



**Seismic vulnerability analysis for masonry hospital structures:
expeditious and detailed methods**

Dissertation

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ABSTRACT

Italy is a territory characterized by a high seismic risk, which is a function of three main aspects: seismic hazard of the territory, vulnerability of the structure and exposure.

The assessment of the structural vulnerability for existing buildings is a key aspect for the seismic risk reduction, in particular for strategic and relevant buildings because of their importance for the civil protection.

There are different approaches for the evaluation of the structural vulnerability: expeditious empirical methods, based on qualitative evaluations, analytical methods, based instead on detailed models, and hybrid methods, combination of the first two methods.

In case of a large sample of buildings (as the Hospital structures of the cities of Florence, Prato and Pistoia, sample of analysis of the present work), a first screening in terms of vulnerability must be performed with an expeditious empirical approach, in order to highlight the most vulnerable objects and, as a consequence, to decide which buildings analyze at first with detailed analyses.

Since the economic resources of each Administration are limited, it is important to understand which are the most exposed to seismic risk buildings with a simple approach, in order to perform more detailed analyses in a second phase.

In this work, the statistical results of an empirical approach (Vulnerability Index Method - Benedetti and Petrini, 1984) will be illustrated for the considered sample of buildings, composed both from reinforced concrete and masonry structures.

Then, focusing on the masonry structures, a comparison among the results of the Vulnerability Index Method and the detailed analytical analyses (pushover analyses) on a subset of structures will be proposed, showing that there is a relation among them, which can be used in order to obtain more information about the behaviour of the structures: indeed, the empirical approach gives as result of the investigation only a Vulnerability Index in the range 0%-100%, where low values correspond to low vulnerable structures, while the detailed analyses, studying the behaviour of the building through structural models, can give some indicators of the seismic risk for the considered structure.

The comparison among the two typologies of approach allowed the construction of a new expeditious vulnerability assessment method, based on the Vulnerability Index Method form: this new method, simply using the same information of the Vulnerability Index Method, allows a first estimation of the global capacity of the structure, expressed in terms of PGA_c (peak ground acceleration of capacity), the related index of risk considering the seismic hazard of the site of construction and an indication on the reliability of this index, due to the possibility of activation of local mechanisms collapses.

A Hospital system must guarantee the life safety of the occupants for a strong seismic event, but it also has to ensure the operational feature even during low intensity seismic events: some considerations have been carried out from the structural point of view, using the results of the detailed analyses.

All these information can help the Administrations in the management of their building heritages, giving some suggestions in order to decide how to use the resources for further specific detailed analyses, which still remain necessary for the correct evaluation of the security of a structure.

CONTENTS

Index of Figures	VI
Index of Tables	IX
1 Introduction	1
1.1 General remarks	1
1.2 Definition of the argument of research	4
1.3 Overview of this work	5
2 Structural vulnerability: definition, state of art, methods for the estimation	7
2.1 Introduction	7
2.2 Seismic action	7
2.2.1 Macro-seismic scales and magnitude	7
2.2.2 Peak ground acceleration	7
2.2.3 Correlation among Macro-seismic scales and peak ground acceleration	8
2.3 Level of damage	8
2.3.1 General definition	8
2.3.2 Damage index - GNDT (1993)	9
2.3.3 Damage level: European Macro-seismic Scale EMS-98	10
2.4 Estimation of the vulnerability: expeditious and hybrid methods	11
2.4.1 Damage Probability Matrix (DPM)	11
2.4.2 European Macro-seismic Scale (EMS-98)	13
2.4.3 Vulnerability Index Method	14
2.4.4 Tuscany Region modifies on the Vulnerability Index Method	22
2.4.5 Other vulnerability forms	23
2.4.6 SAVE Project	24
2.4.7 Last developments on the fast vulnerability assessment	25
2.5 A correlation between seismic action, level of damage and vulnerability	26

3	The sample of analysis: Hospitals structures of Florence, Prato and Pistoia	29
3.1	Introduction	29
3.2	Health Organization of Tuscany Region	29
3.3	DICEA Research Group sample: Hospital structures of Florence, Prato and Pistoia	30
3.3.1	General features of the sample of buildings	30
3.3.2	Technological aspects	32
3.3.3	The methodology of survey	32
3.3.4	The vulnerability assessment: hypothesis	34
3.3.5	Results of the vulnerability campaign and data treatment	35
4	Masonry structures: relation among empirical and detailed vulnerability analyses	39
4.1	Introduction	39
4.2	Hypotheses of the work	41
4.2.1	Local or global analysis?	41
4.2.2	Level of knowledge of structures: materials, soil, topographic conditions	42
4.2.3	Types of global analysis: choice of the methodology	44
4.3	The levels of performance and the resilience concept	46
4.3.1	Introduction	46
4.3.2	Identification of the Structural Performance Levels (SPL)	47
4.3.3	Identification of the Non-structural Performance Levels (NPL)	48
4.3.4	The combination of SPL and NPL: the Target Building Performance Levels (TBPL)	50
4.3.5	Resilience	52
4.4	Typology of modeling: FME approach	53
4.5	Pushover analysis: construction of the capacity curve and treatment of the data	55
4.5.1	Typologies of analysis to perform	55
4.5.2	The creation of the pushover curve	56
4.5.3	From MDOF to SDOF: the bi-linearization of the curve	56
4.5.4	The parameters for the definition of vulnerability: analytical treatment of the curve	57
4.6	Static non-linear analysis: results from a case study	66
4.7	Results of the analytical approach and comparisons with the expeditious approach	70
4.7.1	Comparison among the I_v (11 parameters) and the PGA_c	70
4.7.2	Correspondence of modelling among the two methods	73
4.7.3	Comparison among the I_v (6 parameters) and the PGA_c	75
4.8	Conclusions of the comparison analysis (Life Safety TBPL)	78
4.9	Analysis of the Operational TBPL	79
4.9.1	Acceleration sensitive components	80
4.9.2	Deformation sensitive components	81
4.9.3	Conclusions on the Operational TBPL	82

5	Critical analysis of the Vulnerability Index Method for masonry structures	83
5.1	Introduction: weights of each parameter in the vulnerability estimation	83
5.2	General distributions of the judgments along the Vulnerability classification	84
5.3	Analysis of the distributions of the judgments for each parameter	86
5.4	Distribution of the judgments for each parameter along the vulnerability classification	89
5.5	Relation among the parameters	93
6	Development of a new expeditious Vulnerability assessment method	99
6.1	Introduction	99
6.2	Definition of the new parameters for the global capacity estimation	100
6.2.1	Introduction	100
6.2.2	From parameters 2 and 3 to a new continuous variable: the "lateral resistance indicator"	100
6.2.3	Parameters 5 and 9: study of their relation	102
6.2.4	Parameters 6 and 7	102
6.3	New expeditious vulnerability assessment method: estimation of the global PGA_C	102
6.3.1	Introduction	102
6.3.2	Choice of the scores for each judgment	103
6.3.3	Choice of the weights of each parameter and of the analytical procedure	103
6.4	Test of the method: application on some theoretical case studies	106
6.4.1	Three levels case study	106
6.4.2	Four levels case study	108
6.4.3	Five levels case study	108
6.5	Reliability of the global behaviour PGA_C estimation: local mechanisms influence	109
6.6	Estimation of the Index of Risk	111
6.6.1	Definition of the Index	111
6.6.2	Influence of the seismic hazard	111
6.6.3	Influence of the geotechnical conditions	112
6.6.4	Final consideration on the Index of Risk	113
7	Conclusions and outlooks	115
7.1	Introduction	115
7.2	Principal results of the work	115
7.3	Outlooks	116
7.4	Final considerations	116
8	Bibliography	117

9	Annex A: descriptions and results for the analyzed structures	123
9.1	AUSL 3 SMP 01 03	124
9.2	AUSL 4 MD 01 24	125
9.3	AOUC CAR 4 01	126
9.4	USL 10 IOT ANT 01	127
9.5	AOUC CAR 16 01	128
9.6	AUSL 3 PES 01 06	129
9.7	USL 10 SERR 01	130
9.8	AOUC CAR 13 04	131
9.9	AOUC CAR 13 03	132
9.10	AOUC CAR 26 02	133
9.11	AUSL 3 PES 01 04	134
9.12	AOUC CAR 8b 03	135
9.13	AOUC CAR 8b 04	136
9.14	AOUC CAR 26 03	137
9.15	AOUC CAR 8b 01	138
9.16	AOUC CAR 4 02	139
9.17	AOUC CAR 8b 02	140
9.18	AOUC CAR 13 01	141
9.19	AOUC CAR 26 01	142
9.20	AUSL 3 SMP 01 04	143

Index of Figures

Figure 1: Messina after the earthquake. The Dome (left) and the Hospital (right).....	1
Figure 2: Avezzano after the earthquake. Soldiers helping the community (left) and a destroyed Church (right).....	1
Figure 3: Collapses in two villages of Friuli after the earthquake.....	2
Figure 4: Collapses in the Irpinia territory.....	2
Figure 5: The Cathedral of Foligno before the collapse of the Bell-Tower (left), the Historical Tower of Nocera Umbra (middle) and a particular collapse for a restored building with a reinforced concrete roof (right).	2
Figure 6: School of San Giuliano di Puglia. 27 children and their teacher died due to the collapse of the structure.....	2
Figure 7: Student-House in L'Aquila (left) and village of Castelnuovo (aerial view and historical city centre).....	3
Figure 8: Comparison among the correlations Macro-seismic Intensity - PGA.....	8
Figure 9: Section 8 of Vulnerability assessment form - GNDT (1993).	9
Figure 10: Definition of the levels of damage for masonry structures (left) and reinforced concrete structures (right) - EMS-98.....	10
Figure 11: DPM for the area hit by VII° level of MM scale of San Fernando earthquake (Whitman, 1973).	11
Figure 12: Distributions of the level of damage: comparison among the type of material (steel VS reinforced concrete).....	12
Figure 13: Distributions of the level of damage (r.c. structures): comparison among the age of construction (pre 1933 VS post 1947). 12	12
Figure 14: Vulnerability classes for the different typologies of structures (EMS-98).....	13
Figure 15: 11 parameters Vulnerability Form for masonry structures.	18
Figure 16: 11 parameters Vulnerability Form for reinforced concrete structures.	21
Figure 17: Aerial view of Castelnuovo (AQ) after the 06/04/2009 earthquake. In red, the locations of two collapsed heavy roofs.	22
Figure 18: Two collapsed heavy roofs in Castelnuovo (AQ).	22
Figure 19: Probabilistic correlation law among acceleration and damage (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I). ..	26
Figure 20: Deterministic law among acceleration and damage (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I).....	26
Figure 21: Tri-linear acceleration/damage law (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I).	27
Figure 22: Model of Vulnerability for masonry structures. Relation among damage, acceleration and l_v (Guagenti and Petrini, 1989). 27	27
Figure 23: Comparison in terms of acceleration among the two models of vulnerability.	28
Figure 24: Period of construction of the sample of buildings.	30
Figure 25: Period of construction of the buildings separately analyzed for masonry and reinforced concrete structures.	31
Figure 26: Distribution of the volume of the sample of buildings (the dashed line represents the cumulative curve).	31
Figure 27: Distributions of the volume for masonry and r.c. structures (the dashed lines represent the cumulative curves).....	31
Figure 28: Example of individuation of the three S.U. in the Complex AOUC Careggi - 4.	33
Figure 29: Extract from the photographic documentation of AOUC Careggi-4, U.S.1.....	33
Figure 30: Example of plan with points of view of the photographic documentation (AOUC Careggi-4, U.S.1, ground floor).	34
Figure 31: Distribution of the l_v for the whole sample of buildings (the dashed line represents the cumulative curve).	35
Figure 32: Distributions of the l_v for masonry and reinforced concrete structures.	35
Figure 33: Comparison among the cumulative curves of the l_v distributions for masonry and r.c. structures.	36
Figure 34: Absolute distribution of Vulnerability in the different periods of construction.....	37
Figure 35: Distribution of vulnerability ante and post 1962 (the dashed lines represent the cumulative curves).	37
Figure 36: Distribution of vulnerability ante and post 1975 (the dashed lines represent the cumulative curves).	38
Figure 37: l_v of the subset of masonry S.U. The dashed lines represent the boundaries of the entire sample of masonry structures... 40	40
Figure 38: On the left: out of plane behaviour of a wall. On the right: in plane behaviour. (Touliatos, 1996).....	41
Figure 39: Typologies of out of plane mechanisms (www.reluis.it).....	41
Figure 40: Representation of the box behaviour. (Touliatos, 1996).....	41
Figure 41: In-plane collapse of masonry walls. From the left: bending, sliding and diagonal cracking.	42

Figure 42: Typologies of masonry and roofs directly observable.	43
Figure 43: Thermal images: (left) stone masonry with brick layers, (middle) reinforced concrete floor, (right) brick masonry.	43
Figure 44: Relationship among the capacity curve (black line) and dynamic hysteretic cycles (grey lines).	45
Figure 45: Damages to the San Fernando Veterans Administration Hospital, built in 1926, before the development of earthquake building codes (left). Damages to the Medical Treatment and Care Building (Olive View Hospital complex), completed in 1970: the first storey has nearly collapsed and the towers containing the stairs (structurally separated from the building) have overturned (right).	46
Figure 46: Structural Performance Levels and Damage - Vertical elements - F.E.M.A. 356 (2000).	47
Figure 47: Structural Performance Levels and Damage - Horizontal elements - F.E.M.A. 356 (2000).	48
Figure 48: Non-structural Performance Levels and Damage - Architectural Components - F.E.M.A. 356 (2000).	49
Figure 49: Non-structural Performance Levels and Damage - Contents - F.E.M.A. 356 (2000).	49
Figure 50: Non-structural Performance Levels and Damage - Systems/Components - F.E.M.A. 356 (2000).	50
Figure 51: Target Building Performance Levels (TBPL) and Ranges - F.E.M.A. 356 (2000).	50
Figure 52: Description of the four TBPL and qualitative identification on a general "displacement - total base shear" curve.	51
Figure 53: Graphical definition of resilience.	52
Figure 54: On the left, diagonal cracking; on the right, bending cracking. (Theoretical manual of 3MURI)	53
Figure 55: Equivalent frame structure for a two levels wall with four openings (doors).	54
Figure 56: Description of the macro-element. (Theoretical manual of 3MURI)	54
Figure 57: Elastic - perfectly plastic behaviour of a general macro-element. (Theoretical manual of 3MURI)	54
Figure 58: Difference of behaviour among rigid floors (left) and flexible ones (right). (Theoretical manual of 3MURI)	55
Figure 59: Example of lateral load profiles for a 3 storey building: uniform distribution (left) and 1 st modal shape distribution (right).	55
Figure 60: Example of pushover curve with the bi-linear schematization.	58
Figure 61: Equivalent elastic system for flexible systems on the left ($T^* > T_C$) and rigid systems on the right ($T^* \leq T_C$).	59
Figure 62: Research of the PGA of capacity once that period and spectral acceleration of capacity are known.	61
Figure 63: Variability of the parameter a_g [g] with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).	63
Figure 64: Variability of the parameter F_0 with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).	63
Figure 65: Variability of the parameter T^*_c with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).	63
Figure 66: Visual resume of the concepts of resilience, yielding and life safety accelerations.	65
Figure 67: Planimetric and three-dimensional view of the model of AOUC-CAR-4 01.	66
Figure 68: Pushover curves for the X and Y directions - AOUC-CAR-4 01.	67
Figure 69: Three-dimensional view of the model without the indication of the floors - AOUC-CAR-4 01.	68
Figure 70: Worst analyses in the two main directions - AOUC-CAR-4 01.	68
Figure 71: Graphical convention for the identification of the level of damage on piers and spandrels.	68
Figure 72: Deformed plan view and a significant deformed facade for the analysis 3 - AOUC-CAR-4 01.	69
Figure 73: Deformed plan view and a significant deformed facade for the analysis 5 - AOUC-CAR-4 01.	69
Figure 74: Comparison among the classifications obtained with the expeditious method (I_v - 11 parameters) and the detailed one.	71
Figure 75: Relation among $I_{v,11}$ parameters and PGA_C	71
Figure 76: Relation among $I_{v,11}$ parameters and PGA_C - zoom.	72
Figure 77: Comparison among the classifications obtained with the expeditious method (I_v - 6 parameters) and the detailed method.	76
Figure 78: Relation among $I_{v,6}$ parameters and PGA_C	77
Figure 79: Relation among $I_{v,6}$ parameters and PGA_C - zoom.	77
Figure 80: Relation among $PGA_{C,Life\ Safety}$ and $PGA_{C,Operational}$ TBPL.	80
Figure 81: Maximum interstorey drift for the Operational TBPL along the Vulnerability classification.	81
Figure 82: Distribution of the Index of Vulnerability for masonry structures - 118 buildings are analyzed in this graph.	84
Figure 83: Distribution of all the judgments assigned in the 118 Vulnerability Forms: the statistic is based on 1298 judgments.	84

Figure 84: Distribution of the "A" judgments.....	85
Figure 85: Distribution of the "B" judgments.....	85
Figure 86: Distribution of the "C" judgments.	85
Figure 87: Distribution of the "D" judgments.	86
Figure 88: Distribution of the judgments on the 11 parameters. For each parameter, the sum of the percentages gives 100%.	86
Figure 89: Distribution of the judgments in the Parameter 1 along the Vulnerability classification.....	90
Figure 90: Distribution of the judgments in the Parameter 2 along the Vulnerability classification.....	90
Figure 91: Distribution of the judgments in the Parameter 3 along the Vulnerability classification.....	90
Figure 92: Distribution of the judgments in the Parameter 4 along the Vulnerability classification.....	91
Figure 93: Distribution of the judgments in the Parameter 5 along the Vulnerability classification.....	91
Figure 94: Distribution of the judgments in the Parameter 6 along the Vulnerability classification.....	91
Figure 95: Distribution of the judgments in the Parameter 7 along the Vulnerability classification.....	92
Figure 96: Distribution of the judgments in the Parameter 8 along the Vulnerability classification.....	92
Figure 97: Distribution of the judgments in the Parameter 9 along the Vulnerability classification.....	92
Figure 98: Distribution of the judgments in the Parameter 10 along the Vulnerability classification.....	92
Figure 99: Distribution of the judgments in the Parameter 11 along the Vulnerability classification.....	93
Figure 100: Bubble graphs for high correlation index couples of parameters.....	96
Figure 101: Bubble graphs for low correlation index couples of parameters.	97
Figure 102: Bubble graph for the couple of parameters 2-3.	97
Figure 103: Relation among the lateral resistance indicator and the PGAc.....	101
Figure 104: Comparison among the detailed analyses and the estimation performed with the multi-linear regression.....	105
Figure 105: Contribution of each variable in the multi-linear regression.	106
Figure 106: Plan and 3d view of the three levels case study.	106
Figure 107: Plan and 3d view of the four levels case study.	108
Figure 108: Plan and 3d view of the four levels case study.	108

Index of Tables

Table 1: Scores for the 11 parameters Vulnerability Form for masonry structures.....	17
Table 2: Scores for the 11 parameters Vulnerability Form for reinforced concrete structures.....	20
Table 3: List of the subset of masonry S.U. chosen for the detailed analyses. The list is ordered by the I_v	40
Table 4: Typologies of analysis considered for each structure.	56
Table 5: General data for the 8 performed analyses - AOUC-CAR-4 01.	66
Table 6: Results for the Operational Target Building Performance Level - AOUC-CAR-4 01.	67
Table 7: Results for the Operational Target Building Performance Level - AOUC-CAR-4 01.	67
Table 8: Results of the detailed analyses for the chosen subset of masonry S.U.. The list is ordered by the I_v	70
Table 9: Estimation of the PGA_C using the trend line $I_{v,11parameters} - PGA_C$. Measure of the errors comparing the results with the detailed analyses.....	72
Table 10: Parameters considered in the detailed approach.....	75
Table 11: Index of Vulnerability calculated with 6 parameters, in order to perform a direct comparison with the results of the detailed analyses.....	75
Table 12: Comparison among the classifications of the expeditious and detailed method. The list is ordered by the new values of I_v (6 parameters).	76
Table 13: Estimation of the PGA_C using the trend line $I_{v,6parameters} - PGA_C$. Measure of the errors comparing the results with the detailed analyses.....	78
Table 14: Resume of the results for the Operational TBPL.	79
Table 15: Maximum influence of each parameter in the vulnerability estimation through the Vulnerability Index Method.....	83
Table 16: Influence of each parameter in the estimation of the vulnerability related to the global behaviour.	83
Table 17: Matrix of correlation: indexes of correlation for each couple of parameters.	95
Table 18: Calculation of the "lateral resistance indicator" $\alpha_0 \cdot \tau k$	101
Table 19: Multi-linear regression for the estimation of the PGA_C	104
Table 20: Three levels case study: test of the method.....	107
Table 21: Four levels case study: test of the method.....	108
Table 22: Five levels case study: test of the method.	108
Table 23: Calculation of the "Reliability Index" RI	110
Table 24: Parameters for the definition of the seismic hazard in the sites of construction of each S.U.	112
Table 25: Seismic risk classification.....	114

1 Introduction

In this introduction, the general aspects of the argument of research are proposed. After some general remarks, the main topic of research is defined, identifying the principal targets.

1.1 General remarks

The Italian territory is characterized by a considerable seismic activity, due to its geographical position, located in the area of convergence between the Eurasian (Europe and Asia) and African lithospheric plates.

The frequency of the seismic events, which historically have affected the country, and the intensity of some of them, led the Scientific Community to pay more attention to this phenomenon in recent years.

In addition to this, it is important to remind that many existing structures of the Italian building heritage are not properly designed against seismic actions, since this approach has been introduced throughout the country only in recent years.

Considering the aspects explained above, it is possible to affirm that Italy is a nation with a relevant *seismic risk*. This sentence can be easily confirmed just looking the following list of some of the strongest earthquakes which have occurred in the Italian territory in the last century (information and images from <http://www.protezionecivile.gov.it/> - Department of National Civil Protection). For each event, the place, date and time is reported, together with the number of victims and the intensity of the earthquake measured on the Mercalli Cancani Sieberg scale (MCS), which is a scale that measures the intensity of the seismic event by means of the effects that the event produces on persons and objects.

Reggio Calabria and Messina earthquake, 28/12/1908, h. 5.20 - 86000 casualties - Intensity: XI degree (MCS)



Figure 1: Messina after the earthquake. The Dome (left) and the Hospital (right).

Avezzano earthquake, 13/01/1915, h. 7.53 - 30000 casualties - Intensity: XI degree (MCS)



Figure 2: Avezzano after the earthquake. Soldiers helping the community (left) and a destroyed Church (right).

Friuli Region earthquake, 06/05/1976, h. 21.00 - 965 casualties - Intensity: IX-X degree (MCS)

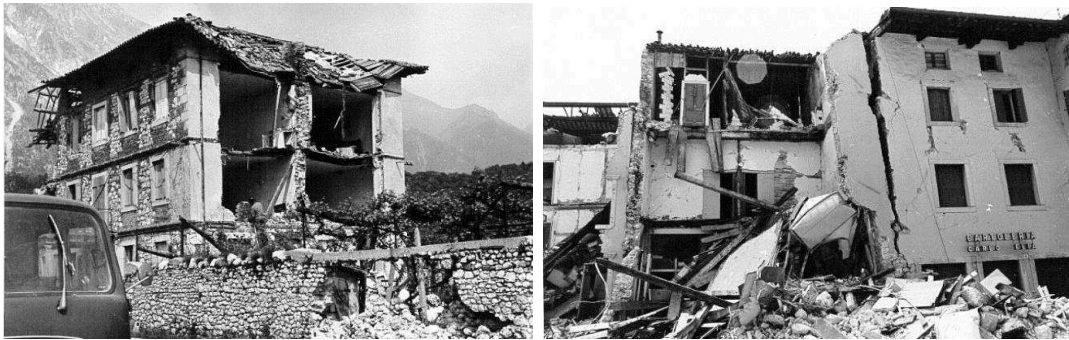


Figure 3: Collapses in two villages of Friuli after the earthquake.

Irpinia Area earthquake, 23/11/1980, h. 19.35 - 2734 casualties - Intensity: X degree (MCS)



Figure 4: Collapses in the Irpinia territory.

Umbria and Marche Region earthquake, 26/09/1997, h. 11.40 - 11 casualties - Intensity: VIII-IX degree (MCS)



Figure 5: The Cathedral of Foligno before the collapse of the Bell-Tower (left), the Historical Tower of Nocera Umbra (middle) and a particular collapse for a restored building with a reinforced concrete roof (right).

Molise Region earthquake, 31/10/2002, h. 11.32 - 30 casualties - Intensity: VII-VIII degree (MCS)



Figure 6: School of San Giuliano di Puglia. 27 children and their teacher died due to the collapse of the structure.

Abruzzo Region earthquake, 06/04/2009, h. 3.32 - 308 casualties - Intensity: IX-X degree (MCS)



Figure 7: Student-House in L'Aquila (left) and village of Castelnuovo (aerial view and historical city centre).

Looking the pictures proposed before, it is possible to observe that earthquakes have destroyed monuments, historical city centres, buildings with public functions (hospitals, schools...) and, in general, structures with high presence of persons (such as the Student-House in L'Aquila - 2009).

The approach to the argument of the seismic risk requires the definition of some elements, in order to identify and quantify the variables which play a significant role in this field.

What is the seismic risk?

From the Scientific point of view, the *seismic risk* is a function of three main aspects:

- the *seismic hazard of the territory*, which expresses the probability that, in a given area and in a certain period of time, an earthquake of a given intensity occurs;
- the *vulnerability of the structure*, which represents the trend of the structure to suffer damage due to a seismic event of a given intensity;
- the *exposure*, which measures the quality and quantity of the "elements" exposed to the risk (in terms of human lives, economic resources, artistic or cultural ones ...).

All of the three elements listed above give their contribution in the definition of the level of risk and they must be considered in a correct seismic analysis, both for a new construction design process and for the evaluation of the level of safety of an existing one.

As written before, many existing structures of the Italian building heritage are not properly designed against seismic actions, since this approach has been introduced throughout the Country only in recent years.

From the engineering point of view, the assessment of the structural vulnerability for existing buildings is therefore a key aspect for the seismic risk reduction.

1.2 Definition of the argument of research

In order to reduce the seismic risk for the Italian building heritage, it is important to invest energies in the vulnerability assessment, trying to highlight the objects which show the most critical conditions from the structural and non-structural point of view in case of a seismic event.

The evaluation of the vulnerability has been an argument of research since the 70's: the first methods have been developed by the observation of the damages of the past earthquakes, in order to understand the main causes of the collapses. Starting from this, many methods have been developed during the last decades, in order to give more detailed procedures of evaluation and more reliable results, considering the effects of the earthquakes occurred in recent times.

There are different approaches for the evaluation of the structural vulnerability; a first subdivision can be performed considering the level of detail of the typology of analysis and the dimension of the sample of buildings which represents the object of the vulnerability evaluation:

- *expeditious empirical methods*, mainly based on qualitative evaluations, necessary in case of the seismic vulnerability analysis of a large number of buildings, for which a detailed survey campaign can be prohibitively expensive; this approach is often used for analysis on a territorial scale;
- *analytical/mechanical methods*, characterized by a direct physical meaning; they allow the study of the seismic vulnerability through analyses of the mechanical behaviour of the considered structure, using models with different levels of complexity;
- *hybrid methods*, which are a combination of the two methods described above.

Usually, the use of expeditious methods represents the first step in case of the analysis of a considerable group of buildings, in order to decide which are the structures (within the considered sample) that require more detailed analyses at first by means of mechanical models; only after the detailed vulnerability assessment phase, it is possible to decide to design retrofitting interventions for the analyzed buildings, in order to reduce the seismic risk.

It is important to remind that empirical methods are useful for a comparative purpose among the structures of the considered sample, but they are not able to give complete information on the level of vulnerability, since they are based on qualitative evaluations.

The field of the empirical methods is a current topic of research in Seismic Engineering: the necessity of the seismic risk evaluation for the built environment throughout the Italian territory makes this argument very actual and interesting for the Scientific Community. The present work tries to give a contribution for this category of methods, using a real case of study.

1.3 Overview of this work

The thesis is organized in 7 chapters and 1 Annex; for each of them, a brief description is here proposed, in order to better understand the structure of the work.

- CHAPTER 1. It is the current introduction of the work, containing the general remarks of the topic.

- CHAPTER 2. In this chapter, the state of art is proposed: in particular, the concepts related to the definition of the seismic vulnerability are described; since the main topic of the work is the comparison among expeditious and detailed methods for the vulnerability estimation, the review of a selection of the most common expeditious methods for the vulnerability assessment is proposed.

- CHAPTER 3. In this chapter, the sample of analysis which represents the basis of this work is described: the DICEA-UNIFI Research Group (Department of Civil and Environmental Engineering of the University of Florence) has investigated, by means of an expeditious method (Vulnerability Index Method), the Hospital buildings of the cities of Florence, Prato and Pistoia during the period 2010-2011. The sample is composed both by reinforced concrete (r.c.) and masonry structures, for a total of 219 elements: 118 units made of masonry and 101 made of reinforced concrete.
The method of survey adopted by the DICEA-UNIFI Research Group is described, as well as the documents produced for each analyzed structure in order to give a report of the performed activity.
The results obtained with the survey campaign are statistically treated in this chapter, in order to obtain general information about the considered sample of buildings: some general conclusions about the vulnerability of the buildings have been found.

- CHAPTER 4. This chapter represents the heart of the thesis: from this point, only masonry structures will be considered, in order to study in detail the expeditious method used in the Chapter 3.
The levels of performance for Hospital structures have been studied, considering the evolution of the scientific developments on the argument: this aspect was necessary in order to define the limit conditions to consider for the evaluation of the vulnerability of this family of buildings.
In order to study in detail the expeditious vulnerability assessment method and its relation with the classic detailed analyses procedures, a subset of buildings has been considered (20 constructions on 118); on these structures, detailed structural analyses have been performed, using the static non linear analysis approach (pushover analysis) for the estimation of the global vulnerability, expressed in terms of peak ground acceleration of capacity, PGA_c .
In this chapter, the analytical procedure has been described and a case study of the subset has been proposed; then, the comparison among the two typologies of analysis has been performed, observing the relation between the results of the expeditious method (I_V) and the detailed approach (PGA_c), at first comparing the methods as they are conceived and later focusing on some particular aspects (considering only the structural global parameters of the Vulnerability Index Method).
The results show that there is a relation among the two approaches. Consequently, the idea of an improvement of the expeditious method has been considered, in order to obtain more information on the vulnerability aspects using the same typologies of data required for the Vulnerability Index Method.

- CHAPTER 5. Before starting to work on the new expeditious vulnerability assessment method, a critical analysis of the original method (Vulnerability Index Method) has been performed, using all the data collected during the survey of the masonry structures described in Chapter 3. These analyses have highlighted advantages and disadvantages of the considered method.
- CHAPTER 6. Considering the results of Chapter 4, where a certain level of correlation has been found among expeditious and detailed analyses, and considering the critical points highlighted in Chapter 5, a new expeditious method has been proposed, in order to estimate the global capacity of the structure (in terms of PGA_c) simply using the same data that must be collected for the Vulnerability Index method. In particular, the layout of the method remains the same as the original one, leading to more detailed results.
- CHAPTER 7. Conclusions and outlooks are illustrated: the final product of the work is described, highlighting advantages which it brings in the seismic vulnerability estimation field; also the limits of the method are mentioned, which are due to its simplified nature.
- ANNEX A. For each structure of the subset of buildings analyzed with the detailed approach, a brief description is proposed, together with the calculation of the Index of Vulnerability, the graphs of the pushover curves performed for the detailed approach and the related value of the estimated capacity, expressed in terms of peak ground acceleration.

2 Structural vulnerability: definition, state of art, methods for the estimation

In this chapter, the principal concepts for the definition of the structural vulnerability are proposed. Then, a general overview on the methods for the evaluation of the vulnerability is proposed, describing the evolution of the empirical methods and focusing on the one which has been used in the present work.

2.1 Introduction

The evaluation of the seismic vulnerability of buildings can be assessed with several methods.

But what is *vulnerability*?

Seismic vulnerability of a building is a characteristic related to the structural features of the building itself; it expresses the trend of a structure to be damaged when a given seismic event occurs.

In other words, it is possible to affirm that vulnerability is a behavioural aspect which describes a cause-effect relation, where the cause is the earthquake and the effect is the damage. It is therefore necessary to find out the correlation between the *seismic action* and the *level of damage*.

2.2 Seismic action

2.2.1 Macro-seismic scales and magnitude

At first, Macro-scale Intensity has been used to identify the seismic action through its effects: there are several Macro-seismic Scales, like MCS (Mercalli - Cancani - Sieberg), MM (modified Mercalli), MSK (Medvedev-Sponheuer-Karnik), etc.... All of them are based on a certain number of defined levels (MCS - 12 levels for example) which describe the effects of an earthquake on the built environment.

The most used scale in Europe is the MCS, composed by 12 levels: from the I level - "very light event", with almost no perception of the earthquake, until the XII level - "heavy catastrophic event", where every built object collapses.

Another way to measure the seismic action of an earthquake is the quantification of its magnitude, which represents an indirect measure of the mechanical energy released by the seismic event in the hypocenter: the most famous method is the Richter Magnitude (also called Local Magnitude), based on the logarithm of the measure of the maximum displacement of a seismograph, compared with the displacement of a conventional reference earthquake (1micrometer registered at a distance of 100 km from the epicentre, considered as the representative displacement for an event with zero magnitude). This scale is independent from the built environment.

2.2.2 Peak ground acceleration

After the Intensity scales, the seismologists have introduced the concept of the ground acceleration, parameter which can be directly measured nowadays through a net of instruments (RAN - "Rete Accelerometrica Nazionale"), placed on the whole Italian territory.

This parameter has not replaced the empirical scales of Intensity described before, since the Intensity of an earthquake does not depend only on the values of acceleration of the ground: in fact, the effects that a seismic event produces on the built environment are not fully describable only by means of the horizontal shaking of the ground, since many other parameters are involved in the response (the features of the buildings, the frequency of shaking, the duration of the event...).

It is anyway recognized that the Macro-seismic Intensity depends on the ground acceleration.

2.2.3 Correlation among Macro-seismic scales and peak ground acceleration

Many are the attempts performed in order to correlate the Intensity of an earthquake with the peak ground acceleration: some correlation laws have been obtained by different authors, considering different Macro-seismic Intensity Scales and different seismic databases of information collected during the past seismic events. The structure of the relation is basically the same: the Intensity is assumed to be proportional to the logarithm (sometimes base-10 log, other ones natural log) of the ground acceleration through the equation:

$$\ln(y) = aI - b$$

y peak ground acceleration (normalized to gravity acceleration)

I macro-seismic intensity

a, b parameters empirically obtained from the analysis of the past events

Just to give an idea of the variability of the relations found by the different authors, some of them are here reported, considering the macro-seismic intensity measured on the MCS scale.

Guagenti - Petrini (1989)

$$\ln(y) = 0.602 \cdot I - 7.073$$

Margottini et al. (1992)

$$\log(y) = 0.220 \cdot I + 0.525$$

Faccioli - Cauzzi (2006)

$$\log(y) = 0.200 \cdot I - 1.330$$

It is possible to plot the graph of these correlations (Figure 8), using the macro-seismic intensity (measured on the MCS Scale) as independent variable and the resulting peak ground acceleration as dependent one, without using a logarithmic scale in order to perform a direct comparison among the correlations.

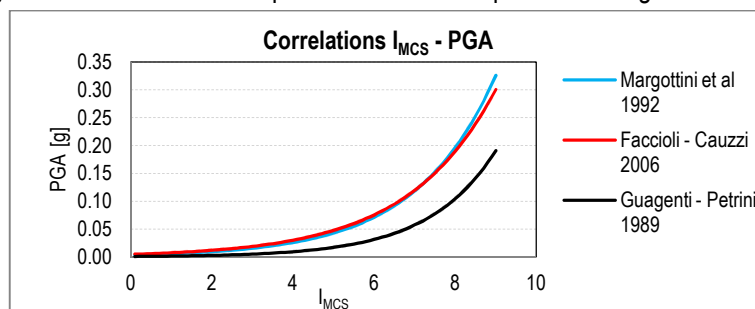


Figure 8: Comparison among the correlations Macro-seismic Intensity - PGA.

The graph shows that the oldest correlation (Guagenti - Petrini) leads to lower values of PGA, while the other two correlations (Margottini et al and Faccioli - Cauzzi) are more similar. More details about this argument are provided in the Deliverable 11 of the project INGV-DPC 2004-2006 (Gómez Capera A.A., Albarello D., Gasperini P. - 2007), where many other correlations are listed and compared.

All these relations have been developed because, from the engineering point of view, the expression of a seismic event in terms of ground acceleration is more suitable than an Intensity measure, in order to perform analyses of seismic risk.

2.3 Level of damage

2.3.1 General definition

It is a measure of the damage that the structure has experienced after a certain seismic event.

It can be generally defined as the ratio between the reparation cost of the observed damage and the entire reconstruction cost of the building. This definition has the advantage to define levels of damage as a continuous variable in the 0 - 1 range; however, it is dependent on the characteristics of the local building market and related technologies in a certain period and in the considered geographic area.

2.3.2 Damage index - GNDT (1993)

Another possibility for the quantification of the damage level is the calculation of a damage index by means of a form: in the work of Angeletti (1984) and Angeletti et al. (1988), a method for the survey of the damage after a seismic event has been presented and developed within the GNDT ("Gruppo Nazionale di Difesa dai Terremoti" - National Group for the Defence from Earthquakes) vulnerability assessment project.

In this method, a form for the vulnerability and damage assessment has been developed: "1° Level assessment Form for the survey of the exposure and vulnerability of the buildings" - GNDT (1993).

In this form, there is a specific section prepared for this purpose: *extension and level of damage* (section 8).

Section 8 - EXTENSION AND LEVEL OF DAMAGE

Event date ³⁰¹

1 earthquake ³⁰⁷
 2 others

Damage level

No damage	A
Light damage	B
Medium damage	C
Heavy damage	D
Very heavy damage	E
Total damage	F

Extension of the damage

E ≤ 10 %	0
10 < E ≤ 20 %	1
20 < E ≤ 30 %	2
30 < E ≤ 40 %	3
40 < E ≤ 50 %	4
50 < E ≤ 60 %	5
60 < E ≤ 70 %	6
70 < E ≤ 80 %	7
80 < E ≤ 90 %	8
90 < E	9

Installations damage 1 yes 2 no ³⁸⁸

Vertical Structures

M	E	L	N°
308			
312			
316			
320			
324			

Horizontal Structures

M	E	L	N°
328			
332			
336			
340			
344			

Stairs

M	E	L	N°
348			
352			
356			
360			
364			

Partition walls

M	E	L	N°
368			
372			
376			
380			
384			

Figure 9: Section 8 of Vulnerability assessment form - GNDT (1993).

As shown in Figure 9, there are four matrices, separately prepared for vertical structural elements, horizontal elements, stairs and partition walls. For each of them, it is necessary to quantify the level of damage with the following procedure, filling the matrices with this information:

- in the column "N", the number of floors with the damage described as written in that row (if the description of the damage is common for all the structure, it is enough to write only one row, the first one, putting the total number of floors in the column "N");
- in the column "M", the maximum level of damage observed in the structure, without any additional indication about the extension of this damage; the level of damage is described with a letter (from A - no damage, to F - total damage; see the definition of the damage level in the section);
- in the column "L", the most extended level of damage observed in the structure, in the same way as column "M";
- in the column "E", the extension of the damage (from 0 - Extension ≤10%, to 9 - Extension >90%); this column is referred to the most extended level of damage (column "L"). The sum of the percentages of this column must be equal to 100%.

For each level of damage, a corresponding numerical value is associated:

A → 0.00, B → 0.20, C → 0.40, D → 0.60, E → 0.80, F → 1.00.

For each of the four analyzed components and for each level, maximum and most extended damage are combined with the following expression:

$$d_{ij} = e \cdot d_e + (1 - e) \cdot d_m / 3$$

- i* analyzed component
- j* analyzed level
- d_e* most extended level of damage
- e* extension of most extended level of damage (as percentage)
- d_m* maximum level of damage observed.

The level of damage for the whole building is defined as a weighted average value:

$$d = \sum_{ij} S_i \cdot F_j \cdot d_{ij}$$

S_i economic ratio between the observed component and the entire building

F_j ratio between the volume of the observed storey and the entire volume of the building.

In this way, it is possible to calculate an index of damage which keeps into account the extension of the damage, the average level of damage and the maximum damage experienced by the structure.

This method has been described in detail because it has been used as a reference method for the calculation of the relation among vulnerability, damage and peak ground acceleration of the ground by Guagenti and Petri (1989), described in the following of this work.

2.3.3 Damage level: European Macro-seismic Scale EMS-98

Another possible method for the identification of the damage is the utilization of the macro-seismic scales for the level of damage: one of the most famous is the European Macro-seismic Scale EMS, developed by Grünthal in 1998, where it is possible to find 5 levels of damage, from "1" - Negligible to slight damage (no structural damage, slight non-structural damage), till "5" - Destruction (very heavy structural damage).

This method is based on the qualitative evaluation of the level of damage, comparing the real situation with the proposed descriptions of each level of damage.




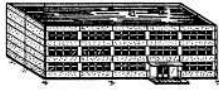






Classification of damage to masonry buildings		Classification of damage to buildings of reinforced concrete	
	Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage) Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.		Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage) Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.
	Grade 2: Moderate damage (slight structural damage, moderate non-structural damage) Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.		Grade 2: Moderate damage (slight structural damage, moderate non-structural damage) Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.
	Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage) Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).		Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage) Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.
	Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage) Serious failure of walls; partial structural failure of roofs and floors.		Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage) Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.
	Grade 5: Destruction (very heavy structural damage) Total or near total collapse.		Grade 5: Destruction (very heavy structural damage) Collapse of ground floor or parts (e.g. wings) of buildings.

Figure 10: Definition of the levels of damage for masonry structures (left) and reinforced concrete structures (right) - EMS-98.

2.4 Estimation of the vulnerability: expeditious and hybrid methods

The seismic vulnerability of the structures is argument of research since the early '70s: considering the level of detail of the method of analysis and the dimension of the sample of building, it is possible to identify three different approaches, as already defined in the Introduction of this work:

- expeditious empirical methods, mainly based on qualitative evaluations; they are necessary in case of the seismic vulnerability assessment of a large sample of buildings, for which the detailed analysis of each structure can be prohibitively expensive to perform. This approach is used for analyses on a territorial scale and it represents the first step in order to highlight the most vulnerable buildings which must be better investigated with subsequent detailed analyses;
- analytical/mechanical methods, characterized by a direct physical meaning; they allow the study of the seismic vulnerability by means of the analysis of the structures' mechanical behaviour through numerical models, which can have different levels of complexity;
- hybrid methods, which are based on simplified quantitative evaluations; they represent a combination of the two categories mentioned above and they are used for the territorial approach, in order to have a first estimation of the structural behaviour for the considered structures.

In order to better understand the definitions proposed above, a review of the principal methods of the vulnerability assessment is here proposed, starting from the empirical ones (developed in the early '70s) and arriving to the hybrid methods, proposed in the recent years.

The mechanical methods will not be described in this chapter: the analytical evaluations of the seismic vulnerability will be explained and applied by means of static non-linear analyses in this work (see Chapter 4).

2.4.1 Damage Probability Matrix (DPM)

The method of the Damage Probability Matrix, developed at first by Whitman et al (1973), provides the use of probabilistic matrices of damage for the prediction of the damage caused by different seismic events; the basic concept of the method is that "*a given structural typology has the same probability of damage for a given earthquake intensity*".

Whitman compiled the matrix for various structural typologies according to the damage suffered by over 1600 buildings after the 1971 San Fernando earthquake (California, 6.6 magnitude - Richter Scale).

The damage has been identified on a scale proposed directly by the author of the method: this scale is composed of 8 different levels of damage, from "0" - no damage, until "7" - destroyed building. Each class has been identified by two different aspects: a description of the physical damage and an objective ratio among the reparation cost of the considered damage and the reconstruction cost of the whole building.

Each damaged building of the sample has been classified by the level of damage, the material of construction, the age of the building and the number of stories.

All the collected data have allowed the creation of matrices of probability of damage. In the following, the DPM for the area hit by VII° level of MM scale of San Fernando earthquake is proposed.

Date Const.	Pre-1933				Post-1947						
No. Stories	5-7		8-13		5-7		8-13		14-18		19+
Type of Const.	Co	St	Co	St	Co	St	Co	St	St	St	
Damage State											
0	16	18	16	6	21	24	27	44	43	21	
1	16	9	12	13	26	28	33	31	43	54	
2	26	46	28	53	16	38	32	6	0	25	
3	21	27	14	16	26	5	8	16	14	0	
4	11	0	21	0	11	5	0	3	0	0	
5	0	0	7	9	0	0	0	0	0	0	
6	10	0	2	3	0	0	0	0	0	0	
7	0	0	0	0	0	0	0	0	0	0	
MDR - %	4.4	1.1	2.7	2.5	1.1	.66	.43	.52	.43	.24	
No. Bldgs.	19	11	43	32	19	21	37	32	14	24	

Figure 11: DPM for the area hit by VII° level of MM scale of San Fernando earthquake (Whitman, 1973).

The buildings have been divided by the age of construction (simply dividing in buildings realized before 1933 and after 1947), the number of stories (dividing the sample in groups of buildings with 5-7 levels, 8-13 levels, 14-18 levels and buildings with more than 19 levels) and the type of construction ("Co" means reinforced concrete structures and "St" means steel structures). All these groups have been ordered by the level of damage (from "0" to "7").

It is possible to highlight some aspects from the matrix proposed in Figure 11, simply plotting the distributions of the levels of damage for each observed parameter: in the following graphs, the comparison among the variables of each parameter is presented, plotting the percentage distributions of the level of damage with continuous lines and the corresponding cumulative distributions with dashed lines.

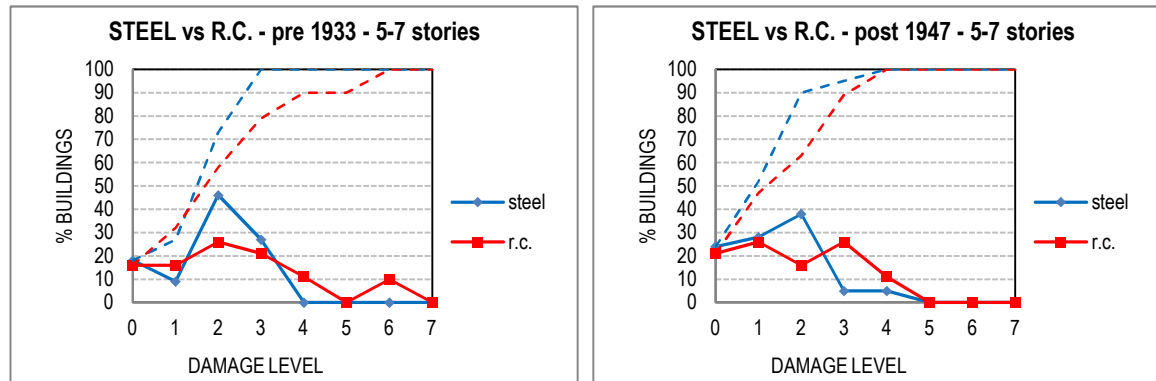


Figure 12: Distributions of the level of damage: comparison among the type of material (steel VS reinforced concrete).

As shown in Figure 12, steel structures are characterized by cumulative curves always higher than those of the reinforced concrete buildings: this means that, on average, steel structures have experienced a lower level of damage; moreover, it is possible to observe that this aspect is more evident for the buildings realized before 1933, since the cumulative curves are really distant each other.

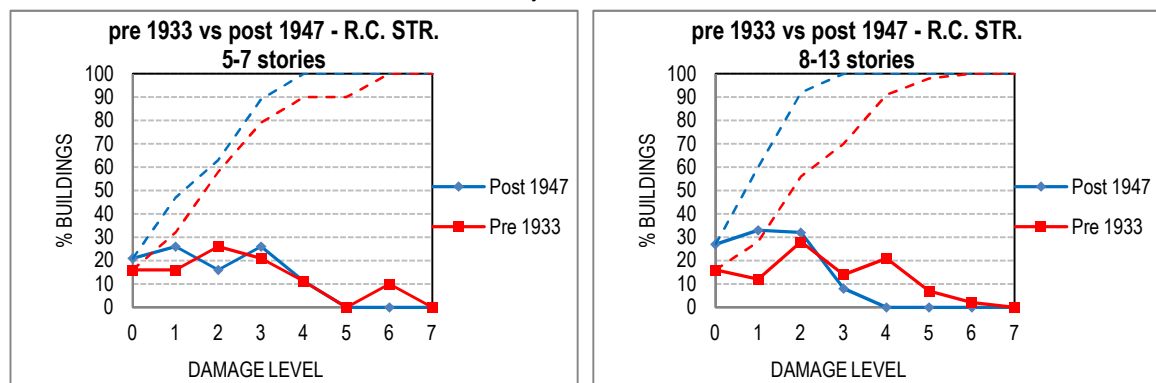


Figure 13: Distributions of the level of damage (r.c. structures): comparison among the age of construction (pre 1933 VS post 1947).

The graphs proposed in Figure 13 show that, considering only the reinforced concrete structures, more recent buildings (post 1947) have experienced, on average, a lower level of damage. This can be a consequence of the development of the construction techniques and of the codes for the structural design.

The same information can be obtained for the graphs related to the steel structures (here not shown).

These are only some of the possible conclusions which can be obtained from the DPM.

Whitman, with this work, has showed some of the most important parameters which influence the behaviour of the structures under a seismic event, obtaining the information directly from a real earthquake.

Many studies have been realized to develop the method in Europe, using at first the MSK macro-seismic scale on damage data of Italian buildings after the 1980 Irpinia earthquake (Braga et al., 1982), later the MCS (Di Pasquale et al., 2005) and finally the EMS-98 (Giovinazzi and Lagomarsino, 2001 and 2004).

2.4.2 European Macro-seismic Scale (EMS-98)

One of the most important Macro-seismic scale used in Europe in the recent years is the EMS-98, (Grünthal, 1998): in this scale, it is possible to find a vulnerability table where the most common typologies of structures are identified in terms of building material, typology of material and level of design of the structure. For each typology, a vulnerability class is assigned within the 7 available ones (from A - most vulnerable, until F - less vulnerable); furthermore, for each assigned vulnerability class, a probable range of variation is identified, differentiating even the exceptional cases which can be reached in terms of vulnerability class.

Type of Structure	Vulnerability Class						
	A	B	C	D	E	F	
MASONRY	rubble stone, fieldstone	○					
	adobe (earth brick)	○	—				
	simple stone	○					
	massive stone			○	—		
	unreinforced, with manufactured stone units			○	—		
	unreinforced, with RC floors			○	—		
	reinforced or confined				○	—	
REINFORCED CONCRETE (RC)	frame without earthquake-resistant design (ERD)			○	—		
	frame with moderate level of ERD			○	—		
	frame with high level of ERD				○	—	
	walls without ERD			○	—		
	walls with moderate level of ERD			○	—		
	walls with high level of ERD				○	—	
STEEL	steel structures			○	—		
WOOD	timber structures			○	—		

○ most likely vulnerability class; — probable range;range of less probable, exceptional cases

Figure 14: Vulnerability classes for the different typologies of structures (EMS-98).

In Figure 14 it is possible to observe that masonry structures represent, in general, the most vulnerable typology of buildings (the worst case is represented by the rubble stone or fieldstone case); different types of masonry are taken into account, showing however some cases with low vulnerability (this is the case of reinforced or confined masonry for example).

For reinforced concrete structures, the image shows different ranges of vulnerability due to the level of the Earthquake Resistant Design (ERD) of the building, both for frame and wall structure typologies.

The last two rows are related to steel and timber structures which show, in general, a better behaviour under seismic action and so they are classified in low vulnerable classes.

The previous Vulnerability Table, as written in the original document, is an attempt to categorize in a manageable way the seismic resistance of the different types of structures, considering the material and the structural typology.

This instrument can be used only in a comparative way, in case of a buildings' sample composed of different structural typologies.

2.4.3 Vulnerability Index Method

This method, developed at first in the '80s by Benedetti e Petrini (1984) and GNDT (1993) for masonry structures, was calibrated on the survey of a large sample of different buildings, damaged by earthquakes.

This method consists in filling in a survey form composed of 11 parameters: for each parameter, the surveyor has to give a judgment (four possibilities, from "A" - optimal condition to "D" - unfavourable condition), taking into account the brief descriptions given in the user manual, which allow a more objective decision.

For each judgment of each parameter, a numerical score value is given by the method. Using the weight coefficients related to each parameter (provided in order to take into account the relative importance of each parameter in the global definition of vulnerability), it is possible to calculate a Vulnerability Index, I_v , usually normalized in a 0%-100% range, where a low index means that the structure is not so vulnerable and therefore it has a high capacity under seismic action.

This method has been developed for masonry structures at the beginning by Benedetti and Petrini, and then it has been extended to reinforced concrete structures later.

In the following text, a brief description of the 11 parameters for the masonry structure Form is presented.

PARAMETER 1 - type and organization of the resistant system

In this parameter, the organization of the structure is evaluated, without taking into account the quality of the masonry (see parameter 2); the most important aspect to analyze is the presence of connections among perpendicular walls (necessary to guarantee the three-dimensional box behaviour of the building). In other words, this parameter must give an indication about the possibility of activation of a global behaviour of the structure.

PARAMETER 2 - quality of the resistant system

In this parameter, the masonry type is qualitatively evaluated: it is necessary to observe the material and the shape of the elements that compose the walls at first, and the homogeneity of the masonry afterwards.

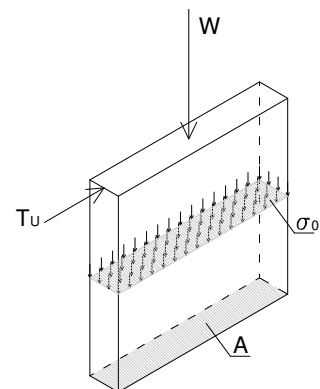
PARAMETER 3 - conventional resistance

The resistance of a building under seismic actions can be evaluated (in a simplified way) with a procedure that takes into account the total horizontal cross-sectional area of the walls in the two main horizontal directions of the building: with this method, it is possible to carry out an estimation of the shear resistance of the structure (on an equivalent model of the whole structure). The ratio between the shear resistance and the weight of the structure gives as result the multiplier of collapse, which allows the decision of the judgment to assign to this parameter. This simplified calculation is based on some hypothesis:

- planimetric and elevation regularity;
- elevation continuity of the masonry piers;
- shear failure mechanism of the piers;
- three-dimensional behaviour of the entire structure.

In the following, a brief analytical description for the calculation of this parameter is proposed.

Considering a singular masonry pier with cross-sectional area A , the shear resistance for the horizontal actions in its plane can be estimated with the original diagonal cracking formula of Turnsek and Cacovic: this approach expresses the shear resistance T_U of the wall in relation to the characteristic shear strength τ_k of the type of material and to the vertical compression stress σ_0 acting on it.



$$T_U = A \cdot \tau_k \cdot \sqrt{1 + \frac{\sigma_0}{1.5 \cdot \tau_k}}$$

The estimation of the shear resistance of the whole building can be performed considering it as an equivalent wall with a horizontal cross-sectional area equal to the minimum total area among the two principal directions of the building.

It is necessary to evaluate:

N	number of levels starting from the one that has to be verified
A_t	area of the considered level (covered area)
A_x, A_y	cross-sectional area of all the resistant elements in the two orthogonal directions
h	interstorey height
p_m	specific weight of the masonry
p_s	permanent load of the floor (including both dead and live loads in the seismic combination)

With the collected data, it is then possible to calculate:

A	minimum value between A_x, A_y
B	maximum value between A_x, A_y

$$a_0 = \frac{A}{A_t}$$

$$\gamma = \frac{B}{A}$$

$$q = \frac{(A + B) \cdot h}{A_t} \cdot p_m + p_s$$

The calculation of the "conventional resistance" (name proposed in the Handbook of the Vulnerability Form by GNDT, 1993) consists in the ratio between the shear resistance of the considered level and the total weight above it:

$$C = \frac{T_u}{W}$$

With some mathematical passages, it is possible to obtain the following equation:

$$C = \frac{T_u}{W} = \frac{a_0 \cdot \tau_k}{q \cdot N} \cdot \sqrt{1 + \frac{q \cdot N}{1.5 \cdot a_0 \cdot \tau_k \cdot (1 + \gamma)}}$$

For this parameter (that has a considerable importance in the vulnerability assessment), it is useful to show which are the numerical values that identify the 4 classes of judgment.

The definition of the performance of the building (necessary for the attribution of the judgment of this parameter) is made by the evaluation of the ratio:

$$\alpha = \frac{C}{\hat{C}}$$

where \hat{C} is a reference value, assumed equal to 0.35 in the survey campaign performed by the DICEA-UNIFI Research Group in 2011.

The identification of the 4 classes for the assignment of the judgment is shown here below:

A class	$\alpha \geq 1$
B class	$0.6 \leq \alpha < 1$
C class	$0.4 \leq \alpha < 0.6$
D class	$\alpha < 0.4$

This parameter gives therefore quantitative information about the vulnerability of the considered structure: in particular, the parameter C represents a first estimation of the maximum acceleration that the total weight of the structure W (considered as a single degree of freedom system) can suffer before the collapse.

PARAMETER 4 - position of the building and foundations

In this parameter, the local morphology of the site of construction and the type of foundations are qualitatively taken into account. In particular, it is important to observe the natural slope of the ground and the possible presence of different levels of foundation.

PARAMETER 5 - typology of floors

This aspect strongly influences the global behaviour of the structure. In particular, two elements must be considered here: the stiffness of the floor in its plane (in order to guarantee a uniform distribution of the horizontal forces among all the vertical elements) and the presence of efficient connections among floors and walls, in order to avoid the unthreading of the floor under horizontal shaking and to reduce the possibility of out of plane mechanisms for the walls, since well connected floors represent a constraint for the walls themselves. In this parameter, it is also important to observe the presence of staggered floors, which can lead to local phenomena of pounding.

PARAMETER 6 - planimetric configuration

The planimetric shape of the building is here analyzed, in order to evaluate its regularity.

PARAMETER 7 - elevation configuration

The presence of different level of stiffness at each level of the structure is here investigated; it is important to underline the presence of porticos or lodges at the ground level or the existence of towers or turrets on the top of the building. It is also necessary to evaluate the difference of masses at each level: differences lower than 10% between two consecutives floors do not need to be considered. If different materials for vertical structures have been used in the construction, a lower judgment must be assigned to this parameter.

PARAMETER 8 - maximum distance among the walls

This parameter gives information about the presence of structural walls orthogonally connected to the considered ones: an excessive distance among them, compared to the thickness of the considered wall, is here investigated. This aspect can lead to local out of plane mechanisms for the considered wall, since the orthogonal walls may not give the necessary constraint for a local overturning for example. This parameter is based on the calculation of the ratio among the distance between the orthogonal walls (d) and the thickness of the analyzed wall (t): each judgment (A, B C or D) is characterized by a maximum value of this ratio, starting from A (maximum value of distance/thickness = $d/t \leq 15$) and arriving to D (maximum value of $d/t > 25$).

PARAMETER 9 - roof

In this parameter, it is important to evaluate the typology of the roof and its weight.

Some elements to analyze are:

- the presence of unbalanced pushing forces of the roof;
- the presence of concrete beams on the top of the walls;
- the presence of metal ties.

PARAMETER 10 - non structural elements

In this parameter, the presence of non structural elements which can cause damages to persons or objects due to their fall is analyzed.

PARAMETER 11 - state of conservation

In this parameter, the actual condition of the building is analyzed. Cracks in the walls or misalignment of the vertical walls must be taken into account. In the following Table 1, it is possible to see the scores related to each judgment of each parameter and the relative weights.

MASONRY STRUCTURES						
PARAMETER		SCORES				WEIGHT
		A	B	C	D	
PAR. 1	Type and organization of the resistant system	0	5	20	45	1.00
PAR. 2	Quality of the resistant system	0	5	25	45	0.25
PAR. 3	Conventional resistance	0	5	25	45	1.50
PAR. 4	Position of the building and foundations	0	5	25	45	0.75
PAR. 5	Typology of floors	0	5	15	45	var.
PAR. 6	Planimetric configuration	0	5	25	45	0.50
PAR. 7	Elevation configuration	0	5	25	45	var.
PAR. 8	Maximum distance among the walls	0	5	25	45	0.25
PAR. 9	Roof	0	15	25	45	var.
PAR. 10	Non structural elements	0	0	25	45	0.25
PAR. 11	State of conservation	0	5	25	45	1.00

Table 1: Scores for the 11 parameters Vulnerability Form for masonry structures.

There are three variable weights:

parameter 5 $P5 = 0.50 \cdot (100/\alpha_0)$

α_0 is the percentage of rigid and well connected floors.

If $P5 > 1$, $P5 = 1$ is assumed.

parameter 7 $P7 = 0.50$ if the non regularity of the building is due only to porticos at ground floor

$P7 = 1.00$ in all the other cases.

parameter 9 $P9 = 0.50 + \alpha_1 + \alpha_2$

$\alpha_1 = 0.25$ for roofs with concrete slabs or with more than $2.0kN/m^2$ of self-weight

$\alpha_1 = 0.00$ for all the other cases

$\alpha_2 = 0.25$ if the ratio "perimeter of roof / length of support of the roof" ≥ 2

$\alpha_2 = 0.00$ for all the other cases

The coefficient α_2 takes into account the distribution of the load of the roof on the walls: when the ratio "perimeter of roof / length of support of the roof" is ≥ 2 , it means that the roof has a structural supporting area that does not interest half of the total perimeter of the roof level, giving a worse distribution of the load itself.

The Index of Vulnerability is obtained as the weighted sum of the scores of each parameter:

$$I_V = \sum_{i=1 \rightarrow 11} V_i \cdot P_i$$

where

V_i are the scores of vulnerability

P_i are the weights related to each parameter.

The Index of Vulnerability belongs to the [0 - 382.5] range and it must be normalized in the [0%-100%] range.

In order to give a complete view of the Vulnerability Index Form for masonry structures, the layout of the form is reported in the following Figure 15. In the 1st column, all the 11 parameters are described, with two boxes where the judgement of the parameter and an index of the quality of the information can be inserted from the surveyor. The 2nd and 3rd columns contain all the necessary information for the definition of the judgments of each parameter.

Cod. ISTAT Provincia		1			Cod. ISTAT Comune	4			N. scheda	7			Squadra	12				
PARAMETRI		Clas- si	Qual. inf.	ELEMENTI DI VALUTAZIONE								SCHEMI - RICHIAMI (MURATURA)						
1	TIPO ED ORGANIZZAZIONE DEL SISTEMA RESISTENTE (S.R.)	14	25	Norm. nuove costruz. (cl. A)	36	1	Norm. riparazioni (cl. A) 2 Cord. o cat. tutti livelli (cl. B) 3 Buoni amm. fra mur. (cl. C) 4 Senza cord. cattivi amm. (cl. D) 5 (vedi manuale) 37								Parametro 3. Resistenza convenzionale			
2	QUALITA' DEL S.R.	15	26	Numero di piani N	38										Tipologia struttura verticale τ_k (t/mq) _____ _____ _____ _____ Minimo fra A_x e A_y A (mq) _____ Massimo fra A_x e A_y B (mq) _____ Coeff. $a_0 = A/A_t$ _____ Coeff $\gamma = B/A$ _____ $q = (A_x + A_y) \cdot h \cdot p_m / A_t + p_s$ _____ $C = a_0 \tau_k / (qN) \cdot [1 + (qN) / (1.5 a_0 \tau_k (1 + \gamma))]^{1/2}$ _____ $\alpha = C / 0.4$ _____			
3	RESISTENZA CONVENZIONALE	16	27	Area tot. cop. A_t (mq)	40		Area A_x (mq) 45 Area A_y (mq) 49 τ_k (t/mq) 53 Alt. media interp. h (m) 56 Peso spec. par. p_m (t/mc) 60 Carico perm. sol. p_s (t/mq) 63				Parametro 6. Configurazione planimetrica							
4	POSIZIONE EDIFICIO E FONDAZIONI	17	28	Pend. perc. terr.	66		Roccia fond. si no 1 2 Terr. sc. non sp. si no 3 4 Terr. sc. sp. si no 5 6 Dif. max. di quota Δh (m) 70											
5	ORIZZONTAMENTI	18	29	Piani sfalsati	74	1	2	Orizz. rig. e ben coll. 75 1 Orizz. def. e ben coll. 2 Orizz. rig. e mal coll. 3 Orizz. def. e mal coll. 4 % or. rig. ben coll. 76										
6	CONFIGURAZIONE PLANIMETRICA	19	30	Rapp. perc. $\beta_1 = a/l$	79		Rapp. perc. $\beta_2 = b/l$ 82								Parametro 7. Configurazione in elevazione			
7	CONFIGURAZIONE IN ELEVAZIONE	20	31	% aumento (+) riduz. (-) di massa	85		Rapp. perc T/H 89 Perc. in sup. port. 92 Piano terra port. si no 1 2								Parametro M9. Copertura Coperture spingenti (tipologia M) 			
M8	D_{max} MURATURE	21	32	Rapp. massimo l/s	96		Coperture poco spingenti (tipologia N) 											
M9	COPERTURA	22	33	Cop. non sp.	99	0	poco sp	1	sp	2	Coperture non spingenti (tipologia O) 							
10	EL. NON STRUTT.	23	34	Cord. in copert.	100	1	no	2	Cat. in copert. si no 1 2 Car. perm. cop. p_c (t/mq) 102 Lungh. app. cop. l_a (m) 106 Perim. cop. l (m) 109									
11	STATO DI FATTO	24	35	(vedi manuale)								(vedi manuale)						

Figure 15: 11 parameters Vulnerability Form for masonry structures.

After the development of the Vulnerability Form for masonry structures, Angeletti and Gavarini (1984) have worked on the definition of an analogous form for reinforced concrete structures. In the following text, a brief description of the parameters of the form for the reinforced concrete structures is presented.

PARAMETER 1 - type and organization of the resistant system

In this parameter, the organization of the building is evaluated, looking in particular to the ductility/fragility of the structural elements.

There are three different behaviours which correspond to the three different classes:

- rigid structure (due to the presence of consistent shear concrete walls); it is possible to consider a maintenance of the resistant characteristics even after a strong seismic event;
- rigid-fragile structures with ductility resources (after the collapse of the most rigid elements, the structure starts to behave in a ductile way with more deformability because of the presence of a structural frame with seismic design);
- rigid-fragile structures, with high stiffness/resistance decay after the seismic event, due to the absence of ductile elements in the structure.

PARAMETER 2 - quality of the resistant system

In this parameter, three aspects are evaluated: the type and quality of the materials, the characteristics of the realization of the structure and the principles of design of the building.

This information can be obtained from the original documents of the design of the structure, or directly by visual inspection when possible.

PARAMETER 3 - conventional resistance

The resistance of a building under seismic actions can be evaluated (in a simplified way) with an equivalent linear static analysis with no eccentricities and planimetric irregularities; it is necessary to evaluate the ratio between the shear resistance (sum of the cross-sectional area of all the columns multiplied for the shear strength) and the seismic forces derived from the simplified analysis.

In this way, it is possible to calculate the multiplier α , which gives information about the ratio "capacity/demand" in terms of shear forces, and consequently the judgment to assign to the structure.

PARAMETER 4 - position of the building and foundations

In this parameter, the type of foundations is taken into account. In particular, it is important to observe the type of the soil and the presence of different level of foundations.

PARAMETER 5 - typology of floors

As seen for masonry structures, two main aspects must be taken into account: the stiffness of the floor in its plane and the presence of good connections of the floor with the vertical elements.

PARAMETER 6 - planimetric configuration

The planimetric configuration of the building is here analyzed, in order to evaluate its regularity. Two aspects must be considered: the distribution of masses and stiffness and the planimetric shape of the building.

PARAMETER 7 - elevation configuration

The distribution of stiffness and masses along the height of the structure is here investigate: a limited reduction of stiffness connected to higher levels is accepted (due to the dynamic response of the system) while the increase of masses or stiffness at higher levels must be penalized, since they lead to a worse dynamic behaviour.

PARAMETER 8 - connections and critical elements

In this parameter, it is important to observe the connections among structural elements (beam-column, floor/beams, foundation-columns...). For the analysis of this parameter, some geometrical rules are contained in the Handbook of the Vulnerability Form.

PARAMETER 9 - low ductility elements

The possible presence of elements with a fragile behaviour is here highlighted.

PARAMETER 10 - non structural elements

In this parameter, the presence of elements (without a structural function) which can cause damages to persons or objects due to their fall is analyzed.

PARAMETER 11 - state of conservation

The current condition of the building is here analyzed. It is important to pay attention to all the structural elements (columns, beams, floors) at first, then to foundations and non structural elements.

For reinforced concrete structures there are no specific weights for each parameter: each of them has the same importance in the calculation of the global Index of Vulnerability.

In the following Table 2, it is possible to see the scores connected to each parameter.

REINFORCED CONCRETE STRUCTURES					
PARAMETER		SCORES			
		A	B	C	D
PAR. 1	Type and organization of the resistant system	0.00	-1.00	-2.00	-
PAR. 2	Quality of the resistant system	0.00	-0.25	-0.50	-
PAR. 3	Conventional resistance	0.25	0.00	-0.25	-
PAR. 4	Position of the building and foundations	0.00	-0.25	-0.50	-
PAR. 5	Typology of floors	0.00	-0.25	-0.50	-
PAR. 6	Planimetric configuration	0.00	-0.25	-0.50	-
PAR. 7	Elevation configuration	0.00	-0.50	-1.50	-
PAR. 8	Connections and critical elements	0.00	-0.25	-0.50	-
PAR. 9	Low ductility elements	0.00	-0.25	-0.50	-
PAR. 10	Non structural elements	0.00	-0.25	-0.50	-
PAR. 11	State of conservation	0.00	-0.50	-1.00	-2.45

Table 2: Scores for the 11 parameters Vulnerability Form for reinforced concrete structures.

The Index of Vulnerability is obtained as the sum of the scores of each parameter:

$$I_V = \sum_{i=1 \rightarrow 11} V_i$$

where

V_i are the scores of Vulnerability of each parameter.

The Index of Vulnerability belongs to the [-9.70; +0.25] range and it must be normalized in the [0%-100%] range.

As already done for the masonry structures, the Vulnerability Index Form for reinforced concrete structures is reported in the following Figure 16. The layout is analogous to the masonry structures Form: in the 1st column, all the 11 parameters are described, with two boxes where the judgment of the parameter and an index of quality of the information can be inserted from the surveyor. The 2nd and 3rd columns contain all the necessary information for the definition of the judgments of each parameter.

Cod. ISTAT Provincia		Cod. ISTAT Comune		N. scheda		Squadra		
PARAMETRI		Clas- si	Qual. inf.	ELEMENTI DI VALUTAZIONE				SCHEMI - RICHIAMI (CEMENTO ARMATO)
1	TIPO DI ORGANIZZAZIONE DEL SISTEMA RESISTENTE (S.R.)	14	25	Pareti in c.a. (cl. A) 36	1	Parametro 3. Resistenza convenzionale		<p>Minimo fra A_x e A_y. A _____</p> <p>Coefficiente $a_0 = A/A_1$ _____</p> <p>$q = (A_x + A_y) \cdot h \cdot p_m / A_1 + p_s$ _____</p> <p>$C = a_0 \cdot \tau_k / (q \cdot N)$ _____ $\alpha = C / (0.4 \cdot R)$ _____</p> <p>Calcolo di R</p> <p>Terreni tipo S_1: $R = 2.5$ ($T < 0.35$ s)</p> <p>$R = 2.5 / (T / 0.35)^{2/3}$ ($T > 0.35$ s) _____</p> <p>Terreni tipo S_2: $R = 2.2$ ($T < 0.8$ s)</p> <p>$R = 2.2 / (T / 0.8)^{2/3}$ ($T > 0.8$ s) _____</p> <p>Parametro 6. Configurazione planimetrica</p> <p>Parametro 7. Configurazione in elevazione</p> <p>Parametro C9. Colleg. ed elementi critici</p>
2	QUALITA' DEL S.R.	15	26	(vedi manuale)	37			
3	RESISTENZA CONVENZIONALE	16	27	Numero di piani N	38			
				Area tot. cop. A_1 (mq)	40			
				Area A_x (mq)	45			
				Area A_y (mq)	49			
				τ_k (t/mq)	53			
				Alt. media interp. h (m)	56			
				Peso spec. par. p_m (t/mc)	60			
				Carico perm. sol. p_s (t/mq)	63			
4	POSIZIONE EDIFICIO E FONDAZIONI	17	28	Pend. perc. terr.	66			
				Roccia	68			
				Terr. sc. non sp.	71			
				Terr. sc. sp.	72			
				Dif. max. di quota Δh (m)	70			
5	ORIZZONTAMENTI	18	29	Piani sfalsati	74			
				Orizz. rig. e ben coll.	75			
				Orizz. def. e ben coll.	76			
				Orizz. rig. e mal coll.	77			
				Orizz. def. e mal coll.	78			
				% or. rig. ben coll.	79			
6	CONFIGURAZIONE PLANIMETRICA	19	30	Rapp. perc. $\beta_1 = a/l$	81			
				Rapp. perc. $\beta_2 = e/d$	82			
				Rapp. perc. $\beta_3 = \Delta d/d$	83			
				Rapp. perc. $\beta_4 = c/b$	84			
7	CONFIGURAZIONE IN ELEVAZIONE	20	31	% aumento (+) riduz. (-) di massa	91			
				Rapp. perc T/H	95			
				Var. in elev. s.r.	96			
				Piano terra port.	97			
C8	COLLEGAMENTI ED ELEMENTI CRITICI	41	34	Rapp. perc. $\gamma_1 = s/b$	100			
				Rapp. perc. $\gamma_2 = e/b'_{min}$	103			
				Rapp. perc. $\gamma_3 = e/b''$	106			
				Rapp. max. h/b_{min}	109			
				% σ/R_c (approssim.)	112			
				Colleg. el. pref.	113			
				Largh. min. b_{min} (cm)	110			
C9	ELEM. BASSA DUTT.	44	33	Rapp. min. h_{min}/b	119			
				Rapp. max. h_{medio}/h_{min}	144			
10	EL. NON STRUTT.	23	34	(vedi manuale)				
11	STATO DI FATTO	24	35	(vedi manuale)				
12	Struttura a telai piani o a telai spaziali			piani	125	spaziali		

Figure 16: 11 parameters Vulnerability Form for reinforced concrete structures.

In case of a mixed sample of buildings (masonry and reinforced concrete structures), it is possible to compare the results in terms of Index of Vulnerability by means of conversion formulas, proposed by the Seismic Service of the Marche Region, modifying the index of the r.c. structures as described in the following:

$$I_{V.r.c.structures} > -6.5 \rightarrow I_V = -10.07 \cdot I_{V.r.c.structures} + 2.52$$

$$I_{V.r.c.structures} < -6.5 \rightarrow I_V = -1.73 \cdot I_{V.r.c.structures} + 56.72$$

These formulas of conversion calculate the equivalent Index of Vulnerability for a reinforced concrete (directly comparable with the masonry indexes), which belongs to the [0%-73.5%] range; masonry structures can reach even the value of 100%: in the definition of these formulas of conversion it was assumed that, statistically speaking, r.c. structures cannot reach levels of vulnerability higher than masonry ones.

2.4.4 Tuscany Region modifies on the Vulnerability Index Method

The observation of the damages of the built environment caused by recent earthquakes allowed a better comprehension of the behaviour of the structures and the influence of each element of a building on the global response to a dynamic solicitation. Using this approach, Tuscany Region has proposed a modification of the Vulnerability Index Method: in particular, the Handbook for the compilation of the vulnerability form has been improved with the addition of some examples (photos and schemes) which can help the surveyor to fill the form; moreover, an update of the weights of the parameters has been done, and in particular for the 1st, 5th and 9th parameter as described below.

- parameter 1: $P1 = 1.50$ (instead of 1.0);
- parameter 5: the calculation of the weight is the same of the original form in the [0.5 - 1.0] range; if the floors belong to a heavy typology (such as concrete slabs) realized on not so resistant masonry walls, the weight coefficient is 1.25;
- parameter 9: the calculation of the weight is the same of the original form in the [0.5 - 1.0] range; if the roof belongs to a heavy typology (such as concrete slabs) realized on not so resistant masonry walls, the weight coefficient is 1.25; if there is also a heavy floor just below the roof, the coefficient becomes 1.5.

The modifications proposed by Tuscany Region in 2003 involve at first the weight of the floors and roofs: this because in the past decades (80's-90's), the substitution of light roofs such as wooden ones, with heavier ones made of concrete slabs, was common. The earthquakes occurred in the last 30 years, through the observation of the damages of the building heritage, have highlighted that this type of intervention, when performed on low quality masonry buildings, can change substantially the dynamic behaviour of the structures since it adds a considerable mass on the top of the building with a high in-plane stiffness: these two aspects can cause the collapse of the structure under seismic excitation.

Some examples of the bad effects of these kind of interventions are visible in the following pictures taken in the village of Castelnuovo, a village located 20 km from the city of L'Aquila, hit by the 06 April 2009 earthquake: some heavy roofs have been realized in the past over really poor quality masonry walls; the seismic event caused the slipping of the roofs and the consequent collapse of the structure.



Figure 17: Aerial view of Castelnuovo (AQ) after the 06/04/2009 earthquake. In red, the locations of two collapsed heavy roofs.



Figure 18: Two collapsed heavy roofs in Castelnuovo (AQ).

2.4.5 Other vulnerability forms

Starting from the Vulnerability Index Method, some other forms have been created for specific cases: an example is the Vulnerability Index Method proposed by Formisano et al (2009), which is related to masonry structure in aggregate. It is well known that the historical city centres are the result of an evolution process, with structural units realized one next to the other during the development of the city/village, realizing complex constructions.

Each structural unit has an intrinsic vulnerability, due to the material of construction, typology of connections among structural elements and so on, and an extrinsic vulnerability, due to the presence of adjacent structural units, which can influence the behaviour of the analyzed building due to the absence of seismic joints. Formisano et al have created a new form, starting from the original one proposed by Benedetti and Petrini (1984) and GNDT (1993), excluding the parameter n.° 3 (conventional resistance) and adding five new parameters which take into account the following features:

- presence of adjacent buildings with different height;
- position of the structural unit in the aggregate;
- presence and number of staggered floors;
- effect of either structural or typological heterogeneity among adjacent structural units;
- difference of the percentage of openings among adjacent facades.

The scores and weights of these new parameters have been calibrated through numerical simulations on some case studies by the authors. The procedure of calculation of the Index of Vulnerability for this form is analogous to the original method proposed by Benedetti and Petrini (1984) and GNDT (1993).

Another attempt to investigate the vulnerability of the masonry aggregates has been performed by Vicente et al (2008), which has started the development of a new form using as reference the Benedetti and Petrini form: the authors added other three parameters related to the following features:

- number of floors;
- aggregate position and interaction;
- wall facade openings and alignments.

All the weights and scores of each parameter have been estimated and the procedure of evaluation of the Index of Vulnerability is even in this case analogous to the one of the original method of Benedetti and Petrini.

There is another typology of form, proposed by Ferreira et al (2012), which tries to estimate the vulnerability (using the same analytical procedure of the other methods mentioned above) of the whole aggregate through the evaluation of five parameters:

- quality of the masonry fabric;
- misalignment of openings;
- irregularities in height;
- plan geometry;
- location and soil quality.

Many other attempts have been developed in order to estimate in an expeditious way the vulnerability of buildings, starting from the method proposed by Benedetti and Petrini (1984) and GNDT (1993). This aspect highlights the importance of the original method used in the present work, since it represents the starting point of many scientific works in the seismic engineering field.

2.4.6 SAVE Project

The methods which have been described until now are mostly empirical ones: in fact, they are based on typological aspects, which give useful information in case of a comparison among different types of buildings: the EMS-98 Scale for example (see § 2.4.2) gives the possibility to compare buildings which show different structural typologies. The Vulnerability Index method can be classified as mainly typological too, since 10 of the 11 parameters are related to typological aspects: indeed, only the parameter n.° 3 requires a simplified quantitative evaluation of the structural elements of the considered building.

Some further developments on the expeditious vulnerability assessment are going in the direction of hybrid methods: these procedures are a combination of the empirical methods (described above) and mechanical ones, based on analytical analyses of structural models.

The earthquake of the 31/10/2002 (epicentre in San Giuliano di Puglia - Molise Region) of magnitude 5.8 in the Richter scale, caused the death of 27 children and their teacher because of the collapse of a school; after that tragic event, lot of attention has been given to the public buildings, in order to assess the level of vulnerability of the public heritage of Italy.

The check of the seismic vulnerability for the public buildings was required with a specific law, the Decree of the President of the Council of Ministers - O.P.C.M. 3274 (2003). It is also important to remind that this document has also modified the definition of the seismic areas in Italy: before that law, only 2965 on a total of 8102 Italian Municipalities were classified as in seismic areas (45% of the total area of the National territory, where the 40% of the population was living), while after the O.P.C.M. 3274 all the Italian territory has been classified as seismic, with 4 different classes identified by the level of the seismic action (from Zone 1 - the most dangerous, until Zone 4 - the least one).

In order to satisfy the requirement of the vulnerability assessment for public buildings, within the SAVE Project (Strumenti Aggiornati per la Vulnerabilità sismica del patrimonio Edilizio e dei sistemi urbani) proposed by the INGV/GNDT Research Group, a new methodology of fast analysis of the vulnerability has been proposed, separately for masonry and reinforced concrete structures. In the volume IV, edited by Dolce and Moroni (2005), the two procedures are described in detail: both of them lead to the evaluation of the peak ground acceleration (PGA_C) of capacity of the structures, calculated with the geometrical and mechanical features of the considered structure.

The vulnerability is referred to two different levels of performance, called "operational" and "incipient collapse": the two levels are distinguished by the amount of damage that the structure can suffer, in order to guarantee the continuity of utilization and the safety in the evacuation phase, without the collapse of the building.

In the following, a brief description of the methods:

- VC (vulnerability for reinforced concrete structures): the resistance shear capacity is calculated, checking if the collapse is due to a flexural or shear mechanism for each column.

By means of a static linear analysis with 1g of horizontal acceleration (9.81m/s^2), the horizontal forces of demand at each level of the structure are calculated: the ratio between the shear capacity and the shear demand gives the spectral acceleration of collapse. From this, PGA_C is calculated by the inversion of the acceleration spectrum formula. For the operational performance level, the 0.5% interstorey drift is assumed as the capacity configuration: the related capacity shear force is calculated with the stiffness of the considered level; the ratio among capacity and demand gives again the spectral acceleration for this performance level. The PGA_C is once again calculated as described before.

- VM (vulnerability for masonry structures) is a similar procedure: it is based on the calculation of the resistance shear capacity of each pier using the Turnsek-Cacovic formula: even in this case, there is a routine which evaluates if the collapse is due to a flexural or a shear mechanism, modifying consequently the resistant parameters in order to obtain a correct estimation of the shear capacity. The procedure is analogous to the reinforced concrete structures method described above, assuming as operational performance level the 0.3% interstorey drift. The ratios among the capacity values and demand ones (even in this case conventionally evaluated through a static linear analysis with 1g of horizontal acceleration) give the spectral accelerations for the two different performance levels ("operational" and "incipient collapse"); it is therefore possible to calculate the related PGA_C values.

As described before, this type of fast vulnerability assessment gives as result the value of the PGA_C of capacity of the structure, which is the maximum acceleration of the ground that the structure is able to suffer before reaching the limit state; this result allows a direct physical comparison among different typologies of structures and an objective estimation of the level of safety or, on the other side, of the level of risk. Indeed, comparing the PGA_C with the PGA_D of demand (the seismic hazard of the site of construction), it is possible to calculate the index:

$$\alpha_{PGA} = \frac{PGA_C}{PGA_D}$$

which is an indicator of the seismic risk.

This type of index is also used in the detailed analyses, in order to give an overall information about the level of safety of the considered structure.

2.4.7 Last developments on the fast vulnerability assessment

The methods described until now represent the fundamental basis for the rapid vulnerability assessment.

Several other methods have been developed starting from these ones, improving the level of detail or proposing different versions of the data treatments.

One method which can be mentioned in this part is the one developed by the University of Bologna by the group of Prof. Savoia: as described in Chinni et al. (2013), a new fast vulnerability assessment form has been developed with the name RE.SIS.TO (acronym of "REsistenza SISmica TOtale" - total seismic resistance), which is based from one side on the SAVE Project proposed by the INGV/GNDT Research Group (see Dolce and Moroni, 2005), because it includes the calculation of the total shear capacity of the structure with a simplified mechanical model, and on the other side on the Vulnerability Index Method (Benedetti and Petrini, 1984), because it provides the calculation of a reduction coefficient of the capacity of the structure based on 10 of the 11 parameters of the 2° Level Vulnerability Form (the parameter n.° 3 is not taken into account since it is connected again to the shear capacity of the structure). Then, the PGA of capacity is calculated in the same way as provided for the SAVE Project.

It is important to remind that the literature review proposed in these last paragraphs must not be considered as complete for the vulnerability assessment approaches, but it has been illustrated in order to give the necessary information for a better comprehension of the contents of the present work.

2.5 A correlation between seismic action, level of damage and vulnerability

In this paragraph, the study on a vulnerability model performed by Guagenti and Petrini (1989) is described; this approach shows the effective meaning of vulnerability through a direct study of correlation among the ground acceleration and the index of damage. In the end of the paragraph, a development of this study is proposed, performed by Grimaz et al (1996), and a comparison among the two definitions is shown.

As described in the beginning of this chapter (see § 2.1), the vulnerability is an aspect related to the features of the structure; it describes a cause-effect relation, where the cause is the earthquake and the effect is the damage. In other words, vulnerability relates the ground acceleration to the level of damage.

The variable y (ground acceleration) can vary between two boundary levels: y_i , acceleration corresponding to the beginning of the damage of a structure and y_c , acceleration corresponding to the collapse.

The variable d (level of damage) can vary in the $[0 - 1]$ space.

The behaviour of structures is quite aleatory, so it is possible to obtain a probabilistic relation between acceleration and damage. For each level of acceleration y , it is possible to find infinite levels of damage d , each of them characterized by a probability density conditioned function $p\{d/y\}$.

The values of y_i and y_c are aleatory variables too.

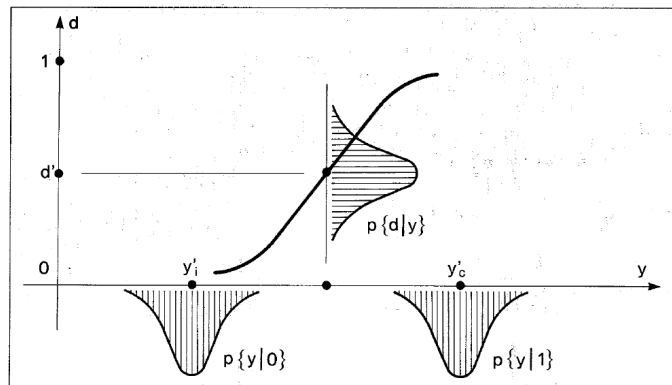


Figure 19: Probabilistic correlation law among acceleration and damage (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I).

In the seismic risk assessment, it is common to assume a deterministic problem instead of a probabilistic one, due to the high level of uncertainties that this aspect can bring to the analysis.

Appropriate acceleration/damage laws can be employed, with the trend shown in the following Figure 20.

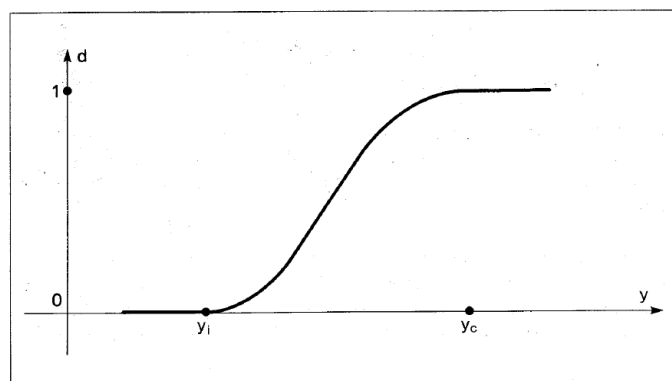


Figure 20: Deterministic law among acceleration and damage (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I).

It is then possible to schematize the law with a tri-linear trend, as shown in the next Figure 21.

The relation is probably not linear, but a reasonable representation of it can be obtained with this approximation.

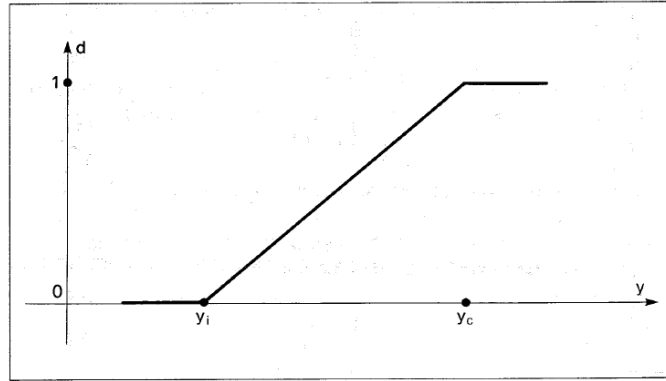


Figure 21: Tri-linear acceleration/damage law (GNDT, 1993 - Rischio sismico di edifici pubblici, Parte I).

The equation for the tri-linear law is:

$$d(y, V) = \begin{cases} 0 & y < y_i \\ (y - y_i)/(y_c - y_i) & y_i \leq y < y_c \\ 1 & y > y_c \end{cases}$$

By means of this equation, the definition of the beginning of the damage and the collapse of buildings has been related to two different values of acceleration.

Guagenti and Petrini (1989) have studied a set of damaged buildings made of masonry: the buildings belong to the historical city centres of the villages of Venzone (Udine, Intensity IX MCS of May 1976 earthquake), Tarcento and San Daniele (Udine, Intensity VIII MCS of May 1976 earthquake); some other buildings have been added from the 1984 Parco d'Abruzzo earthquake (Intensity VII MCS).

The level of damage of each building has been considered, as well as the level of the acceleration of the ground, estimated by the equation (see § 2.2.3): $\ln(y) = 0.602 \cdot I - 7.073$.

Guagenti and Petrini have studied the model of vulnerability described above with the tri-linear law, arriving to the definition of the accelerations y_i and y_c by the following formulas (Guagenti and Petrini, 1989):

$$y_i = \alpha_i \cdot \exp(-\beta_i \cdot I_V) \quad y_c = (\alpha_c + \beta_c \cdot I_V^\gamma)^{-1}$$

with

- $\alpha_i = 0.08, \beta_i = 0.01950$
- $\alpha_c = 1.00, \beta_c = 0.00191, \gamma = 1.80$

It is possible to plot the model of vulnerability for masonry buildings (Figure 22), calculating the accelerations for all the possible values of the Vulnerability Index I_V .

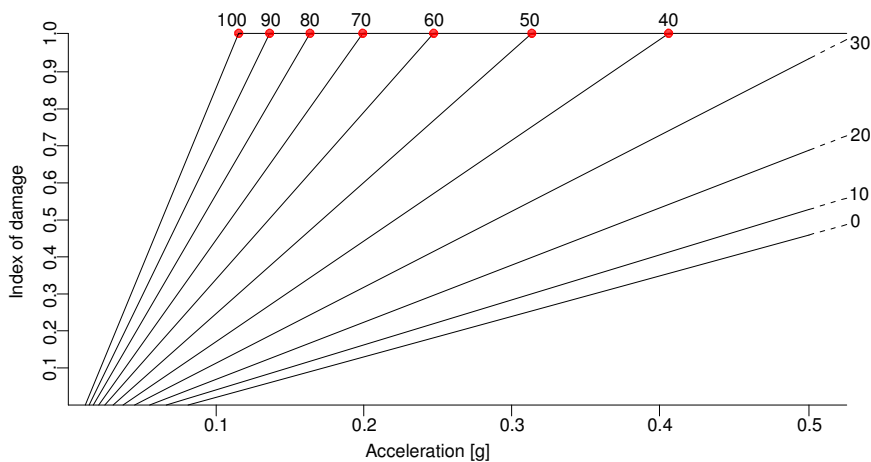


Figure 22: Model of Vulnerability for masonry structures. Relation among damage, acceleration and I_V (Guagenti and Petrini, 1989).

The red points shown in Figure 22 represent the values of acceleration of collapse for each value of the Index of Vulnerability ($I_V=100\%$, 90% , 80% ...), since they are related to an Index of damage equal to 1.0.

Some further studies on this model have been performed by Grimaz et al (1996), starting from the ones of Guagenti and Petrini (1989) and improving the evaluation of vulnerability: in particular, they have studied a more detailed method to define the vulnerability considering some other aspects, like the structural context where the building is (in case of buildings in aggregate), the difference of behaviour of a building due to the main direction of the seismic action, etc...

Analyzing a set of 352 buildings (damaged by earthquakes) both from the vulnerability and damage point of view, the authors have found a new vulnerability model, based on the same mathematical expressions, but with different values of the coefficients:

$$y_i = \alpha_i \cdot \exp(-\beta_i \cdot I_V) \qquad y_c = (\alpha_c + \beta_c \cdot I_V^\gamma)^{-1}$$

with

- $\alpha_i = 0.08$, $\beta_i = 0.013037$
- $\alpha_c = 1.5371$, $\beta_c = 0.00097401$, $\gamma = 1.8087$

In order to compare the two models described above, it is possible to plot the equations which define the two accelerations y_i and y_c , for each value of the Index of Vulnerability.

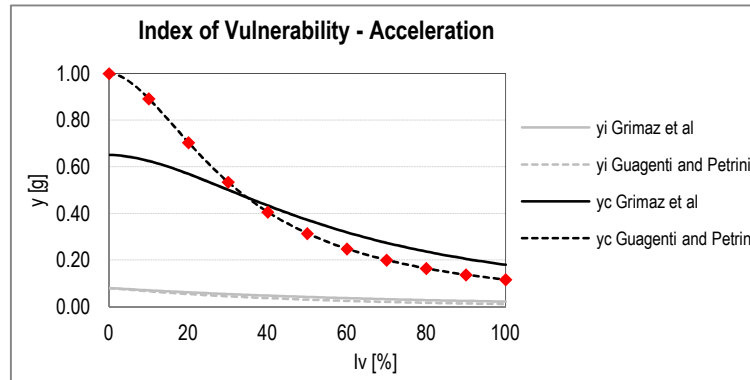


Figure 23: Comparison in terms of acceleration among the two models of vulnerability.

In the Figure 23, for each value of the Index of Vulnerability, the acceleration of beginning of the damage y_i (in grey lines) and the acceleration of collapse y_c (in black lines) are plotted; the two different approaches (Guagenti - Petrini and Grimaz et al.) are plotted with different styles (dashed and continuous lines).

The last graph (Figure 23) is strictly related to the one shown before (Figure 22): in particular, the red points shown in Figure 23 on the line which represents the acceleration of collapse for the Guagenti and Petrini approach are the same of the ones shown in Figure 22. In Figure 23 it is possible to observe, for example, that for an Index of Vulnerability $I_V = 70\%$, the related acceleration of collapse for the Guagenti and Petrini approach is $y_c = 0.20 g$; in Figure 22 instead, looking the tri-linear law of the relation acceleration/damage corresponding to the $I_V = 70\%$, it is possible to see that the third part of the tri-linear graph (horizontal branch that highlights an index of damage of 1.0) starts in correspondence of an acceleration $y = 0.20 g$.

Observing the graph in Figure 23, it is possible to see that the acceleration of beginning of the damage, y_i , does not vary so much between the two different approaches; the acceleration of collapse instead, y_c , shows a sensible variation: in particular, for low values of the Index of Vulnerability, the original formulation led to higher values of accelerations, reaching $1.00 g$ for a building with $I_V = 0\%$, while the modified version of the model arrives to a maximum of $0.65 g$. The original model of vulnerability gives higher values of accelerations than the modified one until $I_V = 35\%$, where the trend changes: the expected accelerations for the original model start to be lower than the modified one, until the end of the possible values of the Index of Vulnerability.

3 The sample of analysis: Hospitals structures of Florence, Prato and Pistoia

In this chapter, the sample of analysis which represents the basis of the present work is illustrated.

The vulnerability of this group of buildings has been analyzed with an empirical approach (Vulnerability Index Method) in 2010-2011 by the DICEA-UNIFI Research Group; this sample of structures belongs to a wider Research Program among the Universities of Florence (Engineering and Architecture Faculties) and Pisa (Engineering Faculty), which have cooperated to assess the vulnerability of the Hospital Structures of all the Tuscany Region.

The method of the survey campaign is here described and the first statistical results on the collected data are proposed, leading to some general conclusions about the vulnerability of the sample.

3.1 Introduction

Seismic prevention in Italy has been performed by means of two different instruments: the classification of the seismic zones and the seismic codes for the design of the constructions.

Considering the first aspect it is possible to observe that, from 1908 (year of the Messina and Reggio Calabria earthquake) until 1974, the Italian Municipalities have been classified as belonging to seismic zones (and consequently subjected to more restrictive rules) only after a destroying seismic event.

In 1974, the law L. 02/02/1974, n.° 64 has established that the seismic classification must be performed only by means of technical-scientific motivations, through Decrees of the Public Works Minister.

Further developments have led to the definition of three different seismic categories, including only the 45% of the Italian territory.

After the earthquake of 2002 of Molise Region, a new structural code, with particular attention to the seismic aspects, has been adopted: O.P.C.M. 3274 (2003). This document has reclassified the entire Italian territory as seismic, identifying four zones with different levels of seismic hazard; the not-classified zones have been cancelled.

The O.P.C.M. 3274 (2003) required also the seismic verification for:

- buildings which have a strategic interest or function for the community;
- infrastructural elements which have a functionality that assumes a fundamental importance for the civil protection in case of emergency;
- buildings or infrastructures which can assume relevance due to the consequences of a collapse.

This verification is mandatory for buildings designed and realized with codes published before 1984.

In order to satisfy these requirements, the Administration of the Tuscany Region, in the year 2009, has requested a collaboration to the Universities of Florence and Pisa in order to start a monitoring campaign of the seismic vulnerability of all its Hospital structures.

3.2 Health Organization of Tuscany Region

The Hospital buildings of the Tuscany Region are subdivided in 16 Healthcare Companies: this is a territorial subdivision, due to the Provincial Administrations; each city of Tuscany has one Healthcare Company at least, some of them have more than one.

Each Company is composed of a certain number of Complexes: this subdivision is both territorial and functional one.

In order to deepen the level of detail, each Complex is structurally composed of a certain number of Structural Units: this subdivision has an engineering meaning, because it is made looking the structural organization and evolution of the Complex. Structural unit is defined as a "building which has structural continuity from the ground until the top for the gravity loads and it is normally delimited by open spaces, structural joints or buildings realized with different construction methods" (D.M. 2008 - § 8.7.1).

The Universities of Florence and Pisa have investigated a total of 80 Complexes, which are composed of 533 Structural Units (in the following, S.U.), corresponding to 5130000 m³ of built volume: this is the original sample of buildings analyzed with an empirical method (2° Level Vulnerability Form - GNDT, 1993) in order to find out a statistical description of the Hospitals of Tuscany Region and in order to make a classification of the buildings most vulnerable and the most exposed to the seismic risk (De Stefano et al, 2012). This classification is a useful tool for the Tuscany Administration, because the Administration itself has to decide where to invest the economical resources for more detailed investigations and, if necessary, for projects of retrofiting: a list of priorities can guide the decisions, allowing a correct use of the monetary resources.

3.3 DICEA Research Group sample: Hospital structures of Florence, Prato and Pistoia

As described before, the DICEA-UNIFI Research Group has participated to the survey campaign of the seismic vulnerability of the Hospital buildings in Tuscany: in particular, the Group has worked in the cities of Florence, Prato and Pistoia.

The analyzed sample of buildings is composed of 4 Healthcare Companies (AUSL 3 - Pistoia, AUSL 4 - Prato, AUSL 10 and AOUC - Firenze), consisting of a total of 32 Complexes; these aggregates can be subdivided in 219 S.U., which correspond to 1240000 m³ of built volume.

In the following, a general overview of the main features of the sample is proposed: at first, the period of construction of the S.U. is investigated as well as the average dimensions; these characteristics are analyzed for the whole sample of buildings at first, considering then if each structural construction material has a particular or different trend comparing to the whole sample.

Later, a general technological description is reported, illustrating the most common structural typologies observed during the surveys: floors, roofs, types of masonry and so on.

3.3.1 General features of the sample of buildings

As written before, the sample of buildings of the present work is composed of 219 Structural Units.

Looking the material of construction, it is possible to divide the sample in two different categories:

- 118 masonry units (54% of the S.U., 49% of the total volume);
- 101 reinforced concrete units (46% of the S.U., 51% of the total volume).

Considering the period of construction, it has been possible to observe the development of realization of the Hospital structures during the last centuries: in particular, looking Figure 24, more than a third of the total volume of the sample has been realized in the period 1963-1975. Starting from 1962, the periods of construction are identified by the history of the seismic codes.

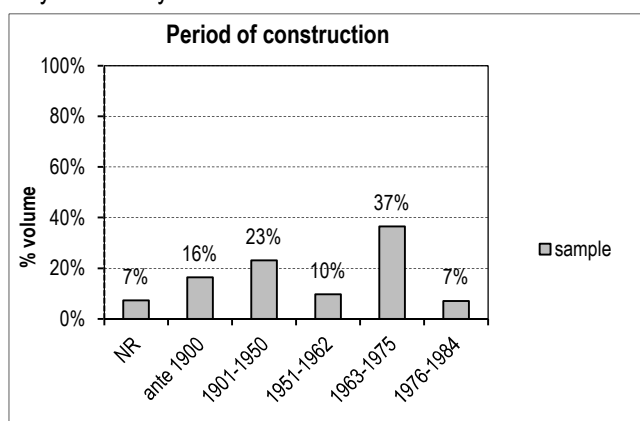


Figure 24: Period of construction of the sample of buildings.

Considering the distributions of the construction period in a separate way for each material of construction (Figure 25), it is possible to see that the reinforced concrete S.U. are more recent on average, mainly realized in the period 1963-1975, while the masonry ones have been realized mainly in the first half of the last century.

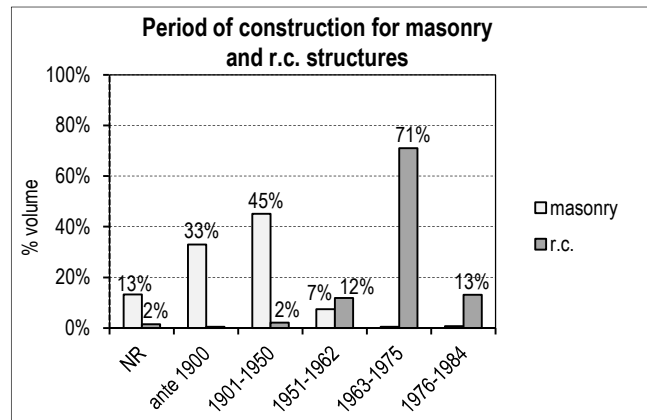


Figure 25: Period of construction of the buildings separately analyzed for masonry and reinforced concrete structures.

Considering the distribution of the volume of the S.U. (Figure 26), it is possible to see that 36% of the sample has a volume lower than 2000 m³, while almost the 85% of the S.U. has a volume lower than 10000 m³. However, there are some cases where the volume of the S.U. reaches higher values, up to 30000 m³.

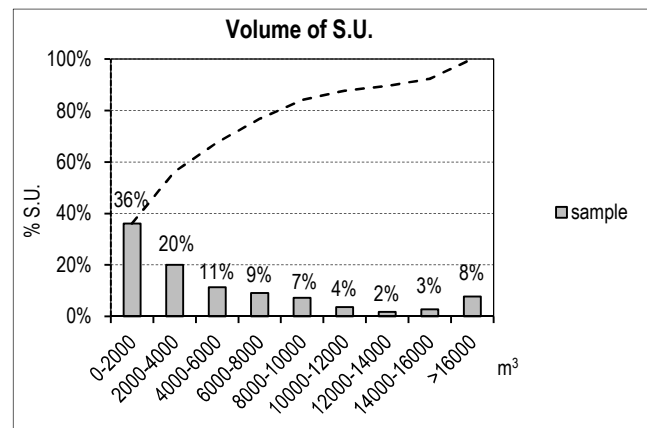


Figure 26: Distribution of the volume of the sample of buildings (the dashed line represents the cumulative curve).

The distributions of volume separately realized for the two materials of constructions (Figure 27) show that the reinforced concrete structures are, on average, bigger than masonry structures, even though volumes bigger than 16000 m³ are possible for both the materials of construction.

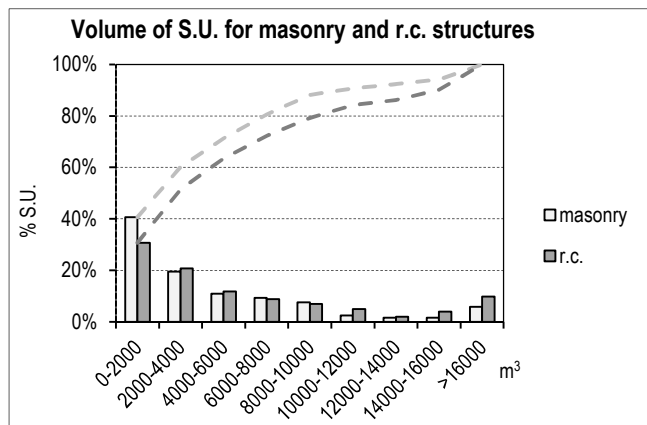


Figure 27: Distributions of the volume for masonry and r.c. structures (the dashed lines represent the cumulative curves).

3.3.2 Technological aspects

From the technological point of view, masonry buildings have different typologies of construction (due to the different periods of realization): in older buildings it is possible to find disorganized stone masonries, flexible floors such as steel beams and hollow tiles floors, masonry barrel and cross vaults; in some cases reinforced concrete with hollow tiles floors have been found, probably realized in subsequent phases of structural interventions on the buildings. Also the roofs are realized with different technologies such as wood, tiles with steel reinforcements (without the concrete slab over the tiles) or reinforced concrete with hollow tiles. More recent structures (built in the second part of the last century) are made of stones with brick layers masonry or directly with brick masonry and, in general, reinforced concrete with hollow tiles floors and reinforced concrete beams on the edges of each floor. In some buildings it was possible to observe the presence of added levels, realized with more recent technologies such as concrete blocks.

Reinforced concrete buildings are composed of framed structures made of parallel frames, with cross connections often realized only with small beams within the thickness of the floor; the floors are generally realized with reinforced concrete with hollow tiles. Stairs and elevators are made of reinforced concrete too. Foundations are realized with beams or with concrete slabs with stiffening ribs. In general, big hospital buildings are realized with independent framed structure blocks, separated by small technical joints (at least 1-2 cm), which are able to allow the thermal deformations but they are not able to avoid the problems connected to the interaction among contiguous S.U. under seismic action (pounding phenomenon).

Generally speaking, it is possible to assess that the masonry buildings of the sample are realized with a medium-high quality level: this aspect is probably due to the fact that these structures were created directly for a public or, more in general, for a relevant function (even though not for Hospital purposes in the beginning); for these buildings the technological level used in the realization was on average higher than the one adopted for normal residential structures.

The reinforced concrete structures instead suffer of all the problems related to their period of construction: indeed, it is well known that frame structures realized in the '60s and '70s of the past century were designed only considering the gravitational load aspects, without taking into account the problems connected to the earthquakes. It is easy to find high r.c. buildings placed one next to the other with only few centimetres of technical joint (which can lead to pounding effects under a dynamic solicitation such as an earthquake) as well as frame structures with very rigid beams and flexible columns (shear type scheme), which nowadays does not represent anymore the correct design philosophy of reinforced concrete structures, since the resistance hierarchy must be adopted for the design of new buildings.

3.3.3 The methodology of survey

A standardized procedure of survey has been developed from the DICEA-UNIFI Research Group in order to obtain a homogeneous description of all the S.U. of the sample; the survey was divided into different parts that have led to the realization of descriptive reports, organized in the following way:

- description of the Hospital Aggregate (or Complex) from the historical - functional point of view, trying to highlight the evolution phases of the aggregate;
- identification of the S.U. which the aggregate is composed of, through the observation of the technical joints or relevant structural discontinuities (Figure 28);
- brief description of each S.U. (number of stories, volume, shape of the building, period of realization, construction typologies, state of conservation etc...);



Figure 28: Example of individuation of the three S.U. in the Complex AOUC Careggi - 4.

- technical summary of the S.U. survey, which is a checklist of the procedures of inspection; in this document, the acquired photographic documentation is listed, the correspondence of the building to the technical documentation is assessed (to be verified by some in-situ measurements), the number of the buildings and every particular feature (presence of added levels, overhangs, internal courts, open spaces etc...), the typological-structural layout of the considered S.U. performed for each storey, the examination of possible local problems and, in the end, the description of particular situations which have relevance from the structural point of view;
- photographic documentation of the S.U. (Figure 29) and technical drawings (Figure 30) with the localization of the points of view for each photo, highlighting the aspects that are relevant for the definition of the vulnerability (stairs, elevators, situations of visible deterioration etc...);



Figure 29: Extract from the photographic documentation of AOUC Careggi-4, U.S.1.

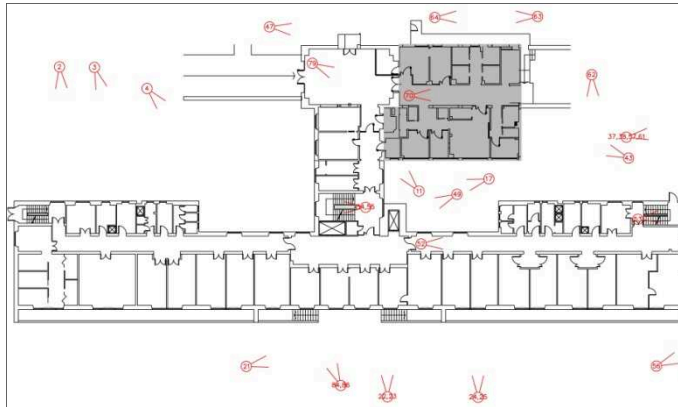


Figure 30: Example of plan with points of view of the photographic documentation (AOUC Careggi-4, U.S.1, ground floor).

- technical plans of the S.U. with the indication of the control measures carried out during the in-situ survey, the typology and direction of the structural elements of the floors, as well as the functional utilization of each room;
- 2° level Vulnerability Form (GNDT, 1993) and calculation of the Index of Vulnerability.

All the collected information have allowed the realization of a knowledge report for each Complex, analyzing each S.U.. The document describes in detail the buildings with historical and functional information, technical drawings, photographic documentation, typological-structural descriptions of the S.U., indications of any possible critical situation or deterioration aspects; the 2° level Vulnerability Form is also reported for each S.U. with the calculation of the Index I_v .

Once that all the I_v have been calculated for each S.U., the vulnerability classification for all the sample has been obtained.

This documentation represents a wide database of information, useful for the Healthcare System of the Tuscany Region in order to manage its building heritage.

3.3.4 The vulnerability assessment: hypothesis

The 2° level Vulnerability Forms proposed by GNDT have been compiled following some assumptions: for both materials of construction, scores and weight coefficients indicated in the Marche Region - Seismic Risk documentation have been used. In the following paragraph, the hypotheses used for this part of the work are described.

For masonry buildings, the mechanical properties shown in Tab. C8A.2.1 of C.M. 617/2009 have been considered in order to evaluate the parameter n.° 3 (Conventional resistance), taking as reference the minimum values of the proposed range for the average shear resistance of the masonry τ_0 .

The confidence factor on the mechanical properties has been assumed as $CF=1$, corresponding to the detailed level of knowledge (called LC3 in the D.M. 2008). Once the Index of Vulnerability has been calculated by the weighted sum of the scores (values in the range [0; 382.5]), a normalization has been performed, in order to obtain values within the [0%; 100%] range.

For reinforced concrete buildings, in order to evaluate the parameter n.° 3 (Conventional resistance), the shear resistance has been considered with the value

$$\tau_k = 1500 \text{ kN/m}^2$$

which is the minimum value of the range [1500; 2500] kN/m^2 proposed by the Handbook of the Vulnerability Form for reinforced concrete structures.

The Index of Vulnerability for reinforced concrete structures is then obtained by the sum of the scores (the weights for each parameter are all equal to 1.0 for reinforced concrete structures); the Index assumes values

in the range [-9.70; +0.25] and it has been converted in a directly comparable masonry structure Index using the conversion formulas proposed by the Marche Region:

$$I_{Vr.c_structures} > -6.50 \rightarrow I_V = -10.07 \cdot I_{Vr.c_structures} + 2.52$$

$$I_{Vr.c_structures} < -6.50 \rightarrow I_V = -1.73 \cdot I_{Vr.c_structures} + 56.72$$

As written in § 2.4.3 of the present work, these conversion formulas calculate the equivalent Index of Vulnerability for reinforced concrete, which belongs to the [0%-73.5%] range, while the masonry structures can even reach the value of 100%: therefore, this method implicitly assumes that, statistically speaking, reinforced concrete structures cannot reach levels of vulnerability higher than masonry structures.

3.3.5 Results of the vulnerability campaign and data treatment

Once that the index I_V has been calculated for each S.U. through the application of the 2° Level Vulnerability Form, the ranking of vulnerability has been obtained for the whole sample of buildings: the values of the Index belong to the range [5%; 81%], where the 1st value corresponds to the least vulnerable S.U. and the 2nd value to the most vulnerable. The analysis of the results for the entire sample of structures (Figure 31) highlights that about 80% of buildings has the index I_V included in the range [30%; 70%]; moreover, a third of the sample belongs to the range of vulnerability [50%; 60%].

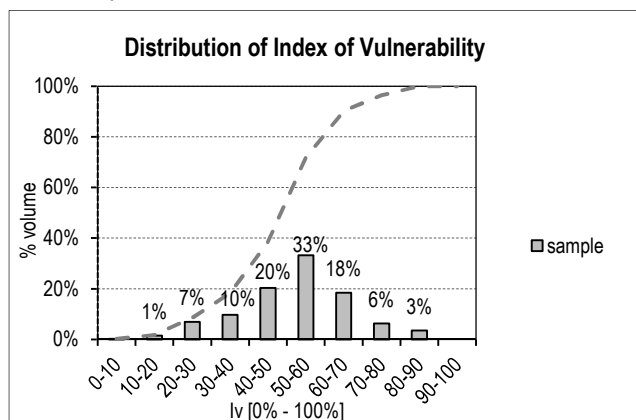


Figure 31: Distribution of the I_V for the whole sample of buildings (the dashed line represents the cumulative curve).

From the trend of the cumulative curve (Figure 31), it is possible to observe that about 40% of the structures has $I_V \leq 50\%$. The cases with high vulnerability index ($I_V \geq 70\%$) represent about the 9% of the sample.

The distributions of the Vulnerability Index have been analyzed in a separate way (Figure 32) for the two materials of construction (masonry and reinforced concrete); in the following Figure 32, the graph is proposed.

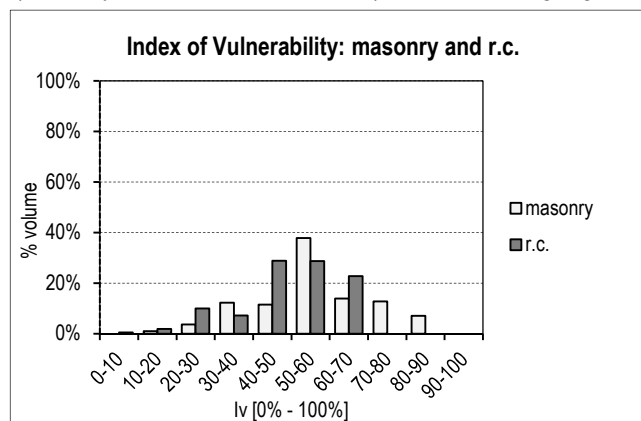


Figure 32: Distributions of the I_V for masonry and reinforced concrete structures.

From the previous histogram, it is possible to observe that:

- the reinforced concrete buildings always show a value of $I_v \leq 70\%$, while the masonry ones have more critical cases, with the Index that reaches values $I_v \geq 80\%$;
- almost 90% of the reinforced concrete structures has the index I_v included in the range [30%; 70%], while only the 75% of the masonry structures has the index belonging to the same range;
- about 40% of the S.U. made of masonry has the Index within the range [50%; 60%], while the reinforced concrete structures do not reach the 30% of the S.U. for the same range.

In order to get a general overview about the relation Vulnerability / Material of construction, it is useful to plot the cumulative curves of the previous histograms, comparing them with the cumulative curve of the whole sample of buildings (Figure 33). Observing the trend of these curves, it is possible to see that the cumulative curve for reinforced concrete structures is always higher than the masonry one: this means that r.c. constructions have, on average, a lower level of vulnerability than masonry buildings.

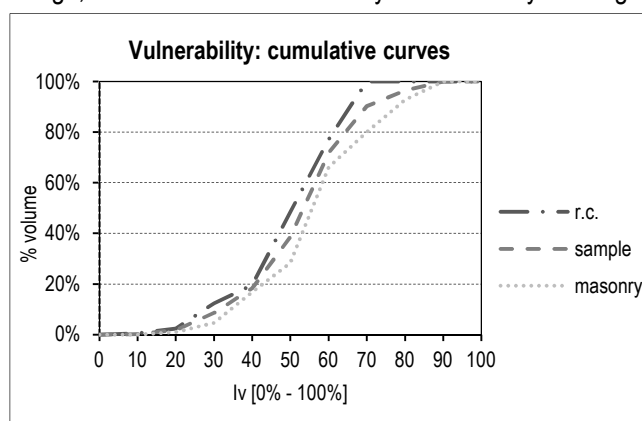


Figure 33: Comparison among the cumulative curves of the I_v distributions for masonry and r.c. structures.

A confirmation of the difference of vulnerability among the two materials comes also from the observation of the percentages of S.U. which show an index $I_v \leq 50\%$: for reinforced concrete structures, the percentage of volume of structures satisfying this condition reaches the value of 48.5%, while for masonry structures this percentage is significantly lower (28.3%).

With the collected data, it has been also possible to study the relation among the Index of Vulnerability and the period of construction. In the following, some conclusions can be obtained just observing this relation through Figure 34.

As seen in the previous part (Figure 24), 37% of the buildings (about 453000 m³) has been realized in the period 1963-1975: observing the graph of Figure 34, an amount of almost 285000 m³ of structures (which represents the 63% of the 1963-1975 Hospital buildings) have a Vulnerability Index in the range [40%-60%]. The presence of almost 130000 m³ of buildings (which represents the 29% of the 1963-1975 Hospital buildings) in the same range of period which show a high level of vulnerability ($60\% \leq I_v \leq 80\%$) is remarkable as well.

Looking to the buildings which are characterized by a high level of Vulnerability ($I_v \geq 80\%$), it is possible to observe that they have been realized in the first half of the last century. On the other hand, Hospital buildings realized after 1976 (which represent only the 7% of the total volume of the sample) are characterized by low levels of vulnerability ($I_v \leq 40\%$).

However, it is interesting to observe that there is a small part of buildings, realized in not recent periods (before 1950), which shows a low Index of Vulnerability ($I_v \leq 40\%$).

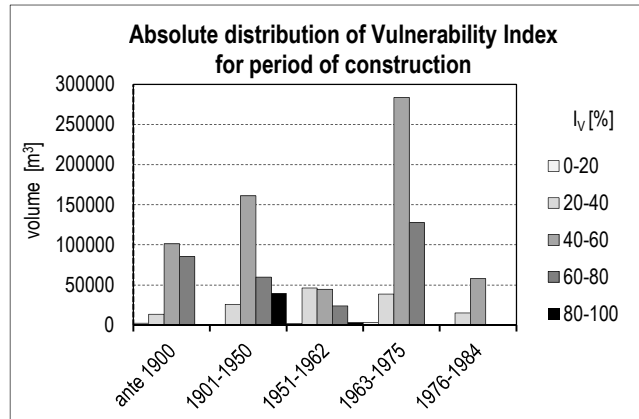


Figure 34: Absolute distribution of Vulnerability in the different periods of construction.

The previous graph (Figure 34) has been realized in absolute terms, in order to highlight also the differences of realization among each period, as already performed above in Figure 24.

The introduction of always more updated seismic codes has led to a change of the tradition of construction: this aspect is clear looking the distributions of the vulnerability (Figure 35) in the two macro-historical periods "ante 1962" and "post 1962" (in that year, the law n.° 1684 of 25/11 has been issued "Provvedimenti per l'edilizia, con particolari prescrizioni per le zone sismiche"). This particular division has been performed in order to separate the total volume of the Hospital buildings in two similar quantities (610000 m³ realized before 1962, 540000 m³ after 1962), even if the considered law has not modified in a significant way the prescriptions for the realization of buildings.

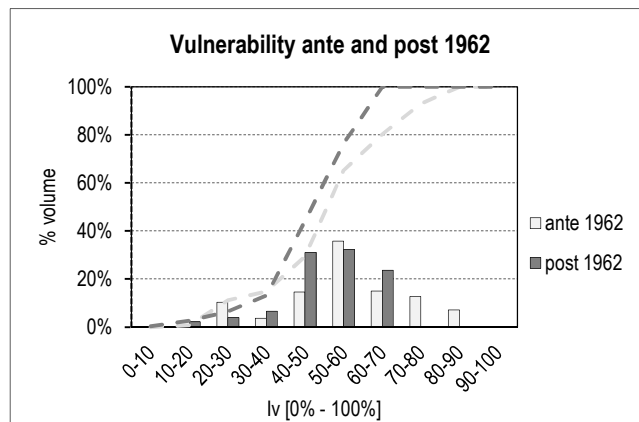


Figure 35: Distribution of vulnerability ante and post 1962 (the dashed lines represent the cumulative curves).

In the previous Figure 35, it is possible to observe that the cumulative curve of the ante 1962 buildings lies almost always under the post 1962 buildings curve: this means that the older buildings have, on average, higher levels of vulnerability. Moreover, it is important to highlight that the highest levels of vulnerability [$I_v \geq 80\%$] are related to buildings realized before 1962.

With the same procedure, the sample has been divided again in two other macro-historical periods "ante 1975" and "post 1975" (Figure 36): in this case, this subdivision has been performed in order to highlight the influence of a significant design code, the L. 02/02/1974 n.° 64 (1974) and its development D.M. 1975, which have strongly modified the requirements for the constructions (introducing, for example, the response spectrum as a function of the natural period of the structure for the calculation of the accelerations on the structure itself).

This temporal subdivision leads to two groups of buildings which do not have the same size: the buildings realized before 1975 in fact represent about the 86% of the entire sample; anyway, the percentage distributions of vulnerability for the two different groups can give useful information about the influence of the development of the codes on the vulnerability aspects.

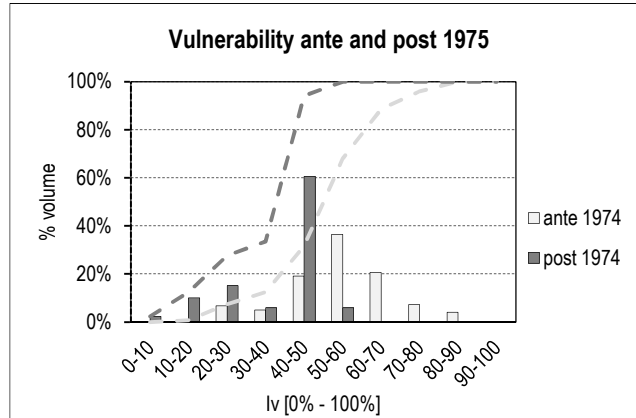


Figure 36: Distribution of vulnerability ante and post 1975 (the dashed lines represent the cumulative curves).

The difference among the two groups is considerable (see Figure 36): the cumulative curve of the ante 1974 buildings lies under the post 1974 one, with a relevant difference.

The buildings characterized by an Index of Vulnerability $I_v \geq 60\%$ belong only to the ante 1974 structures category, while almost all the post 1974 buildings (the 94% exactly) have an $I_v \leq 50\%$.

Resuming what observed in this chapter, it is possible to affirm:

- the sample of buildings is composed of masonry and reinforced concrete structures, each of them representing approximately half of the sample;
- most of the buildings have been realized after 1900 (93% of the total volume). A third of the sample has been realized in the period 1963-1975;
- masonry structures are, on average, older than reinforced concrete ones: in particular, masonry structures have been mostly realized before 1950, while reinforced concrete structures belong to the period 1963-1984;
- the distributions of the Index of Vulnerability, calculated through the 2° Level Vulnerability Form (GNDDT 1993) show that reinforced concrete structures are characterized, on average, by a lower level of vulnerability, even if the difference is not so relevant;
- older buildings are, on average, more vulnerable than more recent ones.

4 Masonry structures: relation among empirical and detailed vulnerability analyses

In this chapter, the relation among the Vulnerability Index Method (empirical expeditious method) and a detailed analytical method for the estimation of the vulnerability of masonry structures is investigated, considering the DICEA Research Group sample (Hospital structures of Florence, Prato and Pistoia).

In particular, static non linear analyses will be carried out on a subset of the masonry sample (20 buildings on a total of 118) and the relation among the results of the two methods will be compared.

4.1 Introduction

The Vulnerability Index Method is a popular method in Italy for the expeditious vulnerability assessment of a wide sample of buildings: since the economical resources of each Administration are limited, in the aim of the seismic risk reduction, it is necessary to obtain a first general overview of the vulnerability for the considered sample, highlighting the most critical cases.

Generally speaking, the Vulnerability Index Method is used to get a classification of vulnerability of the analyzed sample of structures: this ranking is a useful instrument for the Administrations, in order to decide which structures should be better analyzed at first by means of detailed analyses.

Since this method is expeditious, it gives only a first estimation of the level of vulnerability by means of the Index of Vulnerability, I_v , in the range 0%-100%, but it has not a direct meaning looking the level of safety of the singular object: for example, if a building has a $I_v=45\%$, is it possible to consider it as safe against earthquakes?

The original method has been developed in the '80s by Benedetti and Petrini as described above, and it has been improved through the observation of the damages on the structures caused by the seismic events of the last decades: a clear example is the modification performed on the method by Tuscany Region (see § 2.4.4).

The knowledge about the seismic engineering is improving day by day: the performances of the materials have been widely investigated by the Scientific Research all over the world as well as the behaviour of the structural elements in general, the methods of analysis and the numerical codes for them.

The idea of this part of the work is to investigate the features of the Vulnerability Index Method and its relation with detailed analyses, in order to obtain more information about the seismic behaviour of the structures simply using an expeditious method of vulnerability assessment.

*It is important to highlight that, from this point, the remaining part of the present work is focused on the analysis of **masonry buildings**, considering both the expeditious method of the Vulnerability Index and the detailed analyses by numerical models.*

As described in the Chapter 3 of this work, many data have been collected for the Hospital Buildings of the cities of Florence, Prato and Pistoia. In particular, 118 Structural Units (S.U.) have been investigated in order to calculate the Index of Vulnerability and, consequently, the classification in terms of vulnerability has been obtained.

The idea of this part of the work is to analyze a subset of buildings with a detailed approach, in order to investigate the possible relation among the results of this analysis with the one of the Vulnerability Index Method. Since the realization of detailed three dimensional models, the execution of non-linear analyses and the treatment of all the results require a considerable amount of work, it has been decided to investigate a subset composed of 20 S.U., which represents 17% of the entire masonry buildings sample in terms of number of S.U. and 42% of it in terms of volume.

RELATIVE RANKING	S.U. IDENTIFICATION	VOLUME [m ³]	I_v [%]
1	AUSL4 MD 01 24	8 467	70.92
2	AUSL3 SMP 01 03	11 202	69.61
3	FI10 SERR 01	29 655	60.13
4	AUSL 3 PES 01 06	21 280	59.15
5	FI10 IOT ANT 01	11 107	58.82
6	AOUC CAR 16 01	16 056	56.21
7	AOUC CAR 4 01	7 590	52.29
8	AOUC CAR 13 03	8 200	45.42
9	AOUC CAR 13 04	8 527	45.12
10	CAR 8b 03	31 363	44.12
11	AOUC CAR 4 02	2 937	43.14
12	AOUC CAR 13 01	29 500	40.20
13	AOUC CAR 26 02	12 671	38.89
14	AOUC CAR 26 03	12 671	38.89
15	CAR 8b 01	5 271	38.89
16	CAR 8b 02	1 719	38.89
17	CAR 8b 04	8 238	37.58
18	AOUC CAR 26 01	9 945	37.58
19	AUSL3 PES 01 04	14 105	35.62
20	AUSL3 SMP 01 04	4 185	34.97

Table 3: List of the subset of masonry S.U. chosen for the detailed analyses. The list is ordered by the I_v .

In order to understand if this group of buildings is representative for the entire sample of masonry structures, the I_v is plotted in Figure 37, highlighting the boundary values found on the whole sample (118 S.U.).

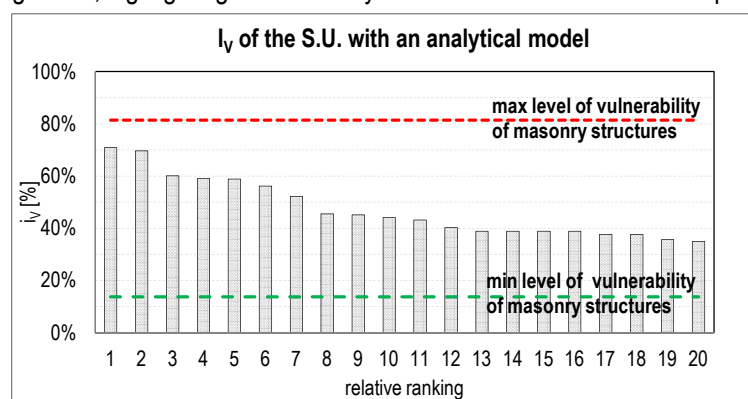


Figure 37: I_v of the subset of masonry S.U. The dashed lines represent the boundaries of the entire sample of masonry structures.

The chosen subset of S.U. covers a considerable part of the I_v range of the entire masonry structures sample (118 S.U.), even if the extreme cases have not been considered as case studies. In particular, the entire sample of S.U. has values of I_v in the range [13.7% - 80.4%], while the subset considers the range [35.0% - 70.9%]. It is important to highlight that, in the total sample, almost 75% of the S.U. belongs to the vulnerability range of the chosen subset. This aspect ensures reliability to the further analyses, without focusing only on a particular "slice" of the I_v .

The subset of S.U. will be analyzed with detailed analyses; an indicator of the vulnerability will be obtained, allowing the creation of another vulnerability classification of the subset and, consequently, a direct comparison with the results of the empirical method, already proposed in Table 3.

4.2 Hypotheses of the work

4.2.1 Local or global analysis?

Generally speaking, a masonry wall works in two different ways, depending on the direction of the solicitation:

- out of plane direction, which represents the weakest direction, since the most resistant section is not oriented along the direction of the solicitation (1st case of Figure 38);
- in plane direction, which is the real resistant behaviour of the wall, because the direction of solicitation is the same of the one with maximum inertia (2nd case of Figure 38).

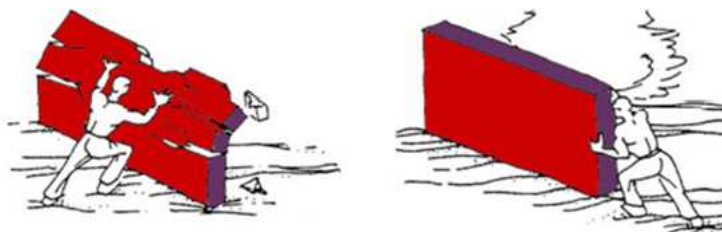


Figure 38: On the left: out of plane behaviour of a wall. On the right: in plane behaviour. (Touliatos, 1996)

Starting from this, it is possible to identify the collapse ways which a wall can experience, due to the level of connection among each wall and among walls and floors:

- 1st failure mode, caused by the realization of out of plane mechanisms. Each wall is not well connected with the other parts of the structure and the horizontal solicitation can cause simple or composed overturning of the wall, horizontal or vertical bending, overturning of the corner and so on.

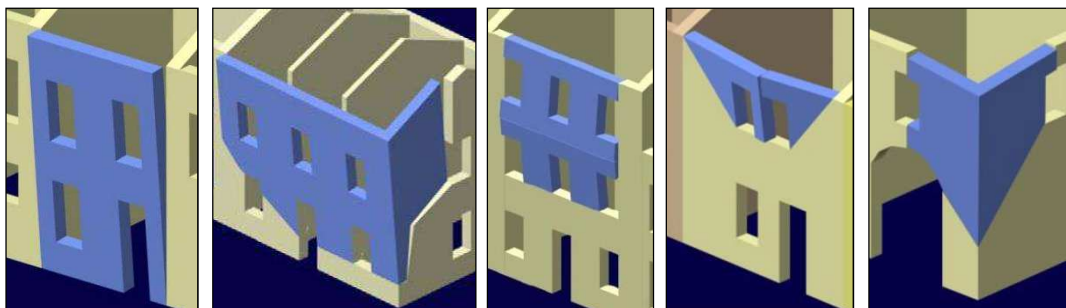


Figure 39: Typologies of out of plane mechanisms (www.reluis.it)

They are called "local mechanisms", in order to underline the local behaviour of a part of the structure, independently from the remaining part: this type of behaviour can be caused by many aspects, such as the lack of connections among perpendicular walls, poor connections among floors or roofs and walls etc... All these aspects contribute to the non-global behaviour of the structure, basic aspect for a good seismic response of masonry structures;

- 2nd failure mode, also called "in-plane mechanisms" of the walls, which can occur only when the structure has a box behaviour (Figure 40), ensured by effective connections among floors and walls. The structure behaves globally and each singular wall can contribute to the resistance of the building: in particular, the walls can work in their plane, having a higher stiffness and resistance than the previous case.

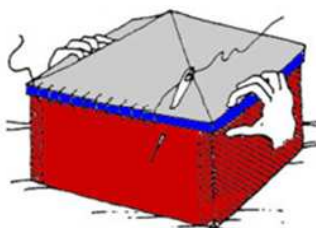


Figure 40: Representation of the box behaviour. (Touliatos, 1996)

The walls collapse after reaching the maximum in-plane shear resistance. The typologies of collapse are: shear due to bending collapse, shear sliding and shear diagonal cracking; these ways of collapse depend on geometry, restraint conditions and material properties of the masonry.

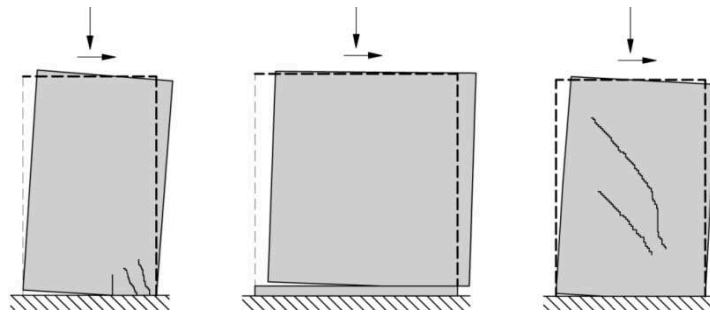


Figure 41: In-plane collapse of masonry walls. From the left: bending, sliding and diagonal cracking.

When an evaluation of the security level of a masonry building has to be performed, local mechanisms must be analyzed at first, in order to avoid local collapses and, as consequence, make a reliable global analysis. Indeed, if local mechanisms occur, a global analysis has no realistic sense.

As described above, local collapses are due to low quality construction methods while the box behaviour can be ensured only by well realized masonry buildings.

Masonry hospital structures are generally located in medium-high quality buildings (as described for the case study in § 3.3.2), probably because of their social function: it is reasonable to assume that the local mechanisms aspects do not represent the main problem of this category of structures, even though the local analysis is always a necessary step in the seismic evaluation of any kind of structure. This hypothesis has been considered also in other works on public buildings such as schools and hospitals: in particular, an important survey campaign has been performed on all the public and strategic buildings of the central and southern Italy in the recent years by INGV and GNDT (Istituto Nazionale di Geofisica e Vulcanologia and Gruppo Nazionale per la Difesa dai Terremoti); starting from that campaign, a new model for the fast vulnerability evaluation, by means of an hybrid approach, has been developed: the SAVE Project, briefly described in § 2.4.6. The method is based on the assumption of a global behaviour of masonry structures, because of the general medium-high quality of the constructive methods which have been observed in the buildings of that sample (Dolce and Martinelli, 2005).

For these reasons, the detailed analyses of the present work will focus only on the global behaviour, considering the local mechanisms problem only with the empirical approach.

4.2.2 Level of knowledge of structures: materials, soil, topographic conditions

After the survey campaign of 2009-2010 on the Hospital structures, performed by the DICEA-UNIFI Research Group for the realization of the vulnerability classification using the empirical approach of the Vulnerability Index Method, an additional detailed survey has been performed for each building of the subset, in order to deepen the level of knowledge of the structural aspects: in particular, the typologies of masonries, floors and roofs have been better investigated, as well as the directions of the structural elements of each floor. Even the level of connection among the walls has been observed, when possible directly from the parts of walls where the plaster was deteriorated. Some information has been obtained even through the use of a thermal image camera, allowing the identification of the typology of masonry (when perfectly covered by plaster), typology and direction of the structural elements of the floors, presence of openings filled with masonry etc...



Figure 42: Typologies of masonry and roofs directly observable.



Figure 43: Thermal images: (left) stone masonry with brick layers, (middle) reinforced concrete floor, (right) brick masonry.

A complete inspection of all the building spaces was sometimes not possible (due to the particular utilizations of some areas) but the technical drawings of buildings were always available and they have been used to create the structural models.

During the survey campaign of 2009-2010 and even during the additional survey campaign on the buildings of the subset, no in situ tests have been performed in order to estimate the mechanical properties of the materials: the numerical models for the detailed analyses have been realized using the categories of materials of the actual Italian Code (D.M. 2008 and C.M. 617/2009).

Considering all these aspects, it is possible to affirm that the level of knowledge reached during the surveys is adequate to realize a detailed analysis, even though not all the information about the structures was directly available. Looking to the Italian Code, it is possible to see which are the requested information to fulfil a specific level of knowledge: three levels are available, from LC1 (lowest level of information) to LC3 (highest level of information); in this case, the LC1 level of knowledge has been selected, with a related Confidence Factor equal to 1.35 (to be applied to the mechanical properties of materials).

From the geotechnical point of view, an accurate investigation of the underground conditions for each building was not possible, so it has been decided to consider a common type of soil for all the buildings, referring to the actual Italian Code (D.M. 2008 and C.M. 617/2009).

In the code, five basic categories of stratigraphic profiles are available, mainly sorted by the average shear wave velocity and the typology of ground:

- A rock or other rock-like geological formation;
- B deposits of very dense sand, gravel, or very stiff clay (thickness more than 30 meters);
- C deposits of dense or medium-dense sand, gravel or stiff clay (thickness more than 30 meters);
- D deposits of low-dense sand, gravel or low consistency clay (thickness more than 30 meters);
- E profiles like C or D with thickness less than 20 meter on reference ground formation A.

There are also two other categories, with lower mechanical characteristics: they must be employed for particular cases. The first typology of ground (A) has been chosen for two main reasons:

- it is necessary to have a common reference soil for the detailed analyses, since the category of soil influences the displacement response spectrum and, consequently, the evaluation of the capacity; since the target of this work is to study the structural vulnerability of the hospital buildings, it is not useful to introduce another variable which is not directly related to the features of the structures;

- there is no specific information on the type of soil of each site of construction; the vulnerability index has been calculated only by visual inspection of the geotechnical condition, considering only the presence of staggered levels of foundation and the natural slope of the ground.

Similar considerations can be performed for the topographic categories of the site of constructions: the actual Italian Code has four types of categories, sorted by the medium inclination of the ground (from horizontal site to slopes higher than 30°): even in this case, the same category has been chosen in order to have a common base for all the models; in particular, the first one (T1 - horizontal soil or sub-horizontal) has been chosen.

Resuming, these choices about the geotechnical conditions were necessary in order to evaluate the vulnerability for all the buildings starting from a common geotechnical framework; the assumption of different categories of soil or of different topographic categories would have influenced the results, excluding the possibility of a direct comparison of the results themselves purely from a structural point of view.

In other words, all the buildings of the subset have been considered as placed in the same geotechnical conditions. The estimation of the vulnerability (expressed in terms of acceleration of capacity - PGA_c) can be then compared in a direct way among all the structures.

4.2.3 Types of global analysis: choice of the methodology

For the evaluation of the safety level of a structure, it is possible to adopt different methodologies of analyses: the actual Italian Code, according to the Eurocode 8 indications, provides four different types of analyses, two linear and two non-linear, briefly listed and described below in order of complexity:

- static linear analysis, based on the calculation of the equivalent static horizontal forces related to the 1st modal shape of the structure in the considered direction. The analysis is linear and the non-linear behaviour of the structure is considered by the adoption of a behaviour factor, which numerically reduces the demand (design spectrum). The loads are applied in a static way;
- dynamic linear analysis, based on the evaluation of the effects of a sufficient number of modes of vibration of the structure, such that the 85% of the global mass is activated (eventually considering additional modes which have a participation mass higher than 5%). The effects of each mode are then combined with a statistical method. As for the previous analysis, materials have linear response and the non-linear behaviour is considered by the adoption of the behaviour factor;
- static non linear analysis, carried out under conditions of constant gravity loads and monotonically increasing lateral horizontal loads. This analysis consists in the plotting of the relation "total base shear - control point displacement" (well known as capacity curve). The verification procedure is performed using an equivalent single degree of freedom (SDOF) system, necessary to find out the demand (in terms of displacement, obtained from the displacement elastic spectrum), point in which structural verifications must be conducted. Materials and structural resistant elements have non-linear behaviour (characterized by an initial elastic phase, a subsequent plastic one until the failure of the considered element) and lateral loads are statically applied;
- dynamic non linear analysis (time-history), obtained through the direct numerical integration of the differential equations of motion, using accelerograms to represent the ground motion and a non-linear model for the structure.

In this part of the work, static non linear analysis has been chosen to investigate the level of vulnerability of the structures: this choice has been adopted because linear analyses on masonry structures are quite simplistic methods for the assessment of the global level of security in such kind of structures; in particular, linear analyses are able to take into account all the non-linear aspects by means of a behaviour factor that is related only to the typology of structure (codes gives some recommended values) but they cannot give exact information about the response in the plastic range.

On the other hand, dynamic non-linear analyses represent the most accurate typology of analysis, but they have some disadvantages connected to the high number of parameters necessary to set the damage criteria (on which no information are available for the sample of structures, so literature values should be assumed); they also have higher computational costs comparing to static non-linear analyses. Since many models will be analyzed, this type of analysis has been considered not so convenient.

Static non-linear analyses give the pushover curve as result, element which represents the simplified envelope of the hysteretic cycles produced during the seismic oscillations; for this reason, the capacity curve is considered a reliable indicator of the post-elastic behaviour of the structure.

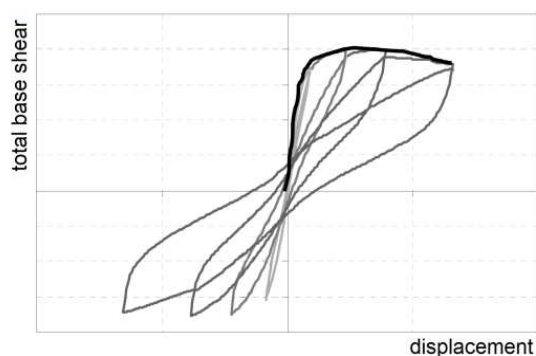


Figure 44: Relationship among the capacity curve (black line) and dynamic hysteretic cycles (grey lines).

Considering the pushover analyses, it is possible to assess that the capacity curve can be mainly obtained with two different approaches:

- Finite Element Method (F.E.M.); the structure is modelled with finite elements (surface or solid), which can be implemented with a damage law that takes into account the decay of the structural masonry properties, reducing the resistance and the elastic properties. This type of analysis requires the definition of a certain number of parameters (for the damage law) that needs a deep knowledge of the materials of the analyzed structure. This method, if properly set, leads to a punctual individuation of the local stress on the masonry;
- Frame by Macro Element (F.M.E.); the structure is modelled as an equivalent frame structure, with non-linear behaviour elements: beams and columns which respectively represent the spandrels and the piers of the building. The required input values are less than the F.E.M. modelling, because the elements have a simple behaviour, composed of an elastic part until the reaching of the maximum shear resistance, and a perfect plastic one, until the reaching of the maximum displacement (conventionally identified in the Italian Code as a percentage of the height of the element, considering the typology of collapse); the output of this methodology is not the local level of stress of the materials but the overall solicitations on each element mentioned above. The verifications are globally made in terms of displacements. The computational cost is lower than F.E.M. modelling, since the model is composed of a smaller number of elements and, consequently, the number of degrees of freedom to consider in the analysis is limited.

The choice among the two types of modelling has been made taking into account some aspects: first of all, the collected information on the sample, as written above, were not mainly focused on the exact definition of the mechanical properties of the materials of each building; a F.E.M. analysis could then bring to not fully reliable results just assuming damage parameters from literature.

Furthermore, the computational cost has been considered as a critical decision point because of the high number of analyses to perform (at least 8 for each building).

For all these aspects, F.M.E. modelling has been chosen to assess the vulnerability of the structures.

4.3 The levels of performance and the resilience concept

4.3.1 Introduction

The definition of the performance expectations for buildings began to evolve in the 1970's in the United States after the 1971 San Fernando earthquake (California, 6.6 magnitude - Richter Scale), event which has caused serious damages to many emergency facilities, in particular hospitals. After that, attention has been given to the buildings considered essential for a post-earthquake scenario (hospitals, fire stations and similar facilities): these buildings must continue to be operative after the occurrence of earthquakes.



Figure 45: Damages to the San Fernando Veterans Administration Hospital, built in 1926, before the development of earthquake building codes (left). Damages to the Medical Treatment and Care Building (Olive View Hospital complex), completed in 1970: the first storey has nearly collapsed and the towers containing the stairs (structurally separated from the building) have overturned (right).

At first, the design of such type of buildings has been modified by the introduction of an "importance factor", coefficient that increases the resistance of the structure by the amplification of the actions on the structure.

Each code has provided its own importance coefficient: for example, in the previous Italian Code (D.M. 1996) there was the definition of the seismic protection index "I", which had to be applied to horizontal forces induced by earthquake; this parameter could assume three values: 1.4 for the buildings which have primary importance for the civil protection, 1.2 for the buildings that have a particular risk connected to their use and 1.0 for all the other buildings (ordinary structures). This method certainly increases the resistance of the structure but gives no insurance about the operational aspect of the essential facilities.

The 1994 Northridge earthquake (California, 6.7 magnitude - Richter Scale) has led several damages again, highlighting some other problems in the design of structures. Some hospitals were not utilizable after the event due to the non structural damages.

Some more developments in the direction of Performance Based Design have been done, understanding that the force based design was not adequate to ensure the full safety and utilization of the structures.

The Federal Emergency Management Agency (F.E.M.A.) gave some funds for the development of this topic to the Applied Technology Council (A.T.C.) and Building Seismic Safety Council (B.S.S.C.): the first result was the "N.E.H.R.P. (National Earthquake Hazards Reduction Program) Guidelines and Commentary for Seismic Rehabilitation of Buildings" - F.E.M.A. 273 (1997), document which has introduced the concept of *building seismic performance levels*. These levels are discrete points on a continuous scale, describing the buildings expected performances, or alternatively, how much damage, economic loss, and disruption may occur for a given earthquake.

Each Building Performance Level (BPL) is composed of a Structural Performance Level (SPL), which describes the damage condition of the structural systems, and a Non-structural Performance Level (NPL), that describes the damage condition of the non-structural systems.

A further development in this direction has been performed by the A.S.C.E. (American Society of Civil Engineers) with the "Pre-standard and commentary for the seismic rehabilitation of buildings" - F.E.M.A. 356 (2000), document which has studied all the levels defined above (described in the next pages), leading to the

definition of some charts where it is possible to individuate the requested performance level for the analyzed structure, looking separately the structural and non-structural components. In particular, for the structural components, there are two tables describing the performances of vertical and horizontal elements, with different descriptions for each material of construction.

For the non-structural components instead, there are three tables which describe separately the behaviour of:

- architectural components;
- mechanical, electrical and plumbing systems/components;
- contents.

4.3.2 Identification of the Structural Performance Levels (SPL)

The Structural Performance Levels concern the structural conditions of a building when a seismic event occurs; F.E.M.A. 273 (1997) has identified six levels, listed below:

- S-1 Immediate Occupancy, definable as the post-earthquake damage state that remains safe for the occupancy. Only very limited structural damages have occurred;
- S-2 Damage Control, defined as the level between Life Safety and Immediate Occupancy Performance Levels, it concerns the conditions when only a not relevant damage is allowed and, consequently, only short repair times and operation interruptions are possible;
- S-3 Life Safety, definable as the post-earthquake damage state that includes damages to structural components but it has a margin against partial or total collapse of the structure;
- S-4 Limited Safety, extends between Life Safety and Collapse Prevention Performance Levels;
- S-5 Collapse Prevention, post-earthquake damage state that includes damage to structural elements such that the structure continues to support gravity loads with no margin against collapse;
- S-6, Structural Performance Not Considered, level necessary to cover the situations where only non-structural aspects have to be considered.

In the following, an extract of the tables concerning the structural performances of vertical and horizontal elements is reported, as defined in F.E.M.A. 356 (2000).

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Unreinforced Masonry Infill Walls	Primary	Extensive cracking and crushing; portions of face course shed.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Secondary	Extensive crushing and shattering; some walls dislodge.	Same as primary.	Same as primary.
	Drift	0.6% transient or permanent	0.5% transient; 0.3% permanent	0.1% transient; negligible permanent
Unreinforced Masonry (Noninfill) Walls	Primary	Extensive cracking; face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor (<1/8" width) cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
	Secondary	Nonbearing panels dislodge.	Same as primary.	Same as primary.
	Drift	1% transient or permanent	0.6% transient; 0.6% permanent	0.3% transient; 0.3% permanent

Figure 46: Structural Performance Levels and Damage - Vertical elements - F.E.M.A. 356 (2000).

Table C1-4 Structural Performance Levels and Damage —Horizontal Elements

Element	Structural Performance Levels		
	Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Metal Deck Diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments.	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck.	Connections between deck units and framing intact. Minor distortions.
Wood Diaphragms	Large permanent distortion with partial withdrawal of nails and extensive splitting of elements.	Some splitting at connections. Loosening of sheathing. Observable withdrawal of fasteners. Splitting of framing and sheathing.	No observable loosening or withdrawal of fasteners. No splitting of sheathing or framing.
Concrete Diaphragms	Extensive crushing and observable offset across many cracks.	Extensive cracking (<1/4" width). Local crushing and spalling.	Distributed hairline cracking. Some minor cracks of larger size (<1/8" width).
Precast Diaphragms	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking (<1/4" width). Local crushing and spalling.	Some minor cracking along joints.

Figure 47: Structural Performance Levels and Damage - Horizontal elements - F.E.M.A. 356 (2000).

4.3.3 Identification of the Non-structural Performance Levels (NPL)

The Non-structural Performance Levels are related to the architectural components such as partitions, exterior coverings, ceilings, mechanical and electrical components, H.V.A.C. (Heating Ventilation and Air Conditioning) systems, plumbing systems, fire suppression systems, and lighting; occupant contents and furnishings are taken into account in some performance levels.

The previous description is written in the F.E.M.A. 273 (1997); it is not specifically related to Hospital non-structural elements because, in addition to these elements, many other aspects must be considered: medical gases systems, patient care equipments and so on. Anyway, it is important to observe these definitions, remembering that Hospitals require more attention to the Non-structural equipments, in order to guarantee the operational function after a seismic event.

The five levels described in the F.E.M.A. 273 (1997) are listed below:

- N-A: Operational, definable as the post-earthquake damage state in which the non-structural components are able to support the pre-earthquake functions present in the building;
- N-B: Immediate Occupancy, post-earthquake damage state that includes damages to non-structural components, but building access and life safety systems (doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems) generally remain available and operable, if electrical power is available. Some systems can be out of service after the seismic event;
- N-C: Life Safety, definable as the post-earthquake damage state that includes damage to non-structural components, but the damage is non-life threatening. From this level, it is obvious that the operational functions are completely out of service;
- N-D: Hazards Reduced, definable as the post-earthquake damage state which includes damages to non-structural components that could potentially create falling hazards, but high hazard non-structural components are secured and will not fall into areas of public assembly;
- N-E, Non-structural Performance Not Considered, level necessary to cover the situation where only structural aspects are considered.

Here in the following, an extract of the tables concerning the non-structural performances of architectural components, mechanical - electrical - plumbing systems/components and contents is reported, as defined in F.E.M.A. 356 (2000). The tables are shown and commented below in order to better analyze which are the requirements for Hospital structures, in analogy to the indications for general buildings.

Table C1-5 Nonstructural Performance Levels and Damage —Architectural Components

Component	Nonstructural Performance Levels			
	Hazards Reduced N-D	Life Safety N-C	Immediate Occupancy N-B	Operational N-A
Cladding	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding, but panels do not fall in areas of public assembly.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding, but panels do not fall.	Connections yield; minor cracks (<1/16" width) or bending in cladding.	Connections yield; minor cracks (<1/16" width) or bending in cladding.
Glazing	General shattered glass and distorted frames in unoccupied areas. Extensive cracked glass; little broken glass in occupied areas.	Extensive cracked glass; little broken glass.	Some cracked panes; none broken.	Some cracked panes; none broken.
Partitions	Distributed damage; some severe cracking, crushing, and racking in some areas.	Distributed damage; some severe cracking, crushing, and racking in some areas.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.
Ceilings	Extensive damage. Dropped suspended ceiling tiles. Moderate cracking in hard ceilings.	Extensive damage. Dropped suspended ceiling tiles. Moderate cracking in hard ceilings.	Minor damage. Some suspended ceiling tiles disrupted. A few panels dropped. Minor cracking in hard ceilings.	Generally negligible damage. Isolated suspended panel dislocations, or cracks in hard ceilings.
Parapets & Ornamentation	Extensive damage; some falling in unoccupied areas.	Extensive damage; some falling in unoccupied areas.	Minor damage.	Minor damage.
Canopies & Marquees	Moderate damage.	Moderate damage.	Minor damage.	Minor damage.
Chimneys & Stacks	Extensive damage. No collapse.	Extensive damage. No collapse.	Minor cracking.	Negligible damage.
Stairs & Fire Escapes	Extensive racking. Loss of use.	Some racking and cracking of slabs. Usable.	Minor damage.	Negligible damage.
Doors	Distributed damage. Many racked and jammed doors.	Distributed damage. Some racked and jammed doors.	Minor damage. Doors operable.	Minor damage. Doors operable.

Figure 48: Non-structural Performance Levels and Damage - Architectural Components - F.E.M.A. 356 (2000).

Looking to the descriptions for each architectural component (Figure 48), it is possible to assess that, in order to guarantee the operational function for a Hospital, Operational Performance level should be considered. Immediate occupancy level is anyway a quite acceptable threshold with the exception for the ceilings, which can show some local falls (this could represent a dangerous aspect in some critical areas of Hospitals).

Table C1-7 Nonstructural Performance Levels and Damage —Contents

Contents	Nonstructural Performance Levels			
	Hazards Reduced N-D	Life Safety N-C	Immediate Occupancy N-B	Operational N-A
Computer Systems	Units roll and overturn, disconnect cables. Raised access floors collapse. Power not available.	Units shift and may disconnect cables, but do not overturn. Power not available.	Units secure and remain connected. Power may not be available to operate, and minor internal damage may occur.	Units undamaged and operable; power available.
Desktop Equipment	Some equipment slides off desks.	Some equipment slides off desks.	Some equipment slides off desks.	Equipment secured to desks and operable.
File Cabinets	Cabinets overturn and spill contents.	Cabinets overturn and spill contents.	Drawers slide open, but cabinets do not tip.	Drawers slide open, but cabinets do not tip
Book Shelves	Shelves overturn and spill contents.	Books slide off shelves.	Books slide on shelves.	Books remain on shelves.
Hazardous Materials	Minor damage; occasional materials spilled; gaseous materials contained.	Minor damage; occasional materials spilled; gaseous materials contained.	Negligible damage; materials contained.	Negligible damage; materials contained.
Art Objects	Objects damaged by falling, water, dust.	Objects damaged by falling, water, dust.	Some objects may be damaged by falling.	Objects undamaged.

Figure 49: Non-structural Performance Levels and Damage - Contents - F.E.M.A. 356 (2000).

Considering now the contents (Figure 49), it is possible to assume that medical equipments (patient care machines) have similar necessities like computer systems: with this assumption, it is possible to see that the Operational Performance level must be reached, in order to ensure the continuous functionality of them.

In the next image (Figure 50) instead, the mechanical, electrical and plumbing systems are analyzed: it is possible to see that the continuous functionality for a building is ensured only for the Operational Performance level, while in the Immediate Occupancy level some problems may occur due to the lack of electricity; there could be also some difficulties caused from the minor leakage of some pipes.

Table C1-6 Nonstructural Performance Levels and Damage —Mechanical, Electrical, and Plumbing Systems/Components

System/Component	Nonstructural Performance Levels			
	Hazards Reduced N-D	Life Safety N-C	Immediate Occupancy N-B	Operational N-A
Elevators	Elevators out of service; counterweights off rails.	Elevators out of service; counterweights do not dislodge.	Elevators operable; can be started when power available.	Elevators operate.
HVAC Equipment	Most units do not operate; many slide or overturn; some suspended units fall.	Units shift on supports, rupturing attached ducting, piping, and conduit, but do not fall.	Units are secure and most operate if power and other required utilities are available.	Units are secure and operate. Emergency power and other utilities provided, if required.
Manufacturing Equipment	Units slide and overturn; utilities disconnected. Heavy units require reconnection and realignment. Sensitive equipment may not be functional.	Units slide, but do not overturn; utilities not available; some realignment required to operate.	Units secure, and most operable if power and utilities available.	Units secure and operable; power and utilities available.
Piping	Some lines rupture. Some supports fail. Some piping falls.	Minor damage at joints, with some leakage. Some supports damaged, but systems remain suspended.	Minor leaks develop at a few joints.	Negligible damage.
Electrical Distribution Equipment	Units slide and/or overturn, rupturing attached conduit. Uninterruptible Power Source systems fail. Diesel generators do not start.	Units shift on supports and may not operate. Generators provided for emergency power start; utility service lost.	Units are secure and generally operable. Emergency generators start, but may not be adequate to service all power requirements.	Units are functional. Emergency power is provided, as needed.
Light Fixtures	Many broken light fixtures. Falling hazards generally avoided in heavier fixtures (>20 pounds) in areas of public assembly.	Many broken light fixtures. Falling hazards generally avoided in heavier fixtures (> 20 pounds).	Minor damage. Some pendant lights broken.	Negligible damage.
Plumbing	Some fixtures broken; lines broken; mains disrupted at source.	Some fixtures broken, lines broken; mains disrupted at source.	Fixtures and lines serviceable; however, utility service may not be available.	System is functional. On-site water supply provided, if required.

Figure 50: Non-structural Performance Levels and Damage - Systems/Components - F.E.M.A. 356 (2000).

Resuming the information obtained from the Non-structural Performance Level tables, it is possible to assess that the continuous operational function for a Hospital is ensured only in the Operational Performance Level.

4.3.4 The combination of SPL and NPL: the Target Building Performance Levels (TBPL)

The information about structural and non structural components can be used together to define the Target Building Performance Level of the structure, which consists in a combination of a Structural Performance level (selected from the six ones available, S1 to S6) and a Non-structural Performance Level (selected from the five ones available, N-A to N-E). These targets are identified by a number (structural level) and a letter (non-structural level).

Combining all the performance levels, the matrix of the possible TBPL is obtained (Figure 51).

Table C1-8 Target Building Performance Levels and Ranges

Nonstructural Performance Levels	Structural Performance Levels and Ranges					
	S-1 Immediate Occupancy	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention	S-6 Not Considered
N-A Operational	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
N-B Immediate Occupancy	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not recommended
N-C Life Safety	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
N-D Hazards Reduced	Not recommended	2-D	3-D	4-D	5-D	6-D
N-E Not Considered	Not recommended	Not recommended	Not recommended	4-E	Collapse Prevention 5-E	No rehabilitation

Figure 51: Target Building Performance Levels (TBPL) and Ranges - F.E.M.A. 356 (2000).

Some of the combinations are identified with "Not recommended" because they are related to particular situations which have no real interest from the engineering point of view: for example, a fully non structural operational performance level (N-A) associated to a collapse prevention level (S-5) has no practical sense.

Four combinations have been better identified with a specific name, because they represent the most likely ones to be selected as the basis for engineering design:

- Operational (1-A);
- Immediate Occupancy (1-B);
- Life Safety (3-C);
- Collapse Prevention (5-E).

These four qualitative levels can be superimposed on the global force-displacement relationship of a generic building. The corresponding levels of damage are sketched in Figure 52.

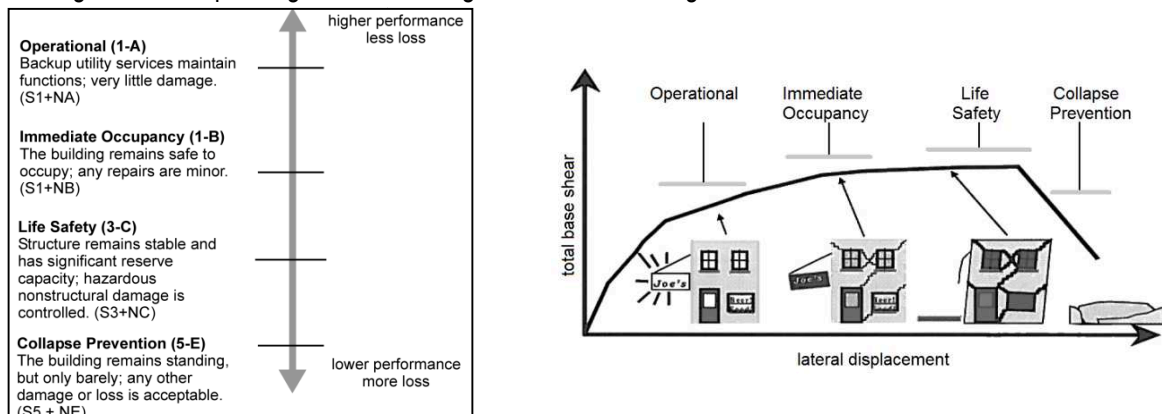


Figure 52: Description of the four TBPL and qualitative identification on a general "displacement - total base shear" curve.

In December 2003 a new document has been produced by F.E.M.A., with specific reference to Hospital buildings: "Incremental Seismic Rehabilitation of Hospital Buildings" - F.E.M.A. 396 (2003), which fully refers to the two documents described above for the definition of the TBPL (see section B of the document) and gives some useful definitions for the risk management for Hospitals.

The present work will refer to two of the four possible Target Building Performance Levels; in particular:

- Operational TBPL, in order to take into account the necessity of a Hospital to be effectively operative after a low/medium intensity earthquake;
- Life Safety TBPL, in order to guarantee the safety for persons inside the Hospital after a medium/strong seismic event.

Therefore, it is important to define parameters which can identify the threshold limits for the two TBPL listed above. This aspect will be performed in the next paragraphs, identifying the analytical translations of the theoretical descriptions written in this paragraph. A first correspondence among the TBPL identified above and the classical Limit State approach for the evaluation of the security of the structure can be found: indeed, the Operational TBPL can correspond to the SLO (Operational limit state) and the Life Safety TBPL to the SLV (Life safety limit state), identified in the actual Italian Code (D.M. 2008 and C.M. 617/2009).

In any case, these are only two particular situations which describe the damage state of a general Hospital during an earthquake. *How does the damage evolve with the increasing of the seismic intensity? Which is the residual operative part of a Hospital for a given earthquake intensity?* A qualitative answer can be offered with the following concepts about resilience.

4.3.5 Resilience

When a certain earthquake occurs with a given intensity, which is the percentage of loss of functionality in a structure? How fast will this building be fully operative again? The answers to these questions are contained inside the definition of the concept of resilience: "*Disaster resilience is the ability of social units (e.g., organizations, communities) to mitigate hazards, contain the effects of disasters, and carry out recovery activities in ways that minimize social disruption, while also mitigating the effects of future disasters. Consequently, strength, flexibility, and the ability to cope with and overcome extreme challenges, are the hallmarks of disaster-resilient communities.*"

This definition has been conceived by the "Multidisciplinary Centre for Earthquake Engineering Research"-MCEER, a national research centre headquartered at the University of Buffalo (The State University of New York) which cooperates with F.E.M.A..

As written in the general description of this research centre, "the overall goal of MCEER is to enhance the seismic resilience of communities through improved engineering and management tools for critical infrastructure systems (water supply, electric power, and hospitals) and emergency management functions."

A general resilient system is characterized by three different aspects:

- reduced failure probabilities - the reduced likelihood of damage and failures for a system;
- reduced consequences from failures - injuries, casualties, damages, economic and social impacts;
- reduced time to recovery - the time required to restore a system to pre-disaster levels of functionality.

Resilience can be improved by reducing both the possibility of failures and the necessary times to recover the functionality when a certain level of damage is reached after a disastrous event.

This concept is multidisciplinary, because it involves a technical aspect (the performance of structural/non-structural elements subjected to a disaster), an organizational one (the management of the functions and the available resources inside the considered system, in this case an hospital), a social one (the emergency measures designed to reduce the disaster impact on the damaged communities) and an economic one (the reduction of direct and indirect losses resulting from the disaster).

Looking to the following image (Figure 53), it is possible to observe a scheme which represents the graphical definition of resilience: in the time t_0 a certain seismic event occurs (with a given intensity) and the analyzed structure, fully operative before that event, suffers a loss of quality (or loss of functionality), reaching the $x\%$ level of "quality"; from this point, the process performed in order to restore the fully operative state of the building starts; it needs a certain amount of time due to the level of the loss, the availability of resources, etc... The trend of this process has been sketched as linear, but it can be any type of function which tries to make the quality of the infrastructure come back to the 100% level.

At the time t_1 the structure is restored to the pre-event situation.

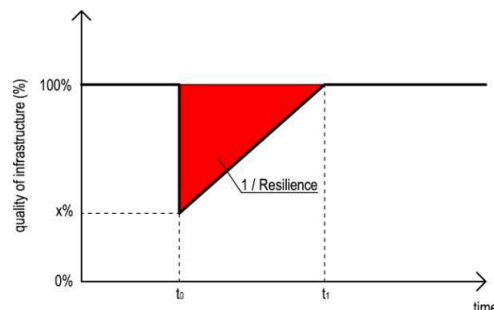


Figure 53: Graphical definition of resilience.

The triangle area highlighted in the graph with the red colour allows a quantification of the resilience: the bigger is this area, the less resilient is the structure. The triangle area depends on the initial loss of quality

(related to the performances of the components of the buildings) and on the necessary time to restore the previous situation.

The goal of this thesis is not to investigate the features of the Hospital systems related to resilience, because this should involve specific studies on the organization of post-earthquake emergency for Hospitals, analysis of the management functions and so on; however, from the engineering point of view, it is possible to assess that the resilient aspect can be only partially (and qualitatively) investigated through the evaluation of the initial loss of quality, as better described in the following paragraphs (see Figure 66).

4.4 Typology of modeling: FME approach

In this paragraph, a brief description of the approach of the software used in this work is proposed: the detailed analyses will be performed with 3MURI, developed and distributed by S.T.A. DATA.

The observation of the damages of masonry structures after seismic events has allowed the classification of the failure in-plane mechanisms of the walls, which occur when the building is able to assume a box behaviour. These failure mechanisms, as seen in § 4.2.1, can be divided in three categories (as illustrated in Figure 41):

- shear failure due to diagonal cracking;
- shear failure due to bending and consequent edge crushing;
- shear failure due to sliding of a portion of wall.

The first two are characteristic of existing masonry buildings, for which a perfect sliding surface is not easily detectable (for example, for not organized stone masonry), while the third is characteristic of more recent typologies of masonry (such as brick masonry). These typologies of damage always occur in well defined portion of the masonry: in particular, these parts are delimited by the openings.



Figure 54: On the left, diagonal cracking; on the right, bending cracking. (Theoretical manual of 3MURI)

Starting from this, the idea of the Frame by Macro Element approach has been developed; in particular, each wall can be divided into elementary components:

- piers, which are the parts of masonry disposed on the sides of the openings;
- spandrel beams, which are located above and below the openings;
- rigid nodes, which are the remaining parts of the walls, not bordering with any openings and, for this reason, assumed as confined from the other two typologies of element.

Theoretical and experimental research has confirmed that piers and spandrel beams' behaviour can be well represented by frame elements, even if they involve surfaces of masonry (Galasco A., Frumento S., 2011).

With these hypotheses, a generic wall can be schematized with a system of the elements mentioned above, creating an equivalent frame structure (Figure 55).

It is important to remind that this typology of approach is efficacious only when a certain regularity of the distribution of the openings can be observed: in this case, the definition of the piers in the structural model has

a direct physical correspondence to the real structure, while for a chaotic distributions of doors and windows the individuation of the piers, spandrel beams and rigid zones starts to be less effective and other typologies of modelling should be used. For the case studies of this work, a general regularity of the distribution of the openings has been observed, leading to the possibility of utilization of this approach.

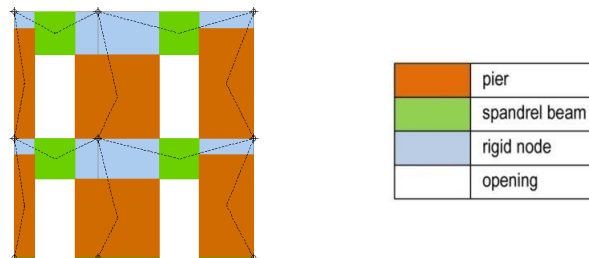


Figure 55: Equivalent frame structure for a two levels wall with four openings (doors).

Each macro-element is numerically treated by means of a finite element with non-linear behaviour (Figure 56), which is composed of three parts:

- axial deformability is concentrated in the two parts placed at the ends of the element, which have infinitesimal thickness Δ , not sensible to shear forces;
- tangential deformability is instead concentrated in the central part of the element with height h , which is not sensible to axial and flexional actions.

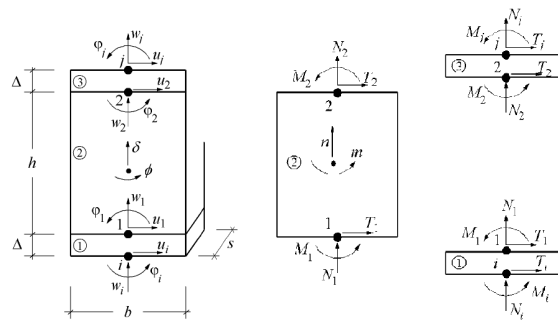


Figure 56: Description of the macro-element. (Theoretical manual of 3MURI)

The elastic - perfectly plastic behaviour is associated to the macro-element: in particular, the initial elastic stiffness is calculated through the flexional and tangential contributions, the bilinear behaviour is identified in agreement with the indications of the actual Italian Code (D.M. 2008 and C.M. 617/2009), defining the maximum shear force that the element is able to carry (evaluated as the minimum value among the ones for diagonal cracking and bending) and the related maximum deformation (drift), associated to the failure mechanism: 6‰h for a bending mechanism and 4‰h for a diagonal cracking mechanism, as written in the Italian Code. If these values are exceeded, the non-linear element is replaced with a truss, able only to transmit normal forces, without any resistance to the horizontal actions.

During the plastic phase, the stiffness of the element degrades until the maximum drift (Figure 57).

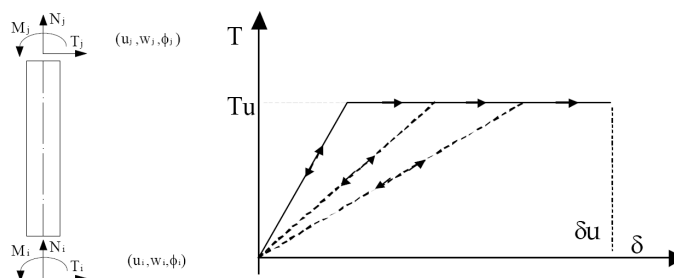


Figure 57: Elastic - perfectly plastic behaviour of a general macro-element. (Theoretical manual of 3MURI)

The resistance and the stiffness in the out of plane direction are not considered in this type of model.

The three dimensional model of the structure is realized assembling each wall with the scheme described above. For each level, the definition of the typologies of floor is required. The floors are the elements which must transmit the horizontal forces to the walls. The in-plane stiffness of the floors strongly influences the global behaviour of the structures: a rigid floor (such as a reinforced concrete with hollow tiles for example) is able to transmit the horizontal forces in a homogeneous way among all the connected walls, while a flexible one (such a wooden floor) spreads the action in a different way among the vertical elements, overloading some of them, without allowing a complete cooperation of all the other nearby walls (Figure 58).

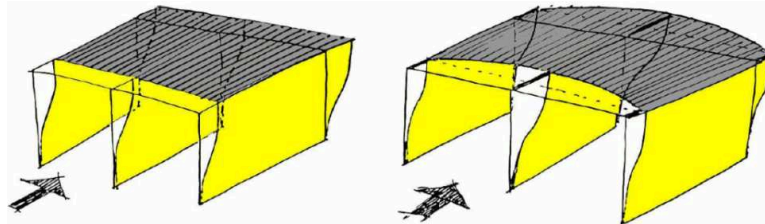


Figure 58: Difference of behaviour among rigid floors (left) and flexible ones (right). (Theoretical manual of 3MURI)

The floors are modelled with superficial finite elements having membrane behaviour, which are only able to transmit forces on their level: these elements have stiffness that is a function of the real constructive features. It is important to highlight that a generic floor can have different values of stiffness in the two horizontal directions, due to the direction of the principal beams of the floor itself.

The three dimensional model is then completely described once the loads for each floor are defined, distinguishing the permanent structural loads, the non structural ones and the live loads.

4.5 Pushover analysis: construction of the capacity curve and treatment of the data

4.5.1 Typologies of analysis to perform

Once the numerical model of the structure is prepared, the types of analysis must be chosen.

The lateral load profile, which has to be applied to the structure, must represent the distribution of the inertial forces produced by the seismic oscillation. In the beginning, the structure will behave as an elastic system, with a distribution of horizontal forces which follows the 1st modal shape; then, as long as the damage increases, the dynamic behaviour of the structure changes, until the limit condition where the structure acts mainly considering the distribution of the masses. In order to reproduce this natural phenomenon, two different typologies of lateral load must be considered (Figure 59):

- a lateral load distribution proportional to the height and the mass of each level; this profile is proportional to the 1st modal shape in the considered direction of analysis;
- a uniform lateral load distribution, proportional to the masses of each storey.

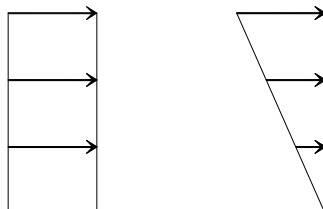


Figure 59: Example of lateral load profiles for a 3 storey building: uniform distribution (left) and 1st modal shape distribution (right).

It is also necessary to consider the possibility of a not-regular distribution of the masses inside the structure, element which can influence the response of the structure itself, since the lateral profile is normally applied in the barycenter of the masses; this aspect is considered by the assumption of an eccentricity of the lateral load

profile (from the barycenter of the masses) at each level equal to 5% of the maximum dimension (orthogonal to the considered direction of analysis) of the building, as requested from the Italian Code. In particular, both the cases +5% and -5% have been analyzed, using the one which leads to a lower capacity for the structure in the following part of the work. This aspect can highlight weaknesses for torsional effects even on buildings which show regular plans.

The necessary analyses for each structure are listed above, considering all previous aspects.

N.	DIRECTION	SEISMIC LOAD	ECCENTRICITY
1	X	1° modal shape	0%
2	X	masses	0%
3	X	1° modal shape	5%
4	X	masses	5%
5	Y	1° modal shape	0%
6	Y	masses	0%
7	Y	1° modal shape	5%
8	Y	masses	5%

Table 4: Typologies of analysis considered for each structure.

4.5.2 The creation of the pushover curve

The pushover curve, as mentioned above, is the representation of the total base shear of the structure related to the displacement of the control point; it is then necessary to choose a control point which can be considered representative for the analysis. This aspect influences the plotting of the pushover curve because, for each point, there is a different displacement response during the analysis.

Usually, the control point has to be chosen among the nodes of the roof, possibly close to the barycenter of it; in any case, it must belong to the last "significant" level of the structure: in case of the roof of a tower which covers only a small part of the whole building, the node must be chosen on the lower level, analyzing in any case the local behaviour of the tower in order to understand its level of capacity.

The software, after an initial gravitational analysis on each element of the equivalent frame structure, performs a modal analysis in order to obtain the participating modal factors of the mass (both dead and live mass).

Then, by the increase of the lateral load profile, the total base shear and displacement of the control point are plotted; in the initial part, most of the elements react with their elastic behaviour, then some of them begin to enter in their plastic range, showing a decrease of stiffness. With the increasing of the multiplier of the lateral load, some of the elements can reach their ultimate acceptable displacement: in that moment, they are not considered in the analysis anymore (they are only able to carry the gravitational loads from this point), leading to a faster decrease of stiffness. When the curve shows a global decrease (with reference to the maximum registered value of base shear) greater than 20%, the analysis stops. This value of decrease has been conventionally adopted as the reduction threshold of the shear resistance; the assumption is in agreement with the indications of the Italian Code (D.M. 2008) and of the Eurocode 8 - EN 1998-3 (2005).

In the following, the procedure of treatment of the curves adopted in the present work is described, referring to the indications of the Italian Code (D.M. 2008 and C.M. 617/2009).

4.5.3 From MDOF to SDOF: the bi-linearization of the curve

Once that the curve for the MDOF (multi degree of freedom) system is plotted, it is necessary to convert it in a SDOF (single degree of freedom) system, using the modal participation factor $\Gamma = \varphi^t M \tau / \varphi^t M \varphi$, which measures the quantity of activated masses with the considered eigenvector (φ is the eigenvector, M the matrix of masses of the structure, τ the drag vector which indicates the masses involved in the analyzed direction); the conversion from MDOF to SDOF curve is performed dividing both X and Y values of the curve with the modal participation factor.

The second step is the calculation of the equivalent bi-linear curve of the SDOF curve, identified by the stiffness of the initial linear phase, the maximum resistance shear (where the elastic phase ends) and the maximum displacement (at the end of the plastic phase). The stiffness of the bi-linear system is calculated imposing the passage of the elastic part at the 70% of the maximum base shear:

$$k^* = F_{70\%}^* / d_{70\%}^*$$

The maximum value of the elastic part is calculated with an energetic balance, assuming that the area below the SDOF curve and the bi-linear one are the same. By a 2nd order equation, it is possible to find:

$$F_y^* = \left(d_u^* - \sqrt{d_u^{*2} - \frac{2Area^*}{k^*}} \right) k^*$$

where *Area* means the measure of the surface below the SDOF curve.

The period of the SDOF system is calculated using the participating mass and the stiffness of the system itself. It is also possible to calculate the displacement of the SDOF system corresponding to the yielding point.

$$T^* = 2\pi\sqrt{m^*/k^*} \quad d_y^* = F_y^*/k^*$$

In this way, the MDOF system is described with an equivalent SDOF one, using a bi-linear description useful for the evaluation of the capacity (and, in case of verification, of the demand) of the structure.

4.5.4 The parameters for the definition of vulnerability: analytical treatment of the curve

When the verification of a masonry structure is required through a pushover analysis, it is necessary to calculate the "displacement demand" using the period of the SDOF system and the displacement spectrum of the considered site of construction of the building: the input values for the construction of the spectrum are related to a certain period of return of the seismic action, depending on the type of limit state to analyze (operational, damage, life safety, collapse prevention) and to the class of the building (rural, ordinary, relevant or strategic buildings).

Using the period of the SDOF system in the displacement response spectrum, it is possible to obtain the displacement demand; multiplying this value with the participation factor, it is possible to obtain the real displacement demand for the analyzed limit state of the MDOF. In this particular point, called performance point (PP), structural verifications must be carried out. In this work, it is not important to assess which are the buildings that fulfil the structural requirements of the current Code, because the goal of this part of research is to obtain a classification of the most vulnerable structures using a detailed analysis approach. For this reason, it is important to find out some parameters which allow the comprehension of the behaviour of these structures and their level of capacity.

The pushover analysis is a procedure which highlights the non-linear behaviour of a structure: starting from a very low lateral forces profile, the multiplier increases the entity of these forces without changing the profile: in this way, the elastic behaviour is activated at first, and then the non-linear one comes further. The characteristics of the buildings (stiffness, ductility and resistance) influence the plotting of the capacity curve.

Which parameter is possible to assume for the evaluation of vulnerability and for the comparison among the different curves of the considered buildings? As described in § 4.3, it is necessary to take into account two different situations, identified by two different Target Building Performance Levels:

- Operational TBPL;
- Life Safety TBPL.

Moreover, it can be useful to try to observe the resilience of the considered system through pushover curves. Looking to a generic capacity curve (and its related bi-linear schematization), it is possible to analyze some aspects which help in the decision of the parameters to consider: in the following text some remarks about the curve and its physical meaning are reported.

The ultimate displacement of the curve (in correspondence of 20% decay of the resistance shear) represents the threshold after which the structure must be considered not safe for the occupants, as commonly assumed during the structural verifications of masonry building: this point can be considered as the Life Safety TBPL indicator. But how fast does a structure, starting from an elastic behaviour, reach the ultimate displacement?

The answer is connected to the concept of ductility: in seismic design, ductility means the ability of a structure to undergo large amplitude deformations in the inelastic range, without suffering a substantial reduction in strength. Ductile structures can dissipate relevant quantities of energy during the deformation by attrition and hysteresis phenomena. The displacement ductility can be measured through the ductility factor, μ , which is the ratio among the maximum displacement, d_u , and the yielding displacement, d_y .

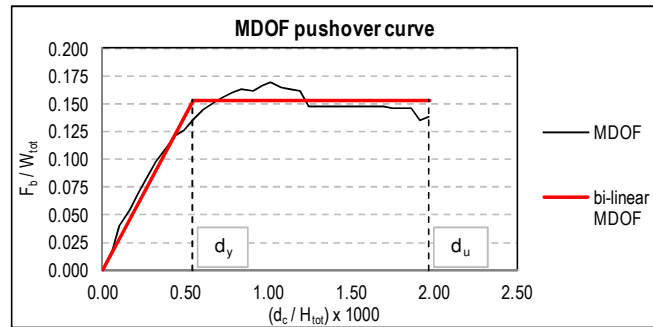


Figure 60: Example of pushover curve with the bi-linear schematization.

During the ductility range (from d_y until d_u), the structure dissipates energy by means of its plastic deformation; this aspect ensures a higher level of safety comparing to a building which shows the same shear resistance with a smaller level of ductility, allowing for example the evacuation of people inside the structure without instantaneous fragile collapse mechanisms.

Considering these aspects, it is possible to affirm that the calculation of the yielding displacement of the bi-linear curve can be considered an important parameter which describes the behaviour of the structure: the stiffness of the system changes significantly because many structural elements enter in their plastic range.

Since the pushover curves are obtained analyzing structural models, the yielding displacement of the bi-linear curve gives information only on the conventional beginning of the structural damage, while the non-structural one is not taken into account, because non-structural elements are not represented inside the model. Anyway, the non-structural damage can happen even before the structural damage, since many fragile and sensible elements are classified as non-structural (such as glazing or computers for example). The two types of damage are connected each other: when some cracking starts to appear on the structural elements, it is reasonable to think that even some partition, cladding or glazing experience some damage too.

In order to compare the behaviour of different structures in relation to the starting phase of the damage, both from the structural and non-structural point of view, the yielding point of the bi-linear curve can be assumed as the Operational TBPL: the equivalent SDOF starts to be in the plastic range, after reaching its maximum elastic deformation. In this way, once the bi-linear curve of the SDOF system is calculated and once the correspondent bi-linear curve of the MDOF system is obtained, the behaviour of the structure is summarily described.

The comparison among different structures is not available yet, because the real displacements are strictly connected to the geometry of the structural elements and to the masses of each structure: it is not possible to compare the absolute results of a three floors masonry building (10 meters high for example) with the ones of a single floor building (3 meters high for example), because the ultimate displacement of the first one will probably be greater than the second one, but this does not mean that it is safer or more ductile. A possibility is offered by the calculation of the accelerations which produce the considered displacement: in this way, the

reference scale becomes unique and comparisons are allowed. Indeed, as described below, the calculation of the acceleration uses the characteristics of the buildings, in terms of mass and stiffness, leading to a common reference system. Resuming the data analyzed before in the bi-linearization procedure, the available elements for each pushover analysis are:

- the MDOF capacity curve;
- the modal participation factor $\Gamma = \varphi^t M \tau / \varphi^t M \varphi$;
- the SDOF capacity curve (simply scaling the MDOF one with the modal participation factor), that ends with the ultimate displacement d_u^* ;
- the mass of the equivalent SDOF system $m^* = \varphi^t M \varphi$;
- the bi-linear curve of the SDOF system, described from k^* , F_y^* and d_y^* (stiffness at 70% of maximum shear, maximum value of the elastic part and displacement of the SDOF system corresponding to the yielding point);
- the period of the SDOF bi-linear system $T^* = 2\pi\sqrt{m^*/k^*}$;
- the available ductility of the SDOF $\mu_D = d_u^*/d_y^*$.

In order to calculate the acceleration which produces a certain displacement of the capacity curve, it is necessary to refer the analytical procedure to an elastic system that is equivalent to the bi-linear one: in this way, it is possible to use the elastic spectrum in terms of displacement and acceleration.

In order to find out the equivalence among an elastic system and a bi-linear one, the observation of the stiffness is necessary: flexible structures usually show deformations equal to elastic systems with the same stiffness, while rigid structures can experience more deformations than the equivalent elastic systems.

This aspect is considered in the analytical treatment of the pushover curve: the period of the structure is compared with a reference period: in the actual Italian Code (D.M. 2008), the comparison is made with the period T_C , corresponding to the end of the plateau in the acceleration spectrum. In particular:

- if $T^* > T_C$, the structure is considered as "flexible";
- if $T^* \leq T_C$, the structure is considered as "rigid".

In Figure 61, the qualitative difference between flexible and rigid systems is shown. On the horizontal axis the displacement of the control point is plotted, while on the vertical one, the total base shear is registered.

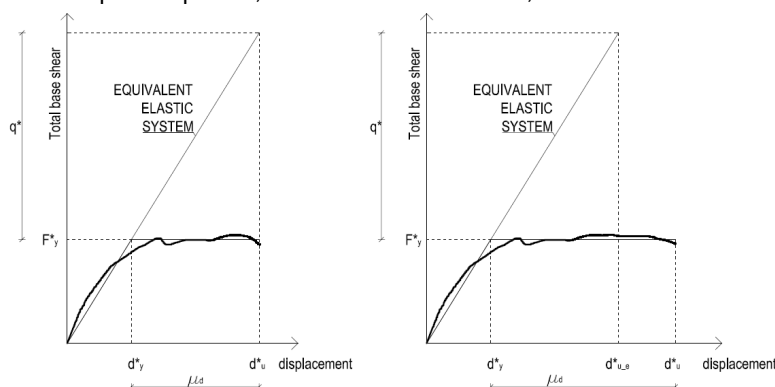


Figure 61: Equivalent elastic system for flexible systems on the left ($T^* > T_C$) and rigid systems on the right ($T^* \leq T_C$).

For flexible systems, it is possible to consider an equivalent elastic system which shows the same maximum displacement of the real system, while for rigid systems, it is necessary to calculate the maximum displacement of the equivalent elastic system through an energy balance between the two systems.

Before explaining the analytical procedure to obtain accelerations of capacity from displacements, another element must be considered: the level of dissipation. Non-linear analyses allow its quantification in order to control the physical meaning of the numerical analysis, as described here in the following.

Figure 61 shows the meaning of ductility (ratio between ultimate and yielding displacement) and of the behaviour factor (ratio between maximum elastic force and yielding force).

In the case of flexible structures (see the previous page for the definition of flexible and rigid structures), ductility and behaviour factor have the same value, while for rigid structures the ultimate displacement of the elasto-plastic system is greater than the elastic one. The relations among the quantities are:

- if $T^* > T_C$, $\mu_d = q^*$
- if $T^* \leq T_C$, $\mu_d = 1 + (q^* - 1) T_C/T^*$

An excessive level of dissipation may point out a not fully reliable analysis and this can cause an overestimation of the capacity of the structure; in order to avoid this, the actual Italian Code (D.M. 2008) gives a threshold limit for this parameter: $q^* < 3$. If the calculated value exceeds the limit, a limitation to the capacity acceleration must be taken into account (as better described in the following).

The SDOF behaviour factor is the ratio among the maximum elastic response force and the yielding one:

$$q^* = S_e(T^*) \cdot m^*/F_y^*$$

but, since the acceleration is not calculated yet, it is possible to obtain the q^* factor from the definition of the ductility, paying attention to the value of the period of the SDOF system:

- if $T^* > T_C$, $q^* = \mu_d$
- if $T^* \leq T_C$, $q^* = 1 + (\mu_d - 1) T^*/T_C$

It is important to remind that all these evaluations are referred to the capacity of the structure.

Resuming the last concepts, four different alternatives are possible for the evaluation of the acceleration:

- flexible structures $T^* > T_C$ and $q^* < 3$ $T^* > T_C$ and $q^* > 3$
- rigid structures $T^* \leq T_C$ and $q^* < 3$ $T^* \leq T_C$ and $q^* > 3$

In the following text, the procedures for the calculation of the acceleration related to the ultimate displacement are explained for the four cases just mentioned.

- **$T^* > T_C$ and $q^* < 3$**
 $d_u^* = d_{u_e}^*$ elastic and real maximum displacement of SDOF are the same value
 $d_{u_e}^* = S_{D_e}(T^*)$ elastic maximum displacement = spectral displacement
 $S_e(T^*) = S_{D_e}(T^*) \cdot \omega^2 = S_{D_e}(T^*) \cdot (2\pi/T^*)^2$
 The last equation allows the calculation of the spectral acceleration from the spectral displacement; this is possible because of the relation among the two spectral quantities through the value of the frequency of the SDOF system " ω ". The calculated acceleration is expressed in [m/s^2]: the non-dimensional value is evaluable dividing this acceleration for the gravity acceleration " g ".
- **$T^* > T_C$ and $q^* > 3$**
 The acceleration cannot be obtained directly from the ultimate displacement of the capacity curve because it is related to an excessive level of dissipation: the displacement must then be reduced because the deformations of the structure cannot be considered as reliable after the limit $q^* = 3$.
 $q^* = S_e(T^*) \cdot m^*/F_y^* \leq 3 \rightarrow S_e(T^*) = 3 \cdot F_y^*/m^*$
- **$T^* \leq T_C$ and $q^* < 3$**
 $d_u^* = (d_{u_e}^*/q^*) \cdot \mu = (d_{u_e}^*/q^*) \cdot [1 + (q^* - 1) T_C/T^*] > d_{u_e}^*$
 It is possible to calculate the maximum elastic displacement from the last equation:
 $d_{u_e}^* = (d_u^* \cdot q^*)/[1 + (q^* - 1) T_C/T^*] < d_u^*$
 If $d_{u_e}^* > d_u^*$, it is necessary to take the value of d_u^* .
 $d_{u_e}^* = S_{D_e}(T^*)$
 $S_e(T^*) = S_{D_e}(T^*) \cdot \omega^2 = S_{D_e}(T^*) \cdot (2\pi/T^*)^2$
- **$T^* \leq T_C$ and $q^* > 3$**
 The procedure, in this case, is the same as the one described for the $T^* > T_C$ and $q^* > 3$ case.

The four cases described above give as result the spectral accelerations related to the displacement capacity of the structures; in the aim of making comparisons between different structures, these accelerations are not directly comparable yet, because they are associated to different dynamic systems (in terms of mass and stiffness). It is possible to avoid this problem calculating the Peak Ground Acceleration of capacity (PGA_c) associated to the spectral acceleration of capacity of the considered structure: in this way, the dependency from the dynamic properties of the considered system is cancelled.

In order to make the comparison among different structures possible, two basic methods can be used:

- calculation of the PGA_c with fixed spectrum shape;
- calculation of the PGA_c and the return period of the event varying the spectrum shape.

The methods are analogous: the goal is to define a spectrum which, for the period of the equivalent SDOF system, gives the same spectral acceleration calculated in the capacity analysis. The difference among the two methods consists in the definition of the spectrum which identifies the capacity of the structure.

An acceleration spectrum is defined when the seismic hazard is identified through three parameters:

- a_g maximum expected horizontal acceleration on A type soil (rock);
- F_0 maximum value of the amplification factor of the horizontal acceleration spectrum;
- T_C^* upper limit of the period of the constant spectral acceleration branch.

These three parameters are defined both in the actual Italian Code (D.M. 2008) and Eurocode 8, even if they assume different values in the two Codes: the present work refers to the indications of the D.M. 2008.

In the "calculation of the peak ground acceleration (PGA) with fixed spectrum shape" method, the spectrum is assumed as defined for two parameters, F_0 and T_C^* , while the $PGA = a_g \cdot S$ is considered as a variable (with S coefficient that keeps into account the soil category and the topographical conditions, considered as constant for the sample of analysis as already mentioned before in § 4.2.2): once that the capacity displacement is obtained from the pushover curve and once that the related spectral acceleration is calculated with the method described above, it is possible to find out the spectrum that, for the period of the equivalent SDOF of the structure, gives the same acceleration. This is possible by varying the value of the PGA inside the equations of the acceleration spectrum; in the following, horizontal spectrum equations are reported and a qualitative description of this method is given in the next image (Figure 62).

$$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_0} \left(1 - \frac{T}{T_B} \right) \right] \quad 0 \leq T < T_B$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \quad T_B \leq T < T_C$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C}{T} \right) \quad T_C \leq T < T_D$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C \cdot T_D}{T^2} \right) \quad T_D \leq T$$

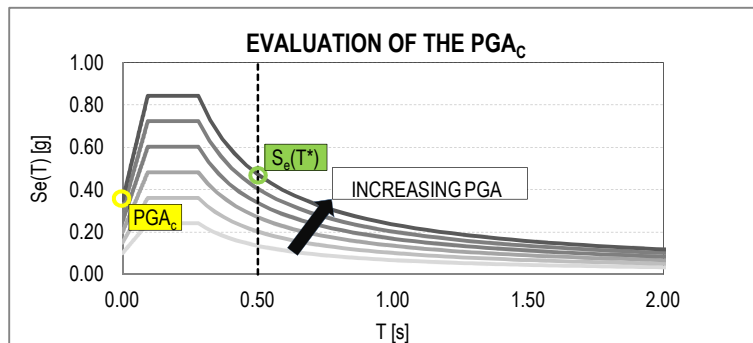


Figure 62: Research of the PGA of capacity once that period and spectral acceleration of capacity are known.

In the previous formulas, the parameter η represents the coefficient that modifies the elastic spectrum in case of conventional viscous damping ξ assumed as different from the usual value of 5%:

$$\eta = \sqrt{10/(5 + \xi)}$$

The analyses carried out in this work have been performed considering the conventional value $\xi = 5\%$.

The PGA_C becomes a univocal parameter which describes the maximum acceleration that the structure can suffer, avoiding the problems related to the different characteristics of the dynamic systems: the definition of the maximum acceleration now is independent from the period of the considered structure.

Since only the PGA changes in this process, a common reference starting spectrum has been chosen for all the structures; considering that most of the analyzed buildings are located in Florence, the following parameters have been assumed:

$V_N = 50 \text{ years}$	(nominal life - ordinary constructions)
$c_U = 2$	(IV class, buildings with public function)
$V_R = V_N \cdot c_U = 100 \text{ years}$	(reference life)
$T_R = -V_R / \ln(1 - P_{Vr}) = 949 \text{ years}$	(return period)
$P_{Vr} = 10\%$	(probability of exceedance in the reference life)
$lat = 43^\circ.7800 - long = 11^\circ.2500$	(city centre of Florence)

The seismic hazard is then defined:

$$a_g = 0.166 \quad F_0 = 2.388 \quad T_C^* = 0.310$$

The geotechnical conditions are assumed as constant, in order not to add another variable, for the moment, in the present work: the analyses will focus on the structural aspects of the masonry buildings, considering all of them placed in the same type of soil; in particular, the A category of soil is assumed, with a topographical condition T_1 . With this assumption, $S = 1$ and, consequently, $PGA = a_g \cdot S = a_g$.

Starting from this acceleration spectrum, the procedure shown in Figure 62 can be applied.

The method described above is direct and it allows the calculation of the capacity PGA_C just reversing the equations of the acceleration spectrum. With this method, it is not possible to obtain the estimation of the return period of capacity of the structure, since only the a_g is assumed as variable in the capacity estimation procedure, while F_0 and T_C^* are fixed (values obtained for $T_R = 949 \text{ years}$).

The "calculation of the PGA and the return period varying the spectrum shape" method is conceptually similar to the previous one: in this procedure, all the three parameters are considered as variable or, just to give the exact definition, the return period is considered as variable.

Even in this method, the acceleration spectrum which contains the value of the capacity spectral acceleration in correspondence to the period of the SDOF structure, has to be obtained. This method is iterative because the shape of the spectrum changes and it is not possible to calculate directly the value of the return period of the seismic action: anyway, with few iterations it is possible to get the correct values that solve the problem. With this method it is possible to calculate both the PGA_C and the return period of capacity T_{R_C} : the second parameter represents another way to estimate the capacity of a structure and it is currently used nowadays for the vulnerability assessment of existing buildings.

In order to better visualize the main differences among the two methods, it is important to observe the following images, where the variability of the three parameters (which identify the seismic hazard) are plotted, in relation to the variation of the return period.

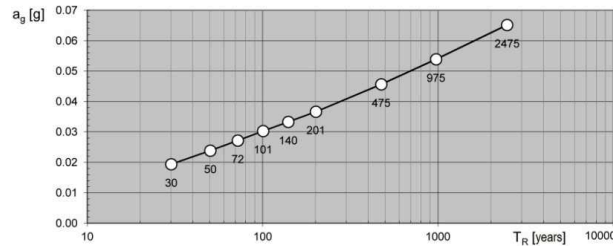


Figure 63: Variability of the parameter a_g [g] with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).

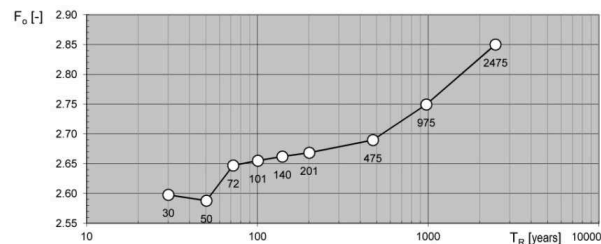


Figure 64: Variability of the parameter F_0 with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).

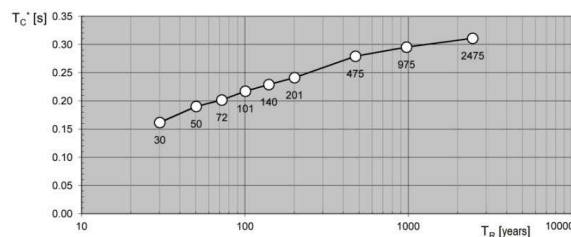


Figure 65: Variability of the parameter T_c^* with the return period (Spettri di risposta v 1.0.3 - C.S.L.P.).

In the first method (calculation of PGA with fixed spectrum shape), the F_0 and T_c^* parameters are assumed as constant, referring to the initial choice of the reference spectrum (in this case, the values of the parameters are the one in correspondence to $T_R = 949 \text{ years}$), while the a_g parameter (and consequently the PGA , since the geotechnical conditions are considered as constant in this work as written before) can vary in order to find the spectrum which passes for the capacity spectral acceleration in the period of the structure T^* .

In the second method instead (calculation of the PGA and of the return period varying the spectrum shape), all the three parameters change, modifying the spectrum shape.

The first method is computationally simpler than the second one, but there are no conceptual or physical differences among them: the fixed shape spectrum method allows the estimation of the acceleration of capacity, while the varying shape spectrum method allows the calculation of the acceleration and return period of capacity. Both of them start the evaluations of the indicators mentioned above from the displacement capacity, obtained with the pushover analysis on the global model of the considered structure.

This choice, as written before, has been made in order to take into account the Life Safety TBPL, which has been assumed as the ultimate condition for the building; the other considered parameter is the yielding displacement of the SDOF system, in order to take into account the Operational TBPL too. For this case, the method described for the Life Safety TBPL is still valid, with some simplifications due to the concept of yielding point: in particular, once that the yielding displacement is calculated, the ductility factor is equal to 1 in that point and, at the same time, the behaviour factor is equal to 1 too as consequence.

In this way, the four cases defined above (rigid or flexible structures, $q^* > 3$ or $q^* < 3$) converge in an unique case: the yielding displacement registered on the elasto-plastic system belongs to the elastic system too and it can be assumed as a spectral value; the spectral acceleration is then calculated with the frequency of the considered dynamic system, as seen for the first of the four cases above:

$$d_u^* = d_{u,e}^* \quad \text{elastic and real maximum displacement of SDOF are the same value}$$

$$d_{u,e}^* = S_{D_e}(T^*) \quad \text{elastic maximum displacement = spectral displacement}$$

$$S_e(T^*) = S_{D_e}(T^*) \cdot \omega^2 = S_{D_e}(T^*) \cdot (2\pi/T^*)^2$$

As described above, even this acceleration is not yet comparable with other structures because of the influence of the system dynamic properties and the calculation of *PGA* is required once again. This is a capacity *PGA* too, but it is referred to another limit state condition (Operational TBPL).

Once that the two Target Building Performance Levels have been taken into account, it is important to see how the resilience can be considered: thinking about the physical meaning of the two accelerations now calculated, it is possible to observe that (see the upper part of Figure 66):

- the one related to the Operational TBPL is associated to the end of the elastic behaviour of a structure and so to the beginning of the plastic range, with non-reversible deformations and damages to the structural and non-structural elements;
- the one related to the Life Safety TBPL is associated to the conventional collapse of the structure, point after which the building must be considered as "not safe". The expected damage in this point can be assumed as the maximum acceptable one; in other terms, 100% of the acceptable level of damage is registered in this point.

Taking into account these two definitions, it is possible to make a comparison to the Guagenti and Petrini (1989) studies and, in particular, to the tri-linear acceleration/damage law: the acceleration related to the start of the damage can be considered as equal to the Operational TBPL one, while the acceleration related to the highest level of damage can be assumed as equal to the Life Safety TBPL one.

The trend of the damage is assumed to be linear in the work of Guagenti and Petrini (1989): this assumption appears not so realistic, because it is necessary to consider the characteristics of the considered building and its structural modifications during the evolution of the damage, which surely follow a not linear law. Since the exact evolution of the damage between these two values of acceleration is not one of the targets of these analyses, this approximation has been considered as acceptable for the present work.

Talking now about the resilience aspect, it is possible to qualitatively describe the behaviour of a structure with the acceleration values defined above; in the following, referring to the scheme proposed in Figure 66, a step-by step description of this aspect is proposed, considering the evolution of the damage caused by the increasing of the intensity of a seismic event, measured through the acceleration:

- the yielding value of acceleration matches with a conventionally 0% level of structural damage; the resilient behaviour is only initially activated, in order to restore the occurred non-structural damages (the operational level of the structure remains full and only light repairing operations are required);
- going on with the acceleration in the tri-linear branch, the level of damage increases and, as a complementary consequence, the operational level decreases: the resilient behaviour is gradually activated and it can be observed that the fully operational feature is quickly restored for low damage cases (small red triangles in Figure 66); the ability of the "repairing function" is faster for low damage cases (for example, a fallen ceiling can be easily repaired, while a broken stair can require lot of time to be restored) and this aspect is shown by the slope of the transition part in the resilience graphs. As the damage increases, the operational level and the slope of the "repairing function" decrease: these two aspects lead to a less resilient building (in Figure 66, the growth of the red triangle area means a less resilient feature);
- the previous behaviour goes on until a certain threshold, after which the operational level cannot be fully restored again, due to the high damage level. This value is certainly lower than the Life Safety

TBPL value and its position inside the sloped branch of the tri-linear acceleration-damage law is a function of the characteristics of the Hospital system and organization.

The concept of resilience ranges over many disciplines and its detailed analysis does not represent one of the main goal of this work: anyway, it is important to understand that the resilient behaviour of a structure is at first connected to the beginning of the level of damage of a structure and so to the yielding acceleration level. The plastic range, identified between the two values of acceleration of the considered TBPL, is another important aspect which gives some information about resilience: in fact, the wider is this range, the slower should be the development of the damage inside the structure (the slope of the tri-linear acceleration-damage law in the central part is the mathematical translation of this concept).

In order to resume the analyzed concepts, resilience can be qualitatively considered with the two values of acceleration used to define the TBPL: the yielding acceleration gives information about the level of seismic action for which resilience starts to be necessary, while the width of the "yielding acceleration - life safety acceleration" range gives information about the rise of the damage and, in the same way, on the level of requested resilience. In the following Figure 66, a scheme is proposed: starting from a pushover curve, the bi-linearization gives the yielding and life safety displacement; with the described analytical procedure, it is possible to obtain the related accelerations: these are the necessary parameters for a simplified tri-linear acceleration-damage law. Below the graph it is shown the qualitative acceleration-resilience relation.

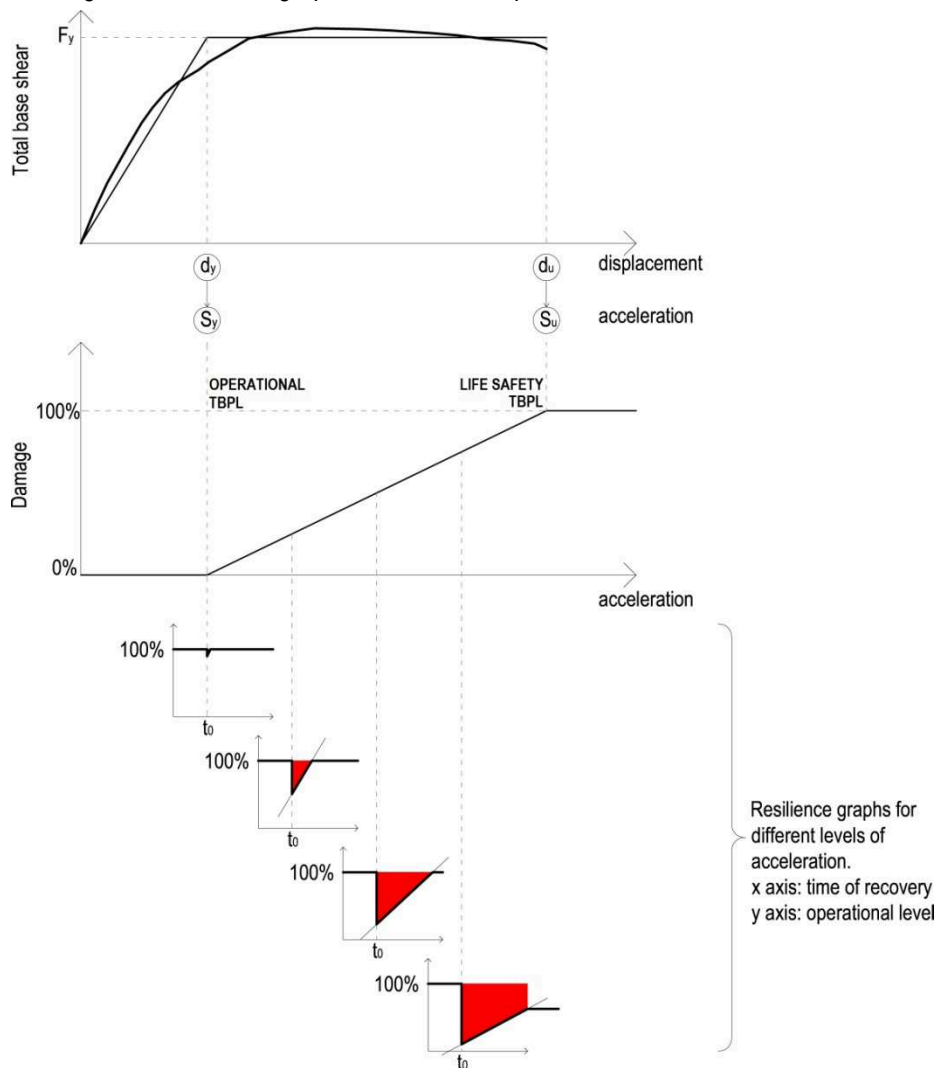


Figure 66: Visual resume of the concepts of resilience, yielding and life safety accelerations.

4.6 Static non-linear analysis: results from a case study

In this paragraph, the analytical results for a case study of the analyzed sample of buildings are proposed.

It is useful to briefly remind what has been done for each structure:

- creation of a three-dimensional model in the software 3MURI, developed and distributed by S.T.A.DATA, in order to perform static non-linear analyses;
- realization of 8 pushover analyses, considering 2 principal directions of the building (called "X" and "Y"), 2 lateral load profiles (proportional to the masses or to the 1st modal shape) and the presence or absence of eccentricity of the load profile in relation to the position of the barycenter of the masses;
- for each pushover analysis, the capacity curve has been obtained and, from that, some response parameters have been calculated by means of the analytical procedure described in § 4.5.4: in particular, the displacements and related PGA of capacity corresponding to the yielding point and to the end of the bi-linear system (equivalent schematization of the capacity curve) have been calculated, both with the constant and variable spectrum shape methods.

In order to give an example of the analytical procedure, the results for the S.U. identified with the name AOUC-CAR-4 01 are proposed in this paragraph. A resume of the results for all the buildings is illustrated in the Annex A, reported at the end of the present work.

The AOUC-CAR-4 01 building is located in Florence, in the Careggi Hospital Company. The S.U. belongs to a Complex composed of 3 units, 2 contiguous made of masonry and 1 more recent, made of reinforced concrete. The considered unit has been built around 1930; it is composed of 3 levels (1 basement and 2 levels out of the ground), all realized with well organized stone masonry (external walls) and brick masonry (internal structural partitions). Most of the floors are realized with reinforced concrete and hollow tiles.

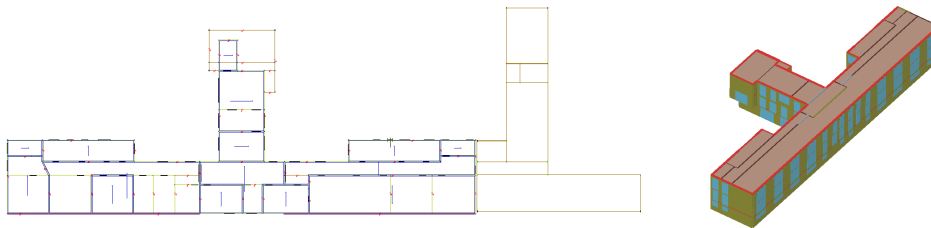


Figure 67: Planimetric and three-dimensional view of the model of AOUC-CAR-4 01.

The following tables contain the results obtained from the treatment of the 8 pushover curves.

In the first table, the data of the 8 performed analyses are shown, with the description of the period, the mass and the participating modal factor of the equivalent SDOF system.

General data						
ANALYSIS	DIRECTION	PROFILE LOAD	ECCENTR. [%]	T* [s]	m* [t]	Γ
1_X_1ms_0ecc	X	1 st modal shape	0%	0.265	2768	1.27
2_X_mass_0ecc	X	masses	0%	0.227	2768	1.27
3_X_1ms_5%ecc	X	1 st modal shape	5%	0.270	2768	1.27
4_X_mass_5%ecc	X	masses	5%	0.227	2768	1.27
5_Y_1ms_0ecc	Y	1 st modal shape	0%	0.364	2803	1.23
6_Y_mass_0ecc	Y	masses	0%	0.312	2803	1.23
7_Y_1ms_5%ecc	Y	1 st modal shape	5%	0.366	2803	1.23
8_Y_mass_5%ecc	Y	masses	5%	0.317	2803	1.23

Table 5: General data for the 8 performed analyses - AOUC-CAR-4 01.

For each analysis, the MDOF system pushover curve has been obtained; after the transformation of the MDOF curve in the SDOF one (using of the modal participating factor), the bi-linearization process has been

performed, leading to the values of the displacement related to the Operational Target Building Performance Level at first (easily evaluable as the end of the elastic branch of the bilinear curve). For each curve, the spectral acceleration has been then calculated and the PGA has been obtained using the two methods described above (with evaluation of return period for the variable spectrum method).

Operational Target Building Performance Level						
ANALYSIS	d_y [cm]	d'_y [cm]	$S_e(T, PGA_{oper})$ [g]	PGA_{oper} ($T_r=cost$) [g]	$PGA_{oper}(T_r)$ [g]	T_r [years]
1_X_1ms_0ecc	0.57	0.45	0.2588	0.1084	0.1040	263
2_X_mass_0ecc	0.55	0.44	0.3402	0.1424	0.1414	592
3_X_1ms_5%ecc	0.56	0.44	0.2425	0.1015	0.0965	217
4_X_mass_5%ecc	0.53	0.42	0.3244	0.1358	0.1346	511
5_Y_1ms_0ecc	0.33	0.27	0.0807	0.0397	0.0460	28
6_Y_mass_0ecc	0.37	0.30	0.1249	0.0526	0.0563	50
7_Y_1ms_5%ecc	0.34	0.28	0.0829	0.0410	0.0470	30
8_Y_mass_5%ecc	0.33	0.27	0.1087	0.0465	0.0515	39

Table 6: Results for the Operational Target Building Performance Level - AOUC-CAR-4 01.

The ultimate displacement has been considered as the one which corresponds to a decay of the maximum resistant shear equal to 20%; with the procedure explained in § 4.5.4, the capacity expressed in terms of peak ground acceleration and return period has been calculated for the Life Safety Target Building Performance Level. In the following Table 7, the values of ductility and behaviour factor are listed after the capacity results.

Life Safety Target Building Performance Level								
ANALYSIS	d_u [cm]	d'_u [cm]	$S_e(T, PGA_{life_saf})$ [g]	PGA_{life_saf} ($T_r=cost$) [g]	$PGA_{life_saf}(T_r)$ [g]	T_r [years]	q^* [-]	μ_d [-]
1_X_1ms_0ecc	2.07	1.63	0.7723	0.3240	0.3163	>2475	3.23	3.61
2_X_mass_0ecc	2.20	1.73	1.0216	0.4280	0.4157	>2475	3.18	3.98
3_X_1ms_5%ecc	2.16	1.70	0.7317	0.3070	0.3001	>2475	3.50	3.87
4_X_mass_5%ecc	2.47	1.94	0.9779	0.4100	0.3983	>2475	3.69	4.68
5_Y_1ms_0ecc	1.62	1.32	0.2474	0.1216	0.1231	403	4.95	4.95
6_Y_mass_0ecc	1.48	1.20	0.3793	0.1597	0.1601	854	3.98	3.98
7_Y_1ms_5%ecc	1.68	1.37	0.2482	0.1226	0.1242	413	4.95	4.95
8_Y_mass_5%ecc	1.28	1.04	0.3192	0.1366	0.1383	554	3.84	3.84

Table 7: Results for the Operational Target Building Performance Level - AOUC-CAR-4 01.

It is important to observe that, in some cases, the capacity expressed in terms of the return period T_r exceeds the maximum value available from the Italian seismic hazard Charts (2475 years): for these cases, the indication ">2475 years" has been adopted and the values of the related PGA are an extrapolation from the available data (see Figure 63). The obtained capacity curves are plotted in Figure 68, distinguishing the direction with a different colour and the type of analysis with a different line style.

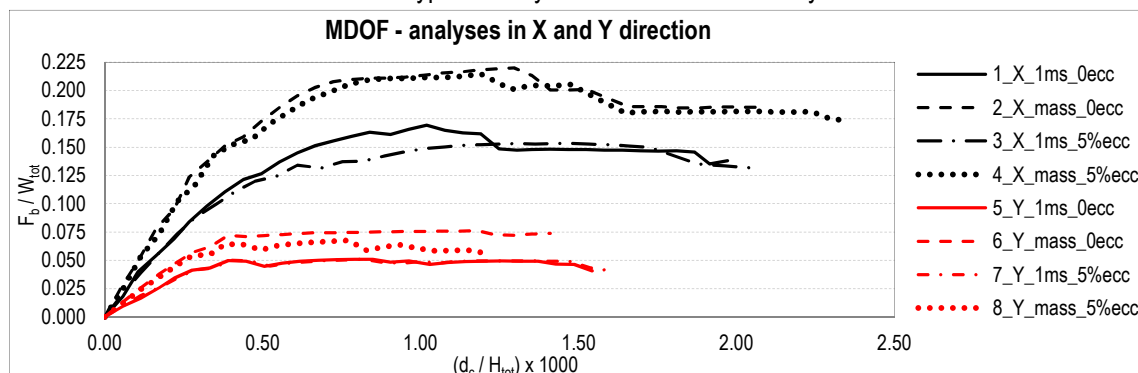


Figure 68: Pushover curves for the X and Y directions - AOUC-CAR-4 01.

As predictable from the observation of the planimetric and 3d view of the model (Figure 67), the behaviour in the two main directions is completely different, due to the different distributions of the resistant systems along them. The longitudinal direction of the building (called "X") shows a higher level of resistant shear, because of the presence of the longitudinal alignments of walls, while the orthogonal direction has few continuous resistant systems (from the bottom to the top of the structure), as clearly visible from the three-dimensional view of the model without the floors (Figure 69).

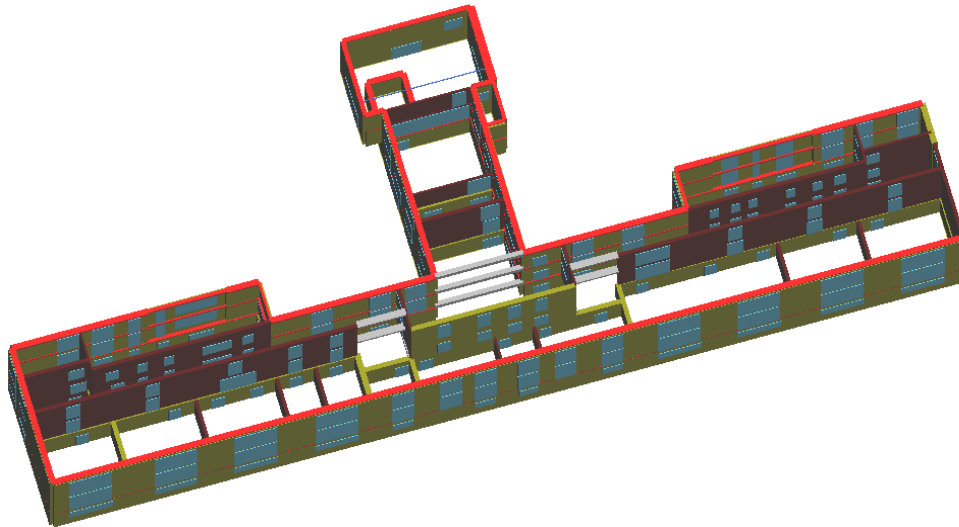


Figure 69: Three-dimensional view of the model without the indication of the floors - AOUC-CAR-4 01.

The analyses which give the worst results in terms of PGA_C are plotted again the following Figure 70, where the bi-linear schematization for each of them is shown. Each curve is plotted with two different values of thickness: the thicker part shows the portion of the curve which has $q^* \leq 3$, while the other part is out of this limit: the capacity has been calculated considering the last displacement of the thicker part of the curves.

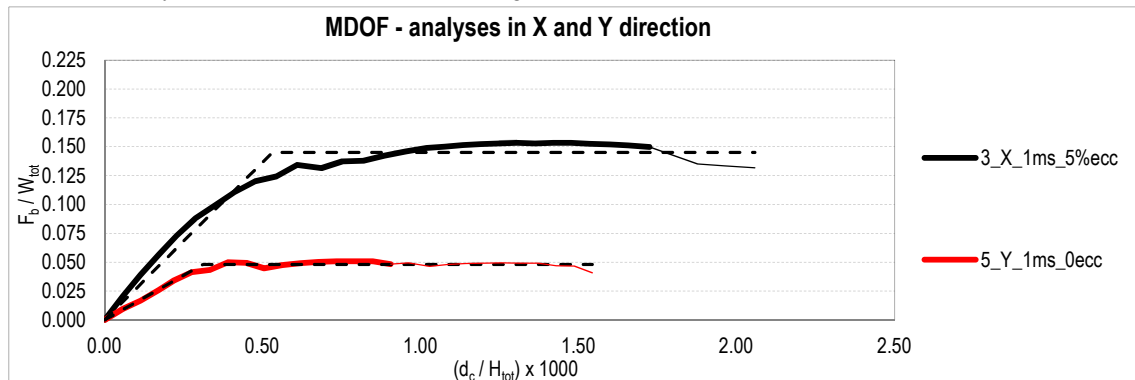


Figure 70: Worst analyses in the two main directions - AOUC-CAR-4 01.

For each of the analyses shown in Figure 70, a deformed plan view and a significant deformed facade is reported (Figure 72 and Figure 73), showing the level of damage in correspondence to the end of the pushover curve. The facades show the level of damage with the following convention (Figure 71):

	Not damaged
	Plastic phase for diagonal cracking
	Collapse for diagonal cracking
	Plastic phase for bending
	Collapse for bending

Figure 71: Graphical convention for the identification of the level of damage on piers and spandrels.

In both cases, the collapse occurs for the 2nd interstorey: for the longitudinal direction, the main collapse typology is the bending one while for the orthogonal direction is the diagonal cracking.

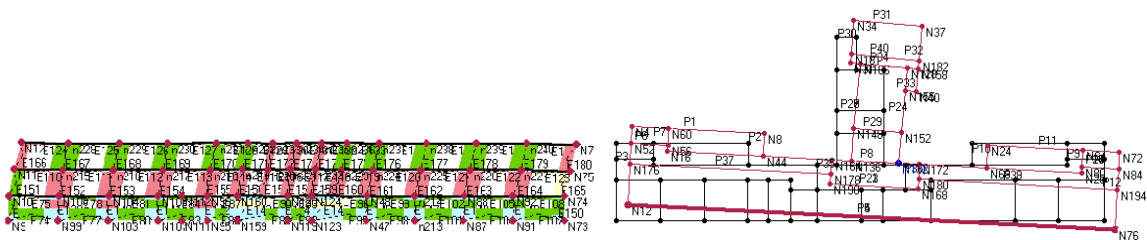


Figure 72: Deformed plan view and a significant deformed facade for the analysis 3 - AOUC-CAR-4 01.

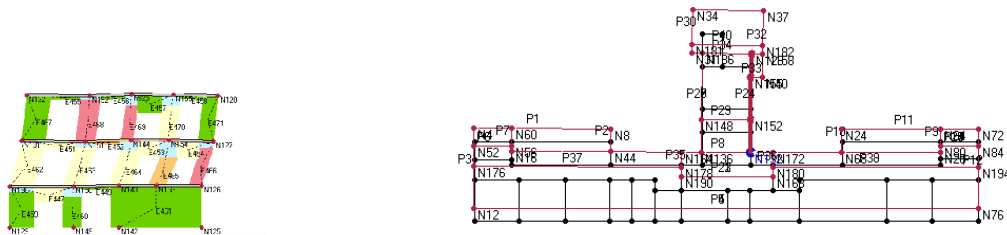


Figure 73: Deformed plan view and a significant deformed facade for the analysis 5 - AOUC-CAR-4 01.

Since some of the buildings show a capacity which exceeds the maximum value of the return period, it has been decided to consider the results of the method "calculation of the PGA_C with fixed spectrum shape": this method is coherent with the other one (calculation of the PGA and the return period varying the spectrum shape) considering the results shown in Table 6 and Table 7 in terms of PGA_C and it has no limits like the other method, which can require an extrapolation of the available results instead.

Considering all these aspects, the capacity for the case study is referred to the analysis n.° 5:

$$PGA_C = 0.1216g$$

With this procedure, all 20 buildings of the sample have been analyzed, leading to a classification of vulnerability expressed in terms of PGA_C .

4.7 Results of the analytical approach and comparisons with the expeditious approach

4.7.1 Comparison among the l_v (11 parameters) and the PGA_c

As described in the previous paragraph, all the 20 buildings of the subset of masonry S.U. (chosen for the detailed analyses) have been investigated with static non-linear analyses. For each of them, a reference value of the capacity has been obtained: in particular, considering the Life Safety TBPL, which corresponds to the ultimate limit state, the minimum value of the PGA of capacity has been calculated; in the following Table 8, the results of these analyses are shown.

For each building, the l_v , obtained from the expeditious vulnerability assessment method, and the PGA_c mentioned above are listed. The classification has been ordered following the expeditious method.

RELATIVE RANKING (l_v 11parameters)	STRUCTURAL UNIT	l_v 11parameters [%]	PGA_c [g]	RELATIVE RANKING (PGA_c)
1	AUSL 4 MD 01 24	70.9	0.115	2
2	AUSL 3 SMP 01 03	69.6	0.097	1
3	USL 10 SERR 01	60.1	0.164	7
4	AUSL 3 PES 01 06	59.2	0.148	6
5	USL 10 IOT ANT 01	58.8	0.130	4
6	AOUC CAR 16 01	56.2	0.144	5
7	AOUC CAR 4 01	52.3	0.122	3
8	AOUC CAR 13 03	45.4	0.177	9
9	AOUC CAR 13 04	45.1	0.172	8
10	AOUC CAR 8b 03	44.1	0.203	12
11	AOUC CAR 4 02	43.1	0.211	16
12	AOUC CAR 13 01	40.2	0.223	18
13	AOUC CAR 26 02	38.9	0.195	10
14	AOUC CAR 26 03	38.9	0.209	14
15	AOUC CAR 8b 01	38.9	0.210	15
16	AOUC CAR 8b 02	38.9	0.220	17
17	AOUC CAR 8b 04	37.6	0.208	13
18	AOUC CAR 26 01	37.6	0.226	19
19	AUSL 3 PES 01 04	35.6	0.203	11
20	AUSL3 SMP 01 04	35.0	0.226	20

Table 8: Results of the detailed analyses for the chosen subset of masonry S.U.. The list is ordered by the l_v .

Looking at Table 8, it is possible to observe that the classification realized with the l_v (expeditious method) does not fully match with the one of the PGA_c (detailed method).

In order to highlight the differences, the positions of each S.U. are plotted in the following Figure 74: the green histogram represents the l_v classification and, since the S.U. are ordered with that ranking, it is simply a step function from 1 to 20 with each step equal to 1 position; the black histogram instead is the PGA classification. It is possible to see that there are some cases where the error among the two classifications reaches values of 8 positions of difference, while only 3 S.U. have the same position in the two classifications. The standard deviation of the position errors is equal to $\sigma = 3.23$.

In any case, a certain level of correspondence can be observed.

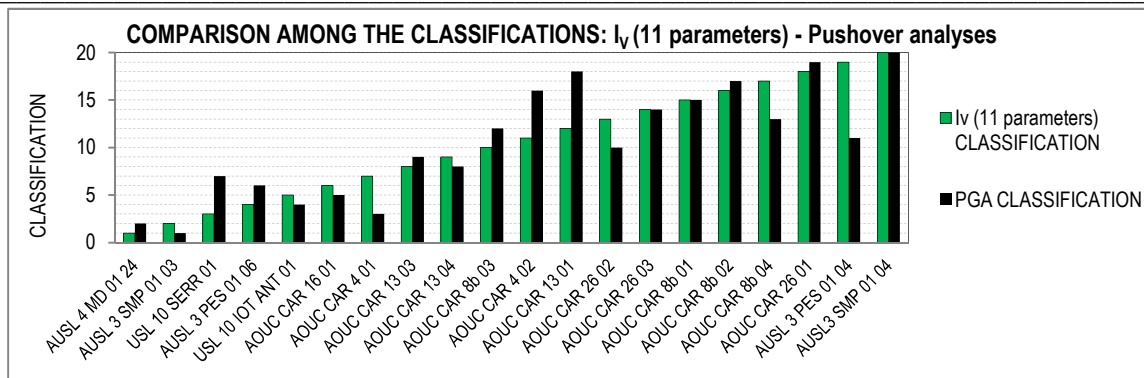


Figure 74: Comparison among the classifications obtained with the expeditious method (I_v - 11 parameters) and the detailed one.

Until now the positions in the two classifications have been observed; in order to observe the possible relation among the quantities I_v - PGA_C , the cloud of points representing the sample of buildings has been plotted and the trend line for this cloud has been obtained (Figure 75).

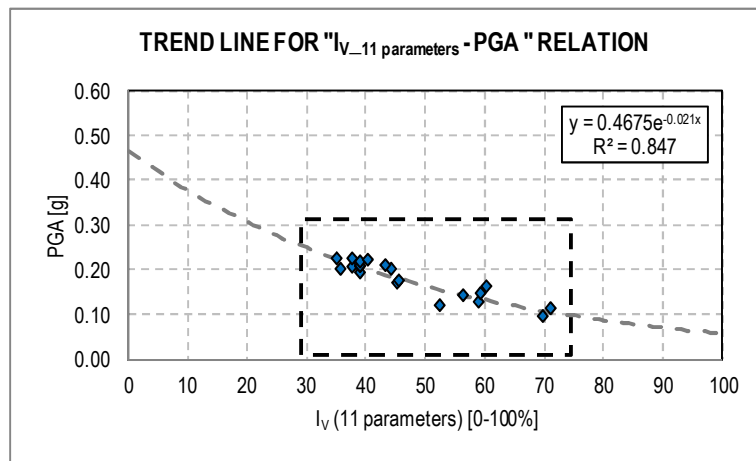


Figure 75: Relation among I_v 11 parameters and PGA_C .

An exponential trend has been chosen because it has been considered as most representative for this type of relation. In particular, the cloud of points is placed close to the line in the central part, and the extreme parts of the trend line give qualitative correct information:

- the most vulnerable buildings (high values of I_v) can still have a certain level of seismic capacity, expressed in terms of PGA_C ; generally speaking, masonry buildings which have high values of I_v are not characterized by a global behaviour, but their vulnerability is mainly related to local mechanisms phenomena, for which the PGA_C can be estimated through kinematic analyses on the singular walls;
- the least vulnerable buildings (low values of I_v) have a capacity which grows as far as the I_v decreases, arriving to the highest value of almost $PGA_C=0.50g$. This value, obtained simply extending the trend line until the vertical axis, should represent the capacity of a "fully not vulnerable" building: considering the seismic hazard of the Italian territory, a building with a capacity equal to $PGA_C=0.50g$ is surely well realized, even if the concept of "fully not vulnerable" building represents only an abstraction. Anyway, it can be assumed that the trend has reasonable values even for very low levels of Index of Vulnerability.

It is important to remind that this study focuses on the central part of the I_v range of variability: for well realized buildings (low values of I_v), the exact estimation of the PGA_C is not so interesting since they do not represent the elements most exposed to risk. On the other hand, the most vulnerable buildings cannot be analyzed with a global analysis, since it is not fully representative for their behaviour; the sample of analysis (as explained in

§ 4.2.1) has, on average, a medium high construction quality which may suggest that local mechanisms aspects do not represent the main problem of this category of structures, even though the local analysis always represents a necessary step in the seismic evaluation of any kind of structure. This argument will be qualitative considered in this work, as described in the following chapters.

Considering again Figure 75, the zoom of the dashed line rectangle is proposed in the following Figure 76, in order to highlight the distances among the cloud of points and the trend line.

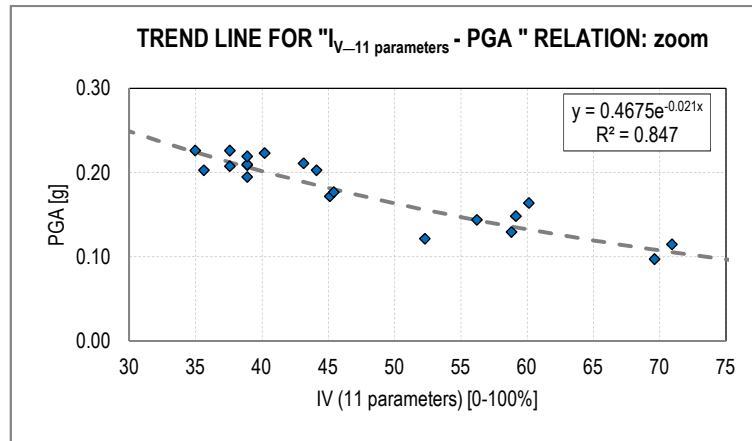


Figure 76: Relation among $I_{v,11}$ parameters and PGA_c - zoom.

Using the trend line equation, for each S.U. the PGA_{c_est} has been estimated and compared with the value obtained with the detailed analyses in the following Table 9.

RELATIVE RANKING ($I_{v,11}$ parameters)	STRUCTURAL UNIT	$I_{v,11}$ parameters [%]	PGA_c [g]	PGA_{c_est} [g]	error PGA_{c_est}/PGA_c	over estimation error	under estimation error
1	AUSL 4 MD 01 24	70.92	0.1149	0.1054	-8.2%		-8.2%
2	AUSL 3 SMP 01 03	69.61	0.0973	0.1084	11.4%	11.4%	
3	USL 10 SERR 01	60.13	0.1640	0.1322	-19.4%		-19.4%
4	AUSL 3 PES 01 06	59.15	0.1484	0.1350	-9.0%		-9.0%
5	USL 10 IOT ANT 01	58.82	0.1295	0.1359	5.0%	5.0%	
6	AOUC CAR 16 01	56.21	0.1440	0.1436	-0.3%		-0.3%
7	AOUC CAR 4 01	52.29	0.1216	0.1559	28.2%	28.2%	
8	AOUC CAR 13 03	45.42	0.1770	0.1801	1.8%	1.8%	
9	AOUC CAR 13 04	45.12	0.1720	0.1813	5.4%	5.4%	
10	AOUC CAR 8b 03	44.12	0.2030	0.1851	-8.8%		-8.8%
11	AOUC CAR 4 02	43.14	0.2112	0.1889	-10.5%		-10.5%
12	AOUC CAR 13 01	40.20	0.2234	0.2010	-10.0%		-10.0%
13	AOUC CAR 26 02	38.89	0.1950	0.2066	5.9%	5.9%	
14	AOUC CAR 26 03	38.89	0.2090	0.2066	-1.2%		-1.2%
15	AOUC CAR 8b 01	38.89	0.2100	0.2066	-1.6%		-1.6%
16	AOUC CAR 8b 02	38.89	0.2195	0.2066	-5.9%		-5.9%
17	AOUC CAR 8b 04	37.58	0.2078	0.2123	2.2%	2.2%	
18	AOUC CAR 26 01	37.58	0.2263	0.2123	-6.2%		-6.2%
19	AUSL 3 PES 01 04	35.62	0.2030	0.2213	9.0%	9.0%	
20	AUSL3 SMP 01 04	34.97	0.2264	0.2243	-0.9%		-0.9%

maximum	28.2%	-19.4%
medium	8.6%	-6.8%

Table 9: Estimation of the PGA_c using the trend line $I_{v,11}$ parameters - PGA_c . Measure of the errors comparing the results with the detailed analyses.

Observing the results of the previous Table 9, it is possible to see that the trend line leads to significant errors; in particular, the over estimation and under estimation errors have been divided, in order to highlight, from the engineering point of view, the errors which are more relevant: an over estimation of 28.2% cannot be considered as acceptable because it influences in a direct way the estimation of the seismic risk of the S.U.. Considering the entire sample of errors, the medium value of the error is $\mu = -0.66\%$, the standard deviation $\sigma = 10.22\%$ and, consequently, the coefficient of variation of the error is $v = \sigma/\mu = 15.46$.

4.7.2 Correspondence of modelling among the two methods

The first comparison among the expeditious method and the detailed one has been proposed in § 4.7.1 considering directly the results from the methods.

In order to understand this relation, it is important to consider some aspects related to the detailed analyses:

- the detailed analyses are performed on numerical three-dimensional models, which allow the realization of static non-linear analyses;
- in the numerical models, non structural elements are not modelled as well as the soil: the structures are considered as fixed at the base level;
- the pushover analysis is realized on a global model of the structure, considering that all the walls and floors are efficiently connected as well as orthogonal walls along the vertical intersections;
- the local mechanisms phenomena are consequently not considered in this type of analysis.

Referring to the last point, it is important to remind that, during the vulnerability assessment of an existing building, local mechanisms must be investigated at first, in order to give reliability to the global analysis, which can be realized after only if the structure does not show considerable local problems.

It has been already explained that, for the sample of analyzed buildings, the local mechanisms phenomena should not represent the main problem of this category of structures since a general high construction quality has been observed. Moreover, the aim of this work is to give a contribution to the estimation of the global capacity estimation of masonry structures through the comparison among expeditious and detailed analysis; in any case, the local mechanisms analysis always represents a necessary step in the seismic evaluation of any kind of structure and it must be performed before the global analysis.

Resuming all the assumptions listed above, it is possible to assess that the detailed results proposed in this work for each structural unit refer to a particular typology of behaviour of the masonry structures, as already explained before (§ 4.2.1): the global behaviour. On the other hand, the expeditious method is based on qualitative evaluations (except the parameter 3 - conventional resistance, which is an estimation of the maximum lateral load) concerning different aspects.

The two investigated methods (the Vulnerability Index Method and the pushover analysis) are then not directly comparable, since they are based on different hypotheses.

In the following list, the differences among the two methods are explained, considering each single parameter of the Vulnerability Index Method:

- PARAMETER 1 - type and organization of the resistant system
In this parameter, the presence and the quality of the connections among the walls are evaluated, without considering the typology of the materials; in other words, the presence of the "box behaviour" is observed.
In the mechanical models, as written before, this aspect is implicitly assumed as in the best condition, because the analytical model has perfect connections among the walls. The quality of the connections must be evaluated in the preliminary phase of the local mechanisms analysis, as already described above.

- PARAMETER 2 - quality of the resistant system
In this parameter, the type of masonry is qualitatively evaluated: it is necessary to observe the type of material, the homogeneity of the masonry, the presence of horizontal brick layers, etc...
In the analytical model, the characteristics of the materials are considered in order to describe the mechanical properties of the structural system. In particular, as mentioned in the previous paragraphs, the classification of the masonry typologies made by the actual Italian Code (D.M. 2008) is considered as reference, with the possibility to add improvement coefficients for the properties in case of particular situations, such as good quality mortar, transversal connection of the masonry layers etc...There are also some pejorative coefficients, in order to take into account, for example, a low quality internal material for a layered masonry.
- PARAMETER 3 - conventional resistance
In the 11 parameters form, this element is calculated with a simplified method; in the analytical model, this aspect is fully considered since it represents an essential part of the definition of the capacity curve.
- PARAMETER 4 - position of the building and foundations
In this parameter, the local morphology of the site of construction and the type of foundations are taken into account. Since the analytical models are realized considering the building as placed on a horizontal field with rock ground characteristics without any particular geotechnical aspects, this parameter is implicitly assumed as with the best condition in the detailed approach.
- PARAMETER 5 - typology of floors
- PARAMETER 6 - planimetric configuration
- PARAMETER 7 - elevation configuration
- PARAMETER 9 - roof
The four parameters listed above are taken into account in the mechanical model, since they represent the importance of the geometry and technology of realization of the considered building.
- PARAMETER 8 - maximum distance among the walls
This parameter, even if the structure is fully modelled in the analytical approach, is mainly referred to the possibility of activation of local mechanisms in the expeditious approach: for this reason, in the detailed approach, it is implicitly assumed as with the best condition.
- PARAMETER 10 - non structural elements
In this parameter, the presence of elements (non structural ones) which can cause damages to persons or objects due to their falling is analyzed. In the numerical model, this aspect is not taken into account, since these elements (for example ceilings, chimneys, etc...) cannot be modelled.
- PARAMETER 11 - state of conservation
In this parameter, the actual condition of the building is analyzed. The presence of cracks on the walls is investigated as well as the degradation of the plasters, the presence of humidity in the walls etc...
This parameter is not considered in the detailed approach, since each typology of masonry is modelled with the mechanical characteristics which are described in the Italian Code (D.M. 2008); in case of particular extended conditions which highlight a clear reduction of the mechanical properties, different materials can be assumed in the modelling phase.

Resuming what has been described until now, the expeditious method for the assessment of vulnerability considers some aspects which cannot be taken into account in the detailed approach; in particular, the parameters which are considered in both methods are the ones listed in the following Table 10:

PARAMETER		INCLUDED IN THE DETAILED APPROACH?
PAR. 1	Type and organization of the resistant system	-
PAR. 2	Quality of the resistant system	X
PAR. 3	Conventional resistance	X
PAR. 4	Position of the building and foundations	-
PAR. 5	Typology of floors	X
PAR. 6	Planimetric configuration	X
PAR. 7	Elevation configuration	X
PAR. 8	Maximum distance among the walls	-
PAR. 9	Roof	X
PAR. 10	Non structural elements	-
PAR. 11	State of conservation	-

Table 10: Parameters considered in the detailed approach.

4.7.3 Comparison among the I_v (6 parameters) and the PGA_C

Considering the aspects described in the previous paragraph, it is possible to conclude that a direct comparison among the Vulnerability Index Method and the results of pushover analyses is not fully reliable, since different aspects are considered in the two methods of vulnerability estimation.

In order to obtain a reliable comparison, it has been decided to consider:

- the parameters which are taken into account in both methods with their scores;
- the parameters which are not directly modelled in the detailed analyses have been assumed as in the best condition for the expeditious method; this choice appears clear considering the parameter 1 for example, which evaluates the connection among the walls: in the three-dimensional model of the structure, the connections are assumed as perfect since the equivalent frame structure does not provide the possibility of out of plane mechanisms while performing the pushover analysis.

Analogous considerations can be extended to the other "not considered" parameters of the Table 10.

With these assumptions, a new Vulnerability Index has been calculated for each structure of the subset (which has been analyzed both with the expeditious and detailed methods): the new Indexes of Vulnerability are based on the original judgments for the parameters included in the detailed approach, while for the other ones the best judgment has been assigned (A judgment).

In the next Table 11, an example of calculation of the new Vulnerability Index is shown (AOUC CAR 4 01).

PARAMETER		I_v (11 parameters)				I_v (6 parameters)			
		SCORE	POINTS	WEIGHT	WEIGHTED POINTS	SCORE	POINTS	WEIGHT	WEIGHTED POINTS
1	Type and organization of the resistant system	C	20	1	20	A	0	1	0
2	Quality of the resistant system	C	25	0.25	6.25	C	25	0.25	6.25
3	Conventional resistance	D	45	1.5	67.5	D	45	1.5	67.5
4	Position of the building and foundations	B	5	0.75	3.75	A	0	0.75	0
5	Typology of floors	C	15	1	15	C	15	1	15
6	Planimetric configuration	D	45	0.5	22.5	D	45	0.5	22.5
7	Elevation configuration	C	25	1	25	C	25	1	25
8	Maximum distance among the walls	D	45	0.25	11.25	A	0	0.25	0
9	Roof	C	25	0.5	12.5	C	25	0.5	12.5
10	Non structural elements	D	45	0.25	11.25	A	0	0.25	0
11	State of conservation	B	5	1	5	A	0	1	0
		Σ POINTS ,WEIGHTS 200 I_v 52.29%				Σ POINTS ,WEIGHTS 148.75 I_v 38.89%			

Table 11: Index of Vulnerability calculated with 6 parameters, in order to perform a direct comparison with the results of the detailed analyses.

From this point, the new Index of Vulnerability, based on this assumption, is called "6 parameters I_V ", in order to highlight that only 6 parameters are variable, while the others are fixed (A judgment).

The maximum value of vulnerability that is possible to reach with these assumptions is $I_V=61.76\%$.

After the calculation explained above, all the Indexes of vulnerability change: the values become smaller or they remain the same at least, since the Index of Vulnerability now concerns only the global behaviour aspect.

The classification realized with the 11 parameters I_V does not coincide with the one of the 6 parameters I_V .

The comparison among the classifications performed with the two methods (expeditious and detailed one), considering the new values of the Index of Vulnerability (6 parameters), is shown in the following Table 12.

Even in this case, the two classifications do not match completely; as done before for the other comparison, the graph with the positions of each S.U. in the two approaches is plotted in Figure 77: the blue histogram represents the I_V (6 parameters) classification and, since the S.U. are ordered with that ranking, it is simply a step function from 1 to 20 with step equal to 1 position; the black histogram instead is the PGA classification.

RELATIVE RANKING (I_V 6parameters)	STRUCTURAL UNIT	I_V 6parameters [%]	PGA _c [g]	RELATIVE RANKING (PGA _c)
1	AUSL 3 SMP 01 03	52.29	0.0973	1
2	AUSL 4 MD 01 24	48.04	0.1149	2
3	USL 10 IOT ANT 01	42.81	0.1295	4
4	AOUC CAR 16 01	38.89	0.1440	5
5	AOUC CAR 4 01	38.89	0.1216	3
6	USL 10 SERR 01	37.58	0.1640	7
7	AUSL 3 PES 01 06	36.93	0.1484	6
8	AOUC CAR 13 03	34.97	0.1770	9
9	AOUC CAR 13 04	34.66	0.1720	8
10	AOUC CAR 8b 03	33.66	0.2030	12
11	AOUC CAR 4 02	29.74	0.2112	16
12	AOUC CAR 13 01	29.74	0.2234	18
13	AUSL 3 PES 01 04	29.08	0.2030	11
14	AOUC CAR 8b 01	28.43	0.2100	15
15	AOUC CAR 8b 02	28.43	0.2195	17
16	AOUC CAR 26 02	28.10	0.1950	10
17	AOUC CAR 26 03	28.10	0.2090	14
18	AOUC CAR 8b 04	27.12	0.2078	13
19	AOUC CAR 26 01	26.80	0.2263	19
20	AUSL3 SMP 01 04	24.18	0.2264	20

Table 12: Comparison among the classifications of the expeditious and detailed method. The list is ordered by the new values of I_V (6 parameters).

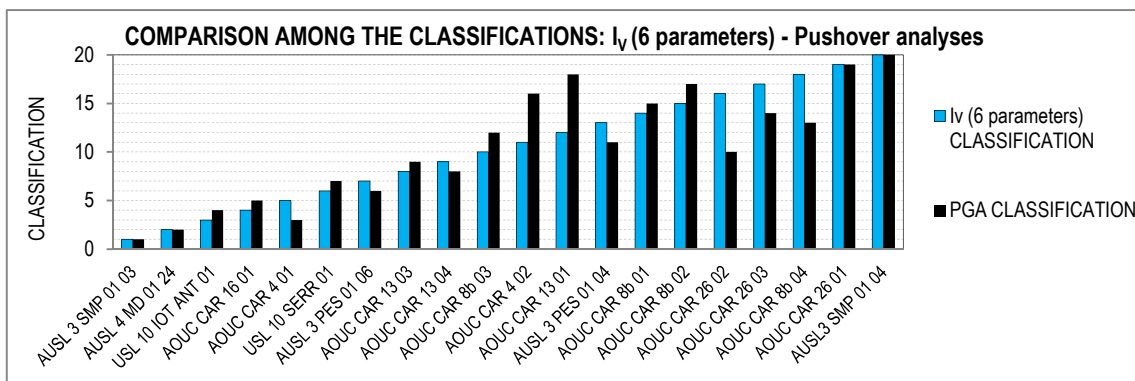


Figure 77: Comparison among the classifications obtained with the expeditious method (I_V - 6 parameters) and the detailed method.

It is possible to see that there are some cases where the error among the two classifications reaches values of 6 positions of difference, while only 4 S.U. have the same position in the two classifications. The standard deviation of the position errors is equal to $\sigma = 2.85$.

Considering the graphs, the values of the standard deviation and the maximum position errors, it is possible to affirm that this new comparison shows a better correspondence than the one observed in § 4.7.1.

As performed before in § 4.7.1, the cloud of points $I_V - PGA_C$, representing the sample of buildings, has been plotted and the trend line for this cloud has been obtained (Figure 78).

The exponential trend has been chosen again and, even in this graph, it gives a trend line which represents correctly the general qualitative relation among the two quantities $I_V - PGA_C$.

Even in this case, for high vulnerable structures (the maximum value, as written before, is $I_V=61.76\%$), a certain level of capacity is still possible, which must be estimated with the local mechanisms analysis.

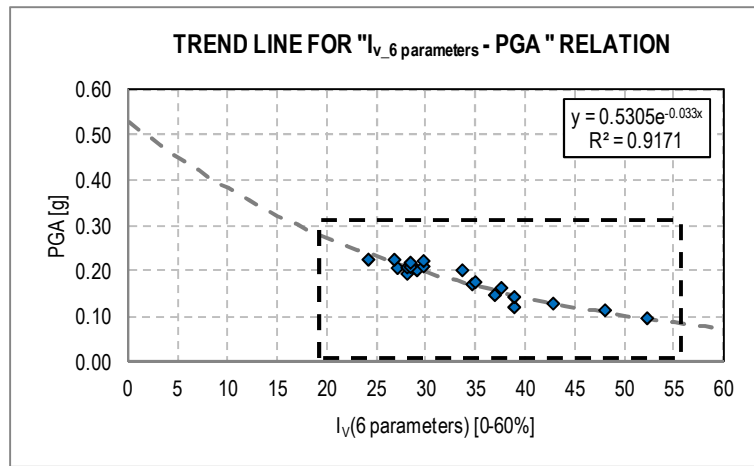


Figure 78: Relation among $I_{V,6 \text{ parameters}}$ and PGA_C .

Considering again Figure 78, the zoom of the dashed line rectangle is proposed in the following Figure 79, in order to highlight the distances among the cloud of points and the trend line.

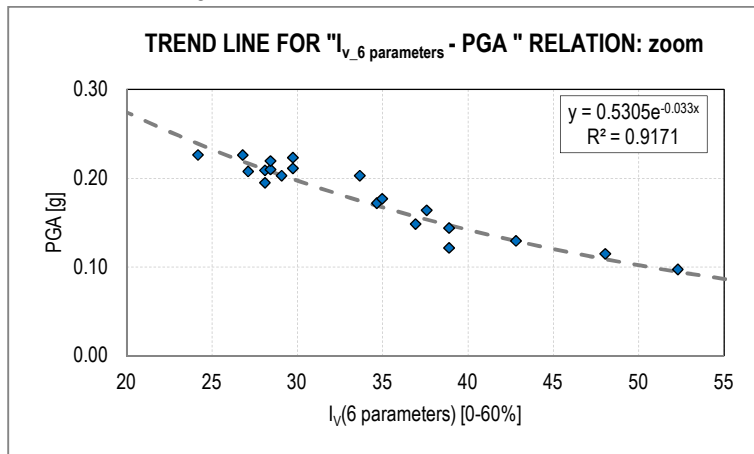


Figure 79: Relation among $I_{V,6 \text{ parameters}}$ and PGA_C - zoom.

Once again, using the trend line equation, the PGA_{C_est} has been estimated for each S.U. and compared with the value obtained from the detailed analyses. In the following Table 13 it is possible to observe the classification of vulnerability listed by the values of the Index of Vulnerability (calculated with 6 parameters), the related value of the seismic capacity expressed in terms of PGA_C (obtained from the detailed analyses), the estimation of the seismic capacity performed with the trend line and the evaluation of the relative errors, distinguishing the over estimation errors from the under estimation errors.

Comparing these results with the ones obtained in the Table 9, it is possible to observe that the maximum errors decrease (maximum over estimation from 28.2% to 20.9%) as well as the medium values (medium over estimation from 8.6% to 5.8%); moreover, considering the entire sample of errors, the medium value of the error becomes $\mu = -0.81\%$ and the standard deviation $\sigma = 7.50\%$, leading consequently to a coefficient of variation of the error equal to $v = \sigma/\mu = 9.31$. In the other correlation analysis, the coefficient of variation was equal to $v = \sigma/\mu = 15.46$: this reduction highlights a better correspondence of the expeditious vulnerability estimation method based on 6 parameters to the static non-linear analyses than the same one based on 11 parameters.

RELATIVE RANKING (lv_6parameters)	STRUCTURAL UNIT	lv_6parameters [%]	PGA _c [g]	PGA _{c_est} [g]	error PGA _{c_est} /PGA _c	over estimation error	under estimation error
1	AUSL 3 SMP 01 03	52.29	0.0973	0.0945	-2.9%		-2.9%
2	AUSL 4 MD 01 24	48.04	0.1149	0.1087	-5.4%		-5.4%
3	USL 10 IOT ANT 01	42.81	0.1295	0.1292	-0.3%		-0.3%
4	AOUC CAR 16 01	38.89	0.1440	0.1470	2.1%	2.1%	
5	AOUC CAR 4 01	38.89	0.1216	0.1470	20.9%	20.9%	
6	USL 10 SERR 01	37.58	0.1640	0.1535	-6.4%		-6.4%
7	AUSL 3 PES 01 06	36.93	0.1484	0.1568	5.7%	5.7%	
8	AOUC CAR 13 03	34.97	0.1770	0.1673	-5.5%		-5.5%
9	AOUC CAR 13 04	34.66	0.1720	0.1690	-1.7%		-1.7%
10	AOUC CAR 8b 03	33.66	0.2030	0.1747	-13.9%		-13.9%
11	AOUC CAR 4 02	29.74	0.2112	0.1988	-5.9%		-5.9%
12	AOUC CAR 13 01	29.74	0.2234	0.1988	-11.0%		-11.0%
13	AUSL 3 PES 01 04	29.08	0.2030	0.2032	0.1%	0.1%	
14	AOUC CAR 8b 01	28.43	0.2100	0.2076	-1.1%		-1.1%
15	AOUC CAR 8b 02	28.43	0.2195	0.2076	-5.4%		-5.4%
16	AOUC CAR 26 02	28.10	0.1950	0.2099	7.6%	7.6%	
17	AOUC CAR 26 03	28.10	0.2090	0.2099	0.4%	0.4%	
18	AOUC CAR 8b 04	27.12	0.2078	0.2168	4.3%	4.3%	
19	AOUC CAR 26 01	26.80	0.2263	0.2191	-3.2%		-3.2%
20	AUSL3 SMP 01 04	24.18	0.2264	0.2389	5.5%	5.5%	

maximum	20.9%	-13.9%
medium	5.8%	-5.2%

Table 13: Estimation of the PGA_c using the trend line lv_6parameters - PGA_c. Measure of the errors comparing the results with the detailed analyses.

4.8 Conclusions of the comparison analysis (Life Safety TBPL)

The results obtained in this chapter have highlighted that there is a certain level of relation among the expeditious vulnerability assessment method (Vulnerability Index Method) and the detailed approach (static non-linear analyses performed on numerical three-dimensional models) considering the capacity related to the Life Safety TBPL: in particular, the relation is stronger if only the parameters of the expeditious method concerning the global behaviour of a structure are considered, since this assumption allows a more direct comparison among the methods. The estimated errors are in any case relevant and they can lead to over estimations of the capacity of a structure (or, on the other hand, an under estimation of the vulnerability).

The study performed in this chapter has been useful to understand which parameters can be considered for the development of a new fast vulnerability assessment method; until now, the results have been obtained using the methods as they are, without modifying their structure.

In the following chapters, specific studies on the expeditious method will be proposed in order to understand the influences of each parameter, their efficacy in the vulnerability comprehension and the general relation among all of them. The final result of this work will propose a new vulnerability method, based on the same information required for the Vulnerability Index Method, which can give some more indications about the level of vulnerability and of seismic risk for the considered structure.

4.9 Analysis of the Operational TBPL

For the Operational TBPL, it has been decided (see § 4.5.4) that the yielding point of the bi-linear equivalent curve is representative for this performance level: in this paragraph, the results obtained on all the structures of the subset of buildings analyzed with a detailed approach are proposed.

First of all, the following Table 14 resumes all the results obtained for this TBPL; for each S.U. are listed:

- the PGA_C related to the Life Safety TBPL (ultimate limit state, calculated considering the maximum displacement of the pushover curve when $q^* < 3$ and imposing $q^* = 3$ for the other cases);
- the PGA_C related to the Operational TBPL (calculated for $q^* = 1$);
- the capacity values of the q^* and μ in correspondence to the Life Safety TBPL displacement. These values coincide when the structure can be assumed as flexible ($T^* > T_C$), as explained in §4.5.4;
- the ratio among the capacity accelerations listed before, which coincides with the value of q^* when $q^* < 3$, otherwise $PGA_{C_Life\ Safety}/PGA_{C_Operational} = 3$;
- the interstorey drift in correspondence of the Operational TBPL, measured in ‰;
- the interstorey displacement in correspondence of the Operational TBPL, measured in mm.

n.°	STRUCTURAL UNIT	PGA_C Life Safety TBPL [g]	PGA_C Operational TBPL [g]	available q^*	available μ	PGA_C : Life Safety / Operational	Interstorey drift Operational TBPL [‰]	Interstorey displacement Operational TBPL [mm]
1	AUSL 3 SMP 01 03	0.0973	0.0500	1.9	2.0	1.9	1.2	6
2	AUSL 4 MD 01 24	0.1149	0.0509	2.3	2.3	2.3	1.5	6
3	AOUC CAR 4 01	0.1216	0.0399	5.0	5.0	3.0	0.5	2
4	USL 10 IOT ANT 01	0.1295	0.0556	2.3	2.3	2.3	0.9	4
5	AOUC CAR 16 01	0.1440	0.0482	3.5	3.5	3.0	0.9	4
6	AUSL 3 PES 01 06	0.1484	0.0498	5.4	5.4	3.0	0.6	3
7	USL 10 SERR 01	0.1640	0.0542	4.7	4.7	3.0	0.4	3
8	AOUC CAR 13 04	0.1720	0.0819	2.1	2.1	2.1	2.8	13
9	AOUC CAR 13 03	0.1770	0.1170	1.5	1.5	1.5	1.4	7
10	AOUC CAR 26 02	0.1950	0.1040	1.9	1.9	1.9	2.8	12
11	AUSL 3 PES 01 04	0.2030	0.0702	2.9	3.0	2.9	1.3	4
12	AOUC CAR 8b 03	0.2030	0.0676	6.4	6.4	3.0	1.0	4
13	AOUC CAR 8b 04	0.2078	0.0691	3.8	3.8	3.0	1.2	4
14	AOUC CAR 26 03	0.2090	0.1021	2.0	2.0	2.0	1.6	8
15	AOUC CAR 8b 01	0.2100	0.0699	4.2	5.6	3.0	0.6	2
16	AOUC CAR 4 02	0.2112	0.0693	6.5	7.2	3.0	1.1	4
17	AOUC CAR 8b 02	0.2195	0.0730	6.0	7.2	3.0	0.7	3
18	AOUC CAR 13 01	0.2234	0.0850	2.6	2.6	2.6	0.9	4
19	AOUC CAR 26 01	0.2263	0.1252	1.8	1.8	1.8	1.3	7
20	AUSL3 SMP 01 04	0.2264	0.0760	8.1	12.6	3.0	0.3	1
	<i>minimum</i>	0.0973	0.0399	1.5	1.5	1.5	0.3	1
	<i>maximum</i>	0.2264	0.1252	8.1	12.6	3.0	2.8	13

Table 14: Resume of the results for the Operational TBPL.

The Operational TBPL can be investigated considering two different approaches: the displacement or the acceleration approach. This is necessary since the non structural elements in general, and for Hospital structures in particular, can be divided in two main categories, as stated in F.E.M.A. 273 (1997):

- acceleration sensitive components, which can show problems if exposed over a certain threshold of acceleration. This is the case of the Hospital technical equipments as, for example, X-rays machines;
- deformation sensitive components, which instead can show problems due to the relative movements of the structure. This is the case of all the net of medical gases, electrical systems and so on, which can stop their functions due to the cut of a line.

These two aspects will be analyzed with the available data obtained from the detailed analyses.

4.9.1 Acceleration sensitive components

From Table 14, it is possible to observe that exactly half of the structures of the sample is characterized by values of $q^* > 3$, highlighting that the analyses lead to high values of ductility consequently.

For these cases, $PGA_{C_Life\ Safety}/PGA_{C_Operational} = 3$.

The other structures are instead characterized by lower values of ductility and, as consequence, $PGA_{C_Life\ Safety}/PGA_{C_Operational} < 3$.

Plotting the cloud of points $PGA_{C_Life\ Safety} - PGA_{C_Operational}$ (each point in Figure 80 identifies a S.U. by means of the accelerations of capacity for the two limit conditions), it is possible to observe that it does not show any particular shape: this means that there is not a direct correlation among these quantities, even if they are obtained both from the same pushover curves.

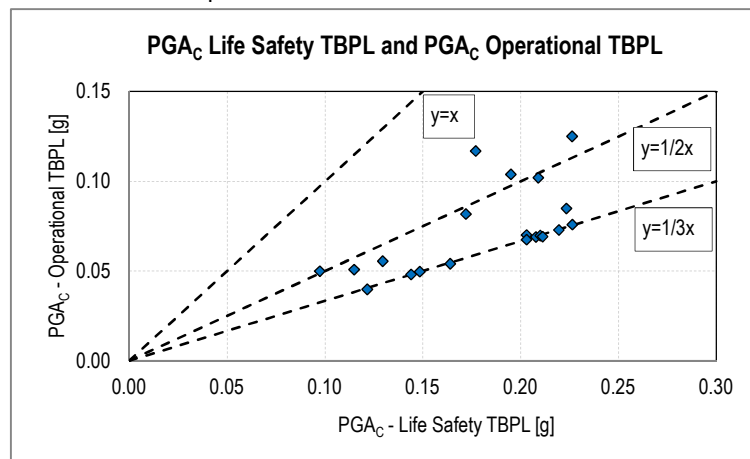


Figure 80: Relation among $PGA_{C_Life\ Safety}$ and $PGA_{C_Operational}$ TBPL.

It is only possible to observe that, since the ratio $PGA_{C_Life\ Safety}/PGA_{C_Operational}$ cannot assume values higher than 3, the line $y = 1/3x$ represents the lowest limit of the points distribution.

The S.U. with $q^* > 3$ have their points directly placed on that line, while the other ones are represented by points located over the line $y = 1/3x$; if the estimation of the $PGA_{C_Operational}$ is performed starting from $PGA_{C_Life\ Safety}$ and using the equation of the limit line $y = 1/3x$, an under-estimation is obtained.

This type of estimation is not useful since it is based on a too simplistic assumption and it cannot give reliable information.

4.9.2 Deformation sensitive components

Considering the pushover curves which have identified the capacity of the structures for the Life Safety TBPL, the maximum "significant" interstorey drift and the relative interstorey displacement, evaluated in the yielding point of the analyzed bi-linear curves, has been registered and listed in the previous Table 14; the adjective "significant" has been written in order to highlight that, since the drift is measured on the masonry alignments and since each singular alignment has its own displacement configuration (due to the possibility of rotation of the building during the pushover analysis and, in general, to the presence of flexible floors which allow differential horizontal movements among the different masonry alignments), the maximum drift related to a significant alignment has been chosen as reference, giving in this way a more global evaluation of the displacement problem, without focusing on a particular portion of a structure. This aspect is even more important considering that the Operational TBPL should give an indication concerning all the considered structure, without focusing on the singular wall.

Looking Table 14, it is evident that the maximum interstorey displacements for the Operational TBPL do not exceed the value of 13 mm: this displacement must be compared with each specific facility that a Hospital can contain, in order to evaluate if the admissible relative displacements of each technology are compatible with the obtained values.

These displacements have been also transformed in interstorey drifts, quantities which are more suitable for engineering considerations and which give information about the possible damage for a family of non-structural elements (for example glazing, partition walls, etc...). The values shown in Table 14 do not exceed the value of 2.8‰, with most of the cases which do not reach a drift of 1.7‰ (18 on 20 structural units).

With these values it is possible to make a first comparison with the indications of the actual Italian Code (D.M. 2008 and C.M. 617/2009): the Operational Limit State is defined as the one which shows a maximum interstorey drift equal to 2‰, so it is possible to observe that most of the buildings show smaller values of drift. It is important to observe that there is not a general trend of the values of drift along the vulnerability classification (Figure 81): once again, this aspect is not so directly related to the Life Safety TBPL since the ductility features as well as the distribution of the resistant system play a relevant role and they cannot allow a direct estimation of the maximum drift starting from the ultimate capacity of the structure (PGA_c).

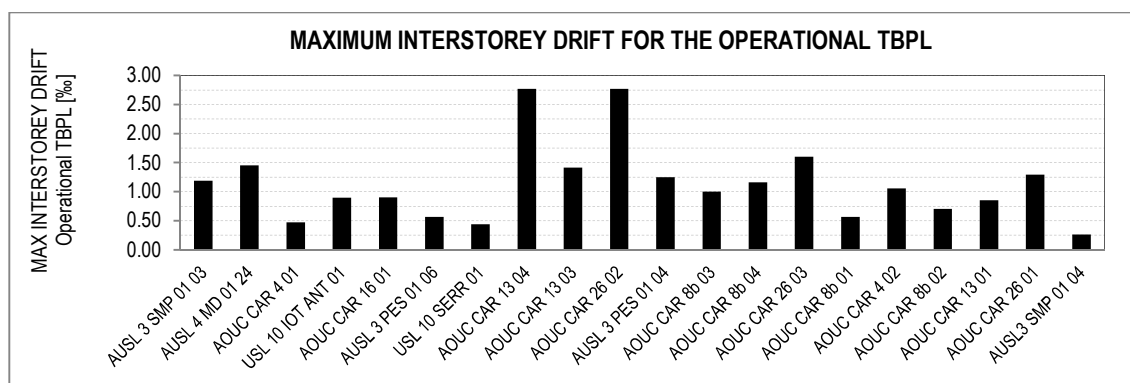


Figure 81: Maximum interstorey drift for the Operational TBPL along the Vulnerability classification.

4.9.3 Conclusions on the Operational TBPL

Resuming the aspects analyzed in this paragraph, it has been observed that the Operational TBPL can be evaluated both from the acceleration and the deformation point of view:

- for the acceleration approach, a generic relation (deriving from the calculation of the factor q^*) has been found, which shows the minimum value of the capacity for this limit state (expressed in terms of PGA_c) starting from the capacity of the ultimate limit state (Life safety TBPL). This estimation is not useful for a fast assessment of the Operational TBPL since it is based on a too simplistic assumption and it cannot give reliable information;
- for the deformation approach instead, no relation has been found, since the calculation of the drift is not directly related to the acceleration of capacity of the Life Safety TBPL.

A general overview of these aspects has been in any case obtained.

Generally speaking, the information obtained from the pushover analyses is not properly indicated for the analysis of the non-structural problems, since a detailed investigation on each singular non-structural system is required, in order to evaluate the level of connection of the non-structural element to the structure, its maximum deformation limit, the acceleration that produces the interruption of its function, etc...

5 Critical analysis of the Vulnerability Index Method for masonry structures

In this chapter, a critical analysis of the Vulnerability Index Method for masonry structures is proposed. The analysis is performed considering the 118 masonry buildings Vulnerability Forms of the Hospital structures sample (Florence, Prato and Pistoia).

5.1 Introduction: weights of each parameter in the vulnerability estimation

The Vulnerability Index Method is based, as explained in detail in § 2.4.3, on the evaluation of 11 parameters through qualitative judgments (A, B, C or D), each of them associated to numerical scores; every parameter has a weight which measures its relative importance. By means of a weighted sum, the I_V is calculated.

How much does each parameter influence the I_V ? Considering the 11 parameters, it is possible to calculate the percentages of importance of each parameter for a building characterized by the worst conditions (all D judgments); the results are listed in Table 15:

PARAMETER		SCORES				WEIGHT	WEIGHTED SCORES	% OF INFLUENCE
		A	B	C	D			
PAR. 1	Type and organization of the resistant system	0	5	20	45	1.00	45.00	11.8%
PAR. 2	Quality of the resistant system	0	5	25	45	0.25	11.25	2.9%
PAR. 3	Conventional resistance	0	5	25	45	1.50	67.50	17.6%
PAR. 4	Position of the building and foundations	0	5	25	45	0.75	33.75	8.8%
PAR. 5	Typology of floors	0	5	15	45	1.00	45.00	11.8%
PAR. 6	Planimetric configuration	0	5	25	45	0.50	22.50	5.9%
PAR. 7	Elevation configuration	0	5	25	45	1.00	45.00	11.8%
PAR. 8	Maximum distance among the walls	0	5	25	45	0.25	11.25	2.9%
PAR. 9	Roof	0	15	25	45	1.00	45.00	11.8%
PAR. 10	Non structural elements	0	0	25	45	0.25	11.25	2.9%
PAR. 11	State of conservation	0	5	25	45	1.00	45.00	11.8%
						8.50	382.50	100.0%

Table 15: Maximum influence of each parameter in the vulnerability estimation through the Vulnerability Index Method.

The parameter 3 (conventional resistance) is the most influent, with a percentage of almost 18%, then there are 5 parameters characterized by a percentage of importance of about 12% (which are all directly related to structural features, except the parameter 11, which can even be related to non structural aspects).

It is interesting to observe that the parameter 2 (quality of the resistant system) has a percentage of importance only of 3% on the global estimation.

Looking now to the 6 parameters which have been considered in the previous chapter for the comparison among the Vulnerability Index Method and the detailed analyses (pushover), it is possible to calculate again the percentages of influence of each of the 6 parameters on the global behaviour, considering the sum of their maximum weighted scores equal to 100%:

PARAMETER		SCORES				WEIGHT	WEIGHTED SCORES	% OF INFLUENCE
		A	B	C	D			
PAR. 2	Quality of the resistant system	0	5	25	45	0.25	11.25	4.8%
PAR. 3	Conventional resistance	0	5	25	45	1.50	67.50	28.6%
PAR. 5	Typology of floors	0	5	15	45	1.00	45.00	19.0%
PAR. 6	Planimetric configuration	0	5	25	45	0.50	22.50	9.5%
PAR. 7	Elevation configuration	0	5	25	45	1.00	45.00	19.0%
PAR. 9	Roof	0	15	25	45	1.00	45.00	19.0%
						5.25	236.25	100.0%

Table 16: Influence of each parameter in the estimation of the vulnerability related to the global behaviour.

In this case, the parameter 3 has a percentage of influence equal to 29%, while the parameter 2 remains the least important one. It is important to highlight some other aspects:

- the planimetric configuration in this approach is less important than the elevation one;
- the typologies of floors have the same importance of the typology of roof, independently from the total number of levels of the considered structure.

All these aspects will be considered in the next chapter, in order to develop a new expeditious vulnerability assessment method.

In order to better understand the importance of each parameter and the relations among them, it has been decided to analyze the data collected during the Survey Campaign performed by the DICEA Research Group on the Hospital structures of the cities of Florence, Prato and Pistoia: in particular, considering all the 118 masonry S.U. of the sample, it has been possible to analyze:

$$n.^\circ \text{ scores} = 118 \text{ structural units} \cdot 11 \text{ parameters} = 1298 \text{ scores}$$

The analyses performed on these data and proposed in this chapter allow a better comprehension of some aspects, related from one side to the general characteristics of the sample of masonry Hospital Structures of the cities of Florence Prato and Pistoia and, from the other side, directly to the general hypotheses and features of the Vulnerability Index Method.

5.2 General distributions of the judgments along the Vulnerability classification

As described in the initial part of the thesis, the survey procedure on all the buildings has allowed the creation of a Vulnerability classification by means of the Vulnerability Index (11 parameters), calculated for each of them: in particular, considering both masonry and reinforced concrete structures (219 structural units in total), the Index of Vulnerability ranges within the interval [81.4%; 5.0%], with a quite uniform distribution on the whole range. Considering only the masonry structures, the interval does not changes at all: in this case, the Index of Vulnerability ranges within the interval [81.4%; 13.7%], as shown in the Figure 82 below.

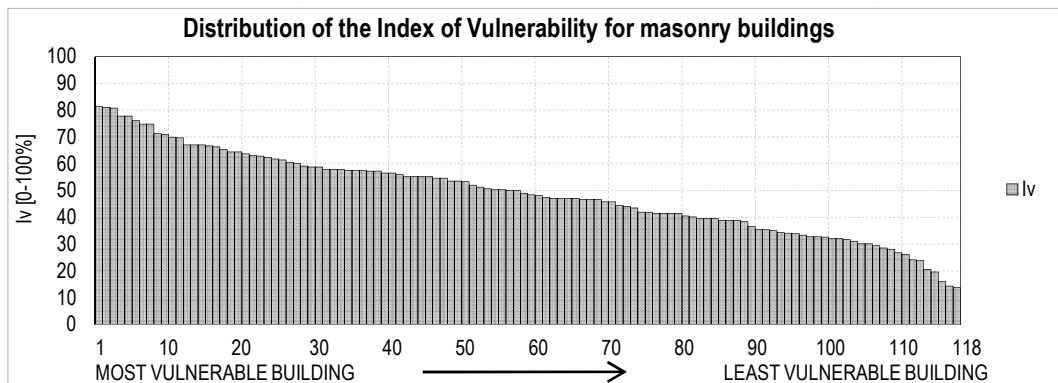


Figure 82: Distribution of the Index of Vulnerability for masonry structures - 118 buildings are analyzed in this graph.

Considering the global number of judgments of all the 118 Vulnerability Forms, it has been possible to observe the distribution of the judgments on the entire sample of structures:

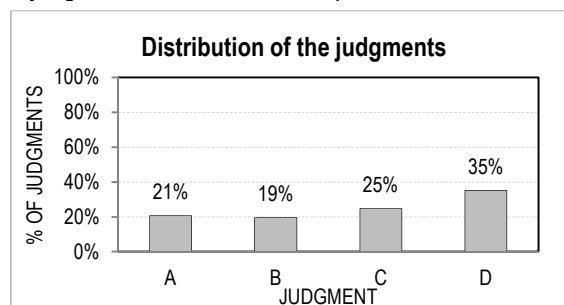


Figure 83: Distribution of all the judgments assigned in the 118 Vulnerability Forms: the statistic is based on 1298 judgments.

The histogram shows that the 4 typologies of judgment are almost equally distributed, except the worst one ("D" judgment) which shows a higher utilization comparing to the other ones.

The next step of the statistical analysis of the data is to observe how the decreasing trend of the Index of Vulnerability (shown in Figure 82) is related to the judgments assigned to each S.U.; some information can be obtained just looking how each building has been evaluated, simply counting the numbers of the different types of judgment, without considering (for the moment) the parameters which these judgments have been assigned to: in other words, how many "A" judgments has each building? And how many "B", "C" or "D"?

In the following graphs, for each structural unit (horizontal axis), the number of the different typologies of judgment is identified: for example, the most vulnerable building (n.º1, on the left side) has 3 "C" and 8 "D" judgments, while the least vulnerable one (n.º 118, on the right side) has 3 "A" judgments, 7 "B" and 1 "D".

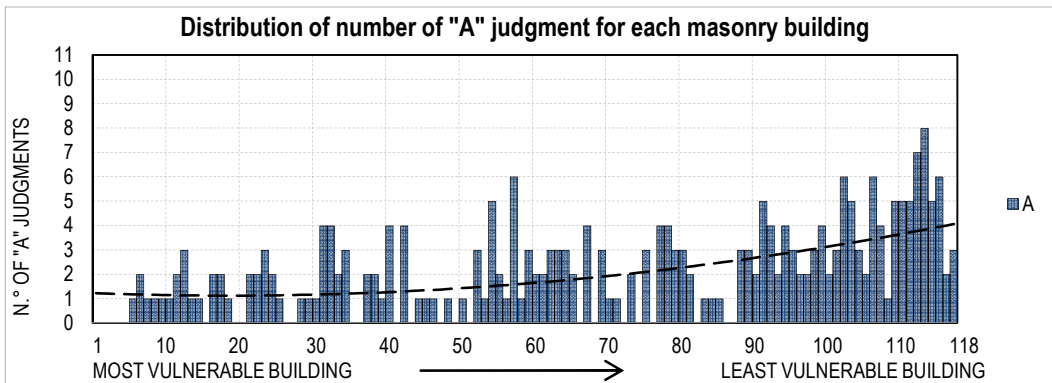


Figure 84: Distribution of the "A" judgments.

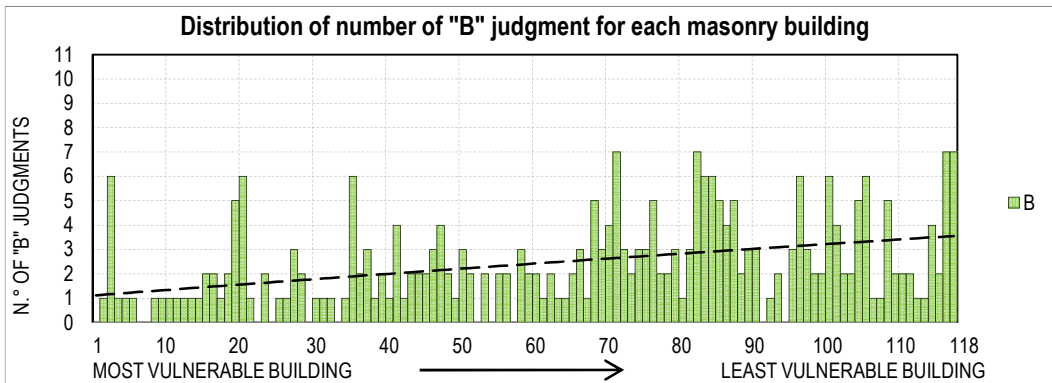


Figure 85: Distribution of the "B" judgments.

As expected, the number of A and B judgments increases as far as the level of vulnerability decreases (moving from the left to right side of the graphs); the trend is analogous for the two typologies of judgment.

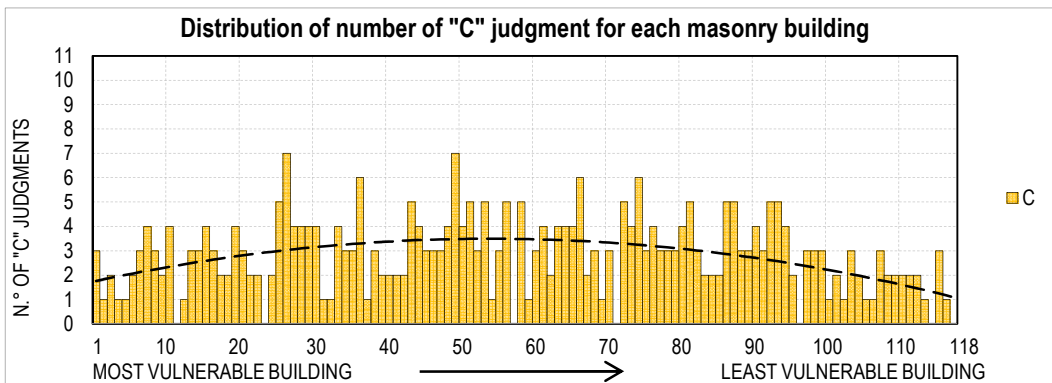


Figure 86: Distribution of the "C" judgments.

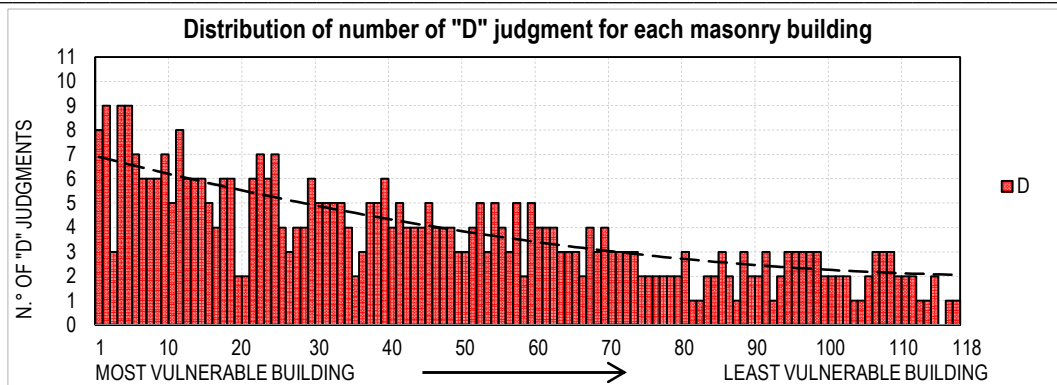


Figure 87: Distribution of the "D" judgments.

The "C" judgment has a variable trend along the vulnerability classification; the "D" one instead has a clearly visible decreasing trend. It is interesting to observe that even in high vulnerable structures, some "A" and "B" judgments are present as well as for low vulnerable structures, "C" and "D" judgments are registered. This happens because the vulnerability aspects are not all related each other, as it will be explained in the following of this chapter: it is possible, for example, to find a well realized structure with some very vulnerable non structural elements, or, on the other way around, high vulnerable buildings can be characterized by the absence of dangerous non structural elements. Analogous evaluations can be proposed for the other features which compose the 11 parameters Vulnerability Form.

5.3 Analysis of the distributions of the judgments for each parameter

Until now, the evaluations on the distributions of the judgments have been performed without considering which parameter the judgment was assigned to: this preliminary analysis has been done just to give an overview of the vulnerability assessment method.

The next steps will analyze in detail each singular parameter. In the next Figure 88, the distribution of the typologies of judgment for each parameter has been investigated, in order to see if the general trend of the judgments distribution (observed for the entire sample - see Figure 83) remains constant for each parameter.

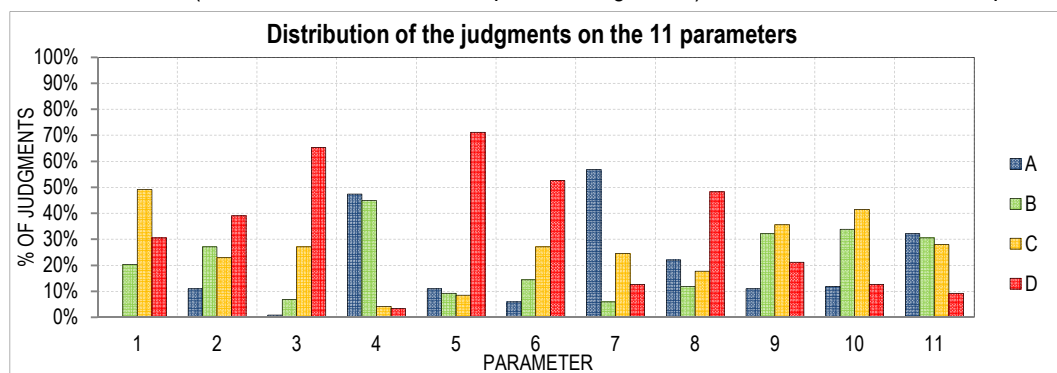


Figure 88: Distribution of the judgments on the 11 parameters. For each parameter, the sum of the percentages gives 100%.

From the previous graph it is easy to observe that the distributions for the different parameters show dissimilar trends: in particular, some aspects can be pointed out for each parameter, as described in the following list:

- PARAMETER 1 - type and organization of the resistant system

There are no buildings which show the best conditions: this is easily understandable simply looking the definition of the "A" judgment written in the Handbook for the compilation of the Vulnerability Form (GNDDT, 1993), where it is explicitly stated that, in this category, only buildings realized in agreement with the current seismic codes for new constructions are allowed. Since the analysis of

the sample of buildings has been focused on buildings realized before 1984, this aspect cannot be fulfilled. More detailed information can be found in the updated Handbook proposed by the Tuscany Region (Regione Toscana, 2003), where there is the reference to the first code to consider, issued in 1986 (D.M. 1986). Moreover, in the detailed description of the "A" typology, there are even some indications about the presence of rigid floors at each level, concrete beams at the intersections among floors and walls, maximum distance of the walls, etc...

Looking the other 3 typologies of judgment, it is possible to observe that 65% of the structures belongs to the medium categories ("B" and "C"), with prevalence of the "C" judgment (50%). Even in this case, it is useful to refer to the definition of the Handbook of the Vulnerability Form (GNDT, 1993) for the "C" category: *"buildings that, even if they do not have concrete beams or metal chains on each level, they are made of orthogonal walls which are well connected each other."* It means that even this category of buildings can guarantee a certain level of box behaviour (depending on the conditions of other aspects such as the floors' connection, the presence of unbalanced horizontal forces caused by pushing roofs, etc...), since the connections among the walls must be ensured.

The histogram shows a percentage of 35% of buildings with the worst judgment ("D"), category of buildings which are made of not well connected walls; this characteristic can lead to local mechanisms so particular attention should be given to this aspect during the detailed analysis phase.

- PARAMETER 2 - quality of the resistant system

This parameter shows a more uniform distribution of the judgments, with a relevant percentage of building in the worst judgment (40.7% in "D"). The first class shows a smaller percentage comparing to the other ones (12.7% in "A") due to its definition, which is composed of buildings made of good quality brick masonry or squared stones, with homogeneity all over the structure. Even in this case, it is easy to understand that these characteristics are not so common to find in the built environment realized before 1984: for example, it is common to find stone masonry buildings with some internal walls realized with brick masonry.

- PARAMETER 3 - conventional resistance

This parameter, based on a simplified calculation which keeps into account the resistant area of the walls against horizontal forces and the shear resistance of the materials, shows that most of the analyzed structures are in a bad condition (92.3% in the last two classes with 66.1% in the last one). Considering the definition of the classes explained in the beginning of the present work, it is possible to better understand this result: in particular, as described in (see § 2.4.3), the 4 classes are distinguished by the value of $\alpha = C/\hat{C}$, where C is the ratio among the ultimate shear resistance of the considered level and the total weight above it, while \hat{C} is a reference value for the resistance of the buildings, assumed as 0.35 in the survey campaign performed by the DICEA-UNIFI Research Group. Once that the α values for each classes are known, it is possible to calculate the correspondent values of C which identify these classes.

A judgment	$\alpha \geq 1$	$C \geq 0.35$
B judgment	$0.6 \leq \alpha < 1$	$0.21 \leq C < 0.35$
C judgment	$0.4 \leq \alpha < 0.6$	$0.14 \leq C < 0.21$
D judgment	$\alpha < 0.4$	$C < 0.14$

The values of C can also be considered as the maximum acceleration (expressed in gravity units) of an equivalent single degree of freedom for the considered structure: with this point of view, it is possible to understand the reason why most of the structures have "C" and "D" judgments, since a capacity acceleration higher than 0.21g is not easy to find on existing buildings. Even if this

calculation has a different theoretical approach comparing to the one of the detailed analyses performed in the previous chapter, it is possible to see that only few structures have a PGA_c higher than 0.21g; it is anyway important to remark that there is not a perfect correspondence among these two capacity estimation methods, since the pushover analyses are performed with non-linear analyses on three-dimensional models, which describe in detail the mass and stiffness organization of the considered structure, while the calculation of the parameter 3 is simply an estimation of the quantity of the resistant system.

In conclusion, the ranges of variation of α proposed by the Vulnerability Index Method are too wide for the considered sample of buildings, so most of the structures fall into the worst category "D".

- PARAMETER 4 - position of the building and foundations

This parameter mainly considers three aspects: natural slope of the ground where the building is located, consistency of the type of soil, presence of the system of foundations; almost all the buildings (94.07%) are classified in the first two categories, highlighting that there are not explicit dangerous situations from the geotechnical point of view. It is even clear that, during an expeditious survey campaign, it is difficult to find detailed information about this parameter, so only explicit vulnerability aspects can be observed.

- PARAMETER 5 - typology of floors

The in-plane stiffness of the floors and their connections with the walls are here considered.

During the surveys, it has been possible to observe (in a direct way, by visual inspection, or in an indirect way, by the observation of the details and by the utilization of the thermal image camera) the typologies of floors: in older buildings flexible floors have been found, such as steel beams and hollow tiles, masonry barrel or cross vaults, while in more recent ones, reinforced concrete with hollow tiles floors have been found. This last typology has been widely employed even in cases of structural interventions on the buildings.

In most of the cases, the connection among floors and walls have been identified as not so efficient (considering the seismic prevention aspects): the reason of this problem is quite evident, since all the buildings have been realized before 1984, period when the attention to the seismic design was not so organized and detailed like nowadays.

The definitions of the 4 classes highlight that it is more important to have a good level of connection among floors and walls instead of having an highly rigid floor which is not well anchored on its perimeter: the first 2 classes ("A" and "B") are identified by rigid and well connected floors (in the "B" group it is even possible to have staggered floors), while in the "C" class, flexible floors are allowed only if they have a good level of connection with the walls; in the last class ("D"), all the typologies of floors, which have a low degree of connection with the vertical structural elements, are allowed. The result of the survey on all the buildings now appears clearer since the connections among floors and walls (from the seismic point of view) represent a weak point for many buildings.

- PARAMETER 6 - planimetric configuration

The result for the 6th parameter shows that the planimetric configuration is usually not so regular; this aspect is partially connected to the age of construction of the buildings: the attention to the seismic behaviour of the structures has been developed in a detailed way only in the recent decades, so it is possible to find old buildings with not regular shapes.

Another aspect to consider is the aggregation of structural units; the planimetric configuration of each S.U. is a function of the subdivision phase of the aggregate: this process, even if performed considering the structural aspects (discontinuities of materials, geometry, stiffness, etc...) cannot be always univocal, since different subdivisions can be sometimes proposed for the same aggregate.

It is important to remind that this parameter, and in general the Vulnerability Index Method, gives information about the intrinsic vulnerability of the singular S.U., without taking into account the effects of interaction among contiguous units, since they are considered as structurally independent. In any case, the pounding effect of adjacent buildings can bring high levels of damage in case of an earthquake: this aspect must be considered when buildings in aggregate are analyzed in detail.

- PARAMETER 7 - elevation configuration

The results for this parameter show that most of the buildings have a more or less regular configuration along their height (63.56% of the S.U. are in the best class); however, a third of the sample has the worst judgments ("C" and "D"), because of the presence of porticos at the ground level or added levels on the top of the buildings, frequently realized with different materials and shapes compared to the ones of the other levels of the structure.

- PARAMETER 8 - maximum distance among the walls

This parameter gives information about the presence of long portions of walls without restraints offered by orthogonal walls: long distances of "free" walls can lead to local mechanisms problems.

The results show that a considerable part of the structures (62.71%) belongs to the worst categories ("C" and "D"), highlighting once again that the attention to the seismic behaviour of the structures was not fully considered during the design and realization of the Hospital buildings.

- PARAMETER 9 - roof

This parameter shows a quite homogeneous distribution of the judgments; in this parameter, the connection of the roof to the walls is evaluated as well as the presence of unbalanced horizontal forces ("pushing roof").

- PARAMETER 10 - non structural elements

This parameter is referred to the general definition of the non-structural elements and it does not take into account all the complex aspects connected to a Hospital system. Generally speaking, the problems related to the non-structural elements have been highlighted, showing the presence of some high vulnerability cases (15% of the structures are in the worst category "D").

- PARAMETER 11 - state of conservation

This result shows that the buildings have, on average, at least an acceptable level of conservation (only the 9.32% of the structural units are in the worst category "D").

All these evaluations have been performed without taking into account the distribution of the judgments along the vulnerability classification: how the judgment trends of each parameter change along the vulnerability classification? In the next step, the trend of each judgment along the vulnerability ranking will be analyzed.

5.4 Distribution of the judgments for each parameter along the vulnerability classification

A more detailed analysis can be performed observing the trend of the judgments of each parameter along the vulnerability classification, starting from the most vulnerable building until the least vulnerable one: which are the parameters that show a trend in agreement with the Vulnerability Index? In the following graphs, the judgments of each S.U. are shown separately for each parameter, using the order given by the vulnerability classification. On the vertical axis there are the four possible judgments, while on the horizontal one the 118 buildings are ordered with the I_v ranking.

PARAMETER 1 - type and organization of the resistant system

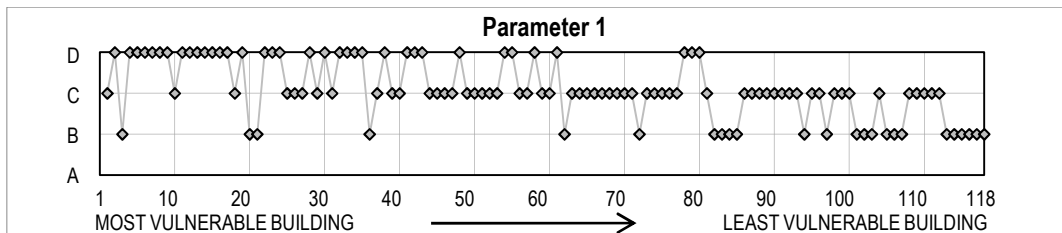


Figure 89: Distribution of the judgments in the Parameter 1 along the Vulnerability classification.

Figure 89 shows a general decreasing trend of the judgments; starting from the most vulnerable buildings (left side of the graph), the distribution is well divided in three different parts: the prevalent "D" judgment part (from the beginning until the 25th building), the prevalent "C" judgment part (until the 80th building) and the mixed "C and B" part, (until the end of the list of structures). The "A" judgment, as explained before, has never been assigned since it requires the complete fulfilment of the indications of the seismic codes after 1986, but all the buildings of the sample have been realized before 1984. The trend of this parameter shows a good correspondence with the one of the I_v .

PARAMETER 2 - quality of the resistant system

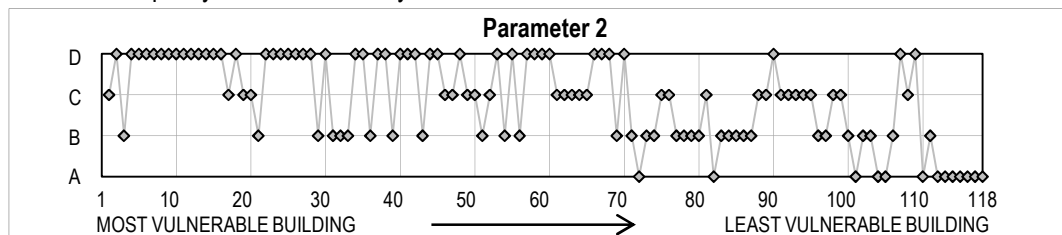


Figure 90: Distribution of the judgments in the Parameter 2 along the Vulnerability classification.

The distribution of the judgments is in general agreement with the trend of the I_v . However, it is important to highlight that even buildings with low values of I_v can be characterized by poor quality materials: this because other parameters influence the vulnerability estimation (structural configuration, typologies of floors, etc...).

PARAMETER 3 - conventional resistance

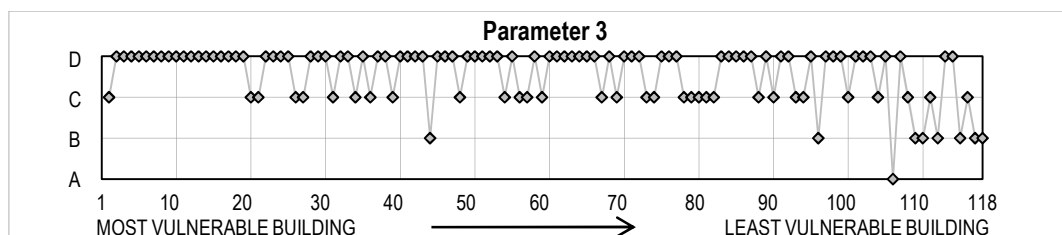


Figure 91: Distribution of the judgments in the Parameter 3 along the Vulnerability classification.

As described before, most of the buildings are in the worst category due to the limit values which identify the boundaries of each class of judgment. Anyway, the trend of this parameter is partially in agreement with the I_v trend: the best judgments ("A" and "B") start to appear in the least vulnerable buildings part of the classification as far as the most vulnerable buildings part is mostly characterized by "D" judgments.

PARAMETER 4 - position of the building and foundations

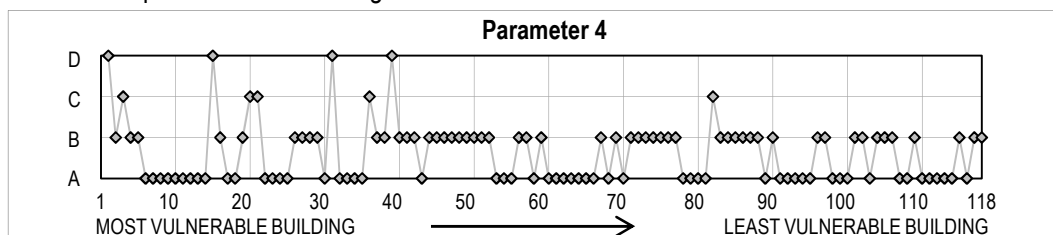


Figure 92: Distribution of the judgments in the Parameter 4 along the Vulnerability classification.

In this case, it is important to remind that almost all the buildings (94.07%) are classified in the first two categories ("A" and "B") as seen in the previous pages (Figure 88).

This graph is not able to give some other information since there is no visible trend in the distribution.

PARAMETER 5 - typology of floors

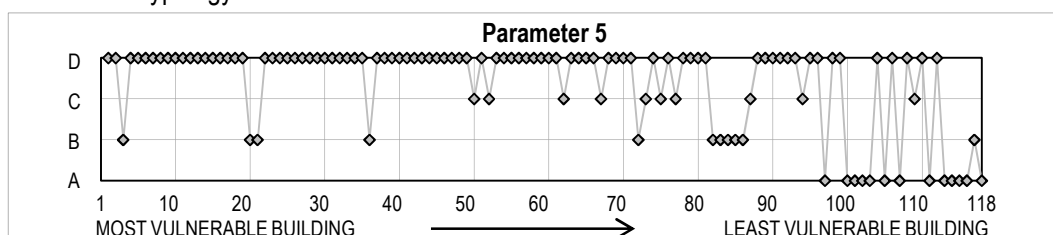


Figure 93: Distribution of the judgments in the Parameter 5 along the Vulnerability classification.

As described before, most of the buildings are in the worst category mainly due to the connections of the floors to the walls and to the low level of in-plane stiffness (75.42% in "D" judgment); even if there is not a uniform distribution of judgments, it is possible to observe that the least vulnerable buildings have floors with better characteristics, arriving up to the "A" judgment in some cases (13.56%).

This graph is in general agreement with the trend of the I_v .

PARAMETER 6 - planimetric configuration

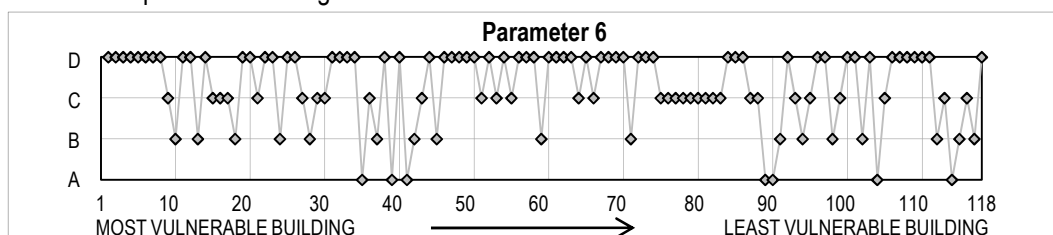


Figure 94: Distribution of the judgments in the Parameter 6 along the Vulnerability classification.

The graph shows that there is not a clear trend of the distribution of the judgments: the planimetric configuration seems to be not so correlated to the I_v trend. This aspect can partially depend on the definition of the structural units, as already explained in § 5.3: the subdivision phase of an aggregate in structural units, based on the observation of the structural aspects of the buildings (materials, construction methods, etc...), cannot be considered as always univocal, since different subdivisions can be sometimes proposed, due to the particular configuration of the considered building.

PARAMETER 7 - elevation configuration

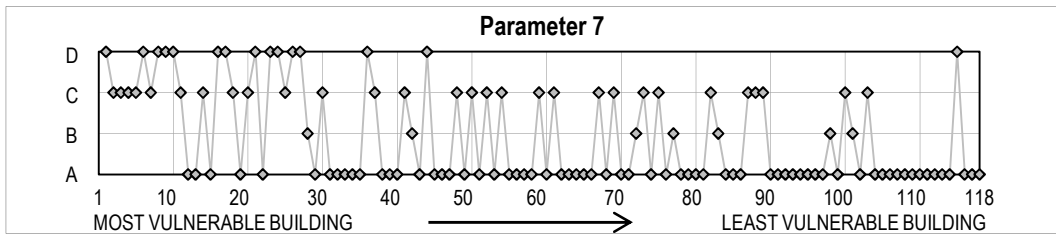


Figure 95: Distribution of the judgments in the Parameter 7 along the Vulnerability classification.

For this parameter, it is important to remind that most of the structural units show a regular elevation shape along their height (63.56% of the buildings in "A" judgment); the graph highlights that the first 30 vulnerable buildings show mainly bad conditions ("C" and "D" judgments), while the remaining part of the sample has, on average, a better judgment. The parameter shows then a quite good accordance with the trend of the I_v .

PARAMETER 8 - maximum distance among the walls

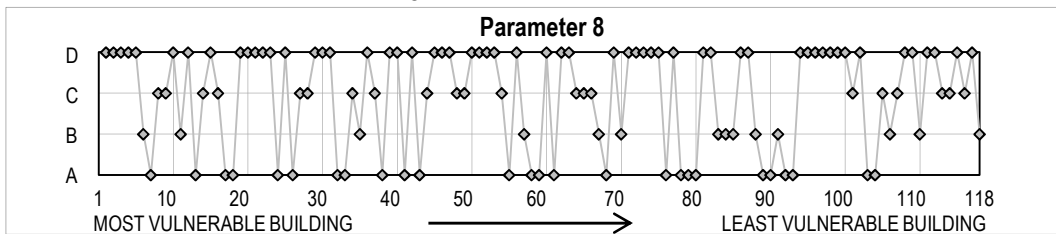


Figure 96: Distribution of the judgments in the Parameter 8 along the Vulnerability classification.

This parameter shows a random distribution of the judgments along the vulnerability classification: this is reasonable since it is mainly connected to local mechanisms problems. Even low vulnerable buildings (characterized by rigid floors, high quality materials, regular shape, etc...) can have at least one or few local situations where a wall alignment has no restraints (orthogonal walls) for a considerable length.

PARAMETER 9 - roof, PARAMETER 10 - non structural elements, PARAMETER 11 - state of conservation

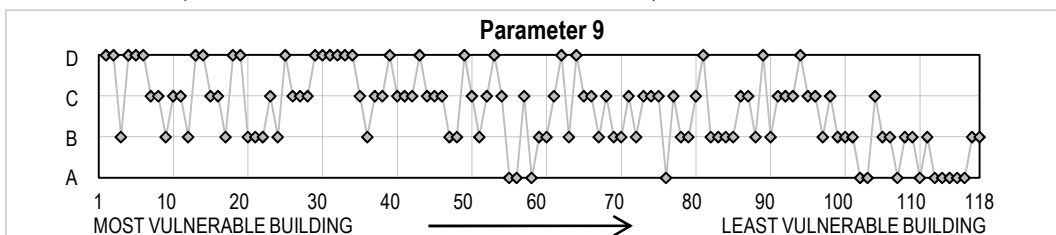


Figure 97: Distribution of the judgments in the Parameter 9 along the Vulnerability classification.

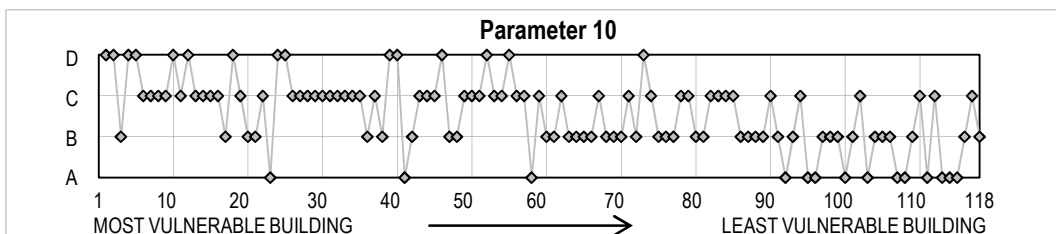


Figure 98: Distribution of the judgments in the Parameter 10 along the Vulnerability classification.

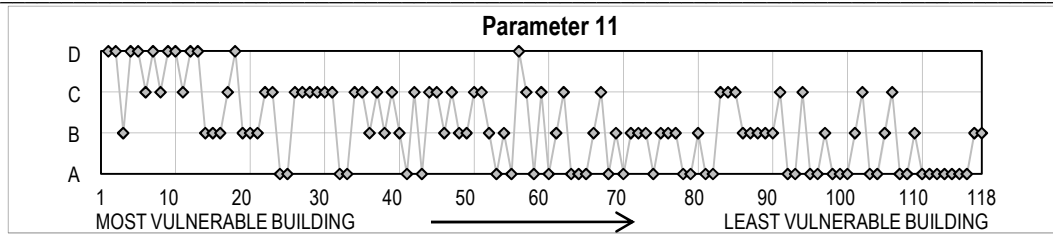


Figure 99: Distribution of the judgments in the Parameter 11 along the Vulnerability classification.

The last three parameters show a general decreasing trend, which is in agreement with the one of the I_v , even if the variability of judgments is relevant.

Resuming all the analyses performed in this paragraph, 8 of the 11 parameters show a general accordance to the trend of the I_v . This means that each of them gives its own contribution in a somehow "coherent" way.

For the other 3 parameters, there is not a clear trend of judgment along the vulnerability ranking; in particular:

- parameter 4 gives only the information that there are not explicit geotechnical problems on average;
- parameter 6 shows that the planimetric shape of the buildings seems not related at all with the I_v ;
- parameter 8 confirms that local mechanisms problems can interest even low vulnerable buildings.

These 3 parameters give their contribution without following the general decreasing trend of the I_v .

5.5 Relation among the parameters

The previous paragraph has described the evolution of the judgments of the parameters along the vulnerability classification of the expeditious method, considering them in an independent way. But how much each parameter is related to the other ones? In other words, are they completely independent?

The relation among the parameters can be discussed from the qualitative point of view simply using the knowledge about the vulnerability argument, but some quantitative information can be obtained even analyzing the collected data during the survey campaign, as described hereafter.

From the qualitative point of view, just to give an example, it is possible to assess directly that the parameter 10 (non structural elements) is not related at all with the parameter 4 (position of the building and foundations), since no reasonable motivations can be found to explain this relation; on the other hand, the parameter 3 (conventional resistance) should be connected with the parameter 2 (quality of the resistant system), since the first one depends on the shear resistance of the masonry.

Some more results and conclusions can be obtained by means of a statistical treatment of the collected data: as written above, for each of the 11 parameters, 118 observations are available (one for each S.U.).

Generally speaking, when the analysis of the relation among "n" parameters is required, it is possible to investigate the distribution of the observations in an "n" dimensions space, plotting the cloud of points; in case of $n > 2$ (as in this case), for a better comprehension of the problem, it is possible to use the technique of the projections of the cloud on a certain number of surfaces, where the axes which identify each surface are 2 of the "n" parameters. The result of each projection is a cloud of point too, where each point corresponds to an observation (a S.U.) which has the values of the 2 considered parameters as coordinates: this cloud of points can have a certain trend or shape and its statistical treatment can highlight a more or less effective level of correlation among the 2 considered parameters, while a random distribution of the points can confirm the independence of them.

In the case of the judgments of the Vulnerability Index Method, the variables (each parameter) cannot assume continuous values, but they are ordinal data: this typology of data is composed of observations which can be counted and ordered referring to a particular scale (in this case, the possible judgments of the method, "A",

"B", "C" and "D", where "A" is the best judgment and "D" the worst), but it is not possible to make direct measurements. In other words, the "distance" among "A" and "B" judgment for example may be not the same as the one among "B" and "C"; it is only possible to assess that the "C" judgment is worse than "B" and "B" is worse than "A". Moreover, the normal procedure of projection using the cloud of points makes no sense in this case, because the points of the cloud can occupy only 16 fixed positions (for each of the 4 possible judgments of the 1st considered parameter, there are 4 possible judgments of the 2nd parameter): since the sample of analysis is composed of 118 observations (each S.U.), a certain number of observations lies in the same position, giving a not correct representation of the distribution of the points (they hide each other).

A possible graphical solution to study these relations is here proposed: each graph shows a grid with the 16 possible positions of each observation and, for each of the 16 possibilities, a bubble which represents the frequency (by the measure of its area) of occurrence of the considered combination is plotted; these bubbles report the percentage value referred to the entire number of observations (in this case, 118 for each graph).

In other words, the "bubble graph" represents the matrix where the frequencies of the possible combinations of judgment are measured.

Each parameter can be analyzed with each one of the other 10. The possible graphs are:

$$n.^\circ \text{ possible graphs} = 10 \text{ graphs/parameter} \cdot 11 \text{ parameters} = 110 \text{ graphs}$$

The number of graphs decreases considering that, for a couple of parameter x-y, the graphs "x - y" and "y - x" are simply inverted and the information about the correlation is exactly the same. For this reason:

$$n.^\circ \text{ graphs} = 110/2 = 55 \text{ graphs}$$

Moreover, since the final goal of this part of the work is to better understand which are the relations among the couples of parameters, it is not important to perform the projections for all of them, but it is necessary to choose the appropriate couples which can give some useful information; coming back to the concepts explained in the beginning of the present paragraph, the knowledge about the vulnerability argument will suggest the couples of parameters to investigate.

Before the analysis of the graphs, an index of correlation for each couple of parameters has been calculated:

$$C(X, Y) = \frac{\sum(x - \bar{x}) \cdot (y - \bar{y})}{\sqrt{\sum(x - \bar{x})^2 \cdot \sum(y - \bar{y})^2}}$$

where: X and Y matrixes of the two considered variables (in this case, vectors with dimension 1x118);
 x and y values of each singular observation for the two considered variables;
 \bar{x} and \bar{y} average values of each variable.

This coefficient allows an estimation of the level of correlation among the two considered parameters; in order to apply the formula, numerical values were required and so an arbitrary analytical translation of the judgments has been adopted ("A"=4, "B"=3, "C"=2 and "D"=1).

The index of correlation can assume all the values within the [-1; +1] interval, where +1 means a perfect direct correlation and -1 a perfect inverse correlation, while values near 0 mean no correlation.

The following matrix is symmetric for the reason explained above: only the upper triangle values are then reported for a better comprehension.

	TYPE AND ORGANIZ. OF THE RESISTANT SYSTEM	QUALITY OF THE RESISTANT SYSTEM	CONVENTIONAL RESISTANCE	POSITION OF THE BUILDING AND FOUNDATIONS	TPOLOGY OF FLOORS	PLANIMETRIC CONFIGURATION	ELEVATION CONFIGURATION	MAX. DISTANCE AMONG THE WALLS	ROOF	NON STRUCTURAL ELEMENTS	STATE OF CONSERVATION
PARAMETER	1	2	3	4	5	6	7	8	9	10	11
1		0.566	0.259	-0.277	0.717	0.037	0.145	-0.268	0.400	0.285	0.175
2			0.343	-0.184	0.574	0.055	0.249	-0.126	0.349	0.270	0.355
3				-0.225	0.090	-0.023	0.006	-0.041	0.238	0.091	0.110
4					-0.145	-0.009	0.144	0.336	0.035	0.187	0.255
5						-0.047	0.008	-0.197	0.494	0.370	0.193
6							0.025	0.148	0.002	-0.012	0.005
7								0.027	0.098	0.141	0.311
8									0.060	0.065	0.109
9										0.387	0.276
10											0.410
11											

Table 17: Matrix of correlation: indexes of correlation for each couple of parameters.

From the observation of the matrix, it is possible to see that:

- the highest value of direct correlation is 0.717 for the couple of parameters 1-5;
- the minimum value, which corresponds to the maximum level of inverse correlation, is -0.277 for the couple of parameters 1-4.

The first result appears reasonable, since the parameter 1 (type and organization of the resistant system) evaluates the level of connections among walls and among walls and floors, while parameter 5 (typology of floors) evaluates the stiffness and the level of connection of the floors with the walls: these two parameters are strictly related.

The second result instead, which highlights a low level of inverse relation among the parameter 1 (type and organization of the resistant system) and parameter 4 (position of the buildings and foundations), reasonably not related at all, ensures that there are no strong inverse relations among the parameters: this aspect is reasonable too, since all the parameters are used within the Vulnerability Index Method to estimate the level of vulnerability by means of a direct sum, so each of them should give a coherent contribution or, at least, a random contribution without any inverse relation with all the other parameters.

Looking the other values of correlation proposed in Table 17 it is evident that, since the possibilities of judgment for each parameter are only 4 ("A", "B", "C" or "D"), it is possible to obtain not negligible values of correlation which have not a specific physical reason; see for example the relation among the parameters 5 (typology of floors) and parameter 10 (non structural elements): the coefficient of correlation is equal to 0.370, but no reasonable motivations can justify this number.

Considering this aspect, only values of correlation higher than 0.50 have been assumed as reliable: this decision has been taken observing the data of Table 17 and using the knowledge about the vulnerability problems in parallel, in order to understand the meaning of the indexes.

Resuming the results obtained from Table 17, the couples of parameters which show a significant degree of correlation are (the following list is ordered starting from the highest level of correlation):

- parameters 1-5: $C(1,5) = 0.717$. This correlation has been justified already in the previous page;
- parameters 2-5: $C(2,5) = 0.574$. This correlation among the quality of the resistant system (parameter 2) and the typology of floors (parameter 5) can be justified with the evolution of the construction technologies: in older buildings, such as not organized masonry walls, the floors were

often realized with masonry vaults, steel beams and hollow tiles, wooden floors, etc... which have all a low level of stiffness in their plane and a not effective connection with the walls; more recent buildings instead, realized for example with brick masonry, are often characterized by the presence of concrete beams and hollow tiles floors, which are usually rigid and connected to the walls by means of r.c. beams; this aspect ensures a higher efficacy of connection under a seismic event;

- parameters 1-2: $C(1,2) = 0.566$. The reason of this correlation is analogous to the previous one: the use of better materials of construction is usually related to a better construction method for the considered masonry structures.

Even if the coefficient of correlation is inferior to the assumed boundary value, it is important to highlight that, following the correlation index ranking, the couple of parameters which come after the last one just described (parameters 1-2) is the couple of parameters 5-9, with $C(5,9) = 0.494$, which concerns the typologies of floors and roof respectively; even this relation is reasonable, since the structural typology of roof is related to the techniques of realization of the floors: in case of rigid and well connected floors, it is difficult to find a pushing roof which has no stiffness in its plane.

In order to confirm what explained above, some of the bubble graphs introduced before will be proposed in the following pages, trying to highlight the differences among the couples of related parameters from the not related ones. In particular, some of the possible graphs which highlight a certain degree of correlation will be proposed at first, describing the common features.

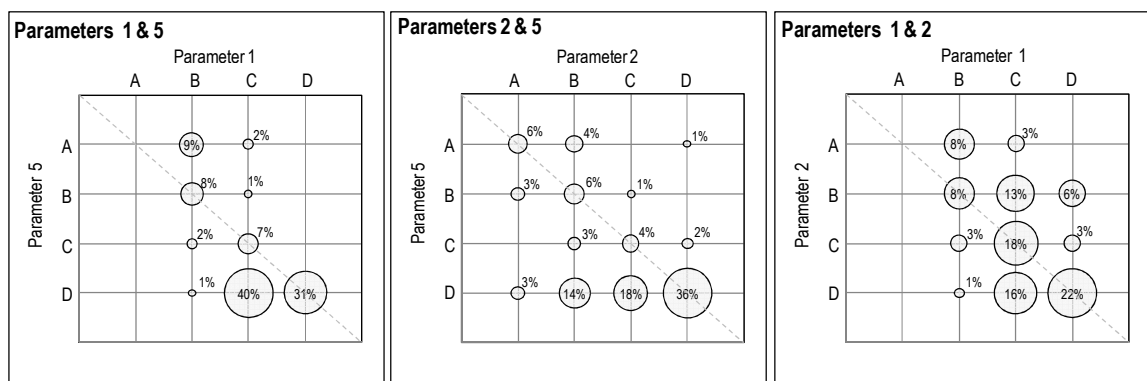


Figure 100: Bubble graphs for high correlation index couples of parameters.

The bubble graphs proposed in Figure 100 show different shapes due to the distributions of the judgments already analyzed in Figure 88, but they all have some common features:

- sum of the percentages along the trace (dashed line in the graphs) of the matrix relatively high (almost 50% of the observations). This sum represents the fraction of S.U. for which the perfect correspondence of judgment has been found;
- sum of the percentages of the bubbles in the places "A-D" and "D-A" relatively low (at least equal to 0). This sum represents the fraction of S.U. which has opposite evaluations in the considered parameters and it is a measure of the possible inverse relation among them;
- most of the percentages lie in the tri-diagonal area of the matrix. This aspect gives some indications about the level of coherence of the judgments among two parameters: the tri-diagonal part contains the S.U. for which only one "position" of difference among the judgments has been used for the evaluation of the two parameters (for example x parameter "B", y parameter "C"). In any case, since the available possibilities are only 4, this characteristic cannot be assumed as a direct correlation

indicator, since it can be even possible to have such kind of feature without any particular relation: this aspect is then subordinated to the preliminary evaluation of the other aspects already mentioned.

In the following Figure 101, some bubble graphs for not related couples of parameters are proposed: considering the aspects listed above, it is easy to understand the difference among this family of graphs and the one proposed above.

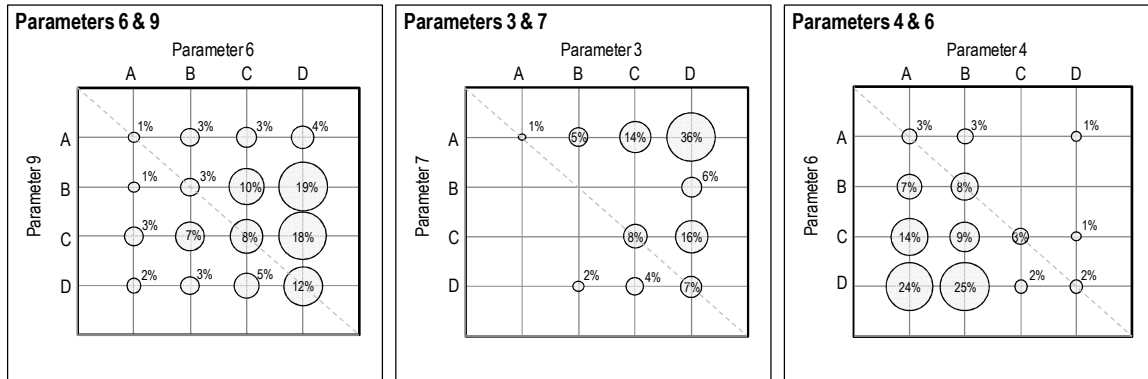


Figure 101: Bubble graphs for low correlation index couples of parameters.

In the previous Figure 101, the following relations have been analyzed:

- parameter 6 (planimetric configuration) and parameter 9 (typology of roof);
- parameter 3 (conventional resistance) and parameter 7 (elevation configuration);
- parameter 4 (position of the building and foundations) and parameter 6 (planimetric configuration).

Even without particular detailed descriptions, it is possible to understand that the parameters of each couple are independent each other. These three proposed graphs have been chosen observing the values of correlation in Table 17 which are the closest ones to 0, highlighting the complete independence among the parameters.

It is important to observe another graph, due to its importance in the study of the relation among the couple of parameters: parameter 2 (quality of the resistant system) with parameter 3 (conventional resistance).

These two parameters should be strongly related since the shear resistance of the materials is a necessary value in order to estimate the conventional resistance, as described in § 2.4.3; the results of the correlation analysis proposed in Table 17 show that $C(2,3) = 0.343$, which highlights a not so strong correlation among the parameters. The bubble graph is here in the following proposed:

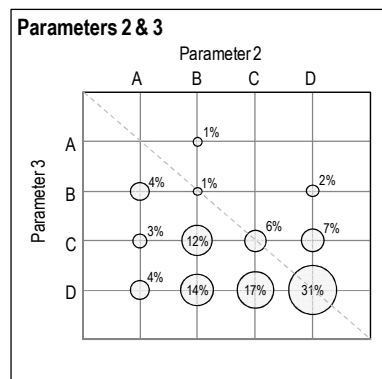


Figure 102: Bubble graph for the couple of parameters 2-3.

Even the graph confirms the not relevant level of correlation (a significant percentage of structural units is outside the tri-diagonal area of the matrix); these results, which can seem not reasonable, find their justification in the following reasons:

- the conventional resistance is based both on the evaluation of the resistant area of masonry and on the use of the shear resistance through the formula of Turnsek and Cacovic, already seen in § 2.4.3; the influence of the first element (the resistant area of masonry) plays an important role and it influences the relative importance of the second one (the shear resistance) on the final result (the conventional resistance);
- the method for the estimation of the conventional resistance, as already explained in § 5.3, has highlighted that the ranges which identify the 4 classes of judgment, proposed by the Vulnerability Index Method, are too wide for the considered sample of buildings, so most of the structures fall into the worst category ("D"); this aspect can be the main reason of the particular shape of the bubble graph, where a lower triangle matrix is displayed.

Resuming the concepts observed in this chapter, it is possible to conclude that:

- the conventional resistance is the most important parameter in the definition of the vulnerability using the Vulnerability Index Method (see Table 15 and Table 16);
- the analysis of the distribution of the judgments for each parameter allows a better comprehension of the general characteristics of the sample of buildings;
- the observation of the trend of the judgments along the vulnerability classification has highlighted that most of the parameters give their contribution in the definition of the vulnerability in a coherent way, except the parameters 4, 6 and 8;
- the correlation analysis among each possible couple of parameters has highlighted that most of the parameters are independent from the other ones; only few couples show a level of relation that can be justified by reasonable aspects, but which cannot allow the definition of "dependent parameters";
- the judgments of the parameter 3 (conventional resistance) seem to be not so well representative of the seismic behaviour of the structure for the considered sample of buildings, because of the particular definition of the classes of judgments.

All these aspects will be considered in the next chapter, in order to propose an alternative method of expeditious vulnerability estimation, using the same information collected for the Vulnerability Index Method.

6 Development of a new expeditious Vulnerability assessment method

In this chapter, a proposal for a new expeditious vulnerability assessment method is described; in particular, this proposal is based on the Vulnerability Index Method and it requires the same information of that method, but it allows a better comprehension of the vulnerability aspects concerning the investigated building and it gives the possibility of a first estimation of the Index of Risk, aspect that was not possible in the original method.

6.1 Introduction

The vulnerability assessment of masonry structures performed through the Vulnerability Index Method (Benedetti and Petrini, 1984) evaluates the Index of Vulnerability, I_V , as already explained in § 2.4.3: this index allows the realization of a vulnerability classification, giving a useful information about which buildings must be analyzed in a detailed way at first.

It has been explained that the I_V keeps into account structural and non-structural aspects and, moreover, it considers both global and local behaviour features through different parameters.

It has been also demonstrated (see § 4.7.1) that the I_V is correlated to the evaluation of the global structural capacity performed with detailed analyses (static non-linear analyses on global models); considering only the parameters referred to the global behaviour, which are directly described in the analytical global models (see § 4.7.2), the level of correlation increases, as seen in § 4.7.3.

Considering the results obtained on the sample of analysis of the present work (Hospital structures of the cities of Florence, Prato and Pistoia), some critical points of the method have been highlighted, as the definition of the conventional resistance, which can be improved in order to better differentiate the behaviour of different structures: as already seen in § 5.3, the conventional resistance of the considered sample of buildings is not well distinguished among different structural units due to the definition of the boundary values of each judgment ("A", "B", "C" or "D"). This aspect becomes even more important if we consider that the parameter 3 (conventional resistance) is the most important one in the definition of the vulnerability through the Vulnerability Index Method (see § 5.1).

Considering all these aspects, it is possible to affirm that the Vulnerability Index Method is a reliable indicator of the seismic vulnerability of a structure; all the performed analyses have highlighted advantages and disadvantages of this method, which will be used in this chapter to propose an improvement of the method.

The idea of this part of the work is to maintain the structure of the method, using the same data in order to obtain more information on the seismic aspects of the investigated building.

In particular, the first aspect is to obtain more information about the structural capacity of the structure, considering the following list:

- estimate the global capacity of the structure expressed in terms of PGA_C , using from one side the parameters referred to the global behaviour and, on the other side, the results of the detailed static non-linear analyses;
- knowing the capacity of the structure and the seismic hazard of the site of construction of the considered building, it is then possible to estimate an index of seismic risk;
- quantify the reliability of this estimation, considering that masonry buildings can show local mechanisms problems which could influence in a substantial way the capacity of the considered structure: in presence of not well organized resistant systems (no connection among orthogonal walls, flexible and not well connected floors and roofs to the walls, long parts of masonry without any constraints, etc...), the global analysis cannot be considered as always reliable since the capacity can be related to the acceleration of activation of the local mechanisms.

In the next paragraph, the descriptions of the parameters for the new expeditious method are proposed.

6.2 Definition of the new parameters for the global capacity estimation

6.2.1 Introduction

In this part of the work, the parameters which give information about the global behaviour of a masonry structure will be considered.

As explained in § 4.7.2 and resumed in Table 10, the parameters of the Vulnerability Index Method which are directly considered in the detailed approach are:

- PARAMETER 2 - quality of the resistant system;
- PARAMETER 3 - conventional resistance;
- PARAMETER 5 - typology of floors;
- PARAMETER 6 - planimetric configuration;
- PARAMETER 7 - elevation configuration;
- PARAMETER 9 - roof.

All of them, in the original method, are considered in a qualitative way except the parameter 3, which requires instead the estimation of the conventional resistance: this numerical evaluation is, in any case, reported to a qualitative one by means of 4 different intervals, which identify the judgment to assign to the parameter.

6.2.2 From parameters 2 and 3 to a new continuous variable: the "lateral resistance indicator"

Since it has been observed that the intervals of the conventional resistance proposed by the original Vulnerability Index Method are too wide for the considered sample of buildings (such that most of the structures fall into the worst category), it has been decided to investigate the direct relation among the variables which identify the conventional resistance and the detailed results of the analytical procedure.

In particular, looking the equation of the maximum shear resistance, which represents the basis of the calculation of parameter 3 (see § 2.4.3), it is possible to see that the conventional resistance is a function of:

- the resistant area of the masonry walls in the considered direction (considering the reference storey which identifies the capacity of the structure), expressed through the unit-less variable $\alpha_0 = A/A_t$;
- the shear resistance which characterize the typology of masonry, τ_k .

In order to analyze the possible correlation among these two parameters and the results of the detailed analyses, a preliminary aspect has been investigated: the direction of collapse found with the two methods (Vulnerability Index Method and pushover analyses) always corresponds for each of the 20 analyzed S.U.; this means that the evaluation of the conventional resistance is reliable in order to individuate the weakest direction of a structure.

Coming back to the relation, it has been decided to evaluate the variable $\alpha_0 \cdot \tau_k$ since it takes into account both the aspects listed above: the "quality" and the "quantity" of the resistant system; this variable has been called "lateral resistance indicator". This evaluation has been performed in the significant level of the structure which can be considered, reasonably, the one where the collapse due to horizontal forces can happen: usually, for regular buildings, this storey is the first one out of the ground but, in case of non-regularity in elevation of the structure, it can be chosen another level.

In the following Table 18, the calculation of the lateral resistance indicator $\alpha_0 \cdot \tau_k$ is explained for all the S.U. of the sample of analysis; in particular, the following elements are listed:

- the quantities of resistant masonry for the level of verification in the two main directions A and B , where A is the minimum one among the two directions; the direction of A is indicated too;
- the global surface of the considered level A_t ;
- the identification of the material and the value of the shear resistance τ_k , evaluated in the LC1 level.

Then, in the last two columns, the lateral resistance indicator and the PGA_C (detailed analyses) are compared. The list is ordered considering the PGA_C values (from the lowest value to the highest one).

n.°	STRUCTURAL UNIT	A [m ²]	dir. A (X/Y)	B [m ²]	A _t [m ²]	a ₀ = A/A _t	τ _k [N/cm ²]	description of the material (D.M. 2008)	a ₀ · τ _k [N/cm ²]	PGA _C [g]
1	AUSL 3 SMP 01 03	36	Y	58	720	0.050	1.48	disorganized stone masonry	0.075	0.0973
2	AUSL 4 MD 01 24	26	Y	37	295	0.087	1.48	disorganized stone masonry	0.129	0.1149
3	AOUC CAR 4 01	21	Y	59	1150	0.018	4.15	well organized stone masonry	0.075	0.1216
4	USL 10 IOT ANT 01	44	Y	73	870	0.050	1.48	disorganized stone masonry	0.075	0.1295
5	AOUC CAR 16 01	40	Y	83	1400	0.029	4.44	brick masonry	0.127	0.1440
6	AUSL 3 PES 01 06	73	X	88	1380	0.053	1.48	disorganized stone masonry	0.079	0.1484
7	USL 10 SERR 01	271	X	297	2185	0.124	1.48	disorganized stone masonry	0.184	0.1640
8	AOUC CAR 13 04	10	X	22	378	0.027	4.44	brick masonry	0.122	0.1720
9	AOUC CAR 13 03	18	Y	33	420	0.042	4.56	well organized stone masonry with brick layers	0.190	0.1770
10	AOUC CAR 26 02	34	X	52	675	0.050	4.56	well organized stone masonry with brick layers	0.227	0.1950
11	AUSL 3 PES 01 04	24	Y	29	910	0.027	8.33	brick masonry (good quality mortar)	0.222	0.2030
12	AOUC CAR 8b 03	92	X	97	1740	0.053	4.56	well organized stone masonry with brick layers	0.242	0.2030
13	AOUC CAR 8b 04	49	Y	62	1085	0.045	4.56	well organized stone masonry with brick layers	0.205	0.2078
14	AOUC CAR 26 03	39	X	52	675	0.057	4.56	well organized stone masonry with brick layers	0.262	0.2090
15	AOUC CAR 8b 01	47	X	64	775	0.061	4.56	well organized stone masonry with brick layers	0.279	0.2100
16	AOUC CAR 4 02	24	Y	24	455	0.052	4.56	well organized stone masonry with brick layers	0.237	0.2112
17	AOUC CAR 8b 02	48	Y	49	705	0.068	4.56	well organized stone masonry with brick layers	0.310	0.2195
18	AOUC CAR 13 01	72	X	73	1180	0.061	4.56	well organized stone masonry with brick layers	0.278	0.2234
19	AOUC CAR 26 01	36	Y	46	550	0.065	4.56	well organized stone masonry with brick layers	0.298	0.2263
20	AUSL3 SMP 01 04	14	Y	26	300	0.045	7.41	"half hollow" brick masonry	0.333	0.2264

Table 18: Calculation of the "lateral resistance indicator" a₀ · τ_k.

Plotting the last two columns in a graph, where the horizontal axis expresses the lateral resistant indicator a₀ · τ_k and the vertical one the PGA_C of the pushover analyses, it is possible to see that there is a certain level of correlation, which can be approximated by a linear trend, as shown in Figure 103.

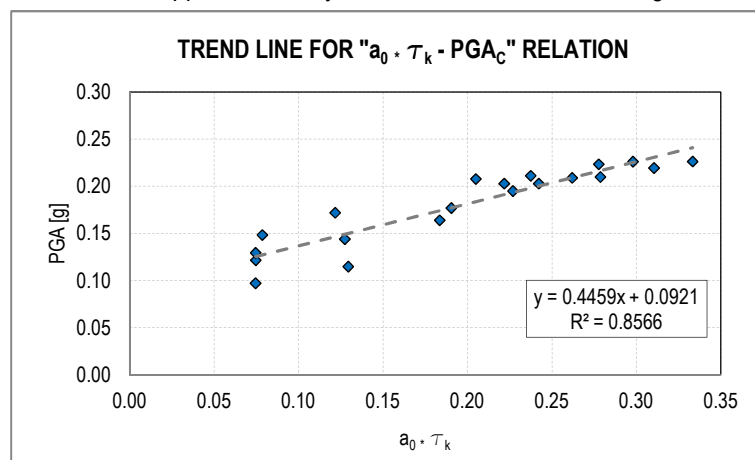


Figure 103: Relation among the lateral resistance indicator and the PGA_C.

It is reasonable that the capacity of the structure is directly proportional to the lateral resistance indicator: the use of good quality materials and the presence of a consistent quantity of resistant elements (referring to the area of the building) generally improve the behaviour and resistance of a structure and, consequently, its capacity against seismic actions.

It is also clear that this parameter is not enough to fully describe the capacity of a structure, since many other aspects must be considered, such as the regularity of the structure, the typologies of floors and roofs, etc...

In any case, this variable (lateral resistance indicator) seems to be more suitable for the vulnerability estimation than the parameters 2 (quality of the resistant system) and 3 (conventional resistance). It has the advantage to be in a direct relation with the detailed analyses results (as shown in Figure 103) and, moreover, it is a continuous variable, which considers the exact amount of masonry piers of the resistant system; this

aspect will ensure a more precise evaluation of the final results, since a continuous variable is generally more useful in the interpretation of another continuous variable (the PGA_c) than a qualitative one (judgments of the Vulnerability Index Method).

It is important to highlight that this new continuous variable does not require more detailed evaluations than the ones which are required in the Vulnerability Index Method form, but it simply uses the same data in a different way.

6.2.3 Parameters 5 and 9: study of their relation

These two parameters will remain qualitative even in the new proposal of the expeditious vulnerability assessment. Without considering, for the moment, the problem related to the definition of the scores related to each judgment and the weights of each parameter (this argument will be studied in the following paragraphs), some general considerations about the importance of the parameters is here described, in order to propose improvements to the organization of the new vulnerability form.

In the original Vulnerability Index Method, the parameter 5 (typology of floors) and 9 (roof) have different scores associated to each of the possible judgments ("A", "B", "C" or "D") and, moreover, they have a different weight due to some particular aspects, as already well described in § 2.4.3; in the worst conditions, as already seen in § 5.1 (see Table 15), the parameters assume the same importance in the evaluation of the vulnerability, due to the maximum score (45 points) and the maximum weight (1.00) of each of them.

Is this assumption always reasonable? Should the two parameters influence in the same way the I_V ?

In this paragraph, a proposal of improvement of these parameters is described.

Considering two different buildings, characterized for example by 2 and 5 levels and by the same typologies of floors and roof, it is reasonable to think that the roof cannot play the same role in the two vulnerability evaluations, since in the 2 levels structure the roof represents the 50% of the horizontal diaphragms, while in the other building (5 levels) only the 20%.

This feature is not considered in the Vulnerability Index Method and, in order to improve the original expeditious method, the calculation of a new dependent parameter is proposed, starting from the parameters 5 and 9: the idea consists in the evaluation of a new total score for the parameters 5 and 9, based on the sum of the original scores weighted considering the number of floors of the structure.

$$score(PAR. 5 + PAR. 9) = \frac{[(n.^\circ \text{ floors} - 1) \cdot score PAR. 5 + (1) \cdot score PAR. 9]}{n.^\circ \text{ floors}}$$

In this way, the relative importance among roof and floors is taken into account.

6.2.4 Parameters 6 and 7

These two parameters, as parameters 5 and 9, will be considered in the new proposal of the vulnerability assessment maintaining the same qualitative features, as already provided in the original Vulnerability Index Method. As written above in the beginning of § 6.2.3, the aspects related to the scores of each judgment and the weights of each parameter will be analyzed in the following paragraphs even for these two parameters.

6.3 **New expeditious vulnerability assessment method: estimation of the global PGA_c**

6.3.1 Introduction

In the previous paragraph § 6.2, the parameters to be considered for the estimation of the global capacity of the masonry structures have been described: in particular, 6 parameters must be considered, 4 of them through a qualitative evaluation (parameters 5, 6, 7 and 9) and the other 2 with a quantitative evaluation (parameters 2 and 3 for the calculation of the "lateral resistance indicator" $a_0 \cdot \tau_k$).

The problems of the definition of the vulnerability estimation procedure are:

- the choice of the scores to assign to each judgment ("A", "B", "C", "D") for the qualitative parameters;
- the choice of the weights to assign to all the considered parameters, in order to take into account the relations among them and their importance in the vulnerability estimation;
- the analytical procedure for the vulnerability estimation.

The total number of variables is high and the number of "observations" (20 structural units analyzed with detailed procedures), which can allow the research of the solution, is not consistent enough in order to solve the problem in a closed form. Some assumptions will be then considered, in order to obtain a reliable estimation procedure: in the following, the description of them is reported.

6.3.2 Choice of the scores for each judgment

Considering the problem related to the scores to assign to each judgment ("A", "B", "C", "D") for the qualitative parameters, it is important at first to look back to the original method (Vulnerability Index Method), which shows different scores for different parameters (Table 1): this decision has been made by the authors of the method considering the effects of past earthquakes on the buildings, in order to give a vulnerability evaluation which was coherent with the observed damages.

Since this work is not based on the evaluation of vulnerability through the direct observation of the damages of a seismic event, the easiest arbitrary assumption has been chosen, in order to not influence the final result with specific hypotheses: in particular, it has been assumed that each class of judgment has the same score in all the considered parameters and, moreover, the "distance" among each class has been assumed as constant. The following scores have been associated to each judgment:

A=4 B=3 C=2 D=1

This assumption has been made considering that, once that these values are used, the relative importance of each parameter in the vulnerability estimation will be consequently estimated by the procedure described in the following paragraphs, calculating the most correct weight for each parameter.

The scores of the original methods have not been used in the new proposal since those values have been calibrated on the 11 parameters evaluation, while in this part of the work only 6 of them are considered and, moreover, with a different approach; the original scores then have no particular reasons to be chosen as reference values.

6.3.3 Choice of the weights of each parameter and of the analytical procedure

The weights of the parameters and the analytical procedure for the estimation of the vulnerability are two aspects strictly related: the weights depend on how the data will be numerically treated.

In this work, it has been decided to adopt the multi-linear regression (MLR) for the estimation of the PGA_C , combining the effects of all the considered parameters in a linear way. In particular, each of the parameters described in § 6.2 has been considered as independent variable: looking back to the § 5.5, it has been observed that all the possible couples of parameters can be considered as mainly independent; the cases which have been analyzed in detail (where the correlation coefficient is higher than 0.50) show that, however, there is not a clear dependent relation among the considered couple of parameters and so the complete independence has been assumed among all of them.

The multi-linear regression (MLR) is based on the identification of a certain number of independent parameters (x_i variables) and on the correspondent dependent parameter (y variable), which can be expressed as a linear combination of the independent parameters:

$$y = m_1 \cdot x_1 + m_2 \cdot x_2 + \dots + m_i \cdot x_i + b \quad \text{with}$$

m_i represent the weights of each independent variable

b is a constant value

The evaluation of the m_i and b is performed with the least squares technique.

For this particular case, the variables to consider are four:

- lateral resistant indicator $a_0 \cdot \tau_k$; (x_1)
- PARAMETER 5+9 - typology of floors and roof; (x_2)
- PARAMETER 6 - planimetric configuration; (x_3)
- PARAMETER 7 - elevation configuration. (x_4)

It is important to remind that the first variable, the lateral resistant indicator, represents the parameters 2 and 3 of the original method combined together and it is a quantitative variable (while all the other ones are qualitative ones); the parameter 5+9 instead is a qualitative variable, which represents two separated variables (considered as independent each other), combined together in order to take into account the influence of the number of levels. The capacity of the structures, expressed through the PGA_c obtained with the detailed analyses, represents the dependent variable (y).

In the following Table 19, the identification of the variables is proposed; the estimation of the PGA_c through the Multi Linear Regression is also shown, using the weights that the procedure has given as result.

The S.U. are listed following the vulnerability evaluation (y) order, performed with the detailed approach.

n.°	STRUCTURAL UNIT	x_1	x_2	x_3	x_4	y	$y'(MLR)$		errors	
		$a_0 \cdot \tau_k$ [N/cm ²]	par. 5+9 [1-4]	par. 6 [1-4]	par.7 [1-4]	PGAc [g]	estimation of PGAc [g]	over estimation error [%]	under estimation error [%]	
1	AUSL 3 SMP 01 03	0.075	1.25	2	1	0.097	0.112	15%		
2	AUSL 4 MD 01 24	0.129	1.00	2	2	0.115	0.131	14%		
3	AOUC CAR 4 01	0.075	2.00	1	2	0.122	0.126	3%		
4	USL 10 IOT ANT 01	0.075	1.33	3	2	0.130	0.123		-5%	
5	AOUC CAR 16 01	0.127	2.00	1	2	0.144	0.143		0%	
6	AUSL 3 PES 01 06	0.079	1.50	3	3	0.148	0.132		-11%	
7	USL 10 SERR 01	0.184	1.33	3	3	0.164	0.165	0%		
8	AOUC CAR 13 04	0.122	2.00	2	2	0.172	0.146		-15%	
9	AOUC CAR 13 03	0.190	2.00	2	2	0.177	0.169		-5%	
10	AOUC CAR 26 02	0.227	3.00	1	4	0.195	0.204	5%		
11	AUSL 3 PES 01 04	0.222	3.00	1	3	0.203	0.197		-3%	
12	AOUC CAR 8b 03	0.242	3.00	1	2	0.203	0.199		-2%	
13	AOUC CAR 8b 04	0.205	3.00	1	2	0.208	0.186		-10%	
14	AOUC CAR 26 03	0.262	3.00	1	4	0.209	0.216	3%		
15	AOUC CAR 8b 01	0.279	3.00	2	1	0.210	0.210		0%	
16	AOUC CAR 4 02	0.237	2.00	1	2	0.211	0.180		-15%	
17	AOUC CAR 8b 02	0.310	3.00	2	1	0.220	0.221	1%		
18	AOUC CAR 13 01	0.278	2.00	2	3	0.223	0.203		-9%	
19	AOUC CAR 26 01	0.298	3.00	2	3	0.226	0.227	0%		
20	AUSL3 SMP 01 04	0.333	3.00	2	2	0.226	0.234	3%		
								15%	-15%	
		0.3364	0.0168	0.0041	0.0053	0.0524				
		m_1	m_2	m_3	m_4	b				

WEIGHTS

Table 19: Multi-linear regression for the estimation of the PGA_c .

The errors in the estimation process have been divided in over-estimation ones and under-estimation ones: as shown in Table 19, the maximum error is 15% both for the over-estimation and the under-estimation.

How good is the regression described above? In order to have a quantitative idea of the efficacy of the analysis, it is possible to calculate an indicator of the quality: the determination coefficient. It is a coefficient which can range in the [0-1] interval: it compares the estimated values y' with the real ones y ; when the coefficient is close to 1.0, the correlation is evident and there is no difference among the estimation y' and the

real values y , while low values of it (close to 0.0) express the lack of correlation among the independent parameters and the result which has to be estimated.

Numerically speaking, the coefficient is calculated as follows:

$$r^2 = \frac{sq_{reg}}{sq_{tot}}$$

where

sq_{tot} is the sum of the squares of y_i

sq_{res} is the sum of the squares of $(y_i - y'_i)$

$sq_{reg} = sq_{tot} - sq_{res}$

For the analyzed case, $r^2 = 0.897$, showing then a good quality of the regression.

In order to visualize the obtained results, an histogram is proposed in Figure 104, comparing the analytical results of the detailed analyses with the estimation obtained through the multi-linear regression.

The reference values of this histogram are contained in Table 19: the colours of the graph identify the columns in the table which the graph refers to.

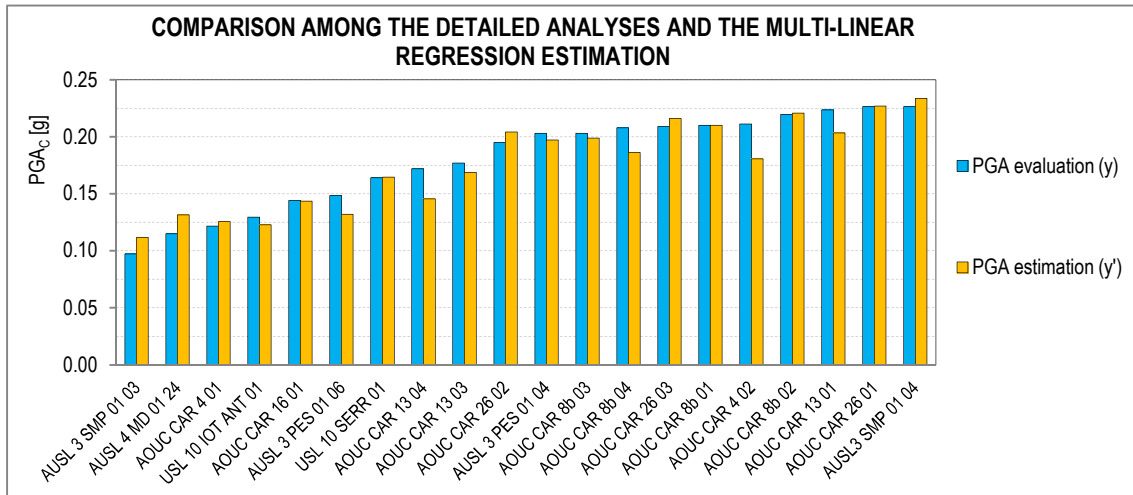


Figure 104: Comparison among the detailed analyses and the estimation performed with the multi-linear regression.

Considering the single contributions of each of the independent variables, it is possible to highlight the relative importance of each parameter in the global estimation: in the following Figure 105, for each S.U. the percentage contribution of each parameter in the estimation of the PGA_C is plotted.

It is possible to observe that:

- the constant parameter b has an influence in the range (24%-50%);
- the variable x_1 , lateral resistant indicator $\alpha_0 \cdot \tau_k$, has an influence in the range (20%-48%);
- the variable x_2 , typology of floors and roof, has an influence in the range (13%-27%);
- the variable x_3 , planimetric configuration, has an influence in the range (2%-10%);
- the variable x_4 , elevation configuration, has an influence in the range (2%-12%).

The percentages are calculated as ratios among the considered quantity and the estimation of the regression. From the data, it is possible to see how the lateral resistance indicator, together with the typologies of floors and roof, rules the estimation of the capacity of the structures.

The configuration of the structures (planimetric and elevation parameters considered together) is able to influence the estimation of capacity in the range (6%-21%): this percentage does not coincide with the sum of the ranges written above, since each singular variable can show its minimum contribution in a different S.U..

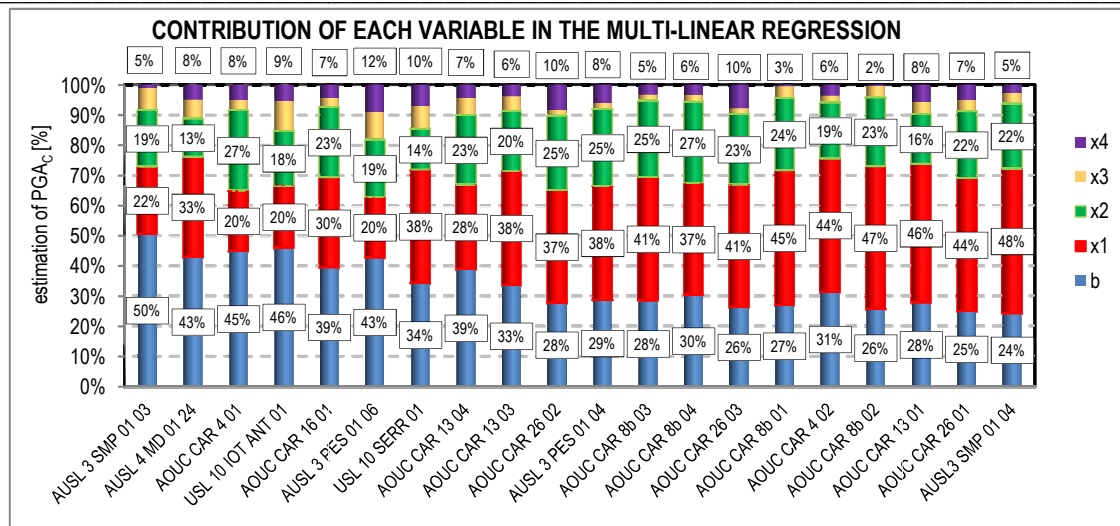


Figure 105: Contribution of each variable in the multi-linear regression.

Looking the results just proposed, it is clear to see that a direct comparison with the weights of the original Vulnerability Index Method cannot be performed, since this new approach has a different philosophy of calculation (estimation of the PGA_C instead of the I_V) and a different numerical approach: in this case, a constant value of capacity is always considered for each S.U. (the value of the constant b), while in the I_V approach the worst condition of evaluation can lead to an Index $I_V=100\%$.

Anyway, it is possible to observe that the "lateral resistant indicator $\alpha_0 \cdot \tau_k$ " has an influence, on average, of 36% on the final result: this parameter has replaced the parameters 2 and 3 of the original method, where the influence of those two parameters has been estimated in $29\%+5\%=34\%$ (see Table 16).

The other parameters have instead a smaller importance in this new approach, comparing to the one that they have in the original Vulnerability Index Method.

6.4 Test of the method: application on some theoretical case studies

In order to observe if the proposed method is suitable even for other typologies of structures, some theoretical case studies have been prepared using dimensions, geometries and structural organization which are coherent with the ones observed in the real analyzed structures. These case studies have been analyzed both with the detailed procedure (pushover analysis) and with the new vulnerability assessment method, in order to estimate the PGA_C : the results have been compared as described in this paragraph.

6.4.1 Three levels case study

The proposed case study has a rectangular shape (31.5m x10.5m), it is composed of three levels, all realized with well organized stone masonry. The internal distribution of the walls, as well as the one of the openings, shows a regular structural organization of the resistant system

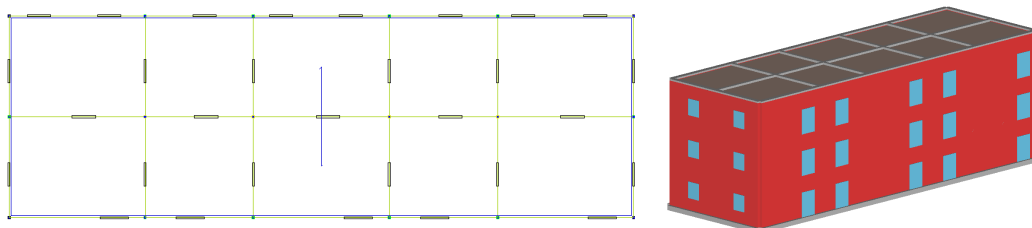


Figure 106: Plan and 3d view of the three levels case study.

The proposed building is characterized by a perfect regularity along its height; the planimetric shape instead, even if it shows regularity too, has a substantial difference among the two horizontal dimensions, leading to a different behaviour of the structure under horizontal forces: for these reasons, the considered building has the best judgment ("A") for the elevation configuration, while the planimetric elevation one is assumed as "C".

All the external masonries have a thickness of 45 cm, while the internal ones 30 cm.

In order to understand if the proposed method is suitable even for other buildings not belonging to the subset of analyzed structures, this case study has been analyzed in different configurations; in particular, in order to highlight the influence of the floors, two conditions have been analyzed:

- rigid floors;
- floors with finite stiffness.

Then, considering the second condition, the influence of the lateral resistance indicator has been analyzed, changing both the type of material of construction and the distribution of the resistant system:

- floors with finite stiffness - brick layers on masonry;
- floors with finite stiffness - wall thickness reduction.

In particular, the first condition has been obtained using the improving coefficients for the mechanical characteristics of the considered typology of masonry, provided in Table C8A.2.2 of C.M. 617/2009: for this case, the table suggests a coefficient 1.1 for the well organized stone masonry in case of brick layers.

The second condition instead has been obtained simply reducing the thickness of all the internal walls, from 30 cm to 20 cm. In all the analyzed cases, the loads on the floors have been maintained as constant.

In the following Table 20, the parameters necessary for the estimation of the PGA_C through the proposed Multi Linear Regression approach are listed, together with the value obtained through detailed analyses; the estimated value of PGA_C and its error are then proposed.

It is useful to remind the scores associated to each judgment: A=4, B=3, C=2 and D=1.

n.°	IDENTIFICATION OF THE MODEL	$a_0 \cdot \tau_k$ [N/cm ²]	par. 5+9 [1-4]	par. 6 [1-4]	par.7 [1-4]	PGA_C [g]	estimation of PGA_C [g]	over estimation error [%]	under estimation error [%]
1	rigid floors	0.187	3	2	4	0.172	0.195	13%	
2	finite stiffness floors	0.187	2	2	4	0.170	0.178	5%	
3	finite stiffness floors - brick layers	0.206	2	2	4	0.178	0.184	3%	
4	finite stiffness floors - walls thickness reduction	0.151	2	2	4	0.161	0.166	3%	

Table 20: Three levels case study: test of the method.

The 1st model is characterized by a "B" judgment for the floors and roofs (par. 5+9): the "A" judgment has not been assigned since in real existing buildings, the presence of non deformable floors, well connected along their entire perimeter and without any staggered levels is a quite difficult condition to find.

The other 3 models have instead a "C" judgment, since the "D" condition can be assigned only when not rigid floors without an effective connection with the walls are observed: this aspect can reasonably lead to local mechanisms problems, making the global capacity estimation a not reliable analysis. Since this work tries to give a contribution for the global behaviour analysis, it is reasonable to suppose a certain level of connection among walls and floors.

The 3rd model has the same judgments of the 2nd model, but a different lateral resistance indicator, due to the improving coefficient mentioned above.

The 4th model is analogous to the 2nd model, but it has the internal walls with lower thickness.

Observing the obtained results it is possible to see that the method of estimation is able to give a value of the PGA_C which has an error lower than the 15%, comparing to the values calculated in the detailed approach.

6.4.2 Four levels case study

In order to test the method on some other structures, the three levels model has been modified, realizing another level on the top of it, creating in this way another set of buildings.

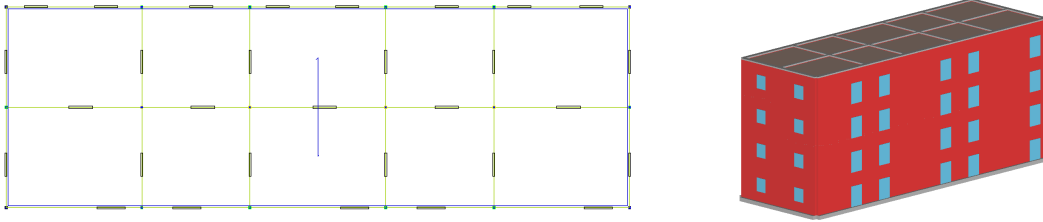


Figure 107: Plan and 3d view of the four levels case study.

With the same assumptions mentioned above, the results for this set of buildings are proposed in Table 21, considering four different cases by the modification of the type of floors, material and thicknesses of the walls.

n.°	IDENTIFICATION OF THE MODEL	$a_0 \cdot \tau_k$ [N/cm ²]	par. 5+9 [1-4]	par. 6 [1-4]	par.7 [1-4]	PGA _C [g]	estimation of PGA _C [g]	over estimation error [%]	under estimation error [%]
1	rigid floors	0.187	3	2	4	0.178	0.195	9%	
2	finite stiffness floors	0.187	2	2	4	0.172	0.178	3%	
3	finite stiffness floors - brick layers	0.206	2	2	4	0.176	0.184	5%	
4	finite stiffness floors - walls thickness reduction	0.151	2	2	4	0.173	0.166		-4%

Table 21: Four levels case study: test of the method.

Even in this case, it is possible to observe that the proposed estimation method is able to give a prediction of the value of the PGA_C with a maximum error of 9%.

6.4.3 Five levels case study

The last set of buildings which has been tested, in order to check the reliability of the proposed method, is the one composed of five levels buildings. The geometry of the building is the same, as well as the load condition and the openings configuration.

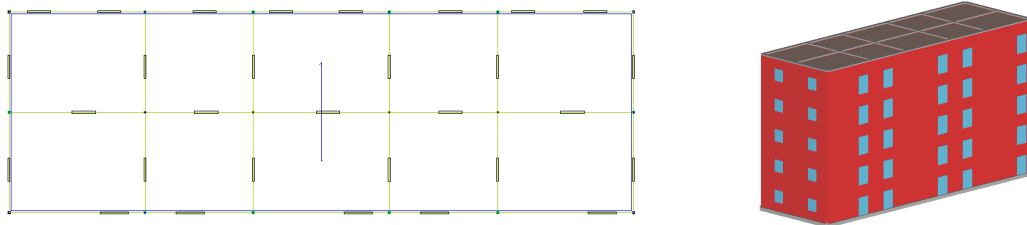


Figure 108: Plan and 3d view of the four levels case study.

In the following Table 22, the results obtained on this set of building are proposed.

n.°	IDENTIFICATION OF THE MODEL	$a_0 \cdot \tau_k$ [N/cm ²]	par. 5+9 [1-4]	par. 6 [1-4]	par.7 [1-4]	PGA _C [g]	estimation of PGA _C [g]	over estimation error [%]	under estimation error [%]
1	rigid floors	0.187	3	2	4	0.188	0.195	4%	
2	finite stiffness floors	0.187	2	2	4	0.186	0.178		-4%
3	finite stiffness floors - brick layers	0.206	2	2	4	0.191	0.184		-3%
4	finite stiffness floors - walls thickness reduction	0.151	2	2	4	0.178	0.166		-6%

Table 22: Five levels case study: test of the method.

As seen for the other sets of buildings, the proposed method gives an estimation of the global capacity, expressed in terms of peak ground acceleration, which has a maximum error of 15% comparing to the results of the detailed analyses.

6.5 Reliability of the global behaviour PGA_C estimation: local mechanisms influence

As already stated in this work, the reliability of the PGA_C estimation related to the global box behaviour depends on the real possibility that the considered structure shows a unitary performance against seismic action, without the activation of local mechanisms.

In order to give a qualitative evaluation of this aspect, it has been investigated which parameters of the 11 ones give some information about the possibility of activation of local collapses; the following parameters have been chosen:

- PARAMETER 1 - quality of the resistant system;
- PARAMETER 5 - typology of floors;
- PARAMETER 8 - maximum distance among the walls.

The parameter 1 concerns the connection among orthogonal walls and among walls and floors, so it is strictly related to the identification of possible local collapses.

The parameter 5 is related to the in-plane stiffness and to the level of connection of the floors to the walls: this parameter, already considered in the global behaviour (since the stiffness of the floors allows a uniform distribution of the horizontal forces among the vertical structural systems), is taken into account again in the local behaviour mainly because of the connections aspect, that is a direct function of the construction typology of the floor itself: flexible floors, such as masonry vaults or wooden floors, are mostly characterized by a low level of connection to the walls, while well realized floors, composed of reinforced concrete beams and hollow tiles for example, are often characterized by the presence of reinforced concrete beams on the edges which ensure (when realized on the whole thickness of the wall) an effective level of connection with the walls.

The parameter 8 concerns only local aspects since it is related to the presence of masonry alignments which have no orthogonal constraints for a considerable length (in relation to the thickness of the considered wall): this aspect can lead to the activation of local mechanisms for the wall itself under horizontal actions, since it is not well anchored to the remaining part of the structure.

In order to give a qualitative evaluation of the local mechanisms problem, it has been decided to fully refer to the Vulnerability Index Method, considering only the three parameters listed above: scores and weights are directly considered from the original method, and the weighted sum related to these parameters is calculated. This weighed sum can lead to a maximum score of 101.25 (considering the "D" judgment for all of them).

With these assumptions, a Reliability Index of the global behaviour of the structure can be calculated, evaluating the ratio among the weighted score for the considered parameters and its maximum value:

$$R_I = 1 - \frac{\sum V_i \cdot P_i}{101.25}$$

This Reliability Index can assume values in the range [0%-100%], where low values are related to conditions that cannot ensure a global behaviour, making the estimation of the global capacity (by means of the PGA_C) not so reliable, while high values guarantee that the structure can behave globally, allowing the use of the in-plane resistance and stiffness of each masonry alignment and making the estimation of the PGA_C a reliable indicator of the capacity of the structure.

In order to give an objective categorization of the index R_I , four different intervals have been identified:

- $R_I < 25\%$
- $25\% \leq R_I < 50\%$
- $50\% \leq R_I < 75\%$
- $R_I \geq 75\%$

The 1st and 2nd classes give information of a low reliability of the global capacity estimation, since local problems may rule the behaviour of the considered structure; in the 3rd class, the global capacity estimation starts to be considered as reliable, even if the local mechanisms problem must be investigated in any case; the 4th class should be characterized by structures realized with technological features which are able to exclude local mechanisms problems.

It is anyway important to remind that, in general, the local mechanisms analysis must be performed for each structure when a detailed structural analysis is required: the indication of this Reliability Index just give some advices on the probable behaviour of a structure, simply through a qualitative evaluation.

In the following Table 23, the calculation of the Reliability Index of the considered sample of structures is proposed: it is important to highlight that only 4 S.U. are characterized by a very low Reliability Index and, considering even the second range, 5 structures are likely to be affected by local mechanisms, which can be predominant in the vulnerability evaluation.

It is also useful to consider the main material of construction of these 5 buildings: all of them are realized with disorganized stone masonry, material which is usually characterized by the presence of low effective connections among orthogonal walls and by flexible floors and roofs.

Once again, the structural units are listed following the vulnerability evaluation (γ) performed with the detailed approach, as already done in Table 19.

n.°	STRUCTURAL UNIT	description of the material (D.M. 2008)	WEIGHTS			RELIABILITY INDEX (R_i) OF THE GLOBAL CAPACITY ESTIMATION			
			PAR. 1 score	PAR. 5 score	PAR. 8 score	<25%	25-50%	50-75%	>75%
1	AUSL 3 SMP 01 03	disorganized stone masonry	45	45	25	5%	30%	54%	
2	AUSL 4 MD 01 24	disorganized stone masonry	45	45	45	0%			
3	AOUC CAR 4 01	well organized stone masonry	20	15	45				
4	USL 10 IOT ANT 01	disorganized stone masonry	20	45	25				
5	AOUC CAR 16 01	brick masonry	20	15	45				
6	AUSL 3 PES 01 06	disorganized stone masonry	45	45	45	0%			
7	USL 10 SERR 01	disorganized stone masonry	45	45	25	5%			
8	AOUC CAR 13 04	brick masonry	20	15	45				
9	AOUC CAR 13 03	well organized stone masonry with brick layers	20	15	45				
10	AOUC CAR 26 02	well organized stone masonry with brick layers	5	5	5				
11	AUSL 3 PES 01 04	brick masonry (good quality mortar)	5	5	45				
12	AOUC CAR 8b 03	well organized stone masonry with brick layers	5	5	45				
13	AOUC CAR 8b 04	well organized stone masonry with brick layers	5	5	45				
14	AOUC CAR 26 03	well organized stone masonry with brick layers	5	5	5				
15	AOUC CAR 8b 01	well organized stone masonry with brick layers	5	5	45				
16	AOUC CAR 4 02	well organized stone masonry with brick layers	20	15	45				
17	AOUC CAR 8b 02	well organized stone masonry with brick layers	5	5	45				
18	AOUC CAR 13 01	well organized stone masonry with brick layers	20	15	45				
19	AOUC CAR 26 01	well organized stone masonry with brick layers	5	5	5				
20	AUSL3 SMP 01 04	"half hollow" brick masonry	5	5	45				

Table 23: Calculation of the "Reliability Index" R_i .

The values of capacity shown in Table 19 (both the estimation and the evaluation with detailed analyses of the PGA_C) are characterized by these coefficients of reliability (Table 23); in the beginning of this work, it has been decided that the local mechanisms analysis would have not been performed since it requires another approach, which introduces a considerable number of other variables.

The proposed results give an estimation of the global capacity of the structure, expressed in terms of PGA_C , and a related Index of Reliability of that estimation; for the cases with a low Index, the value of the estimation is in any case proposed for two reasons:

- it is possible that the detailed analysis of the local mechanisms leads to a capacity value even higher than the global one, since it depends on the particular conditions of the considered structure (type of floors and their particular restraint conditions, eccentricity of the supporting area of the structural elements of the floor in relation to the thickness of the wall, percentage of holes in the wall, position of the barycenter of it, etc...);
- even if the local mechanisms analysis rules the behaviour of the considered structure, the global capacity estimation is an useful indicator of the performance that the structure can reach after some interventions specifically designed in order to improve the connections and, consequently, making the box behaviour possible.

6.6 Estimation of the Index of Risk

6.6.1 Definition of the Index

Once that the seismic capacity estimation of the structure is performed (using the multi-linear regression procedure), the related estimation of the Index of the seismic risk can be obtained: since the site of construction of each S.U. is known, the seismic hazard can be individuated in terms of three parameters:

- a_g maximum expected horizontal acceleration on A type soil (rock);
- F_0 maximum value of the amplification factor of the horizontal acceleration spectrum;
- T_C^* upper limit of the period of the constant spectral acceleration branch.

From these parameters, together with the information about the geotechnical and topographical conditions, the PGA of demand can be obtained:

$$PGA_D = a_{g_D} \cdot S$$

where S is the coefficient which keeps into account both the stratigraphic and topographic conditions.

An indicator of risk is then evaluable in the following way:

$$\alpha_{PGA} = \frac{PGA_C}{PGA_D} = \frac{a_{g_C} \cdot S}{a_{g_D} \cdot S} = \frac{a_{g_C}}{a_{g_D}} = \alpha_{a_g}$$

This index, widely used in the seismic risk analysis field, gives information about the safety of the considered structure:

- values of $\alpha_{PGA} < 1$ are related to not safe structures;
- values of $\alpha_{PGA} > 1$ are instead related to safe structures.

This aspect is really important since it allows the identification of an absolute parameter of risk, which can be compared among different structures located in different sites of construction.

In the new expeditious vulnerability approach, both the location of the considered structure and the geotechnical conditions of the underground can be considered; in order to better understand how they are taken into account, it is useful to describe them separately.

6.6.2 Influence of the seismic hazard

For the demand evaluation, the influence of the site of construction is considered simply using the value of a_{g_D} related to the site of construction itself; this aspect is evaluated as in the case of a detailed analysis.

For the capacity evaluation instead, a common site of construction has been adopted for all the structural units. How much this assumption influences the research of the capacity expressed in terms of a_{g_C} ?

The approach of calculation of the capacity by means of the "fixed spectrum shape method" (already described in § 4.5.4) used in this work is based on the assumption that the capacity spectrum is defined for two parameters, F_0 and T_C^* , while the $PGA = a_g \cdot S$ is considered as the variable.

For the sample of analysis, the following Table 24 shows how the parameters F_0 and T_C^* change in the different sites of construction of each S.U.: the first row has been used in the detailed analysis (definition of the common site) and the differences of all the other values with the reference ones are indicated (% of variation). The differences are really small, with only one percentage of variation equal to 9% for the T_C^* parameter.

Consequently, the capacity values are not influenced in a substantial way by the assumption of a reference site of construction for all the buildings of the sample. This is even reasonable thinking about that the considered cities are placed at a relative maximum distance of 60 km.

S.U.	F_0	T_C^*	% variation (CAR)	% variation (CAR)
AOUC CAR (13 S.U.)	2.391	0.311	-	-
AUSL 3 SMP 01 (2 S.U.)	2.420	0.283	1%	-9%
AUSL 3 PES 01 (2 S.U.)	2.375	0.305	-1%	-2%
AUSL 4 MD 01 24 (1 S.U.)	2.392	0.310	0%	0%
USL 10 IOT ANT 01 (1 S.U.)	2.387	0.309	0%	-1%
USL 10 SERR 01 (1 S.U.)	2.401	0.308	0%	-1%

Table 24: Parameters for the definition of the seismic hazard in the sites of construction of each S.U..

6.6.3 Influence of the geotechnical conditions

The influence of the geotechnical conditions instead are considered in a way which does not exactly correspond to the one of a detailed analysis: in the analytical approach, the assumption of a category of soil or of a topographic condition modifies the parameter S and, consequently, the definition of the reference elastic acceleration spectrum. This aspect influences the research of the capacity spectrum (as seen in Figure 62) and, therefore, the a_{g_c} : considering one building which has to be investigated with two different soil conditions (with two different reference spectra therefore), the ratio among the a_{g_c} calculated in two situations is equal to S (for example, the a_{g_c} calculated with a type of soil "A" or "B", according to indications of D.M. 2008) only if the period of the equivalent SDOF system belongs to the plateau zone of the acceleration spectrum for both the considered conditions; if the period is located in another part of the spectrum, the ratio is not known, since it depends on the relative values of the acceleration measured on the two spectra, which have different shapes. In any case, a certain multiplier value can be obtained, and the two quantities are related through the following equation:

$$a_{g_c_soil"B"} = a_{g_c_soil"A"} / p(S, T^*)$$

where $p(S, T^*)$ is a parameter which depends on the coefficient S and on the period of the SDOF system. This parameter is bigger than 1.0, since it must reduce the best condition values ("A" soil condition).

The calculation of the capacity changes due to the geotechnical conditions, while the demand, expressed in terms of a_g (maximum expected horizontal acceleration on A type soil), remains the same value:

$$a_{g_D_soil"B"} = a_{g_D_soil"A"} = a_{g_D}$$

The calculation of the Index of Risk for the case of "B" type of soil can be expressed as a function of the parameter $p(S, T^*)$:

$$\alpha_{PGA} = \frac{PGA_C}{PGA_D} = \alpha_{a_g} = \frac{a_{g_c}}{a_{g_D}} = \frac{a_{g_c_soil"A"}}{p(S, T^*) \cdot a_{g_D}}$$

The a_{g_D} has no specific reference to the type of soil since it is independent from the category of soil.

In order to take into account this aspect, it is not possible to evaluate in a detailed way the parameter $p(S, T^*)$ for a generic structure, since there is no information about the period of the equivalent system during the expeditious survey of a building (available only after the pushover analysis).

However, it is possible to assume that $p(S, T^*) = S$: in this way, the capacity value of acceleration $a_{g_C_soil\ "A"}$ can still be evaluated through the multi-linear regression approach, while the demand value can be estimated starting from the a_{g_D} and using the information of the coefficient S , in order to consider the geotechnical aspect in a first approximated way.

Analytically speaking, the Index of Seismic Risk for the expeditious method is:

$$\alpha_{PGA} = \frac{PGA_C}{PGA_D} = \frac{a_{g_C}}{a_{g_D}} = \frac{a_{g_C_soil\ "A"}}{S \cdot a_{g_D}}$$

6.6.4 Final consideration on the Index of Risk

As explained above, the Index of Risk is a direct function of the estimation of the capacity, while the demand can be evaluated considering the studies of seismic hazard already performed on the Italian territory and contained in the database of the Italian Code (D.M. 2008).

The estimation of the Index of Risk has the same reliability of the estimation of the capacity (PGA_C): the Reliability Index described in § 6.5 is therefore valid even for it.

In the following Table 25, the resume evaluations about the seismic vulnerability and risk for the subset of buildings (analyzed both with the detailed and the new expeditious method) are proposed; the S.U. are ordered following the estimated Index of risk α_{PGA} for the "A" soil type: the 1st position corresponds to the most exposed to risk building, while the 20th one is related to the building with the lowest level of seismic risk.

For each S.U., the following quantities are listed:

- the ranking in the vulnerability classification, obtained with the estimation of the capacity in terms of $a_{g_C_soil\ "A"}$ (performed with the Multi-linear regression approach); the 1st position corresponds to the most vulnerable structure, while the 20th one is related to the least vulnerable building;
- the value of capacity of the structure, estimated with the Multi-linear regression approach, expressed in terms of $a_{g_C_soil\ "A"}$;
- the value of demand for each S.U., obtained by the observation of the seismic hazard of the site of construction of each of them; it is expressed in terms of a_{g_D} and it has been obtained from the indications of the Italian Code (D.M. 2008).

After these general indications, two hypotheses for the calculation of the index of risk have been done:

- assumption of a category of soil which corresponds to the best possibility among the ones provided from the Italian Code (D.M. 2008): "A" type of soil, which is described as "rock or other rock-like geological formation". With this hypothesis, the coefficient $S = 1$, so the PGA_D and the α_{PGA} are consequently calculated;
- assumption of a "C" category of soil, which is the intermediate possibility provided from the Italian Code: "deposits of dense or medium-dense sand, gravel or stiff clay (thickness more than 30 meters)". In an analogous way, the coefficient S has been calculated following the instruction of D.M. 2008 and, consequently, the PGA_D and α_{PGA} have been obtained.

It is important to highlight that the assumption of the two typologies of soil does not modify the position of the buildings in the seismic risk classification, but it modifies the values of the indexes of risk and the "distances" among the coefficients of each structure.

n.°	STRUCTURAL UNIT	Ranking in Vulnerability Classification [1-20]	estimation of $a_{g,c}$ [g]	DEMAND $a_{g,D}$ [g]	"A" soil type (D.M. 2008)			"C" soil type (D.M. 2008)			
					S "A" soil condition	PGA _D "A" soil condition [g]	α PGA "A" soil condition	S "C" soil condition	PGA _D "C" soil condition [g]	α PGA "C" soil condition	
HIGHEST SEISMIC RISK	1	AUSL 3 SMP 01 03	1	0.112	0.253	1.000	0.253	0.442	1.333	0.337	0.332
	2	AUSL 4 MD 01 24	4	0.131	0.180	1.000	0.180	0.730	1.442	0.260	0.506
	3	AUSL 3 PES 01 06	5	0.132	0.180	1.000	0.180	0.734	1.443	0.260	0.509
	4	AOUC CAR 4 01	3	0.126	0.168	1.000	0.168	0.748	1.459	0.245	0.513
	5	USL 10 IOT ANT 01	2	0.123	0.162	1.000	0.162	0.757	1.467	0.238	0.516
	6	AOUC CAR 16 01	6	0.143	0.169	1.000	0.169	0.849	1.458	0.246	0.582
	7	AOUC CAR 13 04	7	0.146	0.168	1.000	0.168	0.867	1.458	0.245	0.594
	8	AUSL3 SMP 01 04	20	0.234	0.253	1.000	0.253	0.923	1.333	0.337	0.693
	9	AOUC CAR 13 03	9	0.169	0.168	1.000	0.168	1.004	1.458	0.245	0.689
	10	USL 10 SERR 01	8	0.165	0.158	1.000	0.158	1.042	1.472	0.233	0.708
LOWEST SEISMIC RISK	11	AOUC CAR 4 02	10	0.180	0.168	1.000	0.168	1.074	1.459	0.245	0.736
	12	AUSL 3 PES 01 04	12	0.197	0.180	1.000	0.180	1.096	1.443	0.260	0.759
	13	AOUC CAR 8b 04	11	0.186	0.168	1.000	0.168	1.109	1.458	0.245	0.761
	14	AOUC CAR 8b 03	13	0.199	0.168	1.000	0.168	1.184	1.458	0.245	0.812
	15	AOUC CAR 26 02	15	0.204	0.169	1.000	0.169	1.209	1.458	0.246	0.829
	16	AOUC CAR 13 01	14	0.203	0.168	1.000	0.168	1.211	1.458	0.245	0.830
	17	AOUC CAR 8b 01	16	0.210	0.168	1.000	0.168	1.250	1.458	0.245	0.857
	18	AOUC CAR 26 03	17	0.216	0.169	1.000	0.169	1.278	1.458	0.246	0.877
	19	AOUC CAR 8b 02	18	0.221	0.168	1.000	0.168	1.313	1.458	0.245	0.901
	20	AOUC CAR 26 01	19	0.227	0.169	1.000	0.169	1.343	1.458	0.246	0.921

Table 25: Seismic risk classification.

The assumption of a "C" soil is more reasonable than the "A" one for the considered areas (Florence, Prato and Pistoia territories), since rock soil configurations are not so common to be found in these zones, which are instead usually characterized by "B" or "C" types of soil.

With this hypothesis ("C" soil"), it is possible to observe that no one of the structural units can be considered safe (since all the values are lower than 1.0), even if some of them show values close to 1.0.

It is useful to remind that the ranking of the structural units observed in terms of vulnerability and in terms of risk do not coincide, since the acceleration of demand has its independent influence in the definition of the index of seismic risk, without any relation to the vulnerability aspects.

7 Conclusions and outlooks

7.1 Introduction

In the seismic risk evaluation of the built environment, the vulnerability assessment represents a key aspect: it can be performed, as already described in detail in § 2.4, by means of expeditious empirical methods, detailed methods or hybrid methods.

The Vulnerability Index Method, developed at first in the '80s for masonry structures, is one of the most famous expeditious methods used in Italy for the creation of vulnerability classifications in case of large samples of buildings. This method, as fully described in § 2.4.3, gives as result an Index of Vulnerability, I_v , belonging to the range 0%-100%; the index is useful for a comparative purpose but it does not provide any absolute information about the vulnerability.

Nowadays, for masonry structures, the non linear analyses represent the most common procedure for the evaluation of the seismic vulnerability through detailed approaches, using three-dimensional models composed of elements with non linear behaviour.

These two instruments of vulnerability evaluation have been used in this work, in order to propose a new expeditious vulnerability assessment method.

7.2 Principal results of the work

After the literature review on the argument contained in [Chapter 2](#), the application of the Vulnerability Index Method on a real sample of buildings has been proposed in [Chapter 3](#), describing the results for the Hospital structures of the cities of Florence, Prato and Pistoia. It has been possible to analyze some general features of this group of structures (118 buildings), obtaining the vulnerability classification at first just following the order of the Index I_v and then, crossing the collected data (age, material of construction etc...), more detailed aspects have been investigated.

In [Chapter 4](#), the relation among this expeditious approach and a detailed analysis procedure has been investigated, using the static non-linear analysis performed with the F.M.E. (Frame by Macro Element) approach. The detailed procedure has been applied on a subset of 20 structural units (on a total of 118 buildings), performing 8 pushover analyses for each of them.

The comparison analysis has shown that there is a relation among the two considered approaches (see § 4.7): in particular, the relation is even stronger if the comparison is performed considering the parameters of the expeditious approach concerning the global behaviour of the masonry structures (6 parameters have been individuated).

In [Chapter 5](#), a critical analysis of the Vulnerability Index Method has been proposed, in order to highlight advantages and weak points of the method. One of the most important aspects observed in this part of the work is the influence of the parameter 3 (conventional resistance) in the vulnerability estimation: it has a fundamental importance but, for the analyzed sample of buildings, it does not provide an effective differentiation because of the definition of its classes of judgment.

In [Chapter 6](#), the construction of a new vulnerability assessment method has been described, specifically conceived for masonry structures; this new method requires the same amount of information necessary for the Vulnerability Index Method and it allows:

- the estimation of the peak ground acceleration of capacity related to the global behaviour of the structure, using a simple equation;

-
- the evaluation of a reliability index of the capacity estimation mentioned above, in order to consider the possibility of occurrence of local mechanisms collapses;
 - the estimation of the index of seismic risk, considering from one side the PGA_C estimated with the new expeditious method, and from the other side the PGA_D of the site of construction, taking into account even the geotechnical conditions.

Since the required information is the same of the original Vulnerability Index Method, even the Index I_V can still be calculated: it remains a useful instrument for the creation of a first general vulnerability classification, considering both structural (global and local behaviour) and non structural aspects.

The new method provides more information of the original one, giving numerical estimations of the capacity of the considered structures and allowing the calculation of indexes of risk.

7.3 Outlooks

The proposed approach is based on the detailed analysis of 20 buildings on a total of 118 structural units, all characterized by a Hospital function.

A further improvement of the method can be obtained simply increasing the number of buildings analyzed with the detailed approach, providing consequently more stability to the procedure.

Moreover, this approach can be repeated on other categories of buildings and other geographical contexts: the results obtained in this work are related to the Hospital structures realized in Tuscany and it is not possible to extend this method to other samples (School structures or Public Offices for example) without checking at first the compatibility of the buildings' features with the method itself. Only after this passage, the application of the procedure is allowed.

7.4 Final considerations

Generally speaking, the idea which has inspired all this work consists in the desire of obtaining an absolute estimation of the level of risk of a structure through simplified analyses: this aspect becomes important when a consistent sample of structures has to be investigated; this method can help in the first screening phase, giving information about the most vulnerable elements and the most exposed to risk ones through a simple procedure.

Classifications of vulnerability and risk can be obtained: these rankings are useful instruments in order to decide which structures analyze at first with a detailed approach.

Since the economic resources of any Administration are limited, a conscious decision must be performed in order to allocate the funds in the most effective way, trying to reduce as much as possible the seismic risk of the considered built heritage.

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9 Annex A: descriptions and results for the analyzed structures

In this Annex, a resuming form for each structure which has been analyzed both with the empirical approach (2° Level Vulnerability Form - GNDT, 1993) and the detailed analysis procedure (pushover analyses on the Frame by Macro-Element models) is proposed.

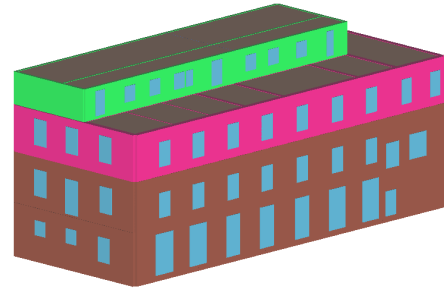
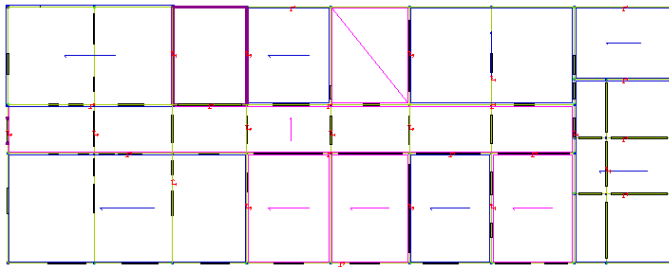
In particular:

- a first brief description of the building is reported;
 - the planimetric view of the unit within the aggregate which it belongs to is proposed, as well as a three dimensional view of the realized analytical model;
 - the Index of Vulnerability I_v is calculated by means of the weighted sum of the 11 parameters; the resuming chart is proposed with the definition of the judgments for each parameter; after that, the Index of Vulnerability specifically evaluated in order to make a comparison with the analytical model is presented (considering 6 parameters);
 - the 8 pushover curves, obtained from the detailed analyses, are plotted in a unit-less graph, distinguishing the directions of analysis with a different colour (red or black), the shape of the lateral load profile and the presence of eccentricity with different line styles. In particular, the following names for each analysis have been used:
 - 1_X_1ms_0ecc X direction, lateral load profile proportional to 1st modal shape, no eccentricity
 - 2_X_mass_0ecc X direction, lateral load profile proportional to masses, no eccentricity
 - 3_X_1ms_5%ecc X direction, lateral load profile proportional to 1st modal shape, eccentricity
 - 4_X_mass_5%ecc X direction, lateral load profile proportional to masses, eccentricity
 - 5_Y_1ms_0ecc Y direction, lateral load profile proportional to 1st modal shape, no eccentricity
 - 6_Y_mass_0ecc Y direction, lateral load profile proportional to masses, no eccentricity
 - 7_Y_1ms_5%ecc Y direction, lateral load profile proportional to 1st modal shape, eccentricity
 - 8_Y_mass_5%ecc Y direction, lateral load profile proportional to masses, eccentricity
- It is important to remind that the eccentricity, when considered, is equal to the 5% of the planimetric dimension of the building which is orthogonal to the considered direction of the seismic action.
- The graphs are all realized with the same maximum values on the horizontal and vertical axis, in order to allow a direct comparison among different structural units;
- the value of the capacity acceleration of the structure, related to the Life Safety TBPL, is written after the graphs. This value has been adopted as the indicator of the vulnerability for the analytical approach.

The structural units here in the following described are ordered considering the classification of vulnerability performed with the detailed approach (pushover analysis), starting from the most vulnerable building and arriving to the least vulnerable one.

9.1 AUSL 3 SMP 01 03

The building is located in San Marcello Pistoiese, in the Hospital Company n.° 3 of Pistoia. The unit belongs to a Complex composed of 4 separated S.U.; it has been built before 1900 (1850 approximately) but the last two levels have been realized in the last century. It is composed of 4 levels: the first 2 are realized with disorganized stone masonry, the third with brick masonry and the last with concrete blocks. The floors of the lower levels are realized with barrel vaults, while the other ones with reinforced concrete and hollow tiles.



CALCULATION OF THE INDEX OF VULNERABILITY

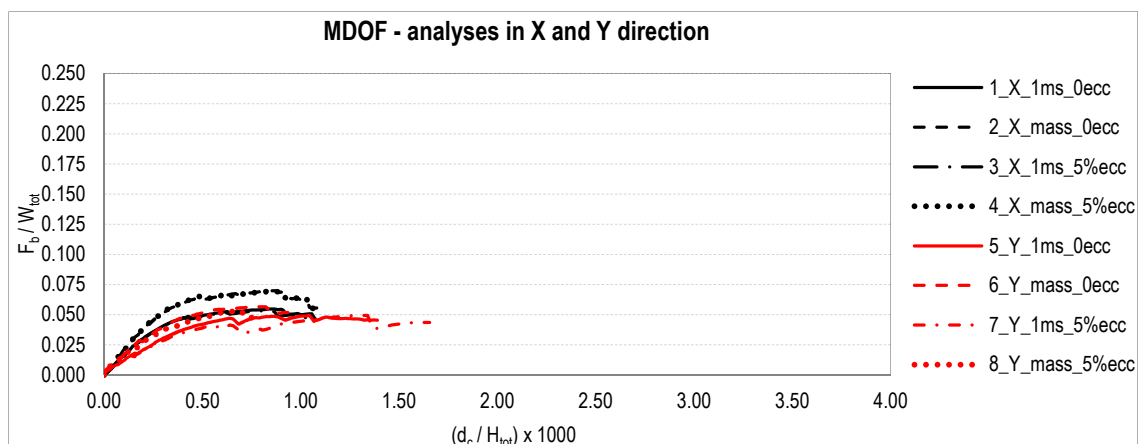
PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	D	45.00	1.00	45.00
2 Quality of the resistant system	D	45.00	0.25	11.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	D	45.00	1.00	45.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	D	45.00	1.00	45.00
8 Maximum distance among the walls	C	25.00	0.25	6.25
9 Roof	C	25.00	0.75	18.75
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	B	5.00	1.00	5.00

266.25

→ I_v **69.61 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=52.29\%$.

PUSHOVER CURVES



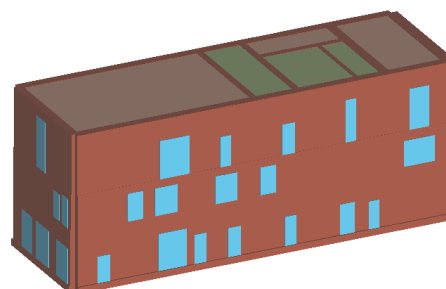
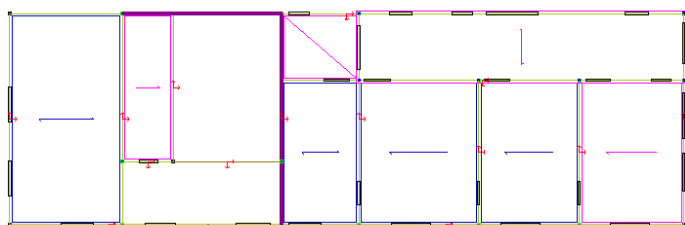
The behaviour in the two direction is similar; the worst analysis is the n.°8, which gives a $PGA_c=0.0973g$.

9.2 AUSL 4 MD 01 24

The building is located in Prato, in the Hospital Company n.° 4.

The unit belongs to a wide Complex (50 S.U.), it has been built before 1900 and it is composed of 3 levels, all of them realized with disorganized stone masonry.

The lower level is mostly characterized by the presence of barrel vaults, while the other parts of the structure have floors composed of reinforced concrete and hollow tiles.



CALCULATION OF THE INDEX OF VULNERABILITY

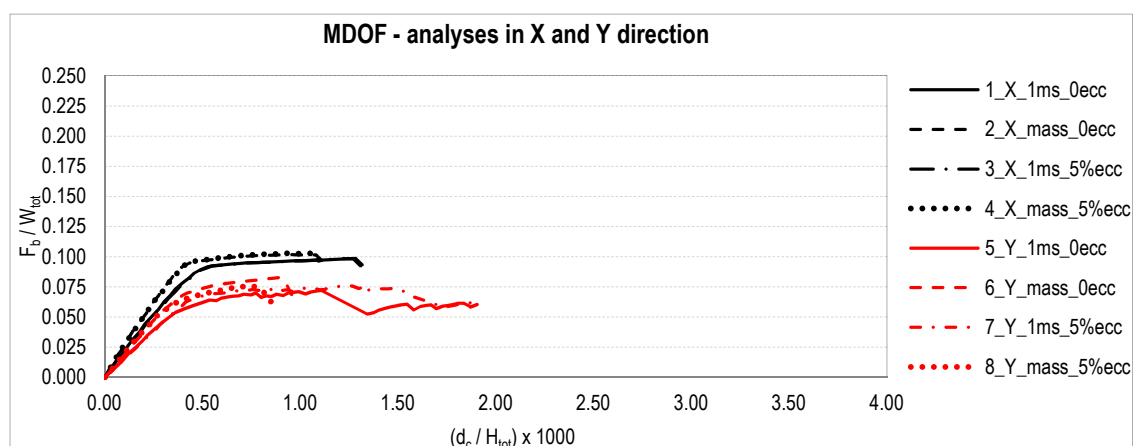
PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	D	45.00	1.00	45.00
2 Quality of the resistant system	D	45.00	0.25	11.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	A	0.00	0.75	0.00
5 Typology of floors	D	45.00	1.00	45.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	D	45.00	0.50	22.50
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

271.25

→ I_v 70.92 %

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=48.04\%$.

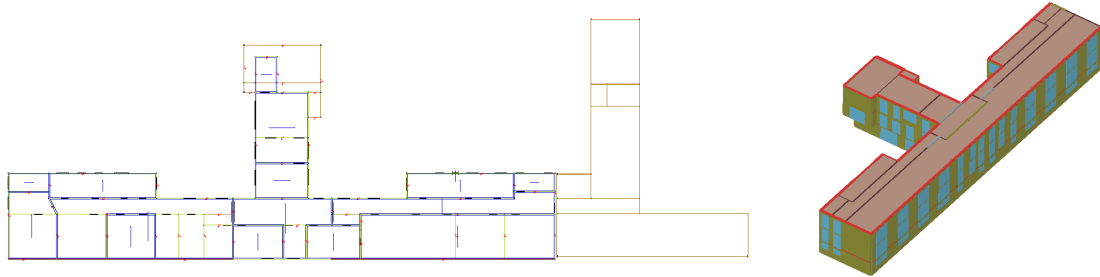
PUSHOVER CURVES



The curves show a better behaviour along the longest direction of the building; the worst analysis is the n.°8, which gives a $PGA_c=0.1149 g$.

9.3 AOUC CAR 4 01

The building is located in Florence, in the Careggi Hospital Company. The S.U. belongs to a Complex composed of 3 units, 2 contiguous ones made of masonry and 1 more recent, made of reinforced concrete. The considered unit has been built around 1930; it is composed of 3 levels (1 basement and 2 levels out of the ground), all of them realized with well organized stone masonry (external walls) and brick masonry (internal partitions). Most of the floors are realized with reinforced concrete and hollow tiles.



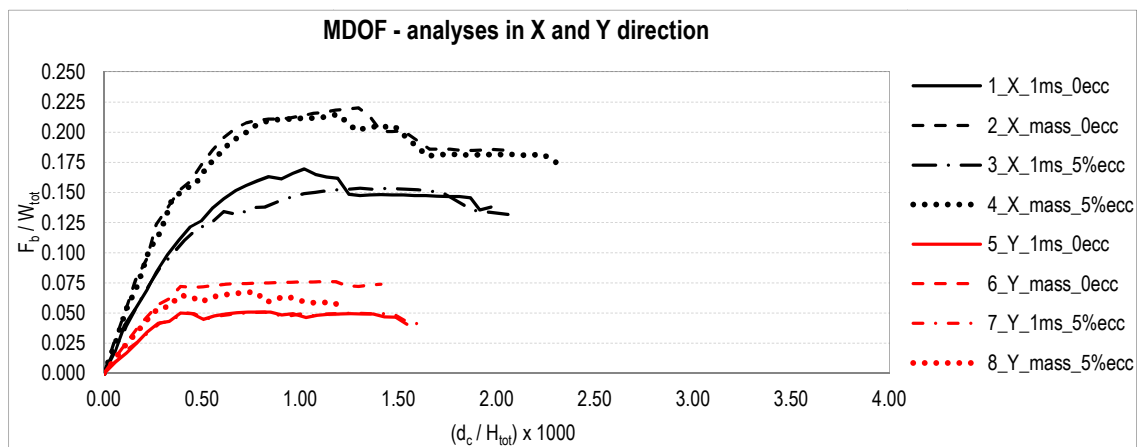
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	C	25.00	0.25	6.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	1.00	15.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	D	45.00	0.25	11.25
11 State of conservation	B	5.00	1.00	5.00

200.00 → I_V **52.29 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=38.89\%$.

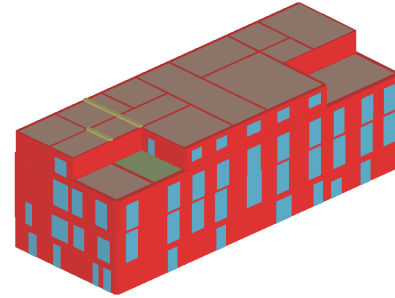
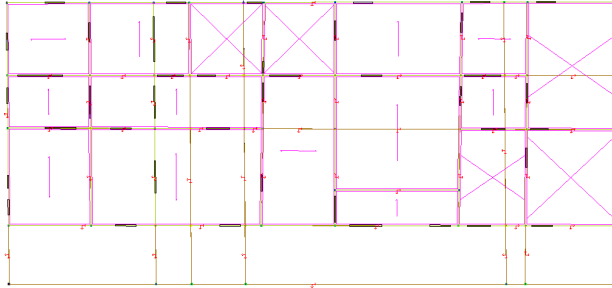
PUSHOVER CURVES



The behaviour is different for the two main directions and for the type of lateral load; the worst analysis is the n.°5, which gives a $PGA_c=0.1216 g$.

9.4 USL 10 IOT ANT 01

The building is located in Firenze, in the Hospital Company n.° 10. The unit belongs to a wide Complex made of 10 S.U.; the considered one is completely independent from the others, it has been built before 1900 and it is composed of 4 levels (1 basement and 3 levels out of the ground), all of them realized with disorganized stone masonry. The lower levels are mostly characterized by the presence of masonry vaults, while the other parts of the structure are mainly realized with wooden floors. The roof is made of wooden beams too.



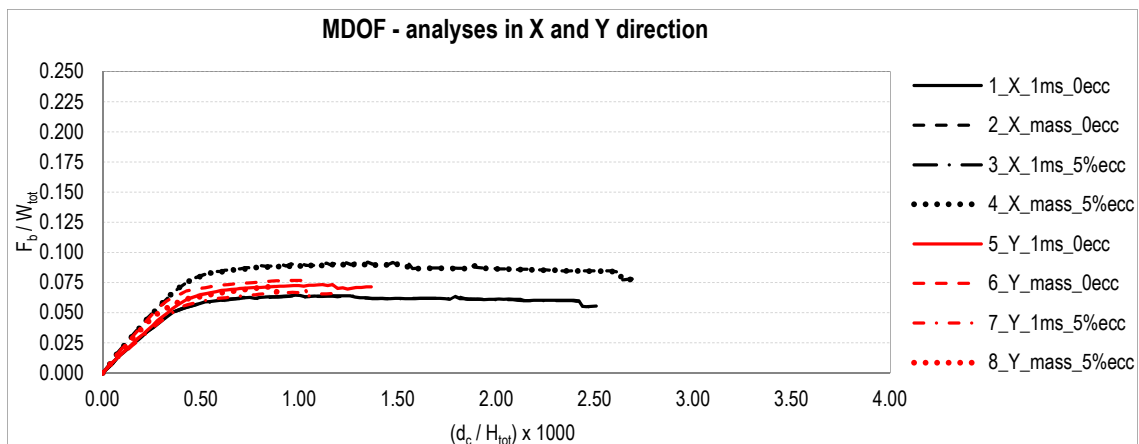
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	D	45.00	0.25	11.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	D	45.00	1.00	45.00
6 Planimetric configuration	B	5.00	0.50	2.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	C	25.00	0.25	6.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

225.00 → I_v **58.82 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=42.81\%$.

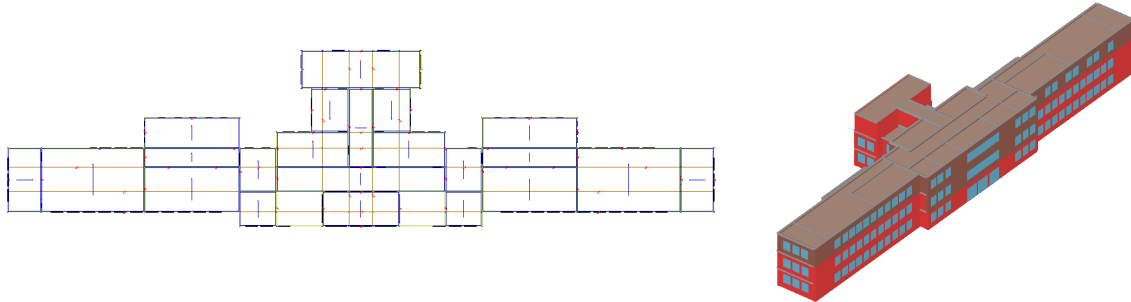
PUSHOVER CURVES



The longest direction of the building shows more ductility; for this direction, the type of lateral load is influent (better behaviour for the "mass" profile). The worst analysis is the n.°8, which gives a $PGA_c=0.1295 g$.

9.5 AOUC CAR 16 01

The building is located in Florence, in the Careggi Hospital Company. The S.U. corresponds to the Complex directly. The S.U. has been realized around 1930 and it is composed of 3 levels: the first is made of brick masonry while the other two (the upper ones) are made of disorganized stone masonry. Most of the floors are realized with reinforced concrete and hollow tiles except some cases where it has been found steel beams and hollow tiles floors.



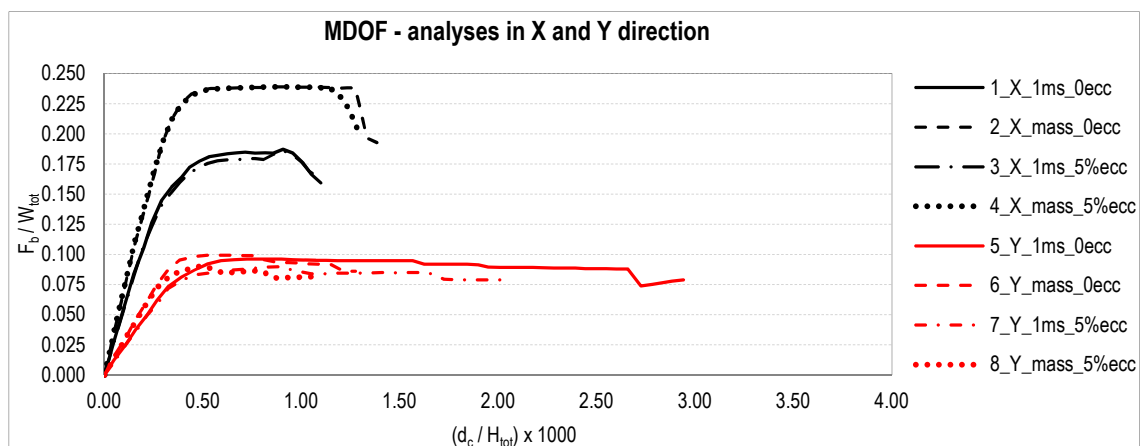
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	C	25.00	0.25	6.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	1.00	15.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

215.00 → I_v **56.21 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=38.89\%$.

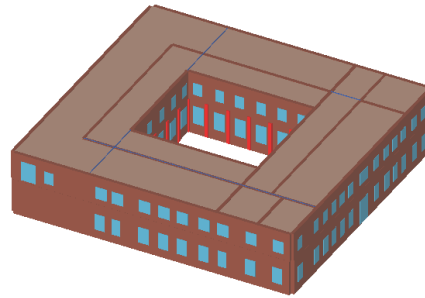
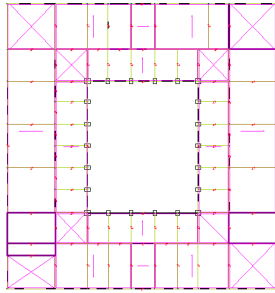
PUSHOVER CURVES



The behaviour is different for the two main directions and for the type of lateral load; the worst analysis is the n.°8, which gives a $PGA_c=0.1440 g$.

9.6 AUSL 3 PES 01 06

The building is located in Pescia, in the Hospital Company n.° 3 of Pistoia. The unit belongs to a Complex composed of 9 S.U. but it is completely independent from the other ones; it has been built before 1900 (XVIII Century approximately) and it is composed of 2 levels, realized with disorganized stone masonry. It has a squared shape with an internal court. The first floor is mainly composed of vaults (cross and barrel ones), while the roof is made of wood with trusses.



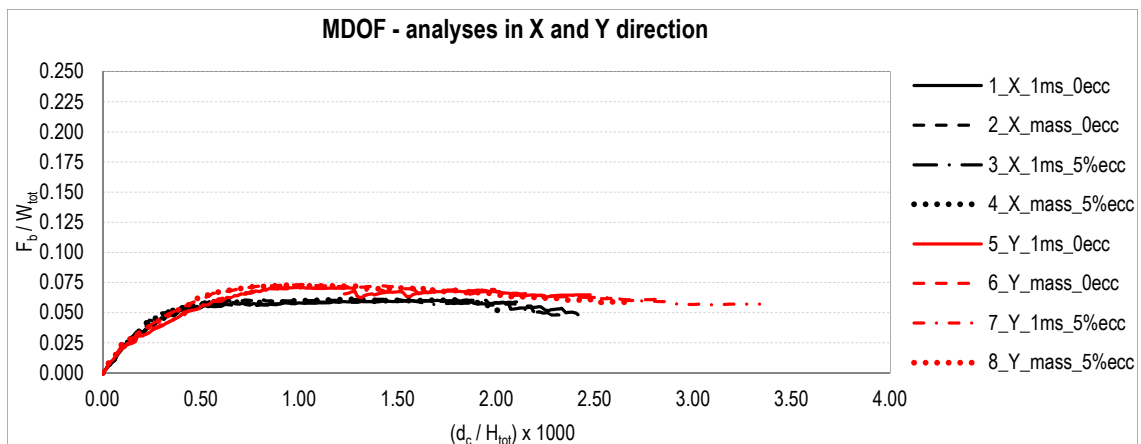
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	D	45.00	1.00	45.00
2 Quality of the resistant system	D	45.00	0.25	11.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	D	45.00	1.00	45.00
6 Planimetric configuration	B	5.00	0.50	2.50
7 Elevation configuration	B	5.00	0.50	2.50
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	C	25.00	1.00	25.00

226.25 → **I_v 59.15 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=36.93\%$.

PUSHOVER CURVES

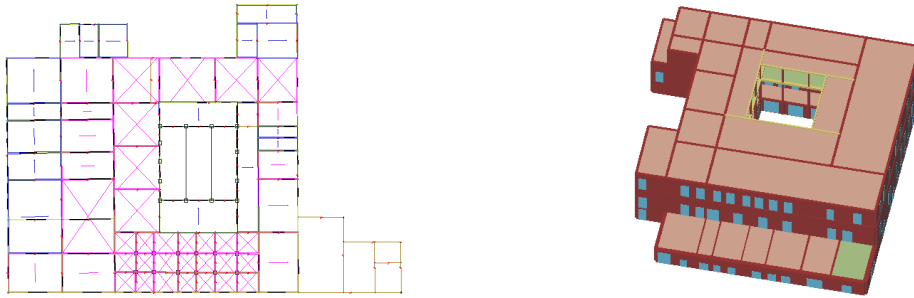


The behaviour in the two direction is similar; the worst analysis is the n.°3, which gives a $PGA_c=0.1484 g$.

9.7 USL 10 SERR 01

The building is located in Firenze, in the Hospital Company n.° 10. The unit belongs to a wide Complex made of 16 S.U.; it has been built before 1900 (XV Century approximately) and it is composed of 3 levels, all of them realized with disorganized stone masonry.

The lower levels are mostly characterized by the presence of masonry vaults, while the other parts of the structure are mainly realized with wooden floors, as well as the roof.



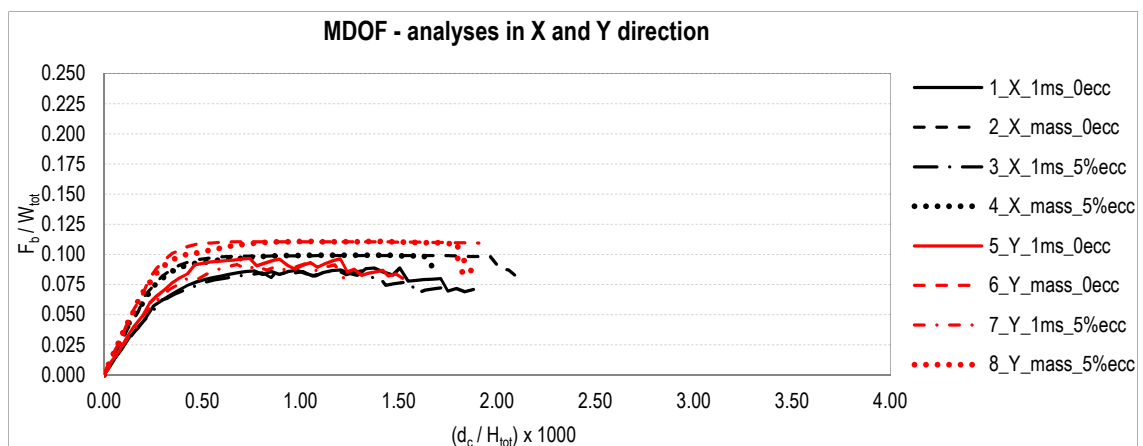
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	D	45.00	1.00	45.00
2 Quality of the resistant system	D	45.00	0.25	11.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	D	45.00	1.00	45.00
6 Planimetric configuration	B	5.00	0.50	2.50
7 Elevation configuration	B	5.00	1.00	5.00
8 Maximum distance among the walls	C	25.00	0.25	6.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

230.00 → I_V **60.13 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=37.58\%$.

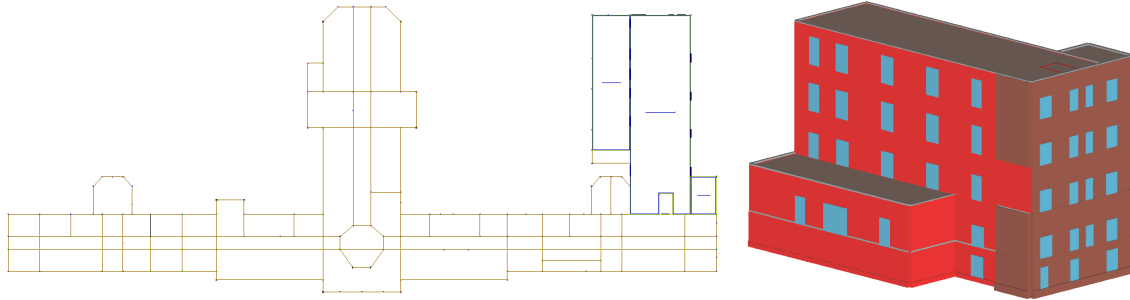
PUSHOVER CURVES



The behaviour in the two main directions is quite comparable; the worst analysis is the n.°3, which gives a $PGA_c=0.1640 g$.

9.8 AOUC CAR 13 04

The building is located in Florence, in the Careggi Hospital Company. The S.U. belongs to a Complex of 4 units, all made of masonry. The S.U. is connected to the other ones of the Complex and it has been realized after the original configuration, before 1950; it is composed of 5 levels (1 basement and 4 levels out of the ground), mostly realized with brick masonry. Most of the floors, as well as the roof, are realized with reinforced concrete and hollow tiles.



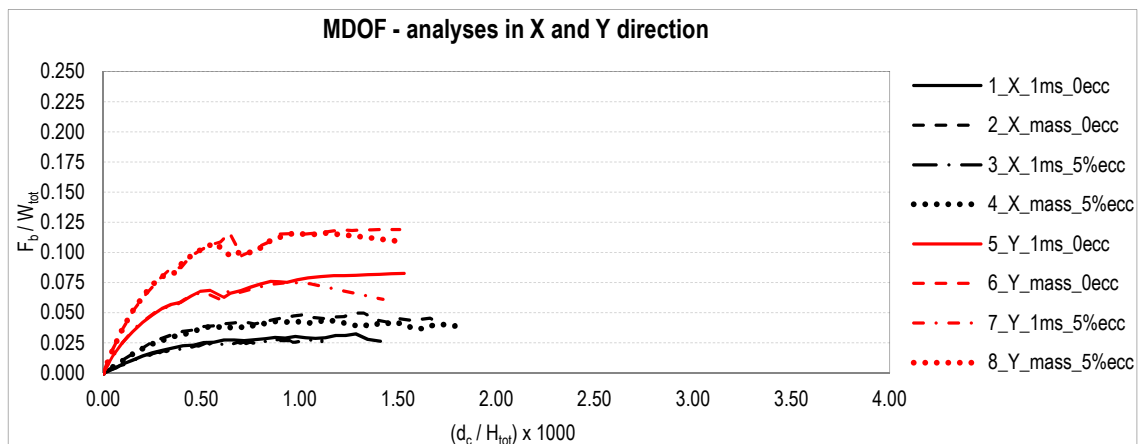
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	C	25.00	0.25	6.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	0.59	8.82
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

172.57 → I_v **45.12 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=34.66\%$.

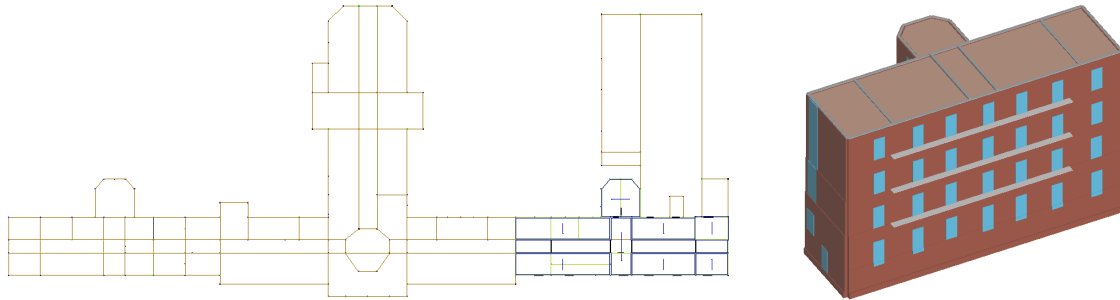
PUSHOVER CURVES



The behaviour is different for the two main directions and even for the two types of lateral load; the worst analysis is the n.°3, which gives a $PGA_c=0.1720 g$.

9.9 AOUC CAR 13 03

The building is located in Florence, in the Careggi Hospital Company. The S.U. belongs to a Complex of 4 units, all made of masonry. The considered S.U. is connected to the other ones of the Complex and it has been realized in the 1930s; it is composed of 5 levels (1 basement and 4 levels out of the ground), all realized with well organized stone masonry. Most of the floors, as well as the roof, are realized with reinforced concrete and hollow tiles.



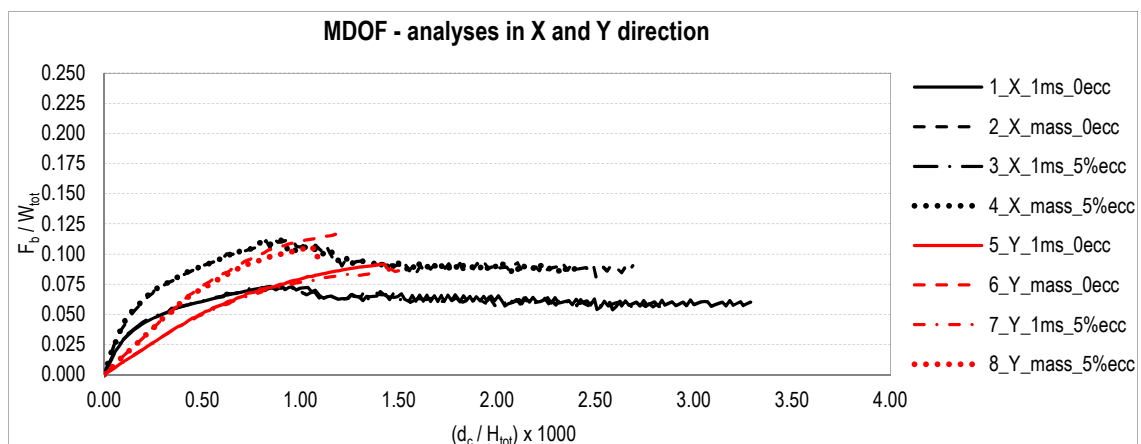
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	1.00	15.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

173.75 → I_V **45.42 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=34.97\%$.

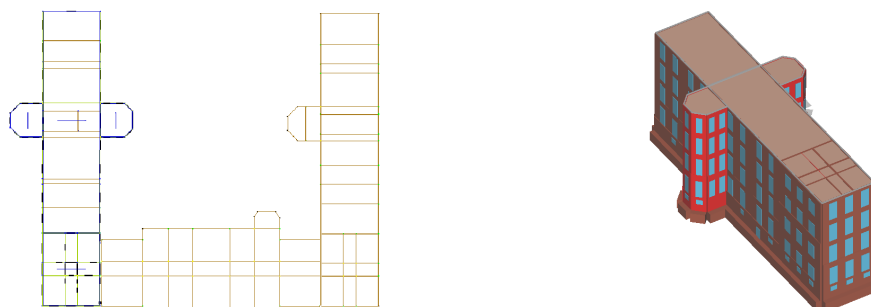
PUSHOVER CURVES



The Y direction analyses show less initial stiffness and ductility comparing to the X ones; the worst analysis is the n.°8, which gives a $PGA_c=0.1770 g$.

9.10 AOUC CAR 26 02

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex composed of 4 S.U., which has been realized in the XX Century, probably before 1950. The considered S.U. is composed of 5 levels, realized with well organized stone masonry with brick layers, except the external semi-circular bodies, made of brick masonry. The floors are made with reinforced concrete and hollow tiles as well as the roof, recently renewed.



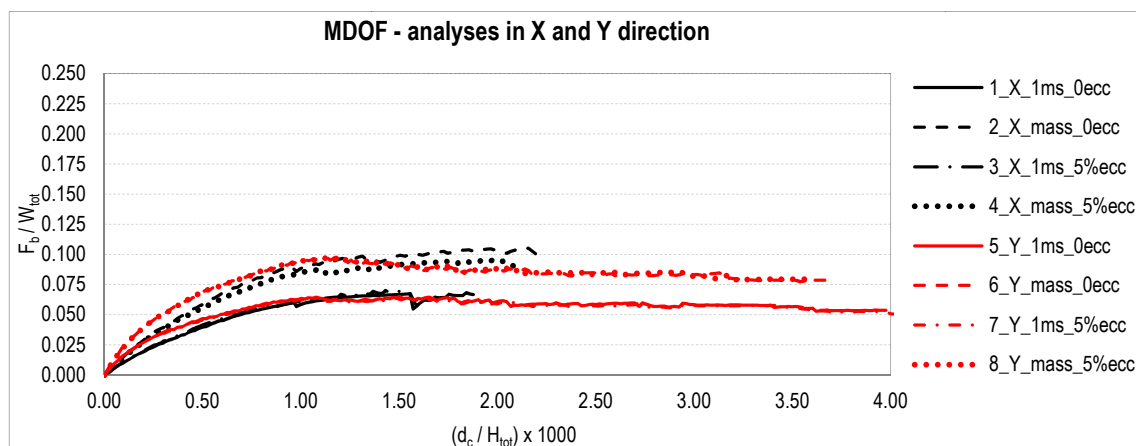
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	A	0.00	1.00	0.00
8 Maximum distance among the walls	B	5.00	0.25	1.25
9 Roof	B	15.00	0.75	11.25
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

143.75 → I_v **38.89 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=28.10\%$.

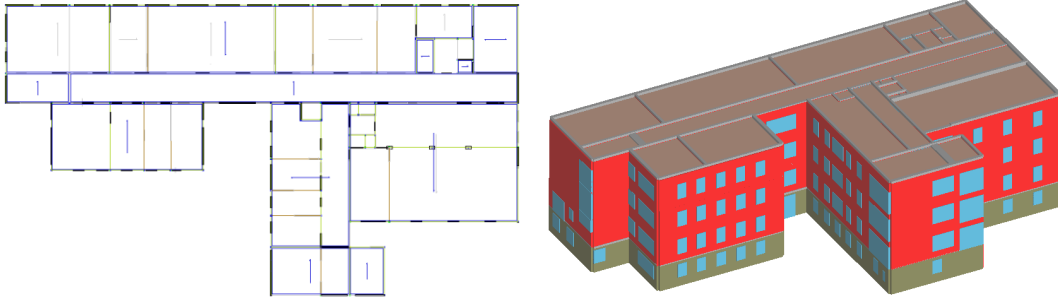
PUSHOVER CURVES



The behaviour in the two direction is similar, even if the Y direction curves show more ductility; the worst analysis is the n.°3, which gives a $PGA_c=0.1950 g$.

9.11 AUSL 3 PES 01 04

The building is located in Pescia, in the Hospital Company n.° 3 of Pistoia. The unit belongs to a Complex composed of 9 S.U. but it is independent from the other ones; it has been built in the 1960s and it is composed of 4 levels: the lower one (basement, partially below the ground level) is made of well organized stone masonry, while the other 3 are realized with good quality brick masonry. All the floors, as well as the roof, are made with reinforced concrete and hollow tiles.



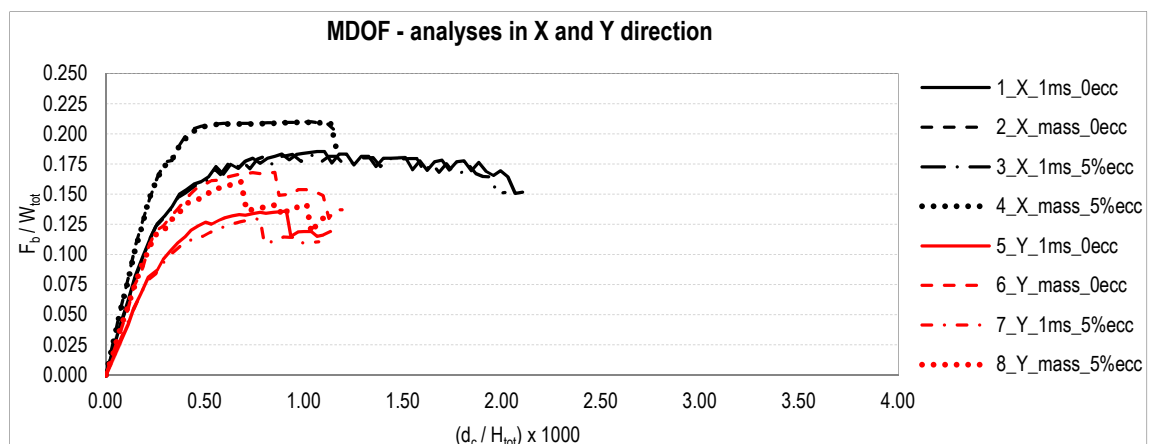
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	A	0.00	0.25	0.00
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	B	5.00	1.00	5.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	0.75	11.25
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

136.25 → I_V **35.62 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=29.08\%$.

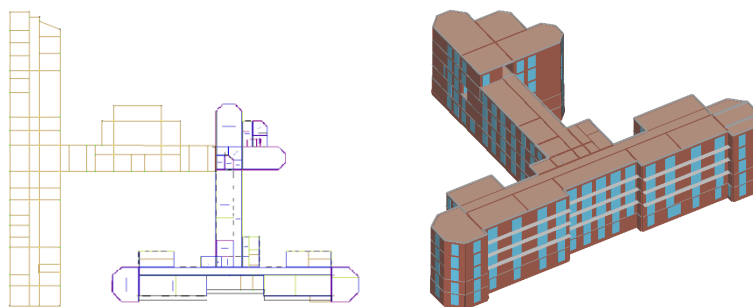
PUSHOVER CURVES



The behaviour in the two direction is different: the X direction shows more rigid and ductile curves than Y one; the worst analysis is the n.°7, which gives a $PGA_C=0.2030 g$.

9.12 AOUC CAR 8b 03

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex originally composed of 4 S.U., realized in the period 1930-1940. The considered S.U. is composed of 6 levels in total, even if almost all the building has 5 levels; the structure is mainly made of well organized stone masonry with brick layers, except some internal walls, made of brick masonry. The floors are made with reinforced concrete and hollow tiles, as well as the roof.



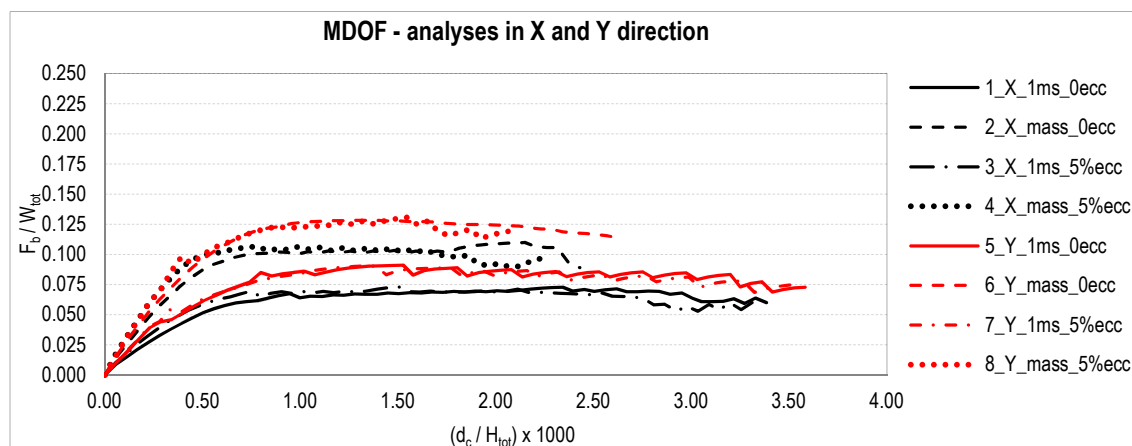
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	C	25.00	0.75	18.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	0.50	7.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

168.75 → I_v **44.12 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=33.66\%$.

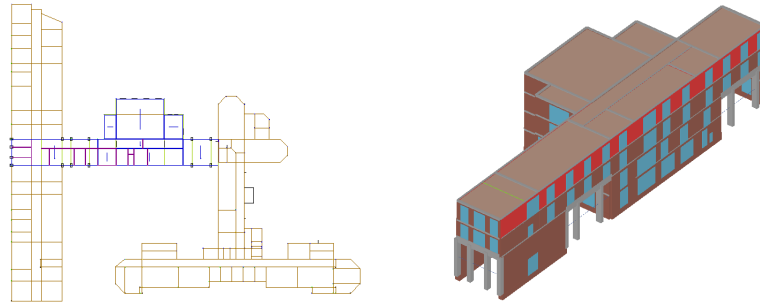
PUSHOVER CURVES



The curves with the 1st modal shape lateral load profile show more ductility than the mass profile ones but they are less rigid; the worst analysis is the n.°3, which gives a $PGA_c=0.2030 g$.

9.13 AOUC CAR 8b 04

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex originally composed of 4 S.U., realized in the period 1930-1940. The considered S.U. is composed of 5 levels, mainly made of well organized stone masonry with brick layers, except some walls, made of brick masonry. The floors are made with reinforced concrete and hollow tiles, as well as the roof. There are some reinforced concrete columns at the ground floor, which sustain a part of the structure.



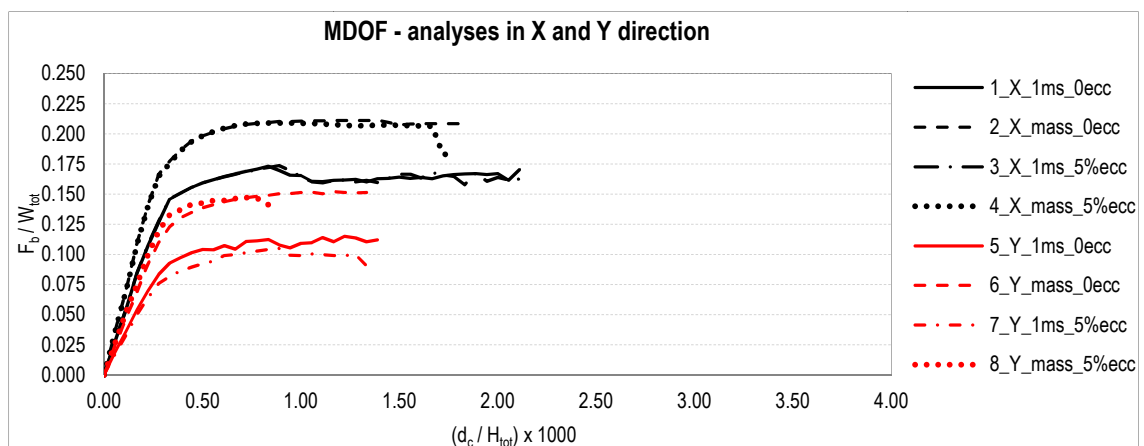
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	C	25.00	0.25	6.25
3 Conventional resistance	C	25.00	1.50	37.50
4 Position of the building and foundations	C	25.00	0.75	18.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	0.50	7.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

143.75 → I_V **37.58 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=27.12\%$.

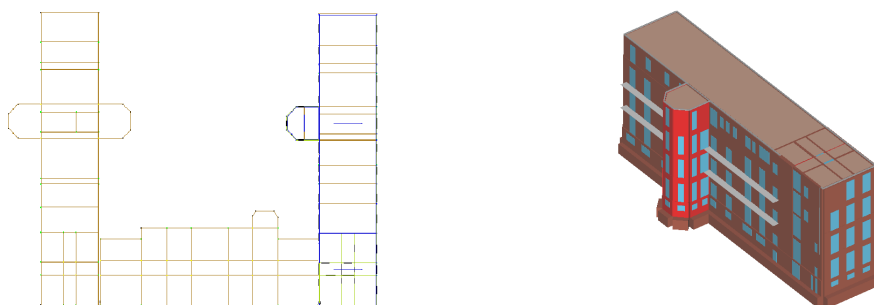
PUSHOVER CURVES



The behaviour is different for the two main directions and even for the two types of lateral load; the worst analysis is the n.°7, which gives a $PGA_c=0.2078 g$.

9.14 AOUC CAR 26 03

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex composed of 4 S.U., which has been realized in the XX Century, probably before 1950. The considered S.U. is composed of 5 levels, realized with well organized stone masonry with brick layers, except the external semi-circular body, made of brick masonry. The floors are made with reinforced concrete and hollow tiles as well as the roof, recently renewed.



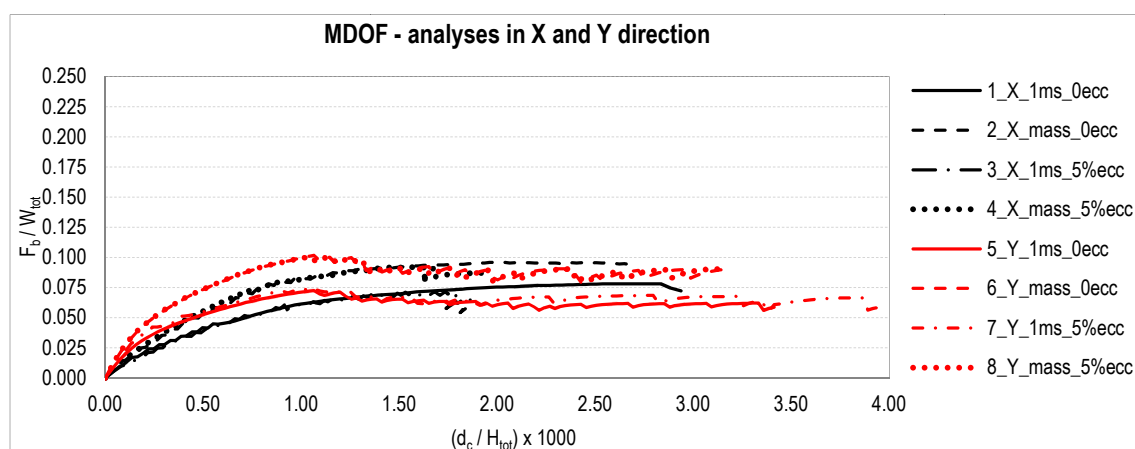
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	A	0.00	1.00	0.00
8 Maximum distance among the walls	B	5.00	0.25	1.25
9 Roof	B	15.00	0.75	11.25
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

143.75 → I_v **38.89 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=28.10\%$.

PUSHOVER CURVES



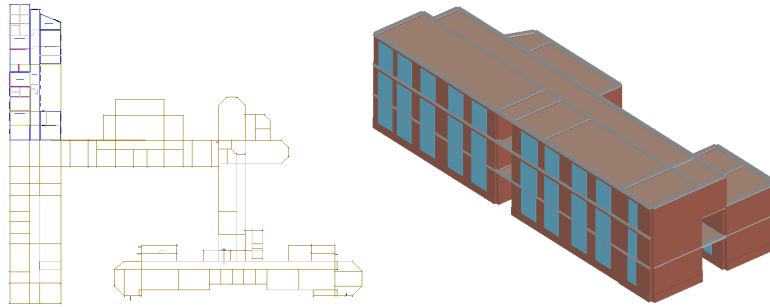
The behaviour in the two direction is similar, even if the curves of the Y direction show a general higher level of ductility; the worst analysis is the n.°3, which gives a $PGA_c=0.2090 g$.

9.15 AOUC CAR 8b 01

The building is located in Florence, in the Hospital Company of Careggi.

The unit belongs to a Complex originally composed of 4 S.U., realized in the period 1930-1940. The considered S.U. is composed of 3 levels, mainly made of well organized stone masonry with brick layers, except some internal walls, made of brick masonry. A part of the structure has only 2 levels.

The floors are made with reinforced concrete and hollow tiles, as well as the roof.



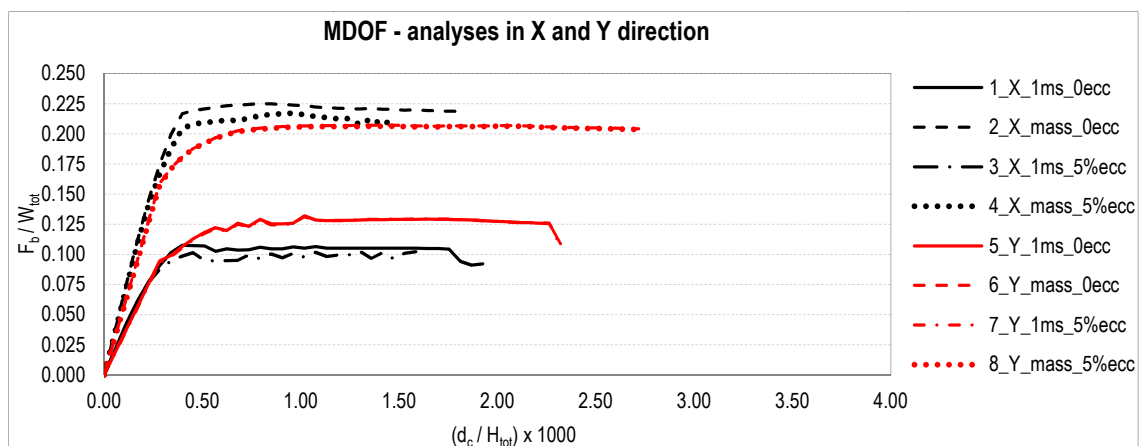
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	C	25.00	1.50	37.50
4 Position of the building and foundations	C	25.00	0.75	18.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	D	45.00	1.00	45.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	0.50	7.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

148.75 → I_V **38.89 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=28.43\%$.

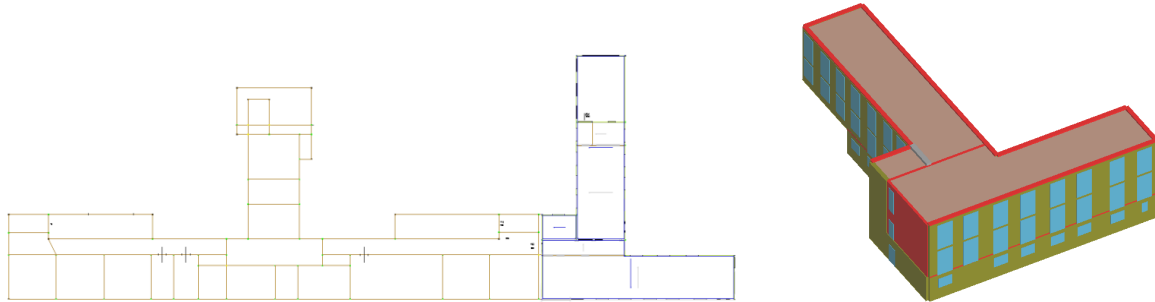
PUSHOVER CURVES



The behaviour is sensibly different both for direction of analysis and type of load profile; the 1st modal shape profile gives always lower shear resistances. The worst analysis is the n.°3, which gives a $PGA_C=0.2100 g$.

9.16 AOUC CAR 4 02

The building is located in Florence, in the Careggi Hospital Company. The S.U. belongs to a Complex composed of 3 units, 2 contiguous ones made of masonry and 1 more recent, made of reinforced concrete. The considered unit has been built around 1930; it is composed of 3 levels (1 basement and 2 levels out of the ground), all of them realized with well organized stone masonry with brick layers (external walls) and some brick masonry walls. Most of the floors are realized with reinforced concrete and hollow tiles.



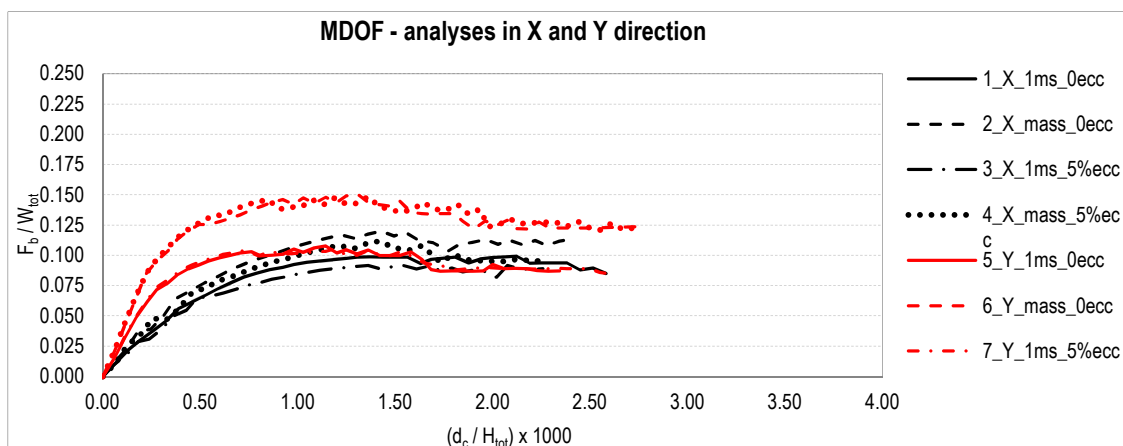
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	C	25.00	1.50	37.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	1.00	15.00
6 Planimetric configuration	D	45.00	0.50	22.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	D	45.00	0.25	11.25
11 State of conservation	B	5.00	1.00	5.00

165.00 → **I_v 43.14 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=29.74\%$.

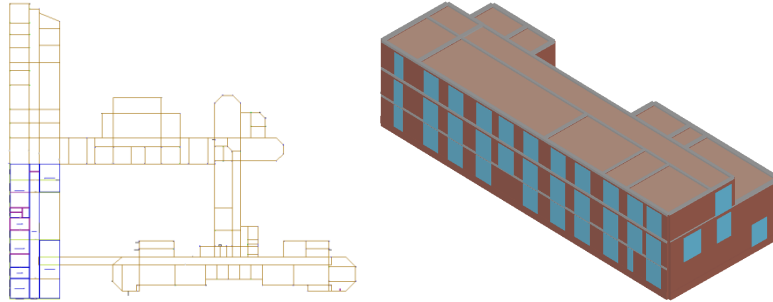
PUSHOVER CURVES



The curves show differences in terms of initial stiffness and maximum shear resistance; the worst analysis is the n.°7, which gives a $PGA_c=0.2112 g$.

9.17 AOUC CAR 8b 02

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex originally composed of 4 S.U., realized in the period 1930-1940. The considered S.U. is similar to the one described before: it is composed of 3 levels, mainly made of well organized stone masonry with brick layers, except some internal walls, made of brick masonry. As seen in the other S.U. (AOUC CAR 8b 02), a part of the structure has only 2 levels. The floors and the roof are made with reinforced concrete and hollow tiles.



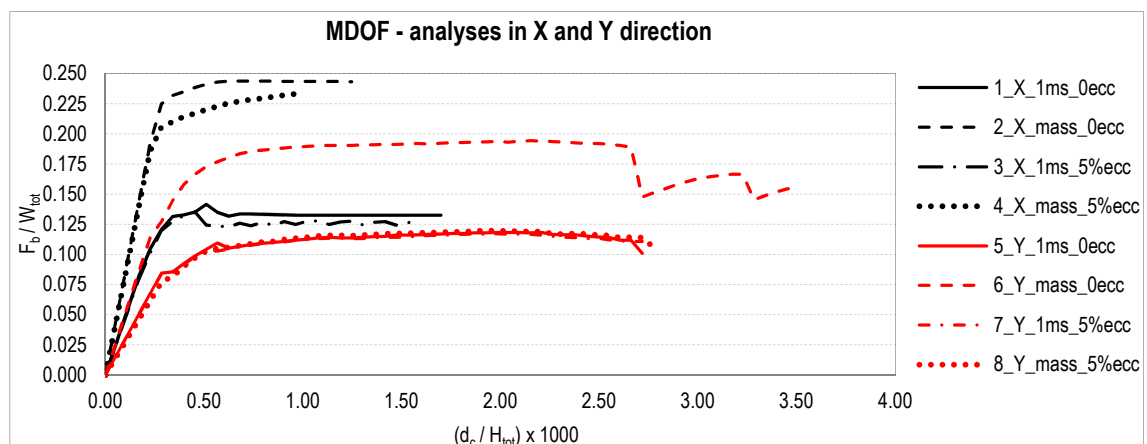
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	C	25.00	1.50	37.50
4 Position of the building and foundations	C	25.00	0.75	18.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	D	45.00	1.00	45.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	0.50	7.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

148.75 → I_V **38.89 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=28.43\%$.

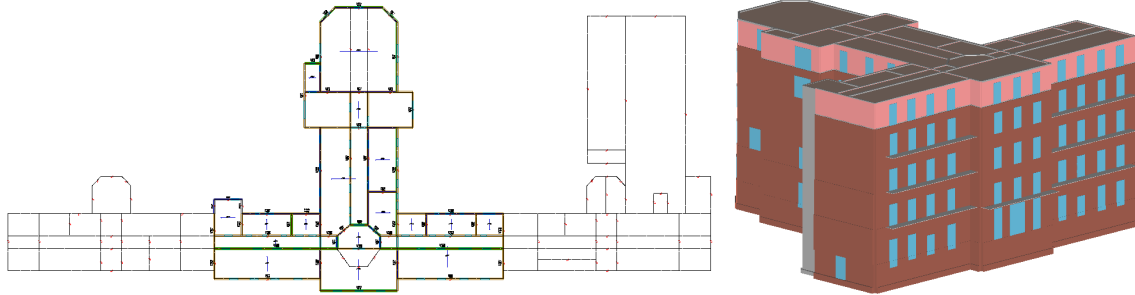
PUSHOVER CURVES



The behaviour is completely different for each curve; the worst analysis is the n.º7, which gives a $PGA_c=0.2195 g$.

9.18 AOUC CAR 13 01

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex composed of a total of 4 structural units; it has been built around the 1930s but the last level has been realized in the 1970s. It is composed of 6 levels (1 basement and 5 levels out of the ground), realized with well organized stone masonry except the last level, made of brick masonry. Floors are made with reinforced concrete and hollow tiles except some cases where there are steel beams and hollow tiles.



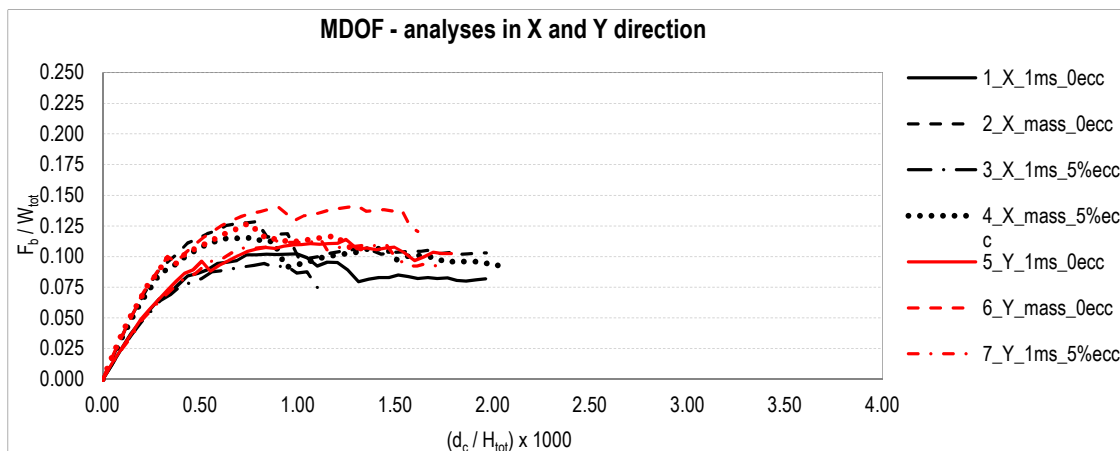
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	C	20.00	1.00	20.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	C	15.00	1.00	15.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	B	5.00	1.00	5.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	C	25.00	0.50	12.50
10 Non structural elements	B	0.00	0.25	0.00
11 State of conservation	B	5.00	1.00	5.00

153.75 → I_v 40.20 %

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=29.74\%$.

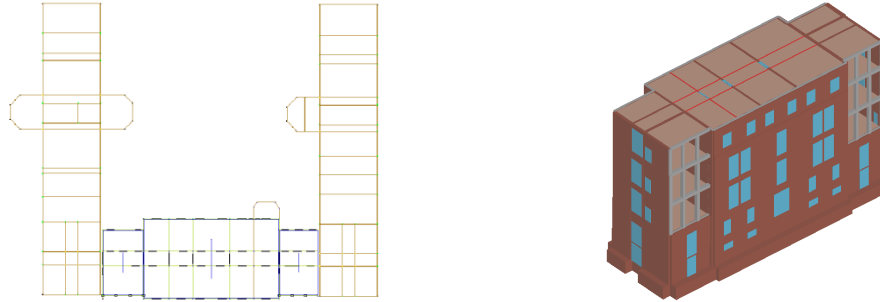
PUSHOVER CURVES



The curves show similar behaviours; the 1st modal shape profile gives lower values of shear resistance; the worst analysis is the n.°3, which gives a $PGA_c=0.2234 g$.

9.19 AOUC CAR 26 01

The building is located in Florence, in the Hospital Company of Careggi. The unit belongs to a Complex composed of 4 S.U., which has been realized in the XX Century, probably before 1950. The considered S.U. is composed of 5 levels, realized with well organized stone masonry with brick layers. For a part of the alignment of the main front, there is one frame structure instead of masonry from the 3rd level. The floors are made with reinforced concrete and hollow tiles while the roof is realized with wooden beams.



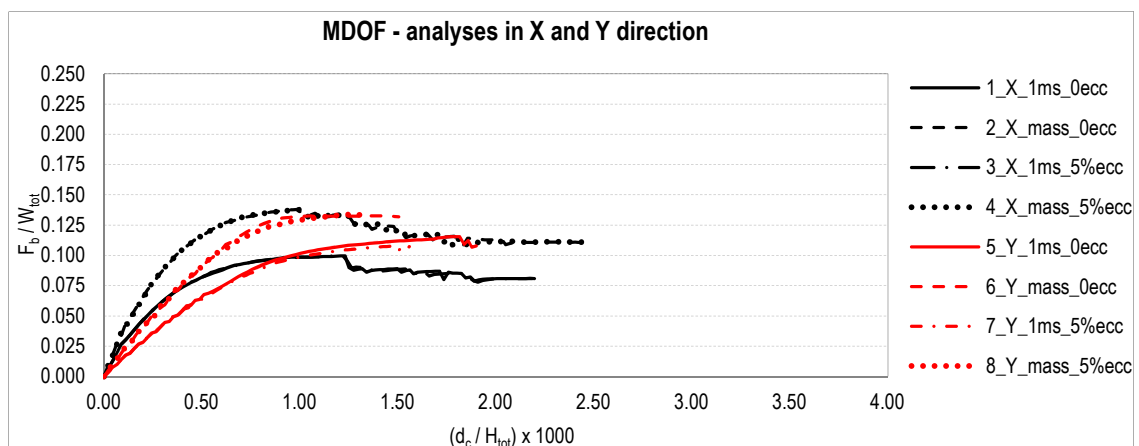
CALCULATION OF THE INDEX OF VULNERABILITY

PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	B	5.00	0.25	1.25
3 Conventional resistance	D	45.00	1.50	67.50
4 Position of the building and foundations	B	5.00	0.75	3.75
5 Typology of floors	B	5.00	1.00	5.00
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	B	5.00	1.00	5.00
8 Maximum distance among the walls	B	5.00	0.25	1.25
9 Roof	B	15.00	0.75	11.25
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	C	25.00	1.00	25.00

143.75 → I_V **37.58 %**

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_V=26.80\%$.

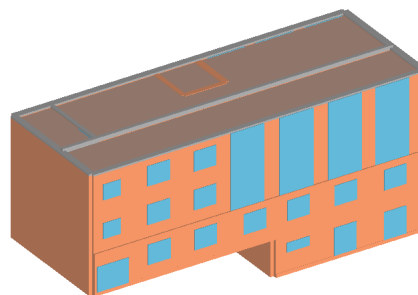
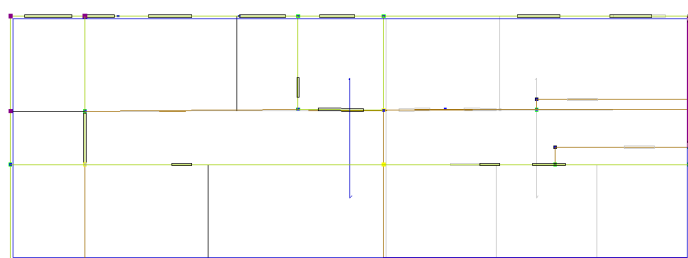
PUSHOVER CURVES



The Y direction has less rigid and less ductile curves than X direction; the worst analysis is the n.°7, which gives a $PGA_c=0.2263 g$.

9.20 AUSL 3 SMP 01 04

The building is located in San Marcello Pistoiese, in the Hospital Company n.° 3 of Pistoia. The unit belongs to a Complex composed of 4 separated S.U.; it has been built in the 1970s as expansion of the neighboring S.U. AUSL 3 SMP 01 03 and it is composed of 4 levels, all of them realized with "half hollow" brick masonry of good quality. All the floors (roof included) are made of reinforced concrete and hollow tiles. The ground floor, due to the natural slope of the land, has a smaller surface than the superior ones.



CALCULATION OF THE INDEX OF VULNERABILITY

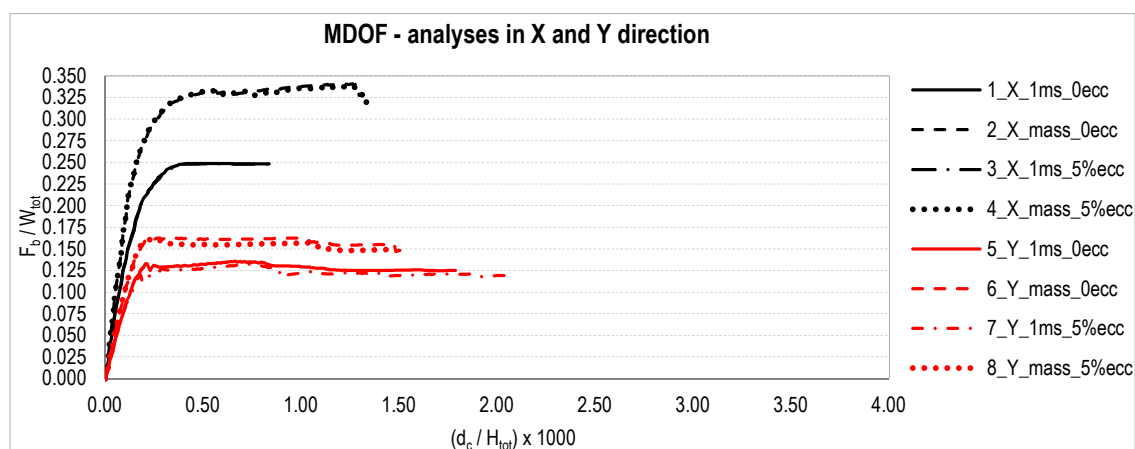
PARAMETER	JUDGMENT	SCORE	WEIGHT	WEIGHTED SCORE
1 Type and organization of the resistant system	B	5.00	1.00	5.00
2 Quality of the resistant system	A	0.00	0.25	0.00
3 Conventional resistance	C	25.00	1.50	37.50
4 Position of the building and foundations	C	25.00	0.75	18.75
5 Typology of floors	B	5.00	0.50	2.50
6 Planimetric configuration	C	25.00	0.50	12.50
7 Elevation configuration	C	25.00	1.00	25.00
8 Maximum distance among the walls	D	45.00	0.25	11.25
9 Roof	B	15.00	1.00	15.00
10 Non structural elements	C	25.00	0.25	6.25
11 State of conservation	A	0.00	1.00	0.00

133.75

→ I_v 34.97 %

Considering the parameters 1, 4, 8, 10 and 11 in the best conditions (for a direct comparison with the analytical model), the index of vulnerability changes value in $I_v=24.18\%$.

PUSHOVER CURVES



The behaviours in the two directions are completely different; even in this case, the 1st modal shape lateral load profile gives lower curves. The worst analysis is the n.°7, which gives a $PGA_c=0.2264 g$.

