Grant Agreement no. 226967
Seismic Hazard Harmonization in Europe
Project Acronym: SHARE

SP 1-Cooperation
Collaborative project: Small or medium-scale focused research project
THEME 6: Environment
Call: ENV.2008.1.3.1.1 Development of a common methodology and tools to evaluate earthquake hazard in Europe

D2.4 – Results from study on minimum hazard levels for explicit structural seismic analysis and design

Due date of deliverable: 31.05.2011
Actual submission date: 16.09.2011

Start date of project: 2009-06-01 Duration: 36

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Revision: 1

Dissemination Level		
PU	Public	X
PP	Restricted to other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the consortium (including the Commission Services)	
CO	Confidential, only for members of the consortium (including the Commission Services)	

Abstract

This deliverable aims to define, through probabilistic structural assessment, the minimum structural capacity levels of typical gravity load designed European buildings. These capacity levels identify values below which the explicit definition of seismic hazard is not necessarily required. A probabilistic framework is used that allows for a consideration of the uncertainty in the displacement capacity that arises when a group of buildings, which may have different geometrical and material properties, is considered together. The structures that have been considered in this study are represented by reinforced concrete structures and masonry structures (both reinforced and unreinforced). The probability density functions and cumulative distribution functions of the displacement capacity for each building typology have been computed and the thresholds of the non-structural damage and the ultimate limit state levels have been estimated.

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1. Introduction

The aim of this deliverable is the definition, through probabilistic structural assessment, of the minimum structural capacity levels of gravity load designed European buildings. These capacity levels identify values below which the explicit definition of different zones of seismic hazard is not necessarily required. The main idea is to define the minimum capacity levels of a new building designed with the minimum design criteria (vertical loads) and from this subsequently identify the lower thresholds of the displacement spectra below which a detailed calculation of the demand on the structure can be avoided. A probabilistic framework is developed and allows for a consideration of the uncertainty in the displacement capacity that arises when a group of buildings, which may have different geometrical and material properties, is considered together. To model the displacement capacity uncertainty, Monte Carlo simulation has been utilized. Monte Carlo simulation has been used to calculate the approximate cumulative distribution function of a non-linear function of correlated random variables. The structures that have been considered in this study are represented by reinforced concrete structures and masonry structures (both reinforced and unreinforced).

The Eurocodes have been used as the reference codes within this project. They have been developed over the past 30 years by the combined experience of the member states of the European Union and they became the standard code for the private sector in Europe. They have been translated in all the main European languages between 2002 and 2007. The Eurocodes are published under 10 area headings. The first two Eurocodes are common to all the types of designs and they concern the basis of structural design (EC0) and the actions on structures (EC1). Six of them describe the building design according to the type of material used to construct the structure: concrete structures (EC2), steel structures (EC3), composite steel and concrete structures (EC4), timber structures (EC5), masonry structures (EC6) and aluminium structures (EC9). Then, EC7 covers geotechnical aspects of the design and EC8 deals with the seismic design.

In seismic risk assessment, the performance levels of a building can be defined through damage thresholds called limit states that define the threshold between different damage conditions. The minimum structural capacity levels expressed as a displacement measure have been computed for the third limit state (ultimate limit state). The three limit states given by the EC8 (CEN, 2004. EN 1998-3: 2005, 2.1) are: Limit State of Damage Limitation (DL), Limit State of Significant Damage (SD) and Limit State of Near Collapse (NC). These limit states are characterized as follows:

1. LS of Damage Limitation (DL): " …*The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures*…" (CEN, 2005. EN 1998-3: 2005, 2.1);
2. LS of Significant Damage (SD). "…*The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of*

moderate intensity. The structure is likely to be uneconomic to repair…" (CEN, 2005. EN 1998-3: 2005, 2.1);

3. LS of Near Collapse (NC). "…*The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity*…" (CEN, 2005. EN 1998-3: 2005, 2.1).

For each of the described limit states a return period for the seismic action have to be selected in order to achieve the appropriate level of protection. In the National Annex of each country it is possible to find instructions about the choice of the return periods. However, for ordinary new buildings the following values can be taken into account (CEN, 2005. EN 1998-3: 2005, 2.1):

1. LS of Damage Limitation. T_R = 225 years. It corresponds to a probability of exceedance of 20% in 50 years;
2. LS of Significant Damage. T_R = 475 years. It corresponds to a probability of exceedance of 10% in 50 years;
3. LS of Near Collapse. T_R = 2475 years. It corresponds to a probability of exceedance of 2% in 50 years.

The limit state for non-structural damage has also been considered in the analyses herein, which is generally for a return period of 95 years.

In the following, a bilinear capacity curve of a building that has been transformed to an equivalent single degree of freedom system is shown as an example and the three limit states are represented on the curve. The aim of this study has been to identify the value of Sd at the limit state NC for typical European gravity load designed structures.

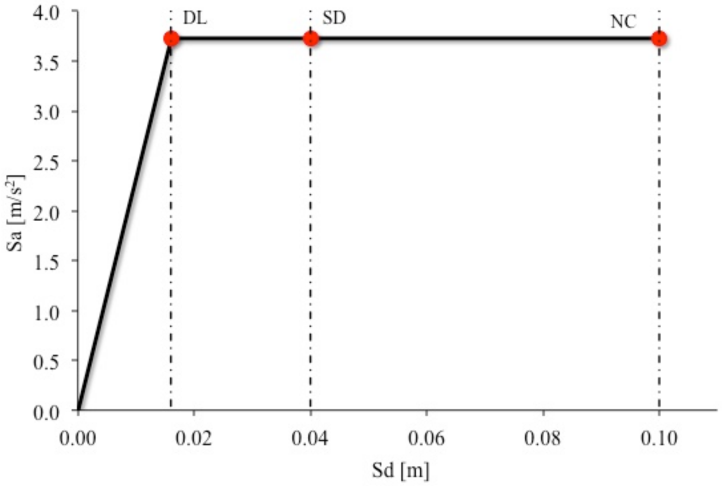


Figure 1.1: Capacity Curve: Three Limit States

Chapter 2 of this deliverable describes the methodology together with the equations used to calculate the displacement capacity for bare framed reinforced concrete structures, reinforced masonry and unreinforced masonry buildings. Both the structural and non-structural elements are analyzed. Chapter 3 presents the characteristics of typical European gravity load designed buildings. Conservative assumptions have been made to estimate the lower thresholds of the displacement spectra. Then, Chapter 4 summarises the results and the thresholds found in this study showing the distribution of the displacement capacity for different building typologies.

2. Calculation of Displacement Capacity

A displacement-based approach has been used to estimate the aforementioned thresholds. It is well known that displacements or inter-storey drifts are more closely correlated to structural and non-structural damage than forces. The procedure presented herein to calculate the displacement capacity of buildings uses mechanics-derived formulae at the three different limit states. These equations are given in terms of material and geometry properties.

2.1 Reinforced concrete buildings (bare frames)

Different formulae are given according to the sway mechanism. There are two sway mechanisms: beam sway and column sway. The former is caused by plastic hinges forming in all the beams above the first floor and in all of the columns at the base of the building (Figure 2.1a), the latter forms when plastic hinges form at both ends of the column usually in the ground floor, leading to what is known as a soft-story (Figure 2.1b).

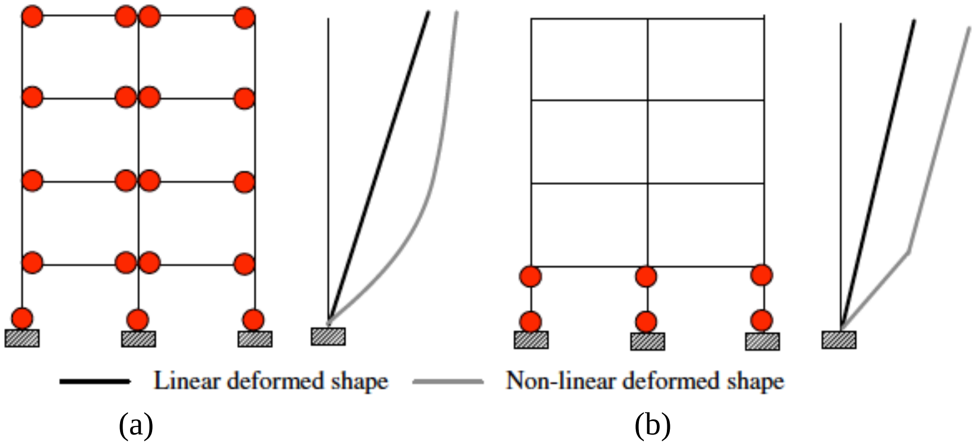


Figure 2.1: Response mechanisms for a frame and associated deformed shapes. (a) Beam Sway mechanism (b) Column Sway mechanism

A simple definition of the displacement capacity of buildings using mechanical material properties and concepts under different limit states is the basis of the displacement-based methodology used herein. The structural and yield displacement capacity and the structural post-yield displacement capacity of a building subjected to a column or beam sway mechanism is a function of the height of the structure. Hazard maps in design codes are generally in terms of peak ground acceleration and spectral acceleration/displacement at a number of period ordinates. Hence, in order to compare the displacement capacity with the levels of hazard, the displacement capacity equations must describe the capacity of a SDOF substitute structure and hence must give the displacement capacity at the center of seismic force of the original structure. The displacement at the center of seismic force is given by multiplying the base rotation by an effective height. The effective height is computed by multiplying the total height of the building by an effective height coefficient (e_{fh}). This coefficient is the ratio between the height to the center of mass of a SDOF substitute structure (H_{SDOF}), that has the same displacement capacity as the original structure at its center of seismic force (H_{CSF}), and the total height of the original structure (H_T). For beam-sway frames, the ratio of H_{CSF} to H_T varies with the height, independently of ductility. The effective height coefficient can then be defined as a function of the number of storeys *n* using the following equations suggested by Priestley (1997):

$$ef_h = 0.64 \qquad n \leq 4 \tag{2.1}$$

$$ef_h = 0.64 - 0.0125(n-4) \qquad 4 < n < 20 \tag{2.2}$$

$$ef_h = 0.44 \qquad n \geq 20 \tag{2.3}$$

For column-sway frames, the elastic and inelastic deformed shapes vary from a linear profile for elastic (pre-yield) limit state to a non-linear profile at inelastic (post-yield) limit states. As suggested by Priestley (1997), the linear profile at pre-yield limit states means that the ratio of H_{CSF} to H_T can be assumed to be 0.67 and this value has to be taken as the effective height coefficient for this case. With regards to the post-yield limit states, the effective height depends on the ductility. However, the ductility cannot be calculated unless the yield displacement at the effective height is known. An iterative procedure should be carried out to find the effective height. Glaister and Pinho (2003) proposed the following equation for the sake of simplicity:

$$ef_h = 0.67 - 0.17\frac{\varepsilon_{S(Lsi)} - \varepsilon_y}{\varepsilon_{S(Lsi)}} \tag{2.4}$$

Where $\varepsilon_{S(LSi)}$ is the sectional limit state for the steel and ε_y is the yield strain of the reinforcement steel. It has to be noted that the sectional limit states for reinforced concrete buildings are different according to the building construction code. Bal *et al*. (2010) estimated a set of second and third limit state strains which have been used within this project. In the following the description of the sectional limit states for reinforced concrete buildings together with their values are shown. The value of the yield limit state is strictly dependent on the type of steel used.

Table 2.1:Description of the sectional limit states for reinforced concrete buildings

	Description	
Exceedance of LS1	Loss of linear elastic response, yielding in the section. The concrete strain limit is checked at the most outer concrete fibres.	
Exceedance of LS2	Member flexural strength is achieved limited ductility developed. Concrete strain is checked at the outer fibres of the core concrete	$\varepsilon_{c(LS2)} = 0.0035$ $\varepsilon_{S(LS2)} = 0.015$
Exceedance of LS3	Wide flexural and/or shear cracks occur, buckling of longitudinal reinforcement may happen. Concrete strain is checked at the outer fibres of the core concrete	$\varepsilon_{c(LS3)} = 0.0075$ $\varepsilon_{S(LS3)} = 0.035$

To estimate the probable response mechanism (beam or column sway) of a structure, a stiffness-based (or deformation-based) sway index is used. The sway index can be related to some general properties of the building. The probability of having a column sway mechanism increases with the increasing beam section depth, with the decreasing column section depth, with the increasing column length (storey height) and with the decreasing beam length. The value of the index for i^{th} joint for a certain floor is:

$$R_i = \frac{\frac{h_{b,L}}{L_{b,L}} + \frac{h_{b,R}}{L_{b,R}}}{2\left(\frac{h_{c,B}}{L_{c,B}}\right)} \quad 2.5$$

where sub-indices 'L' and 'R' refer to 'Left' and 'Right' and 'B' refer to 'Below' respectively. The index per floor could then be obtained by averaging the result of Equation 2.5 for each floor:

$$S_{def,j} = \frac{\sum_{i=1}^{n} R_{i,j}}{n} \quad 2.6$$

Where n is the total number of joints at floor j. The maximum value of the index between floors is the value that represents the structure. In the following table the limits of the index for different building types are shown (Abo El Ezz, 2008, Shah, 2009):

Table 2.2:Limits of the deformation-based sway index for different building types

	Beam Sway mechanism	Column Sway mechanism
Bare Frame structure	$R_i \leq 1.5$	$R_i>1.5$
Fully Infilled Frame structure	$R_i \leq 1$	$R_i>1$
Infilled Pilotis Frame structure	$R_i \leq 0.5$	$R_i>0.5$

Using typical geometrical properties (later presented in Table 3.2), a value of $R_i<1.5$ is calculated for both the steel types that are considered in the project and that are presented in Section 3.1. For this reason, a beam sway mechanism can be confidently assigned to the building class.

Once the effective height has been assigned, the structural yield displacement and the structural post-yield displacement capacity can be calculated using the following formula. To account for shear and joint deformation, empirical coefficients have to be used and the values suggested by Glaister and Pinho (2003) have been applied for the case of a beam-sway mechanism.

$$\Delta S_y = 0.5 ef_h H_T \varepsilon_y \frac{l_b}{h_b} \quad 2.7$$

$$\Delta S_{LSi} = \Delta S_y + 0.5 \cdot \left(\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - 1.7\varepsilon_y\right) \cdot ef_h H_T \quad 2.8$$

Where:

- ef_h is the effective height coefficient;
- H_T is the total height of the building;

- ε_y is the yield strain of reinforcement steel;
- $\varepsilon_{C(LSi)}$ and $\varepsilon_{S(LSi)}$ are the sectional limit states for reinforced concrete buildings;
- l_b is the length of the beam;
- h_b is depth of the beam.

To define the minimum capacity levels of a new building designed with the minimum design criteria (vertical loads) and subsequently identify the lower thresholds of the displacement spectra below which a detailed calculation of the structure can be avoid, the ultimate limit states have to be considered.

Non-structural damage has to be considered too. The non-structural displacement capacity can be found using Equation 2.9:

$$\Delta_{NLSi} = 0.005 \cdot n \cdot h_s \qquad 2.9$$

where:
- n is the number of storeys taken equal to 1 (most conservative assumption);
- h_s is the interstorey height.

2.2 Masonry buildings

For what concerns masonry buildings, formulae for the displacement capacity have been derived from simple structural mechanics principles. As for the reinforced concrete structures, the initial idea is to model a multi-degree of freedom system (MDOF) by means of a single-degree of freedom (SDOF) substitute structure. Masonry buildings are currently assumed to have a storey-sway response mechanism and in Figure 2.2 four different displacement profiles for different limit states and in-plane failure modes are shown. Profile (a) corresponds to the limit state for which none of the members of the structure has reached the yield displacement. Profile (b), (c) and (d) corresponds to significant structural damage and collapse. Profile (b) is the typical situation for low rise unreinforced buildings, profile (c) could also be possible depending on the relative strengths of the stories and profile (d) is possible depending on the relative strengths of spandrel beams and piers.

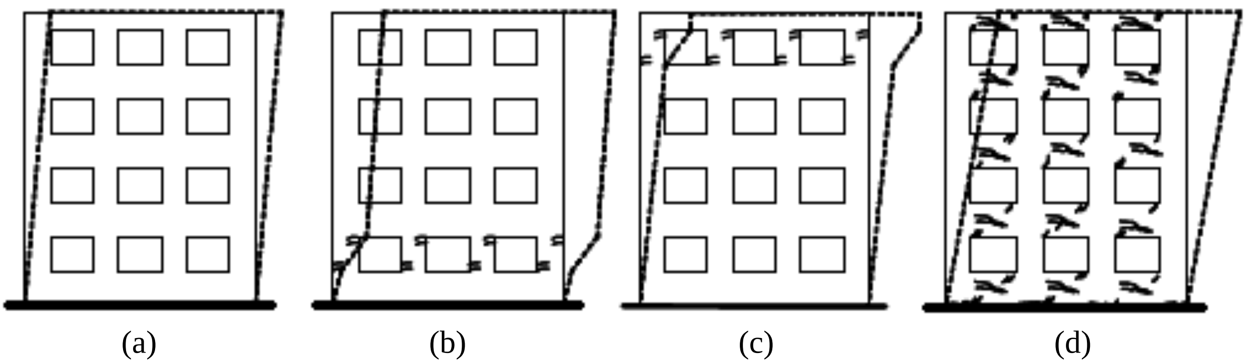


Figure 2.2: Deformed Shape for different limit states and in-plane failure modes (Restrepo-Velez and Magenes, 2004)

The formulae for the displacement capacity at the centre of seismic force of the storey-sway mechanism are given by the following equations:

$$\Delta S_y = \theta_y \kappa_1 H \qquad 2.10$$

$$\Delta S_{LSi} = \theta_y \kappa_1 H + \kappa_2 (\theta_{LS} - \theta_y) h_s \qquad 2.11$$

Where:

- θ_y is the yield rotation capacity;
- κ_1 is the effective height coefficient (to obtain the equivalent height of the deformed SDOF system);
- H is the height of the building;
- κ_2 is the effective height coefficient of the masonry piers;
- θ_{LS} is the second or third limit state rotation capacity;
- h_s is the pier height.

The coefficients κ_1 and κ_2 are based on how the structure deforms and they can be calculated for a given building if the mass distribution and the mechanism shape is known. Restrepo-Velez and Magenes (2004) suggested the values reported in Table 2.3.

Table 2.3:Values of κ_1 and κ_2 for each number of storeys (Restrepo-Velez and Magenes, 2004).

Number of stories	κ_1	κ_2
1	0.790	0.967
2	0.718	0.950
3	0.698	0.918
4	0.689	0.916
5	0.684	0.900
6	0.681	0.881

3. Typical European Building Data

3.1 Reinforced concrete buildings (bare frames)

In the following table the mean values used to compute the structural and non-structural capacity are shown together with the standard deviation value and the probabilistic distributions.

The geometrical properties derive from the study of Bal *et al*. (2007) and poor quality frame emergent beam buildings have been considered. For what concerns the yield strain of reinforcement steel, the values of two different types of steel have been considered which depend on the country of production. It has been possible to divide Europe into two main classes: the first class includes Spain, Portugal and Italy and it is characterized of a nominal yield strength of 415 MPa. The second class includes northeast European regions and it is characterized of a nominal yield strength of 500 MPa. In Table 3.1 the features of these two types of steel are shown. Using an elastic modulus of 210,000 MPa, a value of ε_y equal to 0.0020 and 0.0024 has been calculated respectively. A coefficient of variation between 10% and 15% has been used for the yield strain of the reinforcement steel. Finally the values of the ultimate concrete and steel strain have been taken equal to the values reported previously in Table 2.1 and a coefficient of variation of 30% has been considered.

Table 3.1: Features of the types of steel considered in the analyses

Parameter	Type 1 Value	Type 2 Value
f_{yk}, nominal yield strength (MPa)	415	500
f_{ym}, mean yield strength (MPa)	487	561.5
f_{tm}, mean tensile strength (MPa)	665.5	658

Table 3.2: Mean and standard deviation values together with the probabilistic distribution of the necessary parameters to compute the displacement capacity of a reinforced concrete building class

Parameter	Mean	Standard Deviation	Probabilistic Distribution
$\boldsymbol{\varepsilon}_{\mathbf{y}}$, yield strain of reinforcement steel (Type 1)	0.0020	0.00028	Normal
$\boldsymbol{\varepsilon}_{\mathbf{y}}$, yield strain of reinforcement steel (Type 2)	0.0024	0.00032	Normal
$\boldsymbol{\varepsilon}_{\mathbf{C(LSu)}}$, ultimate concrete strain	0.0075	0.00225	Normal
$\boldsymbol{\varepsilon}_{\mathbf{S(LSu)}}$, ultimate steel strain	0.035	0.0105	Normal
$\mathbf{l_b}$, length of the beam	3.37 m	1.28 m	Gamma
$\mathbf{h_b}$, depth of the beam section	0.60 m	0.096 m	Lognormal
$\mathbf{h_s}$ interstorey height	2.84 m	0.23 m	Lognormal
Ground floor height	3.23 m	0.48 m	Lognormal

3.2 Masonry buildings

A one-storey building has been considered in the analyses. In the following tables the mean values used to compute the structural capacity are shown together with the standard deviation values and the corresponding probabilistic distributions.

As described in Section 2.2, the κ_1 and κ_2 values are taken from Restrepo-Velez and Magenes (2004) and they are reported in Table 2.3. In Bal *et al.* (2008b) a study about the structural characteristics of Turkish RC building stock for loss assessment models has been presented and the statistics corresponding to masonry buildings have been used herein. Eurocode does not provide values of reference for the drift capacity. For this reason, the values proposed in the OPCM 3274 (OPCM, 2003) have been implemented. Different values according to the masonry type (reinforced and unreinforced) have been used. They are reported in Table 3.3 and Table 3.4.

Table 3.3: Mean and standard deviation values together with the probabilistic distribution of the necessary parameters to compute the displacement capacity of unreinforced masonry building class

Parameter	Mean	Standard Deviation	Probabilistic Distribution
θ_y yield rotation capacity	0.003	-	-
θ_{LSU} third limit state capacity rotation	0.004	-	-
κ_1 effective height coefficient	0.790	-	-
κ_2 effective height coefficient of the masonry pier	0.967	-	-
H is the height of the building	2.62 m	0.210 m	Lognormal
hs is the pier height	2.40 m	0.36 m	Normal

Table 3.4: Mean and standard deviation values together with the probabilistic distribution of the necessary parameters to compute the displacement capacity of a reinforced masonry building class

Parameter	Mean	Standard Deviation	Probabilistic Distribution
θ_y yield rotation capacity	0.004	-	-
θ_{LSU} third limit state capacity rotation	0.006	-	-
κ_1 effective height coefficient	0.790	-	-
κ_2 effective height coefficient of the masonry pier	0.967	-	-
H is the height of the building	2.62 m	0.210 m	Lognormal
hs is the pier height	2.40 m	0.36 m	Normal

4. Minimum Displacement Capacity of European Buildings

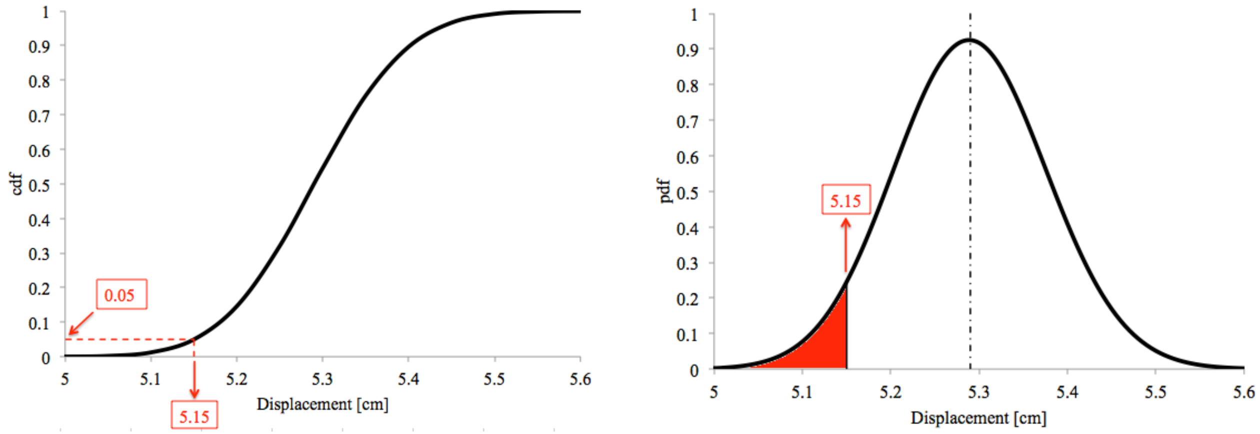
Due to the fact that it is of common use to define the characteristic strength of the materials using the 5th percentile, this percentile has been selected as the reference herein. This latter percentile has been chosen to represent the lower bound threshold capacity. In the following subsections, the probability density and cumulative distribution functions of the displacement capacity and the corresponding 5th percentiles are reported for the analyzed classes, assuming 1 storey throughout in order to obtain the lowest displacement capacity.

4.1 Reinforced concrete buildings (bare frame)

The threshold of non-structural damage is equal to 1.42 cm, irrespective of the type of steel used in the structures. Instead, the threshold corresponding to the ultimate limit state level is equal to 5.29 cm with a standard deviation of 0.0163 if a steel Type 1 is considered and it is equal to 5.36 cm with a standard deviation of 0.0153 if a steel Type 2 is taken into account. These values have been found using the equations described in Section 2.1 and the data reported in Section 3.1.

Once these two parameters are known, the probability density and the cumulative distribution functions can be developed. A population of 500 synthetic one storey height reinforced concrete buildings have been used to estimate the mean and the standard deviation of the displacement capacity and a lognormal distribution has proved to be the best fit. As mentioned before, the 5th percentile has been chosen as a reference value. In the following figures, the probability density and cumulative distribution functions are shown according to the type of steel and the values of the threshold corresponding to the ultimate limit state level are reported. In Table 4.1 and Table 4.2 a summary of the results are also shown. It has to be noted that the red area reported in the probability density function is equal to 5% of the total area under the distribution.

Type 1



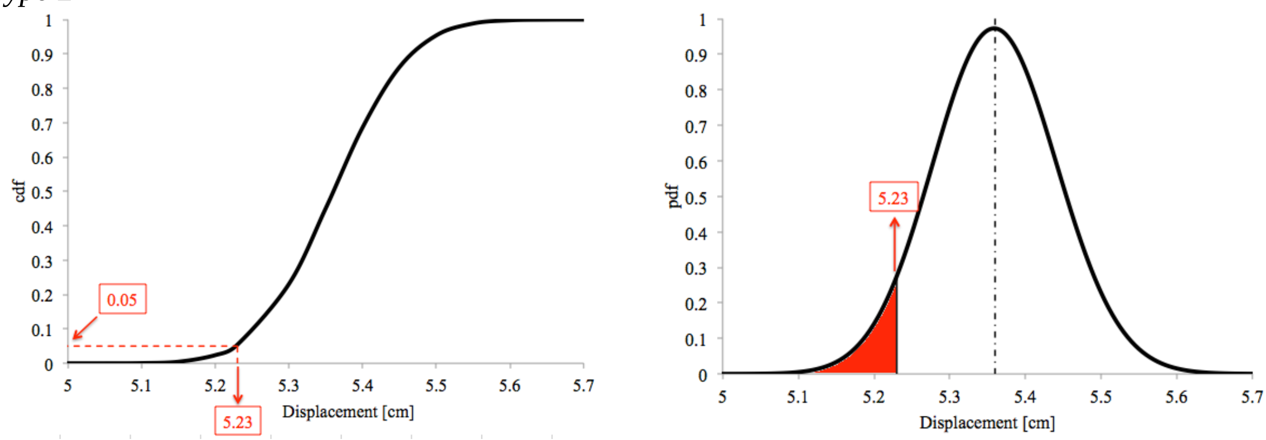
(a) (b)

Figure 4.1: (a) Cumulative distribution function and (b) probability density function of the displacement capacity of a reinforced concrete building class (Steel Type 1)

Table 4.1: Statistics of reinforced concrete building class with a steel of Type 1

Parameter	**Value**
Distribution	Lognormal
Mean	5.29 cm
Standard deviation	0.0163
5^{th} percentile	5.15 cm

Type 2



(a) (b)

Figure 4.2: (a) Cumulative distribution function and (b) probability density function of the displacement capacity of a reinforced concrete building class (Steel Type 2)

Table 4.2: Statistics of reinforced concrete building class with a steel of Type 2

Parameter	**Value**
Distribution	Lognormal
Mean	5.36 cm
Standard deviation	0.0153
5^{th} percentile	5.23 cm

4.2 Masonry buildings

The same procedure can be followed for masonry buildings. A population of 500 synthetic one storey high masonry buildings have been used to estimate the mean and the standard deviation of the displacement capacity and in both cases (reinforced and unreinforced masonry) a normal distribution has proved to be the best fit. The threshold corresponding to the ultimate limit state level is different according to the type of masonry analyzed. For what concerns an unreinforced masonry building class, a mean of 0.82 cm with a standard deviation of 0.00085 has been found using the equations described in Section 2.2 and the data reported in Section 3.2. With regards to reinforced masonry buildings, a mean of 1.25 cm with a standard deviation of 0.0012 has been estimated. Once these two parameters are known, the probability density and the cumulative distribution functions can be developed.

In the following figures, the probability density and cumulative distribution functions are shown according to the type of masonry and the values of the threshold corresponding to the ultimate limit state level are reported. In Table 4.3 and Table 4.4 a summary of the results are also shown.

Unreinforced Masonry

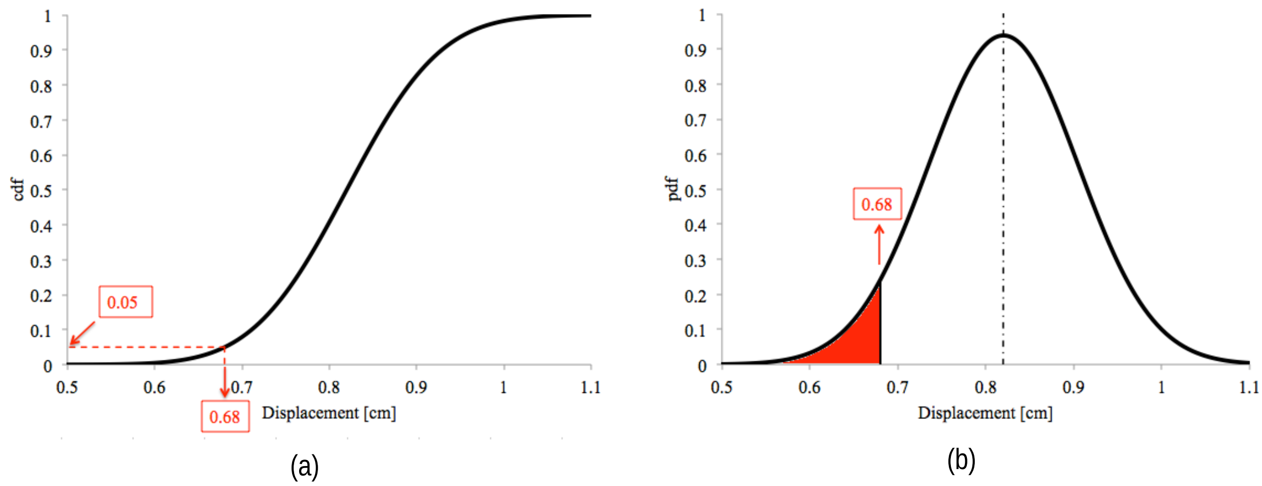


Figure 4.3: (a) Cumulative distribution function and (b) probability density function of the displacement capacity of an unreinforced masonry building class

Table 4.3: Statistics of unreinforced masonry building class

Parameter	**Value**
Distribution	Normal
Mean	0.82 cm
Standard deviation	0.00085
5^{th} percentile	0.68 cm

Reinforced Masonry

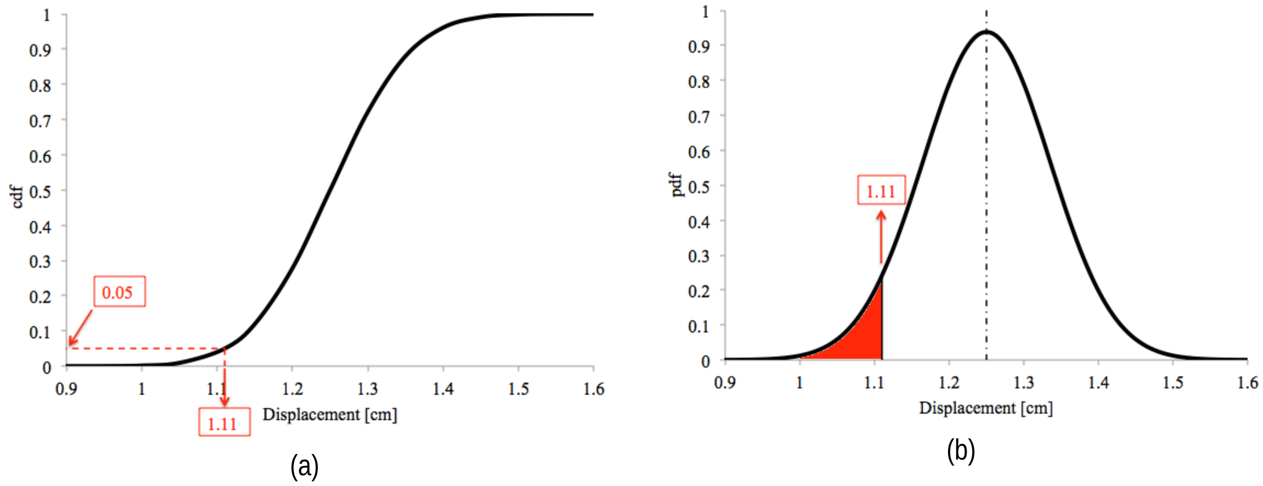


Figure 4.4: (a) Cumulative distribution function and (b) probability density function of the displacement capacity of reinforced masonry building class

Table 4.4: Statistics of reinforced masonry building class

Parameter	**Value**
Distribution	Normal
Mean	1.25 cm
Standard deviation	0.0012
5^{th} percentile	1.11 cm

5. Conclusions

Typical European buildings have been used in this project to estimate the minimum structural capacity levels below which the explicit definition of seismic hazard is not necessarily required. The study by Bal *et al*. (2008b) has been used as a reference for what concerns the building stock characteristics, and additional studies on steel types have been carried out as part of this deliverable. In their study they analyzed a significant number of existing buildings that have led to statistics about geometry and material properties. They provide geometrical data, in particular mean values and their corresponding coefficient of variation and probabilistic distribution.
Different thresholds depending on the structural types have been found herein. Two different types of steel have been used as a reference for Europe. For what concerns masonry buildings, two main classes have been considered as more representative for Europe, reinforced masonry and unreinforced masonry structures. Both the damage level and the ultimate state level are analyzed and the distributions of the displacement capacity have been provided.
Once the hazard maps for Europe are available, the values presented herein can be used to identify the areas of Europe where 95% of gravity load designed buildings will have sufficient capacity to withstand seismic actions and thus further hazard classification and zonation within these areas will not be necessary.

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