *I* value is used as a decimal value and not as an integer (in Roman) as usual. These relations are:

$$a_{ci} = a_{c(475)} \left(\frac{RP_i}{475} \right)^{0.37} \text{, with Poisson distribution} \quad (1)$$

$$\log a_c = 0.301030 \cdot I - 0.2321 \text{, with } a_c \text{ in gals (from NCSE-02)} \quad (2)$$

$$M = 0.552 \cdot I + 1.34 \text{ (from NCSE-02)} \quad (3)$$

$$P_i = 1 - e^{\left(-T/RP_i\right)} \text{, with } T\text{=50 years (structural service life, from FEMA 356 and EHE)} \quad (4)$$

Values found for the eight levels are shown in Table 4.

3.3 Ground motion definition. Searching and scaling of records

For the spectral definition of the seismic action, the acknowledged function of attenuation of Ambraseys *et al*. (1996) is utilized. This model has demonstrated to be sufficiently reliable for south-west Europe. Here, an epicenter distance Δ =15 km is adopted to minimize the influence of directivity from near field sources, local ground effect and important increase of the vertical components of the seismic action (Table 4). The found spectra are shown in Fig. 3. Next, from all the spectra the values S_a/g are found at the referential period T = 1.474 s (Fig. 3 and Table 5).

With this S_a/g value found for each level a thorough searching of records was carried out in a seismic record database. The records used in this study were obtained from the database of the European earthquakes (Ambraseys *et al*. 2004). This database contains 462 records (1386 accelerograms) of magnitude ranging from M = 3 to M = 6, and epicenter distances from Δ=12 km to Δ=500 km. The records with epicenter distances less than 15 km and records on very soft or very stiff soil stations were not included in this study. Only the horizontal components were included.

A specific searching criterion of records was applied. Only records were selected whose values of dispersion Sa/g in T was minor than 2. In other words, it was selected the records with $\varepsilon \leq 2$ on T, being ε the dispersion as defined by Baker *et al*. (2005). It is recognized that $\varepsilon > 2$ can produce a lot of dispersion taking into account that the scaling will be done in only one value of T and not in a range. The amounts of records found are shown in Table 5. A total n = 3423 records has been selected for the eight levels. All records were scaled next to their correspondent spectral level in $S_a(T)$.

Table 5 S_a/g values and record numbers found on T=1.474 s

Level	S_a/g	Records numbers
S_1	0.031	529
S_2	0.040	504
MS_1	0.048	482
MS_2	0.0587	446
LS_1	0.0703	414
LS_2	0.0812	384
C_1	0.0895	359
C_2	0.124	305

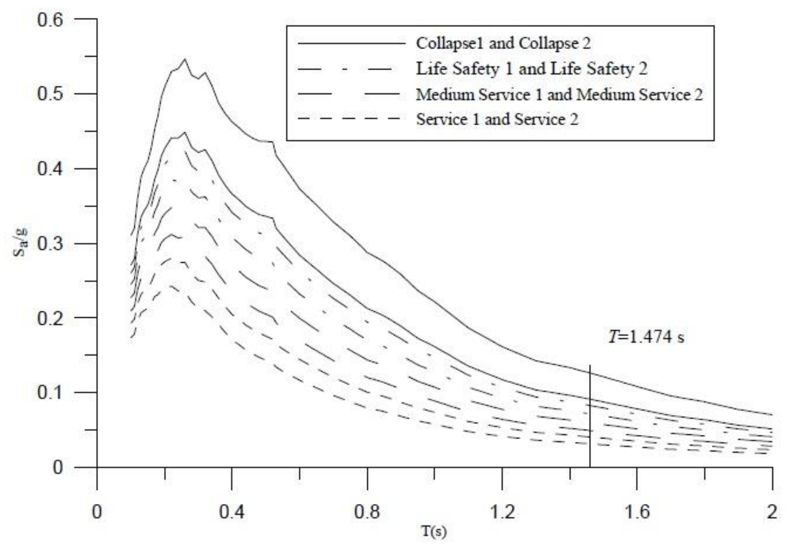

Fig. 3 Spectra obtained on multilevel PBEE.

3.4 General control indexes and threshold values adopted

The most accepted parameters to assess the structural behavior in each level have been discussed above. Among them, the selected in this work are: percentage of collapse, interstory drift ratio, roof drift ratio and modified overall Park Ang's index of damage (Park and Ang 1985; Park and Ang 1987). The global displacement ductility (μ) is not used due to its difficult representation of dissipation on MDOF but it has been used as a referential or indicative value in preliminary design. A ductility of hysteretic type -in an energetic way- should be used but it goes farther the aims of the present study. The modified Park and Ang index of global damage, although indirect, can be an indicator in order to deduce the inelastic dissipation capability of the structure. Although the work can be extended considering other types of damage indexes accounting also low cycle fatigue based on probabilistic approach (Zambrano and Foti 2013). The definition of this index is well known and it is not indicated here.

Table 6 Adopted control index and threshold values

Level	RP (years)	P_C (%)	Δ_{di} (%)	Δ_{roof} (%)	DI_{PA}	Principal Design Type	Observations
S_1	50	5	0.20	0.10	0.10	stiffness	No struct. damage
S_2	72	10	0.30	0.20	0.20	stiffness	Onset yielding
MS_1	175	15	0.40	0.30	0.30	stiffness	
MS_2	300	20	0.50	0.40	0.40	strength	Repairable limit
LS_1	475	40	1.00	0.80	0.50	strength	
LS_2	750	50	1.50	1.20	0.80	strength	
C_1	975	75	2.00	1.50	1.00	NL dis./Stab.	
C_2	2475	100	2.50	2.00	1.50	NL dis./Stab.	Final damage

NL dis./Stab.= Non-linear dissipation/Stability

A reasoned study on threshold values of the control index must be carried out for each level. This is a very complex work. These limit values sometimes are rather subjective and they must take into account many implicated variables. Without a deeper and more detailed description of this study (that exceeds this work) mean values by practice and standards or more acceptable ones are adopted (see Table 6).

The maximum interstory drift and the maximum roof drift adopted at each level, are based on ATC40, FEMAs and EC8 general recommendations. Generally, values are proposed for RP = 72 years and 475 years and they are been deduced to other levels on approximated way. The overall damage index of Park and Ang is deduced considering that the value of 0.40 is the repairable limit and that a great energy dissipation and deformation level (from RP = 975 to 2475 years) would lead to a number greater than one (near collapse). The adopted probability of collapse is found to take into account that it is a probabilistic assessment methodology (with numerous records) and an amount of collapse exist for all the levels.

In Table 6, P_C is the probability of collapse, Δ_{di} is the interstory drift, Δ_{roof} is the roof drift and DI_{PA} is the overall damage index of Park and Ang.

The structural philosophy of these values is based on: enough stiffness in little deformations with non-structural damages own of low levels, strength that permits a great energetic dissipation and stable behavior, saving lives and with structural repairable (if possible) damage, and finally, last dissipation to the collapse of the structure (no repairable damage) with guarantee of stability.

3.5 Collapse criterion. Statistical analyses

A coherent statistical application must be done to the results obtained. Although it is debatable in high levels with a strong non-linear behavior, a statistical lognormal distribution function is supposed. In the initial stage of study of the numerical results, attention was given only to whether the frame collapsed or not under the selected earthquake records. The probability of collapse, P_C, is defined as:

$$P_C = P(C / Sa(T) = x) = \frac{number \cdot of \cdot records \cdot that \cdot cause \cdot collapse}{total \cdot number \cdot of \cdot records \cdot applied} \tag{5}$$

Records, analyses and results of calculation are processed with temporal step of 0.005 s. All the examined levels present cases of collapse and some cases have a loss of data. These should be considered as censored data (CD). These are statistically defined when the number of data is known, but the values of the observations are unknown.

The collapse has been considered to occur under two possible conditions: (1) if the adopted limits values of Δ_{di}, Δ_{roof} or DI_{PA} are reached, and (2) when, due to a numerical instability with a loss of convergence in the algorithm of calculation, a sudden finalization of calculation is produced that leads to a loss of results (CD). In these cases the structure was first re-calculated with a temporary smaller increment (of acceleration and result) of Δt = 0.001 s, attempting to find a numerical convergence when possible.

In the first case and for the purposes of this study, the structure is assumed to collapse when one of the following limit situations occurs: (i) when the interstory drift exceeds 2.5 % of the story height; (ii) when the roof drift exceeds 2 % of the building height; or (iii) when the overall damage index Park and Ang reaches the value of 1.5.

If, for the examined level of the earthquake, the number of cases of collapse is equal to or smaller than 50% with respect to the total number of cases analyzed, the so-called Model of Distribution of Three Parameters is used (Shome and Cornell 2000; Shome and Cornell 1999). This model takes into account the found proportion of collapses or unavailable data in each case. The model finds a value of complete median and equivalent dispersion considering lost data as from three parameters: the probability of no collapse (P_{NC}), and the median and dispersion of the no-collapse results. These median and dispersion values are obtained in a logarithmic way.

3.6 Results and comments

After the complete individual calculate process of all records (record-to-record) is concluded, the results of each one are kept and then the values of P_C, Δ_{di}, Δ_{roof} and DI_{PA} are extracted for further processing.

Next, a statistical process is carried out and values of median and equivalent dispersion are obtained. All results are shown in Table 7 where the median value and the equivalent dispersion (in brackets) are indicated.

Table 7 Threshold values and results for control index in multilevel PBEE

Level		P_C (%)		Δ_{di} (%)		Δ_{roof} (%)		DI_{PA}	
Nº	Key	Thres.	Result	Thres.	Result	Thres.	Result	Thres.	Result
1	S_1	5	3.96	0.20	0.14(0.53)	0.10	0.09(0.39)	0.10	0.12(0.59)
2	S_2	10	6.35	0.30	0.17(0.52)	0.20	0.12(0.41)	0.20	0.21(1.26)
3	MS_1	15	13.28	0.40	0.20(0.67)	0.30	0.14(0.56)	0.30	0.25(1.40)
4	MS_2	20	19.06	0.50	0.24(1.06)	0.40	0.17(0.88)	0.40	0.33(1.45)
5	LS_1	40	35.75	1.00	0.30(1.19)	0.80	0.21(1.02)	0.50	0.42(2.84)
6	LS_2	50	48.18	1.50	0.46(0.80)	1.20	0.32(0.70)	0.80	1.50(1.22)
7	C_1	75	52.09*	2.00	2.77(0.05)	1.50	1.13(0.08)	1.00	2.39(0.30)
8	C_2	100	73.12**	2.50	NA	2.00	NA	1.50	NA

NA: Not Available (P_C>50%)
*It is approximated to 50%
**The p-value 0.05 range for P_C on C_2 is [0.69-0.79]

A first analysis appears with the comparison of P_C with the admissible values adopted. It has been commented that a reasoned study for these adopted values is necessary, always bearing in mind a posterior statistical analysis and having many records to process. This study and decision belongs to the structural designer. Here, the P_C of level 7 (C_1) has been approximated to 50% to be able to process this level since the difference is very small.

Fig. 4 shows the results of the analysis for the eight levels. A first observation is that in all levels of analysis collapses are generated (Fig. 4a) even in the lowest level. This is explained by the great record-to-record variability and the record selection utilized (by only one T), but it is acceptable due to the quantity of records used. The distribution of these collapses in relation to the maximum values is satisfactory and it is shown that in the 4-level (MS_2) a close value to the maximum is reached (20%). Observing that if few records are chosen it would lead to incorrect results. In levels 7 and 8, the P_C values are 52.09% (~50%) and 73.12%, respectively (Table 7). Only the range of dispersion of the collapse for a p-value 0.05 is given for level 8. This range value is 0.69-0.79 (%). This means that other control values do not exist in this level. This is a censored data case with more than 50% of collapses, and the impossibility to apply the Model of Three Parameters.

If the conditions of P_C imposed are not verified, it would implicate in this initial stage a re-calculation influencing stiffness, strength or dissipation. Obviously, all the process of calculation should be carried out again as T_1 would change.

In the analysis of Δ_{di} and Δ_{roof} , (Figs. 4b and 4c, Table 7), it is shown that to level 6 (LS_2) the median deformations are lower than the maximum, but with some values of equivalent dispersion exceptionally high. This highlights the great record-to-record dependence that the analysis has. However, it is noticed that both values increase significantly in level 7 and the Δ_{di} increases the maximum value meaningfully.

In Fig. 4d, the evolution of DI_{PA} is analyzed. Up to level 5 (LS_1) the index is more or less maintained in admissible values, while in levels 6 and 7, its value grows suddenly over the maximum allowed values. For level 6, DI_{PA} value is 1.50 >1.0 and clearly indicates collapse, something that should not be allowed in level with RP=750 years.

It can even be seen that in low levels, a relatively high DI_{PA} value is produced for the values of displacement that are detected, and that is due to damage for insufficient or bad distribution of hysteretic dissipation energy or very concentrated energetic dissipation. Considering that the structures analyzed in the present study consisted in a regular 5-bay 8-story frame system, it could be deduced that dissipation in plastic hinges of beams and columns should be satisfactory but everything suggests that some plastic hinges have an excessive demand of dissipation (and damage) and others do not. It leads to the conclusion that the configuration of plastic hinges is not adequate and therefore, the plastic hinge formation sequence produced is incorrect and/or insufficient.

The point 1 in Δ_{di} and point 2 in DI_{PA} show the beginnings of inadmissible behavior. From the point 2 (level 5, approximate) the structure does not verify all conditions and this is a critical point.

From the observation of graphics found with every index evolution it can be deduced that the structure presents an adequate stiffness in low levels, although somewhat over- dimensioned. Strength relatively adapted in the intermediate levels but an insufficient energetic dissipation in the high levels leads to a high value of damage being produced. In a simplistic way, it could be said that the structure is structurally adequate only to an intermediate level between 5 and 6, e.g., between RP=475 years and RP=750 years. For example, the point 2 represents a RP close to 545 years. This would indicate a level 8.77/50/545 with a_c=0.258g.

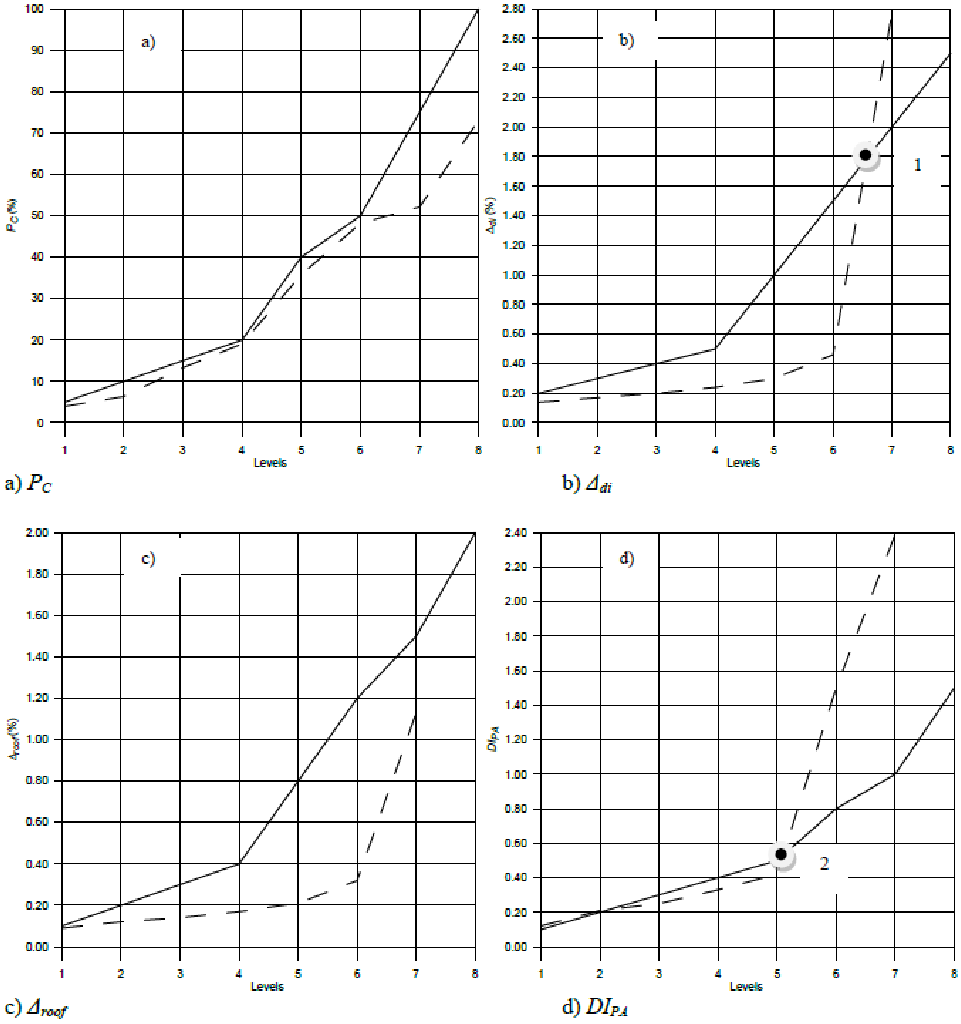

Fig. 4 Curves of control index for multilevel PBEE analysis. Threshold values are solid lines and results values are dashed lines.

From here, the calculation would continue in an iterative process to verify all the levels (see 2.1.7). Next, the more restrictive level should be found and finally, the process of optimization would consist in adjusting this level ensuring that the other levels verify the imposed conditions. These calculations are excluded from the scope of this investigation.

4. Conclusions

The emerging PBEE methodology has produced advances in knowledge in all important aspects of earthquake engineering and by societal demands for accountability for decisions that affect life safety and economy. Nowadays the 4-levels PBEE has become a used methodology. In this paper a study of PBEE analysis with many levels, a statistical evaluation of results, and finally a coherent analysis for possible optimization is proposed. The methodology is applied in an orderly way and here the multilevel design is understood as more complete than the frequently adopted 4-level one. The application of a multilevel PBEE approach in a zone of low-to-moderate seismicity has been investigated. In a low-to-moderate seismic zone (although a more active seismic zone) the economic importance of levels lower than RP=475 years must be carefully assessed.

Eight PBEE levels have been considered, from *RP*=50 years to *RP*=2475 years. The values of P_C, Δ_{di}, Δ_{roof} and DI_{PA} have been adopted as control indexes and for each one some maximum admissible values in each level of study have been proposed. More than 3400 dynamic response analyses were carried out with one structure subjected to a large number of records obtained from a European database. The European Ambraseys attenuation prediction (Ambraseys *et al*. 1996) with different seismic levels as referential spectra has been applied. A statistical evaluation of results has been carried out with the application of the Method of Three Parameters, able to predict a value when cases of data are missing (censored data) by the effect of loss of data due to collapse.

An application of the proposed method has been accomplished in a Spanish seismic zone. A regular 5-bay 8-story frame, designed and calculated according to the current seismic Spanish code has been examined. As graphic output, four curves with maximum adopted values of calculation and found values for each parameter have been done. These curves permit to see a complete perspective of all the levels for a determined parameter of control and to deduce values of another different level, at different *RP*, in a "continuous" way.

From Table 6 and Fig. 4, it can be seen that the percentage of collapses, the values of the interstory drift, the roof drift and the Park and Ang damage index, are acceptable to a level of earthquake 5 (LS_1) of *RP*=475 years. Starting from level 5, a great value of structural damage appears. This indicates that in low levels, the structure has enough stiffness (even excessive) and that in high levels a great damage for incorrect handling of energetic dissipation exists. Even in low levels a great value of damage is produced. This can be due to insufficient energetic dissipation or very concentrated energetic dissipation. Taking into account that the structure is a 5-bay 8-story frame, with a great quantity of possible plastic hinges; everything indicates that some plastic hinges have an excessive demand of dissipation (and damage) and others do not. It leads to the conclusion that the configuration of plastic hinges is not adequate and, therefore, the plastic hinges formation sequence produced is incorrect and/or insufficient. From here, it is inferred that the structure calculated with the Spanish seismic code has a satisfactory behavior to level 5 (LS_1), the level utilized as a preliminary design. Also it is observed that the analyzed structure is excessively stiff in low levels and lacking dissipation, while shows a great damage in high levels. Obviously if the PBEE behavior is analyzed as a whole, the conclusion is that the structure has a deficient behavior. If it is considered as a typical analysis imposed by code rules the results would be satisfactory.

In this work has been demonstrated that the proposed methodology is very useful although, more extensive examples of analysis must be carried out to justify completely the most complete multilevel seismic performance. Finally, from the means, resources and capabilities available at this time and based on the results of the present study, the use of a methodology of type multilevel PBEE analysis could be applied.

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