Seismic Retrofit of Concrete Buildings

(Summary)

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1. Introduction

The main objective of this dissertation is to describe the guidelines and proceedings for the Seismic Assessment and Retrofitting of Existing Buildings, according to the Eurocode 8 part 3. This thesis also intends to give out the fundamental notions in part 1 of Eurocode 8, which are useful for the assessment and retrofit of buildings. In addition, presents the main techniques for the seismic retrofitting of buildings and applies this knowledge to an existing building.

In recent years, the seismic behaviour of structures has been an increasing concern for the civil society. The most recent seismic events, such as the Northridge earthquake in 1994 and Kobe earthquake in 1995, were evidence of seismic vulnerability of structures when submitted to strong and rare seismic actions.

Based on that awareness, a new set of regulations and methods of analysis, such as the Eurocode 8, were developed. Only after 1983, when the “Regulation of Safety for Actions in Buildings and in Bridges” was implemented, the seismic action was considered in similar fashion to the new eurocodes. Despite not fully accomplishing the performance requirements demanded in the eurocodes, the seismic behaviour of structures in this period, in respect to their seismic strength and ductility, offers satisfactory results.

In the other hand, the structures prior to that period have a number of deficiencies in their detailing and designing. The most common are the lack of reinforcement in structural elements and the great spacing between stirrups, lap splice failures, column failures, irregularity in mass and in stiffness, interruption of load paths.

The purpose of the new Eurocode 8 part 3 is to upgrade the seismic capacities of existing structures to a level in which they can sustain the seismic demands according to the performance requirements defined in the part 3 of the Eurocode 8, Assessment and Retrofit of Buildings [4].

2. Seismic Behaviour of Structures

A set of dynamic displacements or a quantity of energy transmitted to the structure can translate the effects or demands of the seismic action. The structure must have the ability to sustain these seismic demands and if not, the structure is considered vulnerable.

However, the concept of vulnerability is not absolute, as a building may be vulnerable to a type of seismic action and seismic resistant to another. It all depends on the properties of the structural system and on the characteristics of the seismic action.

Another essential feature is to learn about the mistakes of the past, to improve the design and detailing of future buildings. For that purpose, the deficiencies that led do local or global failure of the structural system are analysed.

It is possible to classify the reasons for the collapse of a building in two groups; one regarding the internal conditions of the building and another concerning the external conditions of the building. The external conditions concern the type of foundation, the type of soil and the
interaction with adjacent buildings. The internal conditions concern the regularity in terms of mass and stiffness, the ability to dissipate energy, the concentration of stresses in singular zones, the deficient detailing and design of structural elements and non-structural elements.

3. Seismic design and analysis, general rules for buildings

Objectives and performance requirements
The main purpose of the Eurocode 8 part 1, Design of structures for earthquake resistance, is to assure the protection of human lives, the limitation of damages and to guarantee that the most important structures for the civil protection remain actives. As a result, buildings in seismic regions should be design and built, in accordance with the Performance Requirements defined in the Eurocode 8 part 1, the No-Collapse requirement and the Damage Limitation requirement.

The No-Collapse requirement guarantees the capacity of the structure to withstand the design seismic action without local or global collapse. In addition, the structural integrity and the residual load bearing capacity is guaranteed after the seismic event.

For ordinary structures, this requirement should be satisfied for a reference seismic action with 10 % probability of exceedance in 50 years (with 475 years return period).

The Damage Limitation requirement guarantees the ability to withstand a more frequent seismic action, without significant damage for structural and non-structural elements.

For ordinary structures, this requirement should be satisfied for a seismic action with 10 % probability of exceedance in 10 years (with 95 years return period).

Reliability Differentiation
The Eurocode 8 implements a Reliability Differentiation by classifying the structures into Importance Classes, depending on the consequences of the structure’s failure. An importance factor $\gamma_1$ is assigned to each class. This classification should be the responsibility of the National Authorities, and its values are the National Determined Parameters (NDP).

It is possible to enhance the seismic performance of the structure, multiplying the reference seismic action by the desired importance factor [9].

$$a_g = \gamma_1 \times a_{gR} \quad (E. 1)$$

Where, $a_{gR}$ is the ground peak acceleration.

Compliance Criteria
Furthermore, the EN 1998-1: 2004, defines a Compliance Criteria to verify the performance requirements. For the Damage Limitation requirement, the inter-storey drifts should be checked, according to the ruling exposed in the EN 1998-1: 2004. In the other hand, the No-Collapse requirement, must guarantee the structural resistance and the capacity to dissipate energy according to the ruling exposed in the EN 1998-1: 2004 [1].
Seismic action

The seismic action depends on the Seismic Zonation, on the Ground Conditions and on the type of Seismic Action. In fact, the seismic zonation is different for each type of seismic action, depending on the local hazard. The national authorities have the responsibility to divide the territory in seismic zones with the seismic hazard assumed constant in each zone.

![Seismic zones diagram]

**Figure 1: Seismic zones according to the seismic action type 2 and to the seismic action type 1[3].**

The ground types A, B, C, D, E, described by the stratigraphic profiles and by the parameters defined in Table 3.1 of the part 1 of the Eurocode 8. These ground types may be used to account for the influence of local ground conditions on the seismic action.

In addition, two other classes, S₁ e S₂, were created according to the rulings displayed in EN 1998-1: 2004 [1].

**Representation of the seismic action**

The basic representation of the seismic action is the elastic response spectrum for 5% damping. In Portugal, the seismic action may be represented by the elastic response spectrum type 1 and by the elastic response spectrum type 2, to consider the different seismic conditions. The design of structures should consider the seismic action most hazardous for the structure.

The elastic response spectrum comprehends the horizontal and the vertical components. The parameters and expressions that define those spectrums are in the relevant parts of the EN 1998-1: 2004 [1] and in the National Norm [3].

In the elastic analysis, the structure’s capacity do dissipate energy is not considered. Therefore, to take advantage of the structure’s ductility without any explicit inelastic structural analysis, it is possible to perform an elastic analysis based on a response spectrum reduced with respect to the elastic one, hereafter called “design spectrum”. This reduction is accomplished by introducing a reduction factor denominated behaviour factor q. This coefficient assumes the value from 1,5 for concrete structures [1] [7].

The parameters and expressions that define this spectrum may be found in the relevant parts, of the EN 1998-1: 2004 [1] and in the National Norm [3].

Apart from the basic representation of the seismic action, the Eurocode 8 part 1 also contemplates alternative representations of the seismic action such as time-history representations, artificial accelerograms and recorded or simulated accelerograms [1].
The seismic action is combined with other relevant actions according to the following expression:

\[ \sum G_{K,j} A_{Ed} \text{"} + \text{"} \sum \varphi_{2,j} Q_{k,j} \]  

(E. 2)

Where, \( G_{K,j} \) is the nominal value of permanent action \( j \), \( A_{Ed} \) is the design seismic action, \( Q_{k,j} \) is the nominal value of the variable action \( i \) and \( \sum \varphi_{2,j} Q_{k,j} \) is the quasi-permanent value of variable action \( i \). Coefficients \( \varphi_{2,j} \) are defined in Normative Annex A1 of EN 1990 [4] as National Determined Parameters (NDP).

**Methods of analysis**

It is possible to determine the effects of the seismic action combined with the other actions present in the seismic design combination by the following methods of analysis:

a. Linear static analysis or "lateral force method".

b. Modal response spectrum or "dynamic static analysis".

c. Non-linear static or "pushover analysis".

d. Non-linear dynamic or "time history analysis".

The reference method for the seismic design of new buildings is the modal response spectrum analysis with the design spectrum by means of an elastic-linear model. It is also possible to apply other methods if they respect the applicability conditions defined in the relevant parts of the EN 1998-1: 2004 [1].

**Design philosophy**

The guiding philosophy of the new Eurocodes is the "Capacity Design". The main objective of this philosophy is to take advantage of the ductility properties of the structures, thus preventing the brittle failures and the concentration of stress in singular zones. For that purpose, the beam-column joints are over-designed comparatively to the beams, making it possible to control the formation of plastic hinges and to control and preview the behaviour of the structure [5]. Therefore, guaranteeing that the structures possess capacity to dissipate the energy transmitted by the seismic action without losing its strength capacity, it is possible to guarantee the structural safety taking advantage of the structural’s properties.

**Base isolation**

This latest version of the Eurocode 8 part 1 [1] contemplates, for the first time, the base isolation of structures. This isolating system is located below the main mass of the structure and its objective is to reduce the seismic response of the lateral-force resisting system.

The reduction of the seismic response of the lateral-force resisting system is obtained by:

- increasing the fundamental period of the seismically isolated structure, modifying the shape of the fundamental mode, increasing the damping or by a combination of these effects [11].
The Eurocode 8 part 1, section 10, defines the performance requirements, compliance criteria, seismic action, type of analysis, safety verifications, as well as other significant notions for the design of base isolating system.

4. Seismic Assessment and Retrofit

The Eurocode 8 part 3, Assessment and Retrofitting of Buildings, reflects the latest tendencies in the seismic design of structures, especially the displacement-based approach [7] [8].

Performance requirements

The performance requirements correspond to the hazard level of the structure. There are three hazard levels, known as limit states or damage states.

The hazard is described in the form of elastic, five percent damping response spectra having specified average return periods.

<table>
<thead>
<tr>
<th>Hazard (return period of the design spectrum)</th>
<th>Required performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_R = 2475 ) years, corresponding to a probability of exceedance of 2% in 50 years.</td>
<td>Near Collapse (NC)</td>
</tr>
<tr>
<td>( T_R = 475 ) years, corresponding to a probability of exceedance of 10% in 50 years.</td>
<td>Significant Damage (SD)</td>
</tr>
<tr>
<td>( T_R = 225 ) years, corresponding to a probability of exceedance of 20% in 50 years.</td>
<td>Damage Limitation (DL)</td>
</tr>
</tbody>
</table>

National authorities may select lower values for the return periods, and require compliance with only two limit states.

The seismic action is applied to the structure without any ductility-related reduction factor, and the state of the structure, displacements or stresses, are evaluated by means of linear or non-linear types of analyses, depending on the characterisation of the structure [7].

Compliance criteria

The verifications of the structural elements/mechanisms vary, depending on their nature. If they qualify as ‘ductile’, bending with and without axial force, the deformation demands must be smaller than the admissible deformation for the considered performance level.

If they are of the ‘brittle’ type, shear mechanisms, the strength capacity should not exceed the seismic demands in terms of resistance.

Information for the Structural Assessment

The structural assessment should include information regarding the building’s history, such as the original drawings, contemporary codes and standards, field investigations and in-situ and/or laboratory tests. This information should cover the; characteristics of the structural system; type of foundations; information regarding the materials; the detailing and the overall dimensions of elements; information regarding previous alterations or damages to the structural system, etc.
There are several new features in this regulation; in first place the introduction of new safety coefficients; secondly, new expressions that define ultimate flexural deformation of concrete elements; and finally, the possibility of using non-linear static analysis as standard tool for assessment purposes.

The introduction of new safety elements called ‘confidence factors’ (CF), which take into account the different degree of knowledge achieved regarding the; geometry, amount and quality of the details, and the materials. Each confidence factor is associated to a knowledge level, Those levels reflect the quality and quantity of the information resultant of the seismic assessment. The Knowledge level also determinates which analyses are possible to perform [2].

<table>
<thead>
<tr>
<th>Knowledge level</th>
<th>Analysis allowed</th>
<th>CF</th>
</tr>
</thead>
<tbody>
<tr>
<td>KL1: Limited knowledge</td>
<td>Linear</td>
<td>1,35</td>
</tr>
<tr>
<td>KL2: Normal knowledge</td>
<td>All</td>
<td>1,20</td>
</tr>
<tr>
<td>KL3: Full knowledge</td>
<td>All</td>
<td>1,00</td>
</tr>
</tbody>
</table>

The values of the material strengths must be divided, by the confidence factor, in case of the ductile elements, and additionally, by the respective partial factor, in case of the brittle elements.

**Structural Assessment**

The structural assessment is a quantitative procedure in which the seismic capacity, of the existing building, under the appropriate limit state, is assessed. Subsequently, if there is a need for structural intervention, it should also contemplate the retrofits design and the effects of such intervention on the rest of the structure [2] [6].

The methods of analysis are the same applicable to the new structures. The only difference is that the linear methods have to fulfil an extra applicability condition, clause 4.4.2(1) of the EN 1998-3: 2005[2]:

“The ratio between the demand $D_i$ and the capacity $C_i$, $\rho = D_i / C_i$, must be sufficiently uniform across all primary resisting members, e.g.: $\rho=\rho_{\text{max}}/\rho_{\text{min}}< 2$ to 3. Where, $\rho_{\text{max}}$ and $\rho_{\text{min}}$ are the maximum and minimum value of the ratio $\rho$”.

For brittle elements is evaluated the resistance capacity, and for ductile elements is evaluated the deformation capacity. The values of the seismic demands are obtained from the analysis, under seismic load combination and the values of the capacities are obtained from the Capacity Models, presented in Annex A of EN 1998-3: 2005 [2].

**Capacity models for RC members**

The capacity models for the assessment of the flexural deformation (E.3) and for the shear strength (E.4) may be found in Annex A of EN 1998-3: 2005 [2]. The most important are, the ultimate deformation and the shear resistance, respectively, as follows:

$$\theta_{\text{ult}} = \frac{1}{\gamma_{\theta}} + 0.016(0.3^{+})^{\left[\max(0,01; \omega)\right]} f_{t}^{0.225} \left[\left(\frac{L_{c}}{h}\right)^{0.35} \left(\frac{\sigma_{\omega}}{f_{t}}\right)\left(1.25\rho_{\omega}\right)\right]$$ (E. 3)
\[ V_R = \frac{1}{\gamma_{Ch}} \left[ \frac{h - x}{2L_V} \min \left( N; 0,55A_c f_{c'} \right) + \left( 1 - 0,05 \min \left( 5; \mu_{s} ^{\text{fr}} \right) \right) \right] \]

(E. 4)

5. Retrofitting techniques

The decision on the type of retrofitting technique must be substantiated by; the results of the structural assessment; the type and extension of the damage; the technical criteria; the social influence of the intervention; and the costs of such intervention [2].

Depending on the level of damage in the structure, two types of intervention may be defined; local reinforcement of elements, and/or global reinforcement of the structural system.

When the target is to enhance the properties of a few elements, a local intervention like jacketing of elements or the introduction of resistant elements, may be used. When the damage is more generalized along the structure, a global intervention is more advisable. The most common are; base isolation; dissipation energy systems; or a combination of techniques.

Independently of the type of intervention, local or global, the effects of the seismic action should be considered along the whole of the structural system. Therefore, avoiding the concentration of stresses in singular zones, and improving the seismic performance of the structure [10].

6. Case Study

![Figure 2: Plant of the building in study.](image)

The building's location is in Lisbon, the standard and rules used for its design and detailing, are from 1967 (namely the Regulation for Concrete Structures). Its implantation area is 477 m². It has five storeys, with the total height of 23.75m.

Information for the structural assessment

This information gathers the original architectural plants and the detailing of the structural elements. It was assumed that the inspections were carried through 50% of the elements, that is, an intensive inspection to the building, with extensive testing to the materials.

From that information, it is concluded that the building presents a normal level of knowledge (KL2), therefore being possible to employ any method of analysis. The corresponding
The confidence factor is 1.2. According to the EN 1998-3: 2005 [2], the information for structural assessment and for the seismic retrofit is as follows:

- The structural system is a typical concrete building, having its slabs with beams. The building fulfils the regularity criteria defined in EN 1998-1: 2004 [1].
- The ground type C is according to EN 1998-1: 2004 [1]. In addition, the foundations are direct (footing).
- The dimensions of the cross-sections, of the structural elements, are the same as the original drawings.
- The structural materials are; C20/25 ($f_{ck}=20$ MPa; $E=30$GPa) and A400 ($f_{syd}=348$ MPa). They present good conditions of conservation and there are no signs of intense cracking, corrosion in the reinforcement or deterioration of the concrete cover.
- The structural elements were designed and detailed according to the regulations in use in that period (RSEP and REBA). From the structure’s analysis the following deficiencies are distinguishable:
  - The columns exhibit lack of stirrups, and the spacing between them is too long. Therefore, the confinement is very weak and inefficient to avoid the brittle failure of the columns. The elements have an insignificant amount of ductility and the flexural reinforcement is insufficient to sustain the prescribed seismic action (figure 3).

![Figure 3: Detailing for column type P1.](image)

- The beam-column joints present serious deficiencies in its detailing, such as the shortness of the lap-splices.
- The beams detailing is very poor and the flexural strengths demands are much bigger than the flexural capacity of the beams. In addition, these elements have little ductility, on account of the lack of stirrup confinement (figure 4).
- The structure, was not designed and detailed in order to guarantee its ductile behaviour neither to allow the redistribution of stresses. Thus, the columns are vulnerable to mechanisms of brittle failure due to the shear strength demands.
- The structure’s design, inspired in the RSEP and in the REBA, considers a seismic action that is insufficient to the real demands, but is enough to provide some seismic resistance to the structure.
The building’s current use is for offices ($\psi_0=0.7; \psi_1=0.5; \psi_2=0.3$) with a corresponding importance class II ($\gamma_1=1$). There is no plan to change the utilisation or the structural system of the building and it never went through significant modifications or repairing.

Analysis and structural modelling
The structural modelling was done with the automatic calculation program SAP2000. The chosen method of analysis was the modal response spectrum with the design spectrum and a behaviour factor of 1.5. The building has a fundamental vibration period of 1.46 seconds, the first vibration mode occurs in the building’s longitudinal direction. The torsional vibration modes are relatively high.

The most hazardous seismic action for the building is the type 1. Therefore, the combinations of actions for the assessment of the structure and for the retrofit used the seismic action type 1.

Capacity assessment
The building’s capacity assessment was made according to the Eurocode 8 part 3, Assessment and retrofit of buildings.

For the brittle structural elements, columns, was analysed the shear demand. For the ductile elements, beams, was analysed the ultimate deformation demands. Both demands were taken directly from the analysis. With those seismic demands, along with capacity defined by the capacity models, it is possible to verify the compliance criteria. Another aspect taken from the analysis is the verification of the applicability condition of the linear methods ($\rho_{\text{max}}/\rho_{\text{min}}< 2$ or 3).

Afterwards the flexural capacity is evaluated, with and without axial force, of columns and beams.

The results of structural assessment were not surprising as the deficiencies are extremely serious and generalized along the building. In fact, the building’s seismic resistance is well below 50% of the demanded prescribed values.

The majority of the columns does not have the ability to sustain the seismic deformations and its shear capacity is insufficient to support the seismic shear demands. Only 10% of the columns
have the capacity to sustain the seismic shear demands and in some cases, they have a resistance debt of 75%.

The longitudinal reinforcement is not enough to sustain the stress form the seismic flexural demands as only 40% of the column’s cross-sections analysed, have enough reinforcement. The beams also do not have the ability to sustain the seismic deformations as no more than 10% have an ultimate deformation capacity greater than the seismic demands. Furthermore, the flexural capacity of these elements is inadequate. In some cases these deficiencies reach 70% in comparison to the seismic demands.

Due to the extent of the deficiencies detected in the building, the global retrofitting of the structural system is the best option. Consequently, the building is retrofitted through the introduction of concrete walls (figure 5). As a result, the building’s fundamental frequency increases to 1.33 seconds and the inter-storey drifts respects the 1% (height between storeys) limit for the service seismic action, in all storeys. The columns verify the safety to flexure moments with axial force.

Despite the overall improving of the structure’s performance, it is almost impossible to verify the local safety in every cross-section without the local reinforcement of those sections. The shear demands in columns, in the upper floors, are much bigger than its resistance. In addition, the flexure moments in beams, essentially in the column-beams joints, and bigger than the resistance, in spite of the redistribution of stresses that was effectuated.

7. Conclusion

The assessment is very conclusive as the deficiencies are throughout the whole building. The resistance capacity of the structure is less than 50% of the needed seismic capacity to sustain the seismic demands. It is logic to assume that this level of deficiencies is common in buildings constructed in the same period, from 1960 to 1983, due to the identical standards and codes.

The adopted retrofit intervention allows the global safety verifications to be fulfilled, even if at a local level it is impossible to satisfy all the safety criteria, like the shear criteria in the upper storeys and the flexural demands on the beam-column joints. The best solution will be either to consider a reduced seismic action or to reinforce locally those elements.
References


