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SIMIT

Development of an integrated cross-border Italian-Maltese civil protection network

RESEARCH REPORT

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Sections: Structural Analysis
Seismic Analysis

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1. INTRODUCTION

The research activity carried out within the research project, in perspective of the final goal of the development of an integrated of Italian-Maltese civil protection network, regarded essentially the study, the characterization, the localization and the quantification of seismic and hydro-geological risk.

The main objective declared cannot be in fact achieved without reaching a deep knowledge of the risk scenarios involved in the area that is object of investigation.

This statement arises from the fact that seismic and hydro-geological risks constitute the major component of the activities involving assistance actions carried out by civil protection bodies because of their repetitiveness and the amount of human resources needed to deal with emergencies.

In this context, the possibility of coordinated actions and cooperation between different countries can be an element of fundamental importance, especially if the procedures are based on standardized rules and civil protection plans are characterized by consciousness of the territory and of the possible risks.

The promptness of the response of the entities involved in emergency management is essential to the success of the operations. This feature is however not only achievable by practice exercises aimed to implement a responsiveness system to emergencies, but also through a deep understanding of the existing risks and the major exposure recognized for the urbanized contexts.

This unit has carried out a research activity related to the seismic risk and with particular reference to the assessment of the vulnerability of buildings belonging to a small urban context, in order to define a vulnerability map having territorial scale validity and whose reliability is based on the combination of results coming from different typologies of investigations, experimental and analytic.

The constitution of a map results particularly useful when coordinated emergency actions should be planned, providing a framework of urban areas subjected to the major risks.

The test site chosen for the definition of the vulnerability map is the city centre of the island of Lampedusa. The choice of this site is particularly suitable for the prefixed purposes because of the chance to operate on a large quantity of buildings in a short time and provide reliable assessments through the use of proper tools appropriately calibrated and validated.

The research activity carried out on the island has been divided in 4 phases, characterized by a progressive level of depth of the analysis, listed below:

- Historical, critical, and typological analysis of the urban centre and buildings;
- Assessment of seismic vulnerability (by means of simplified assessments forms);
- Calibration and validation of the adopted vulnerability model (by means of structural identification and analysis of prototype buildings)
- Definition of fragility functions and possible damage scenarios.

The research phases above reported are discussed in detail through the sections of this report. A brief summary of them within the framework of the research work is instead reported in this introductive chapter.

The historical-critical study was aimed at the recognition of the urban evolution of the city centre of Lampedusa over the time and of the regulations succeeded which have changed the constructive and typological framework of buildings.

The subsequent typological analysis of the buildings, performed through several surveys, made it possible to categorize the recurring structural types within the city centre of the island and their similarities and differences in relation to periods of construction. This activity was of particular importance, allowing guiding the choice of the most appropriate tools (to the typology of buildings) for the subsequent phase of vulnerability assessment.

The latter, covered the most of the activity, and has been carried out by the application of evaluation forms already known in the literature and commonly used in Italy for the fast assessment of the vulnerability single buildings and building aggregates. The major output coming from the use of such kinds of vulnerability evaluation forms is constituted by possibility to determine a numerical vulnerability index, suitable to be adopted for the definition of the vulnerability maps themselves and the prediction of damage scenarios (being this functional to the definition of vulnerability (or fragility) curves).

The definition of the fragility functions passes through the preliminary calibration of a certain number of parameters necessary to adapt the vulnerability model to the characteristic building context. For this purpose, the later phases of the research activity regarded the calibration and validation of the vulnerability model used on the basis of direct surveys on buildings and numerical modelling. A verification of the reliability of the methodology for the recognition of the vulnerability is in fact necessary in these cases, in order to support the extrapolation of results coming from the models adopted. In the current study, the empiric calibration operations were carried out by performing the experimental dynamic monitoring of two prototype buildings, through a monitoring system based on the use of tri-axial accelerometers. The analysis of the accelerometric signals recorded on the buildings subjected to environmental noise allowed to identify and characterize their dynamic response and consequently their structural behaviour. Simultaneously, numerical structural models of the buildings have been developed in SAP 2000 NL program in such a way to be consistent with the results of the experimental investigation. The definition of complex models of the prototype buildings made it possible to get as first a validation of the fast-procedure used for the evaluation of the vulnerability indexes. Secondarily, the non-linear analyses performed, allowed the calibration of the fragility functions used for urban context of the island of Lampedusa.

The final outputs of the research are the vulnerability maps for the urban area of the island of Lampedusa, presented in terms of index of vulnerability and peak ground accelerations (associated to early damage and collapse).

2. HISTORICAL AND CRITICAL ANALYSIS OF BUILDINGS

The evolution of the urban settlement of Lampedusa is concentrated in port area, the seat of trade, tourism and fishing activities (Fig. 1).

Fig. 1. Urban centre of area of Lampedusa Island.

The absence of a specific cartography until the first half of the nineteenth century, shows a limited strategic importance of the island from the point of view of the commercial network in that period.

One of the first cartographic representations, dating back to 1843, due to D.B. Sansevite is shown in Fig. 2, a period in which the Bourbon colonization was started. Since it was necessary to give accommodation to 120 people arrived to start the settlement and cultivation of the island, it was initiated the construction of the so-called "seven palaces" (Fig. 3), aligned on a main road axis, and five other buildings on a second line parallel to the first.

With the advent of the Bourbon domination, there was an impetus for the development of the urban centre. The regular meshes arrangement of the building of the area immediately behind the seven buildings (Fig. 4) was delineated in that period.

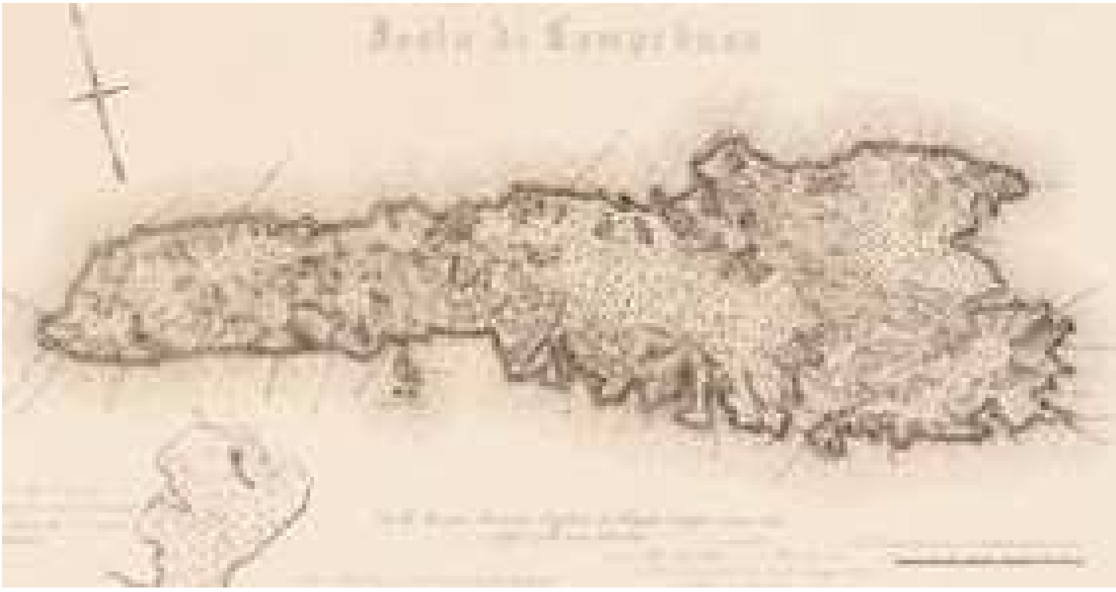


Fig. 2. Cartographic representation of the island of Lampedusa. D.B. Sanvisente, 1843

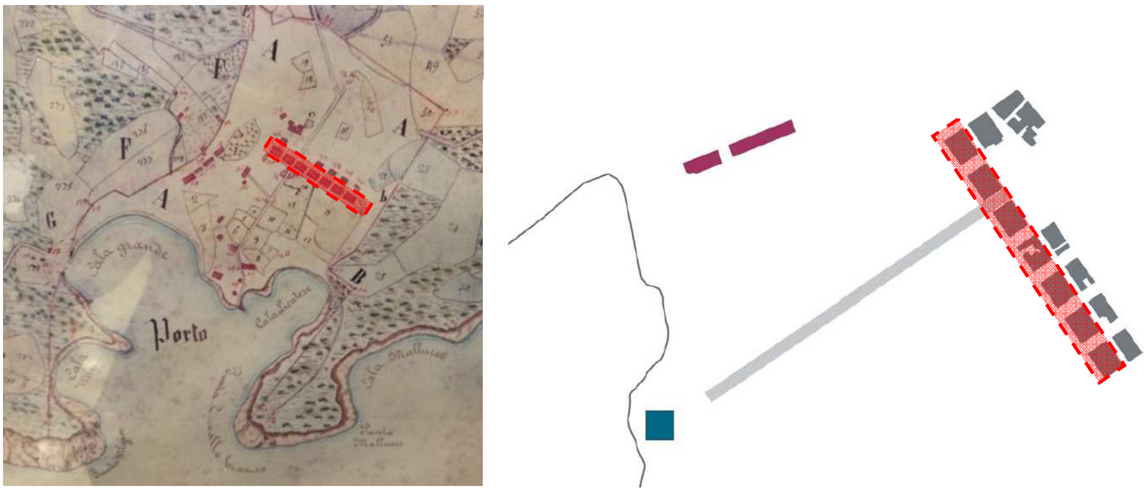


Fig. 3. Native urban arrangement. The “seven palaces”.

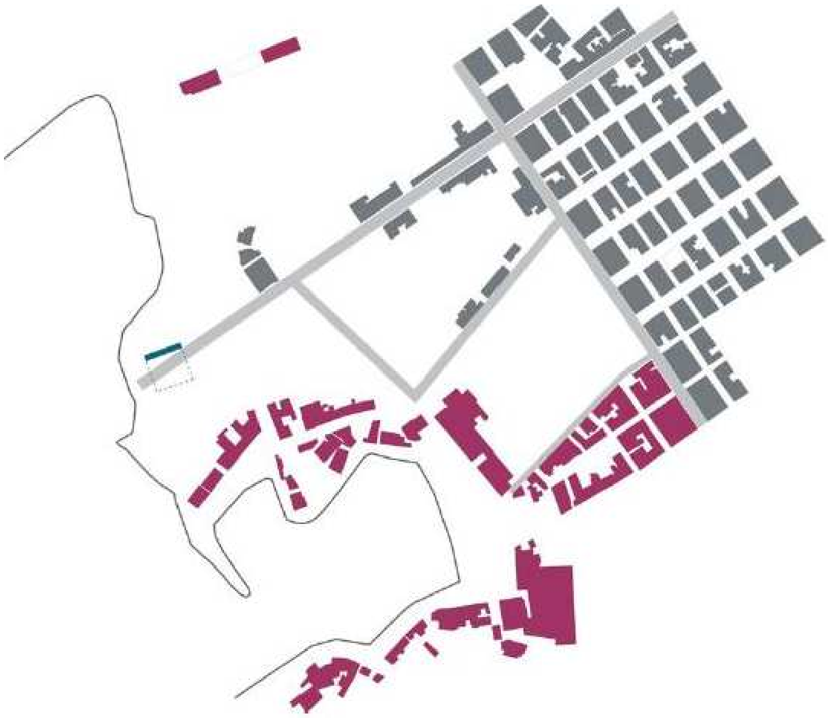


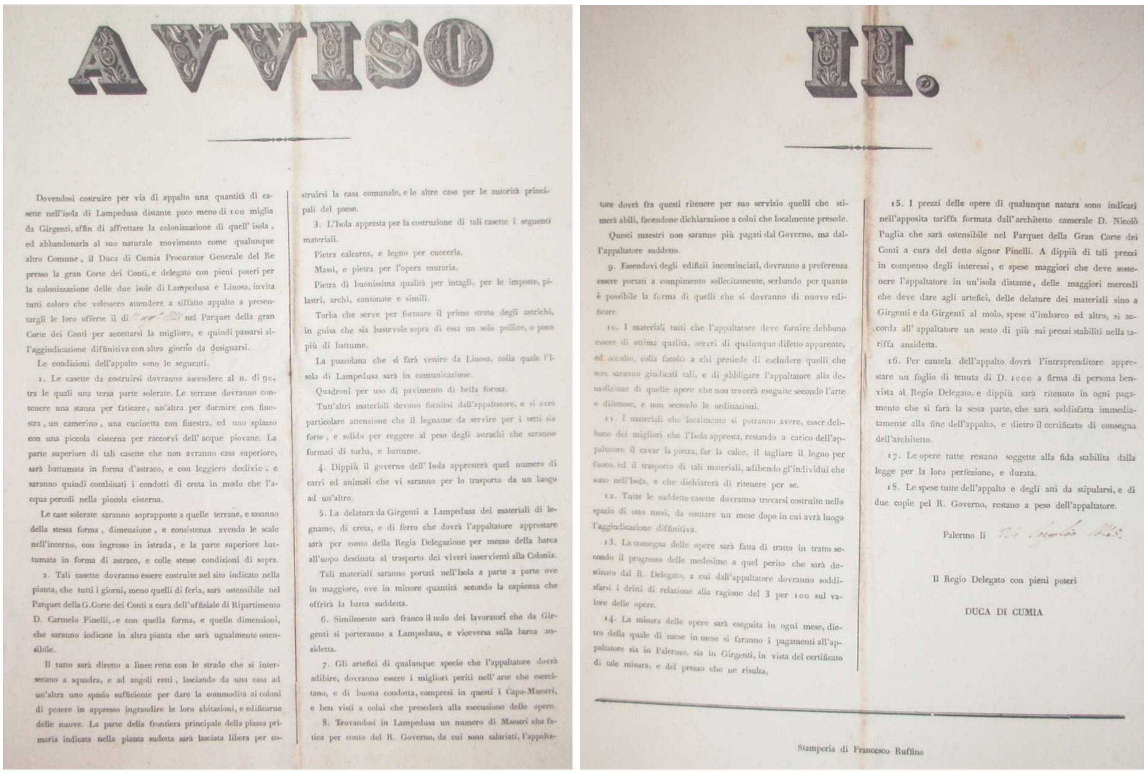
Fig. 4. Regular mesh arrangement of buildings during the Bourbon domination.

One of the finds that tell about the evolution of urban development is an announcement of 1845 for the construction of 90 buildings for the residence of the new settlers of the island (Fig. 5).

At the end of this period the city centre takes a well-defined conformation as well as it is visible by a picture of 1945 (Fig. 6).

In the following period (1950-1970) the island has experienced strong growth due to the increase of tourist flows associated with a further growth of the city centre.

Following the construction of the airport in 1968, the links with the mainland became much more stable having as effect a further rapid expansion up to the present day. A comparison between the configuration of the old town in 1945, 1970, and up to the current zoning is illustrated in Fig. 7.



AVVISO III.

Dovendosi costruire per via di appalto una quantità di casette nell'isola di Lampedusa distante poco meno di 100 miglia da Girgenti, affin di affrettare la colonizzazione di quell'isola, ed abbandonarla al suo naturale movimento come qualunque altro Comune, il Duca di Cumia Procurator Generale del Re presso la gran Corte dei Conti, e delegato con pieni poteri per la colonizzazione delle due isole di Lampedusa e Linosa, invita tutti coloro che volessero attendere a siffatto appalto a presentargli le loro offerte il dì [illegible] nel Parquet della gran Corte dei Conti per accettarsi la migliore, e quindi passarsi all'aggiudicazione diffinitiva con altro giorno da designarsi.

Le condizioni dell'appalto sono le seguenti.

1. Le casette da costruirsi dovranno ascendere al n. di 90, tra le quali una terza parte solerate. Le terrane dovranno contenere una stanza per faticare, un'altra per dormire con finestra, un camerino, una cucinetta con finestra, ed uno spiazzo con una piccola cisterna per raccorvi dell'acque piovane. La parte superiore di tali casette che non avranno casa superiore, sarà lastricata in forma d'astraco, e con leggiero declivio, e saranno quindi combinati i condotti di creta in modo che l'acqua percoli nella piccola cisterna.

Le case solerate saranno soprapposte a quelle terrane, e saranno della stessa forma, dimensione, e consistenza avendo le scale nell'interno, con ingresso in istrada, e la parte superiore lastricata in forma di astraco, e colle stesse condizioni di sopra.

2. Tali casette dovranno essere costruite nel sito indicato nella pianta, che tutti i giorni, meno quelli di feria, sarà ostensibile nel Parquet della G. Corte dei Conti a cura dell'uffiziale di Ripartimento D. Carmelo Pinelli, e con quella forma, e quelle dimensioni, che saranno indicate in altra pianta che sarà ugualmente ostensibile.

Il tutto sarà diretto a linee rette con le strade che si intersecano a squadra, e ad angoli retti, lasciando da una casa ad un'altra uno spazio sufficiente per dare la commodità ai coloni di potere in appresso ingrandire le loro abitazioni, e edificarne delle nuove. La parte della frontiera principale della piazza primaria indicata nella pianta suddetta sarà lasciata libera per costruirsi la casa comunale, e le altre case per le autorità principali del paese.

3. L'Isola appresta per la costruzione di tali casette i seguenti materiali.

Pietra calcarea, e legno per cuocerla.

Massi, e pietra per l'opera muraria.

Pietra di buonissima qualità per intagli, per le imposte, pilastri, archi, cantonate e simili.

Torba che serve per formare il primo strato degli astrichi, in guisa che sia bastevole sopra di essa un solo pollice, o poco più di battume.

La puzzolana che si farà venire da Linosa, colla quale l'Isola di Lampedusa sarà in comunicazione.

Quadroni per uso di pavimento di bella forma.

Tutt'altri materiali devono fornirsi dall'appaltatore, e si avrà particolare attenzione che il legname da servire per i tetti sia forte, e solido per reggere al peso degli astrachi che saranno formati di torba, e battume.

4. Dippiù il governo dell'Isola apresterà quel numero di carri ed animali che vi saranno per lo trasporto da un luogo ad un'altro.

5. La delatura da Girgenti a Lampedusa dei materiali di legname, di creta, e di ferro che dovrà l'appaltatore apprestare sarà per conto della Regia Delegazione per mezzo della barca all'uopo destinata al trasporto dei viveri inservienti alla Colonia.

Tali materiali saranno portati nell'Isola a parte a parte ove in maggiore, ove in minore quantità secondo la capienza che offrirà la barca suddetta.

6. Similmente sarà franco il nolo dei lavoratori che da Girgenti si porteranno a Lampedusa, e viceversa sulla barca anzidetta.

7. Gli artefici di qualunque specie che l'appaltatore dovrà adibire, dovranno essere i migliori periti nell'arte che esercitano, e di buona condotta, compresi in questi i Capo-Maestri, e ben visti a colui che presiderà alla esecuzione delle opere.

8. Trovandosi in Lampedusa un numero di Maestri che fatica per conto del R. Governo, da cui sono salariati, l'appaltatore dovrà fra questi ritenere per suo servizio quelli che stimerà abili, facendone dichiarazione a colui che localmente presede.

Questi maestri non saranno più pagati dal Governo, ma dall'appaltatore suddetto.

9. Essendovi degli edifizii incominciati, dovranno a preferenza essere portati a compimento sollecitamente, serbando per quanto è possibile la forma di quelli che si dovranno di nuovo edificare.

10. I materiali tutti che l'appaltatore deve fornire debbono essere di ottima qualità, scevri di qualunque difetto apparente, ed occulto, colla facoltà a chi presiede di escludere quelli che non saranno giudicati tali, e di obbligare l'appaltatore alla demolizione di quelle opere che non troverà eseguite secondo l'arte o difettose, e non secondo le ordinazioni.

11. I materiali che localmente si potranno avere, esser debbono dei migliori che l'Isola appresta, restando a carico dell'appaltatore il cavar la pietra, far la calce, il tagliare il legno per fuoco, ed il trasporto di tali materiali, adibendo gl'individui che sono nell'Isola, e che dichiarerà di ritenere per se.

12. Tutte le suddette casette dovranno trovarsi costruite nello spazio di otto mesi, da contare un mese dopo in cui avrà luogo l'aggiudicazione diffinitiva.

13. La consegna delle opere sarà fatta di tratto in tratto secondo il progresso delle medesime a quel perito che sarà destinato dal R. Delegato, a cui dall'appaltatore dovranno soddisfarsi i dritti di relazione alla ragione del 3 per 100 sul valore delle opere.

14. La misura delle opere sarà eseguita in ogni mese, dietro della quale di mese in mese si faranno i pagamenti all'appaltatore sia in Palermo, sia in Girgenti, in vista del certificato di tale misura, e del prezzo che ne risulta.

15. I prezzi delle opere di qualunque natura sono indicati nell'apposita tariffa formata dall'architetto camerale D. Nicolò Puglia che sarà ostensibile nel Parquet della Gran Corte dei Conti a cura del detto signor Pinelli. A dippiù di tali prezzi in compenso degli interessi, e spese maggiori che deve sostenere l'appaltatore in un'isola distante, delle maggiori mercedi che deve dare agli artefici, delle delature dei materiali sino a Girgenti e da Girgenti al molo, spese d'imbarco ed altro, si accorda all'appaltatore un sesto di più sui prezzi stabiliti nella tariffa anzidetta.

16. Per cautela dell'appalto dovrà l'intraprenditore apprestare un foglio di tenuta di D. 1000 a firma di persona benvista al Regio Delegato, e dippiù sarà ritenuto in ogni pagamento che si farà la sesta parte, che sarà soddisfatta immediatamente alla fine dell'appalto, e dietro il certificato di consegna dell'architetto.

17. Le opere tutte restano soggette alla fida stabilita dalla legge per la loro perfezione, e durata.

18. Le spese tutte dell'appalto e degli atti da stipularsi, e di due copie pel R. Governo, restano a peso dell'appaltatore.

Palermo li [illegible]

Il Regio Delegato con pieni poteri

DUCA DI CUMIA

Stamperia di Francesco Ruffino

Fig. 5. Announcement for the construction of 90 residential buildings in the city centre of Lampedusa.

Fig. 6. A picture of the city center of Lampedusa in 1945.

1945 Configuration   1970 Configuration   Current Configuration

Fig. 7. Expansion of the city centre of Lampedusa island since 1945

A simplified representation on the evolution of the of the island surfaces destination (Longhi. et al. (2006)) is also shown in Fig. 8, witnessing a growth of about 7.5 times of the urban area since 1850.

The area of the city center because of the tourist season is subject to widely varying levels of population density. The presence of buildings belonging to different periods of construction substantially, leads to the conclusion that the seismic safety of these buildings is significantly different. The intention to define of a map of vulnerability needed to assess the risk scenarios and the most exposed areas is therefore pertinent and necessary to plan assistance interventions in case of natural disasters.

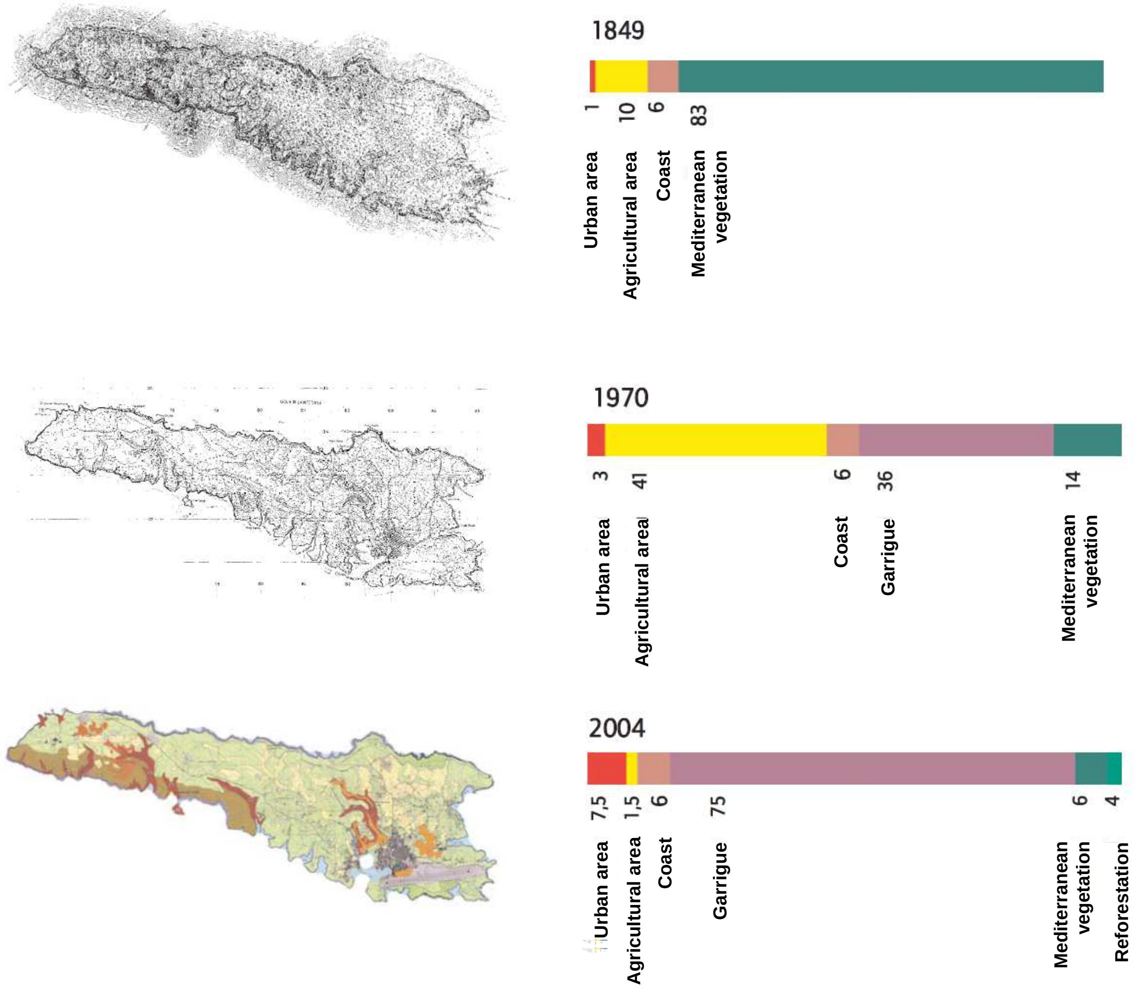


Fig. 8. Expansion of urban area of Lampedusa island (Longhi et al. 2006).

3. ANALYSIS OF BUILDING TYPOLOGIES

The study of the building typologies in the urban settlement is as a fundamental basis for the selection of the most suitable tools for the assessment of the seismic vulnerability and at the same time it allows you to get a framework for the classification of buildings, necessary to facilitate the operations of survey and collection of information. The identification of details and construction methods is often not easy during the investigations in situ and may produce significant slowdowns in the formulation of judgments.

In this way, the chance to take advantage of a preliminary study aimed to associate building typologies and details to certain historical periods, greatly simplifies the procedure and allows a smaller margin of error.

From an early examination on the structural types, it results that over 85% of the buildings in the area of the city centre have masonry primary structure. The remaining 15% are reinforced concrete buildings or in a few cases have mixed primary structure.

This large prevalence of masonry buildings, is due primarily to the availability of natural resources on the island which has a rich geological formation of limestone rock in the subsoil. This circumstance provided the primary building material directly from the quarries (Fig. 9) for many years and especially since the period going from the construction of the "seven palaces" up to 1970, when two factories for the production concrete block are built in the island. The following images (Figs. 10-12) show examples of buildings belonging to the period 1850-1970.

Fig. 9. A limestone quarry


a)

b)

Fig. 10.a-b. Limestone masonry buildings previous to 1970.


a)


b)

Fig. 11.a-b. Limestone masonry buildings previous to 1970.


a)


b)

Fig. 12.a-b. Limestone masonry buildings previous to 1970.

The establishment of factories for the production of lightweight concrete blocks determines from 1970 onwards a change of trend in the choice of the basic building material. Concrete blocks are in fact more light and easy to produce and transport and guarantee a better thermal insulation. Simultaneously to reinforced concrete structures, several lightweight concrete masonry structures are built (Fig. 14 ab).

a)

b)

Fig. 13.a-b. Lightweight concrete masonry buildings after 1970.

The quality of the construction and resistant systems, that greatly influence the vulnerability of the buildings, was subject to a careful analysis aimed at the characterization of some fundamental aspects in the recognition of the vulnerability. As first it is observed that the majority of the buildings characterizing the urban centre of Lampedusa, does not exceed two floors above the ground. The walls are typically compact and the thickness of the walls seems to be adequate. The combination of these two elements allows to state, that in general the stress rate of materials is quite low and the primary structures are generally under safety conditions at least with respect to gravity loads. The reduced height of the buildings also limits the extent of the possible seismic involvement of structures.

By performing a quality analysis of the structural systems, it is observed that the primary structural elements, beyond a degradation due to the aging, present construction details of good workmanship. The floors are made from reinforced concrete slabs (or mixed RC-clay block) which provide a rigid behavior and a suitable distribution of seismic forces between the walls. It is also noted that both the limestone and concrete block masonries are featured with reinforced concrete curbs at each level (Fig. 14-15) able to confer a greater degree of solidification of the walls and reducing the possibility of collapse for out of plane actions.

The surveys also allowed to state that the orthogonal walls are effectively clamped at the corners of the building, ensuring a box-like behavior of the wall structure (Fig. 16).

Fig. 14. RC curbs in limestone masonry buildings.

Fig. 15. RC curbs in concrete block masonry buildings.

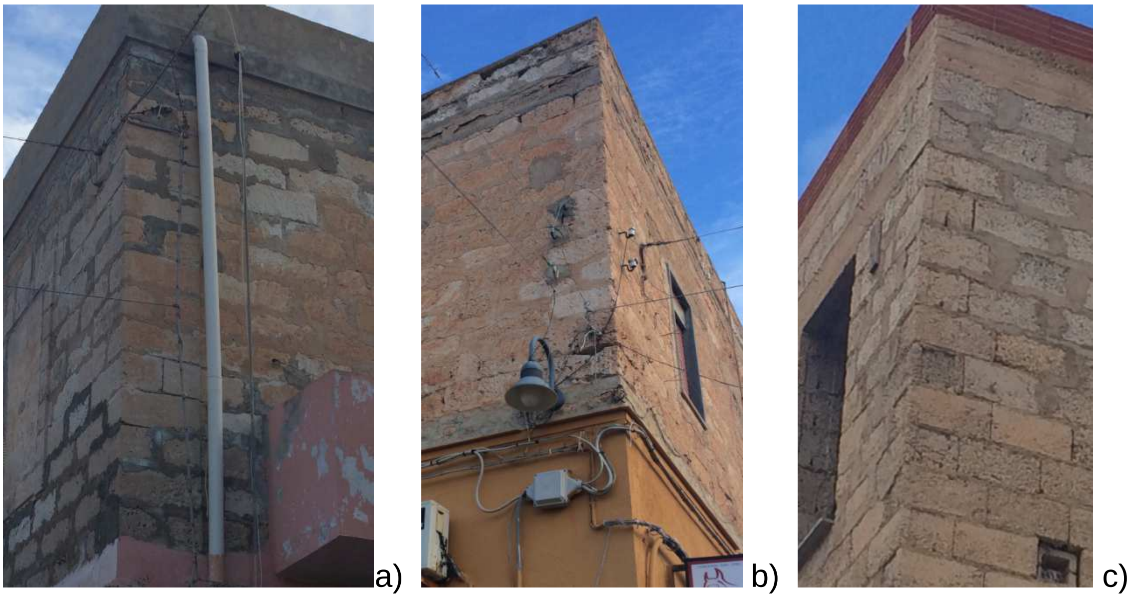


Fig. 16. Effectiveness of corner clumping of buildings. a-b) Limestone masonry; c) Concrete block masonry.

It is also detected the absence of thrusting roofs, a reduced slenderness of the walls and a good planar regularity for almost all of the buildings.

Finally, the walls do not typically exhibit signs of structural instability such as cracks of foundation subsidence.

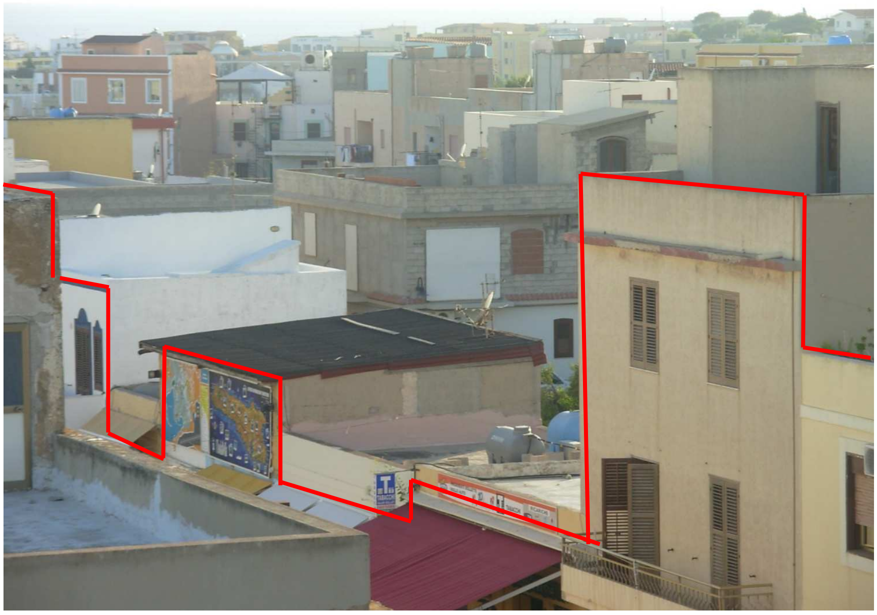
On the other hand it should be observed that in most cases the buildings take the conformation of building aggregates. This is testified by the relevant heterogeneity of the facades (Fig. 17) both from a purely formal point of view, that in the articulation and volume sizes.

The aggregate buildings, that were built over the years with a progressive addition of adjacent bodies, were often also object floor raising interventions.

The buildings resulting by these circumstances are configured as structural bodies having substantially different heights, but also with a different conception of the constitution and distribution of the internal walls. This condition of irregularity over the height, due to significant variations of lateral stiffness and strength from one floor to the next, is recognized as the primary and most important element of vulnerability of buildings belonging to the city centre.

Finally, with regard the reinforced concrete buildings, it can be said that these are characterized by a low rise and not equipped with special seismic detailing. The primary structures often have the typical appearance assumed by frames designed for gravity loads only (Fig. 18). This condition is certainly due to the older non-seismic classification of the site. The RC buildings, although certainly not seismically performing, present anyway a good state of preservation, and have a sufficient regularity in plan and elevation (Fig. 19), which allows to state that they don’t present a critical structural condition.

a)



b)

Fig. 17 a-b. Building aggregates in the city centre of Lampedusa.

As before mentioned, the presence of RC buildings within the city centre of Lampedusa is limited and not representative of the overall vulnerability. The surveys included the however the recognition of both RC and masonry buildings by means of evaluation forms suitable for the detection of the vulnerability of these structural types.

Fig. 18. RC building with frames designed only for gravity loads.

Fig. 19. Three storey RC building.

4. STRATEGIES FOR THE ASSESSMENT OF VULNERABILITY OF THE CITY CENTRE OF LAMPEDUSA

As defined by Dolce and Martinelli (2005), the seismic vulnerability of a building indicates its propensity to suffer damage as a result of a stress state induced by an earthquake. More properly the seismic vulnerability of a building is a characteristic behavior that can be described by a of cause-effect law in which the cause is the earthquake and the effect is the damage. From this definition it follows, the need to identify a parameter for measuring the severity of the earthquake *S* and one measuring the damage *D*, and then to establish a correlation law *D(S)* that is able to provide the level of damage building for each earthquake of a given intensity. There are different possibilities to choose the parameters *S* and *D* and there are many methods that can be used to derive the relationship between the severity of an earthquake and the related damage.

Even with regard to methods for assessing the seismic vulnerability several strategies can be followed, aiming to achieve different purposes, with appropriate tools that, precisely on the basis of their characteristics, can also be distinguished and classified appropriately.

In this case the research for an appropriate tool to assess the vulnerability of the built has led to the choice to use the evaluation forms developed by INGV / GNDT - National Group for the Defence against Earthquakes. In particular, the need correlate scientific information with on-site surveys has requested the use of forms defined "second level forms" since their compilation requires the definition and evaluation of some parameters by simplified numerical calculations.

The assessment of the vulnerability based on the 2nd level GNDT forms is an indirect method, since it is based on the evaluation of a vulnerability index which is a conventional measure of the propensity to damage; the correspondence between severity and damage in this case is deterministic and is represented by the fragility curves associated to each index value, that correlate, the seismic ground acceleration (or the macro-seismic) with the level of damage expressed as a percentage of loss of economic value. This methodology makes use of a numerical index of global vulnerability, calculated by summing the contributions of vulnerability scores of 11 parameters measured and related to some characteristic features of typical seismic behavior of masonry buildings. As common to all methods based on the index of vulnerability, it has the disadvantage of a step more than the methods of the direct type, and also involves a more laborious recognition phase.

However, the amount of information contained in the GNDT forms allows to make more proper judgments and also to use different techniques of investigation to define the severity-damage laws. In this way, the methodology can be defined as hybrid-type. The vulnerability index that is obtained also allows to compare buildings and to establish graded lists or maps of vulnerability, as in the case of the present study. In particular, the choice of GNDT 2nd level forms has been basically determined on the basis of the following requirements that have been placed at the base of the research:

• Possibility of detecting pre-earthquake vulnerability

• Adequate amount of information about the parameters that affect the vulnerability;

•Compilation without specific investigations or detailed surveys on buildings;

• Consolidated use of the forms on the national territory;

• Possible adaptation of the forms to particular needs found in the area;

• Availability of the same type of forms masonry and RC structures.

4.1 GNDT 2nd level vulnerability assessment forms for masonry buildings

The GNDT vulnerability assessment form for masonry buildings (Fig. 20) is composed of 11 parameters below described in detail. Each parameter is associated with a class of vulnerability between A and D, taking into account that A represents the best condition and D the worst. At the same time it is assigned a class of quality of the information used to establish the class of vulnerability. The vulnerability classes are characterized by increasing scores identified by the symbol c_{vi}, while individual parameters are weighted by a numerical weight (p_i), which would establish the influence within the overall assessment of the vulnerability. The Tab. 2 shows the list of the 11 parameters of vulnerability, the scores assigned to the classes and weights. The parameters of vulnerability and scores associated refer to those proposed in the standard GNDT procedure. Regarding the weights, the procedure provides that only those relative to the parameters 1, 2, 3, 4, 6, 8, 10 and 11 are established, while those related to the parameters 5, 7 and 9 have to be calibrated according to the conditions detected, and therefore those highlighted in red in Tab. 1 are the values which in this case it was considered suitable to assume on the basis of the conditions of greater or lesser criticalities detected for the buildings. In particular, it was decided to penalize the conditions of irregularity in elevation that may be the cause of activation of storey mechanisms in the presence of seismic actions.

The vulnerability index V is defined as

$$V = \sum_i c_{vi} \, p_i$$

Taking values between 0 and 328.5. The vulnerability index is however usually expressed in cents, so it can be also defined a normalized vulnerability index $\overline{V}$ as

$$\overline{V} = \frac{V}{382.5} \times 100$$

The attribution vulnerability class for the individual parameters may come from simple observations on the structure or may involve simplified calculations to determine unambiguously the class and its associated score. In the following pages, the operations necessary for the identification of classes of vulnerabilities, related to the above parameters therefore, are described in detail. It should be specified that the compilation of the forms can be done with different levels of detail of the information, for sake of time or for logistical reasons. Therefore, for each parameter, the compilation of the forms provides to assign a rating (E, M, B, A) on the quality of information that has allowed the assignment of the class according to the scale described below.

E – High Quality:

Information predominantly direct (measurements carried out on site, reliable reading of drawings, direct vision of the information elements) with a degree of reliability near certainty.

M- Medium Quality:

Information mainly derived (indirect readings such as those derived from photographs, measurements derived from non-executive drawings, non-destructive surveys of poor reliability, direct readings of similar situations, oral information from people) with a degree of reliability is intermediate between the previous (E) and the following (B).

B – Low Quality:

Information mainly assumed (measures derived from reasonable assumptions, such as those on the usual manner and the most frequent design choices, oral information) with a degree of reliability little more than a purely random selection of the class.

A – Missing information:

With a degree of reliability around the limit of a random choice. In these cases, the evaluation of the detector is only for reference.

PARAMETER		Class C_{vi}				Weight
		A	**B**	**C**	**D**	$\mathbf{p_i}$
1	Type and organization of the resisting system	0	5	20	45	**1,00**
2	Quality of the resisting system	0	5	25	45	**0,25**
3	Conventional resistance	0	5	25	45	**1,50**
4	Position of the building and foundations	0	5	15	45	**0,75**
5	Floors	0	5	25	45	**0,75**
6	Configuration in plan	0	5	25	45	**0,50**
7	Configuration in elevation	0	5	25	45	**1,75**
8	Walls maximum interaxis	0	5	25	45	**0,25**
9	Roof	0	15	25	45	**0,5**
10	Non-structural elements	0	0	25	45	**0,25**
11	Current conditions	0	5	25	45	**1,00**

Table 1. Parameters for the identification of vulnerability of masonry buildings and related scores and weights.

G.N.D.T. – SCHEDA DI VULNERABILITÀ DI 2° LIVELLO (MURATURA)

GNDT

Codice ISTAT Provincia [1]|__|__|__| Codice ISTAT Comune [3]|__|__|__| Scheda N° [7]|__|__|__|__|__|

	PARAMETRI	Classi	Qual. Inf.	ELEMENTI DI VALUTAZIONE
1	TIPO ED ORGANIZZAZIONE DEL SISTEMA RESISTENTE (S.R.)	11|__|	22|__|	Norme nuove costruzioni (Clas. A) 33 [1] Norme riparazioni (Clas. A) [2] Cordoli e catene tutti i livelli (Clas. B) [3] Buoni ammorsam. fra muri (Clas. C) [4] Senza cordoli cattivi ammors. (Clas. D) [5]
2	QUALITÀ DEL S.R.	12|__|	23|__|	(vedi manuale) 34|__|
3	RESISTENZA CONVENZIONALE	13|__|	24|__|	Numero di piani N 35|__|__| Area totale coperta A_t (mq) 37|__|__|__|__.| Area A_x (mq) 41|__|__|__.| Area A_y (mq) 44|__|__|__.| τ_k (t/mq) 47|__|__|__.| Alt. media interpiano h (m) 50|__.|__| Peso specifico pareti p_m (t/mc) 52|__.|__| Carico permanente solai p_s (t/mq) 54|__.|__|
4	POSIZIONE EDIFICIO E FONDAZIONE	14|__|	25|__|	Pendenza percentuale del terreno 56|__|__.| Roccia Fondazioni: Si [1] No [2] Terr. sciolto non sping Fond. Si [3] No [4] Terr. sciolto spingente Fond. Si [5] No [6] Differen. max di quota Δh (m) 59|__|__|__.|
5	ORIZZONTAMENTI	15|__|	26|__|	Piani sfalsati Si [1] No [2] Orizzontamenti rigidi e ben collegati 63 [1] Orizzontam. deformabili e ben collegati [2] Orizzontam. rigidi e mal collegati [3] Orizzontam. deformabili e mal collegati [4] % Orizzontam. rigidi e ben collegati 64|__|__.|
6	CONFIGURAZIONE PLANIMETRICA	16|__|	27|__|	Rapporto percentuale $\beta_1 = a/l$ 66|__|__.| Rapporto percentuale $\beta_2 = b/l$ 70|__|__.|
7	CONFIGURAZIONE IN ELEVAZIONE	17|__|	28|__|	% aumento (+) o diminuzione(-) di massa 74|__|__|__.| Rapporto percentuale T/H 77|__|__.| Percentuale superficie porticata 79|__|__.| Piano terra porticato Si [1] No [2]
8	D_{max} MURATURE	18|__|	29|__|	Rapporto massimo l/s 82|__|__.|
9	COPERTURA	19|__|	30|__|	Copert. non sp. 84 [Ø] poco sp. [1] sp. [2] Cordoli in copertura Si 85 [1] No [2] Catene in copertura Si 86 [1] No [2] Carico perman. coper. p_c (t/mq) 87|__.|__|__| Lungh. appoggio coper. l_s (m) 90|__|__|__.| Perimetro copertura l (m) 93|__|__|__.|
10	ELEM. NON STRUTT.	20|__|	31|__|	(Vedi manuale)
11	STATO DI FATTO	21|__|	32|__|	(Vedi manuale)

SCHEMI – RICHIAMI

Parametro 3. Resistenza convenzionale

Tipologia strutture verticali τ_k (t/mq)

________ ______

________ ______

________ ______

________ ______

Minimo tra A_x ed A_y A (mq) ______

Massimo tra A_x ed A_y A (mq) ______

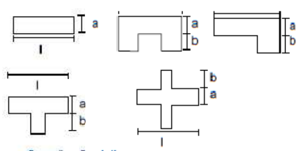
Coeff. $a_0 = A/A_t$ ______ Coeff. $\gamma = B/A$ ______

$q = (A_x + A_y)\, h\, p_m / A_t + p_s$ ______

$$C = \frac{a_0 \tau_k}{qN} \sqrt{1 + \frac{qN}{1{,}5\, a_0\, \tau_k\, (1+\gamma)}}$$

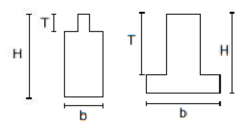
$\alpha = C/0{,}4$ ______

Parametro 6. Configurazione planimetrica

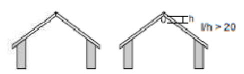


$\beta_1 = a/l$ $\beta_2 = b/l$

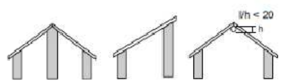
Parametro 7. Configurazione in elevazione



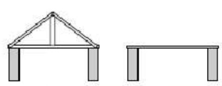
Parametro 9. Copertura



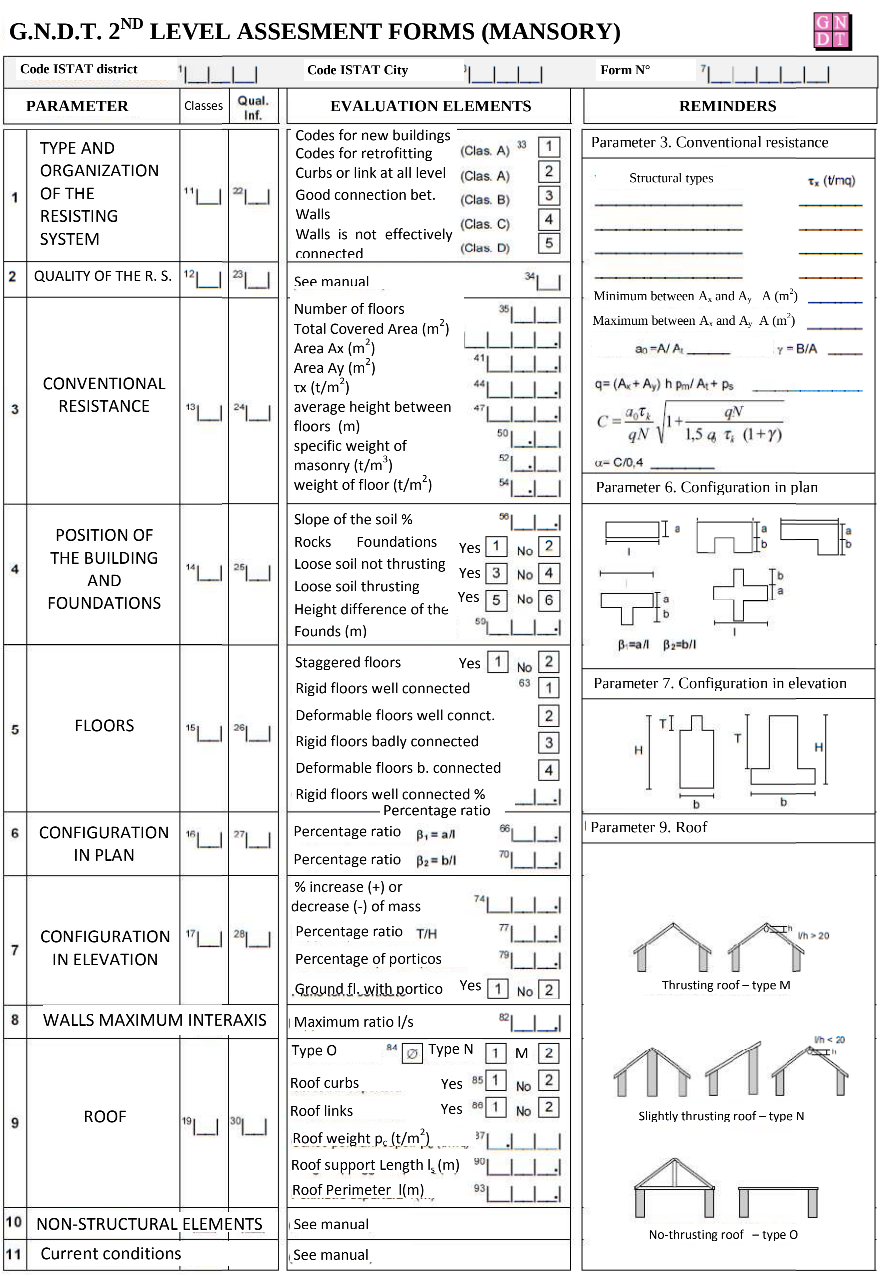
Coperture spingenti (tipologia M)



Coperture poco spingenti (tipologia N)



Coperture non spingenti (tipologia O)



G.N.D.T. 2ND LEVEL ASSESMENT FORMS (MANSORY)

GNDT

Code ISTAT district 1 | Code ISTAT City | Form N° 7

	PARAMETER	Classes	Qual. Inf.	EVALUATION ELEMENTS
1	TYPE AND ORGANIZATION OF THE RESISTING SYSTEM	11	22	Codes for new buildings / Codes for retrofitting (Clas. A) 33 [1]; Curbs or link at all level (Clas. A) [2]; Good connection bet. Walls (Clas. B) [3]; Walls is not effectively connected (Clas. C) [4]; (Clas. D) [5]
2	QUALITY OF THE R. S.	12	23	See manual 34
3	CONVENTIONAL RESISTANCE	13	24	Number of floors 35; Total Covered Area (m^2); Area Ax (m^2) 41; Area Ay (m^2) 44; τx (t/m^2) 47; average height between floors (m) 50; specific weight of masonry (t/m^3) 52; weight of floor (t/m^2) 54
4	POSITION OF THE BUILDING AND FOUNDATIONS	14	25	Slope of the soil % 56; Rocks Foundations Yes [1] No [2]; Loose soil not thrusting Yes [3] No [4]; Loose soil thrusting Yes [5] No [6]; Height difference of the Founds (m) 59
5	FLOORS	15	26	Staggered floors Yes [1] No [2]; Rigid floors well connected 63 [1]; Deformable floors well connct. [2]; Rigid floors badly connected [3]; Deformable floors b. connected [4]; Rigid floors well connected % — Percentage ratio
6	CONFIGURATION IN PLAN	16	27	Percentage ratio $\beta_1 = a/l$ 66; Percentage ratio $\beta_2 = b/l$ 70
7	CONFIGURATION IN ELEVATION	17	28	% increase (+) or decrease (-) of mass 74; Percentage ratio T/H 77; Percentage of porticos 79; Ground fl. with portico Yes [1] No [2]
8	WALLS MAXIMUM INTERAXIS			Maximum ratio l/s 82
9	ROOF	19	30	Type O 84 [∅] Type N [1] M [2]; Roof curbs Yes 85 [1] No [2]; Roof links Yes 86 [1] No [2]; Roof weight p_c (t/m^2) 37; Roof support Length l_s (m) 90; Roof Perimeter l(m) 93
10	NON-STRUCTURAL ELEMENTS			See manual
11	Current conditions			See manual

REMINDERS

Parameter 3. Conventional resistance

Structural types — τ_k (t/mq)

Minimum between A_x and A_y A (m^2)

Maximum between A_x and A_y A (m^2)

$a_0 = A/A_t$ — $\gamma = B/A$

$q = (A_x + A_y)\, h\, p_m / A_t + p_s$

$$C = \frac{a_0 \tau_k}{qN}\sqrt{1 + \frac{qN}{1{,}5\, a_0\, \tau_k\, (1+\gamma)}}$$

$\alpha = C/0{,}4$

Parameter 6. Configuration in plan

$\beta_1 = a/l$ $\beta_2 = b/l$

Parameter 7. Configuration in elevation

Parameter 9. Roof

Thrusting roof – type M

Slightly thrusting roof – type N

No-thrusting roof – type O

Fig. 20. GNDT 2nd level assessment forms (masonry).

PARAMETER 1 - TYPE AND ORGANIZATION OF THE RESISTING SYSTEM.

This item assesses the degree of organization of the vertical elements, regardless of the material and the respective characteristics of masonry: the most significant element is the presence and effectiveness of the connections between orthogonal walls, able to ensure box-type behaviour of the structure. Therefore, the four classes are defined as follows:

Class A: Buildings constructed in accordance with the seismic regulations for new buildings

-Buildings with consolidated and/or repaired masonries in accordance with the requirements of the codes in force;

Class B: Buildings presenting at all levels and all free sides, connections made through external curbs or links and connections able to transmit vertical shear actions

Class C: Buildings that, while not presenting curbs or links at all levels, are provided by effective connections between orthogonal walls;

Class D: Buildings with orthogonal walls is not effectively connected

PARAMETER 2 - QUALITY OF THE RESISTING SYSTEM.

This item takes into account the different types of masonry most frequently used, differentiating, in a qualitative way, the characteristics of strength, in order to assess the efficiency. The attribution of the building to one of four classes is carried out as a function of two factors: on the one hand the type of material and shape of the elements constituting the walls, on the other hand the homogeneity of material and size for the whole extension of the wall. With regard to the second factor it should be noted that the presence of recurring bricks extended to the whole thickness of the wall does not constitute an element of inhomogeneity for a stone masonry. Similarly the presence of stones of substantially greater size at openings or corners of a building is not considered an inhomogeneity element.

Class A: Clay brick masonry of good quality, stone or calcarenite masonry well squared, homogeneous in their whole extension; Double curtain masonry well meshed and homogeneous, provided with links between the two sheets;

Class B: Clay brick, calcarenite or stone masonry, well squared but not homogeneous; double curtain masonry provided with connections between the two sheets.

Class C: Stone masonry stone roughly squared or clay brick masonry of bad quality; double curtain masonry (stone or calcarenite) well meshed but without links between the two sheets.

Class D: Masonry with irregular stones; brick masonry of poor quality with inclusion of pebbles; Double curtain masonry badly meshed with no links between the two sheets.

PARAMETER 3 - CONVENTIONAL RESISTANCE

Assuming a perfect box-type behavior, the assessment of the strength of a masonry building with respect to seismic actions can be carried out with reasonable reliability. The procedure described below is a necessary simplification and requires the collection of data specified below relating to the floor at which the verification is carried out:

N Number of floors including the from the one verified;

A_t average covered area above the verified floor;

A_x, A_y total resisting area of the walls in two orthogonal directions

The length of the resisting elements is measured between the interaxis of the orthogonal walls. If one indicates:

A the minimum value between A_x and A_y; B the maximum value between A_x e A_y; $\mathbf{a_o}$ = A/A_t; $\boldsymbol{y}$= B/A, it can be demonstrated that the ratio C between the ultimate shear at verification floor and the weight P of the portion of the building above is given by:

$$C = \frac{a_0 \tau_k}{qN} \sqrt{1 + \frac{qN}{1.5 a_0 \tau_k (1+\gamma)}}$$

In the previous expression, besides the parameters already defined, it appears the value of the shear strength, τ_k, associated to the masonry typology and the value q of the average weight per unit of covered area, a building level (sum of the weight of a floor and a masonry inter-floor). The parameter q is evaluated as a function of the average specific weight of the masonry p_m, the average weight per unit area of the floor p_s and the average height of an inter-floor as follows

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

For the determination of the reference shear strength values, τ_k, in the absence of direct experimental information, the reference is made to the average or minimum values of shear strength in the absence of vertical loads specified in the Ministerial Decree - DM 14.01.2008 §C8A.2.1 (Tab. 2).

The attribution of a building at one of four classes is made on the basis of the ratio α = C/C between the value of C, obtained as above indicated, and the C reference value, assumed as 0. The four classes are defined in terms of α in the following way:

Class A: - α ≤ 1

Class B: - 0.6 ≤ α < 1

Class C: - 0.4 ≤ α < 0.6

Class D: - α < 0.4

Tipologia di muratura	f_m (N/cm^2) Min-max	τ_0 (N/cm^2) min-max	E (N/mm^2) min-max	G (N/mm^2) min-max	w (kN/m^3)
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	100 180	2,0 3,2	690 1050	230 350	19
Muratura a conci sbozzati, con paramento di limitato spessore e nucleo interno	200 300	3,5 5,1	1020 1440	340 480	20
Muratura in pietre a spacco con buona tessitura	260 380	5,6 7,4	1500 1980	500 660	21
Muratura a conci di pietra tenera (tufo, calcarenite, ecc.)	140 240	2,8 4,2	900 1260	300 420	16
Muratura a blocchi lapidei squadrati	600 800	9,0 12,0	2400 3200	780 940	22
Muratura in mattoni pieni e malta di calce	240 400	6,0 9,2	1200 1800	400 600	18
Muratura in mattoni semipieni con malta cementizia (es.: doppio UNI foratura ≤ 40%)	500 800	24 32	3500 5600	875 1400	15
Muratura in blocchi laterizi semipieni (perc. foratura < 45%)	400 600	30,0 40,0	3600 5400	1080 1620	12
Muratura in blocchi laterizi semipieni, con giunti verticali a secco (perc. foratura < 45%)	300 400	10,0 13,0	2700 3600	810 1080	11
Muratura in blocchi di calcestruzzo o argilla espansa (perc. foratura tra 45% e 65%)	150 200	9,5 12,5	1200 1600	300 400	12
Muratura in blocchi di calcestruzzo semipieni (foratura < 45%)	300 440	18,0 24,0	2400 3520	600 880	14

Type of Mansory	f_m (N/cm^2) Min-max	τ_0 (N/cm^2) min-max	E (N/mm^2) min-max	G (N/mm^2) min-max	w (kN/m^3)
Stone masonry disorderly arronged (pebbles, irregular stones)	100 180	2,0 3,2	690 1050	230 350	19
Masonry made of large square-cut stones, inner layer of limited thickness	200 300	3,5 5,1	1020 1440	340 480	20
Square cut stone mansory with good texture	260 380	5,6 7,4	1500 1980	500 660	21
Soft stone block mansory (tuff, calcarenite stone)	140 240	2,8 4,2	900 1260	300 420	16
Masonry of square cut stone blocks	600 800	9,0 12,0	2400 3200	780 940	22
Solid bricks mansory and lime mortar	240 400	6,0 9,2	1200 1800	400 600	18
Masonry of hollowed clay blocks and lime (ex. double UNI hollow perc. ≤ 40%)	500 800	24 32	3500 5600	875 1400	15
Masonry of clay hollowed blocks (hollow perc. ≤ 45%)	400 600	30,0 40,0	3600 5400	1080 1620	12
Masonry of clay blocks with dry vertical joints (hollow percentage < 45%)	300 400	10,0 13,0	2700 3600	810 1080	11
Masonry of concrete block or expanded clay (hollow percentage beetween 45% and 65%)	150 200	9,5 12,5	1200 1600	300 400	12
Masonry of hollowed concrete block (hollow percentage < 45%)	300 440	18,0 24,0	2400 3520	600 880	14

Table 2. Reference mechanical values for exiting masonry (minimum and maximum) (DM 14.01.2008).

PARAMETER 4 – POSITION OF THE BUILDINGS AND FOUNDATIONS

This item evaluates, as far as possible with a visual investigation, the influence of soil and foundations. For this reason only some aspects are considered:

- Consistency and slope of soil;
- Foundations at different heights
- Unbalanced forces by embankments

For the attribution of classes, the following table was used

SOIL AND FOUNDATIONS	SOIL SLOPE	FOUDATION HEIGTH DIFFERENCE	CLASS
1 ROCK WITH FOUNDATION	P <=10	-	A
	10<P<=30	-	B
	30<P<=50	-	C
	P>50	-	D
2 ROCK WITHOUT FOUNDATION	P <=10	-	A
	10<P<=30	-	B
	30<P<=50	-	C
	P>50	-	D
3 NO-PUSHING SOIL WITH FOUNDATIONS	P <=10	Δh=0	A
	P <=10	0<Δh<1	B
	10<P<=30	Δh<=1	B
	30<P<=50	Δh<=1	C
	P>50	-	D
	-	Δh>1	D
3 NO-PUSHING SOIL WITHOUT FOUNDATIONS	P <=10	Δh=0	A
	P <=10	0<Δh<1	B
	10<P<=30	Δh<=1	B
	30<P<=50	Δh<=1	C
	P>50	-	D
	-	Δh>1	D
5 PUSHING SOIL WITH FOUNDATION	P <=50	Δh<=1	C
	P>50	-	D
	-	Δh>1	D
6 PUSHING SOIL WITHOUT FOUNDATION	P <=30	Δh<=1	C
	P>30	-	D
	-	Δh>1	D

Table 3. Identification of the class for parameter 4.

PARAMETER 5 - FLOORS

The quality of floors has a significant role in ensuring the good behaviour of the vertical resisting elements; on the other hand it is not unusual the internal collapse of the floors, with substantial consequences in terms of damage and victims. In the attribution of the classes both these factors are taken into account. In particular, it is important to verify the following requirements for each floor:

a) floor rigidity and plate behaviour (so good connection of structural elements);

b) effective connection of the vertical resisting elements;

The four classes are defined as follows:

Class A: - Buildings with floors of any typology satisfying these three conditions:

  a. negligible in the plane deformability of the slab;

  b. effective links between floors and walls;

  c. absence of staggered floors;

Class B: - Building with floors as the previous category not satisfying the condition c

Class C - Buildings with floors having a significant deformation in plan but well connected to the walls

Class D - Buildings with floors of any typology badly connected to the walls.

PARAMETER 6 – CONFIGURATION IN PLAN

The seismic behavior of a building depends even the layout of the plan. In the case of rectangular buildings is significant relationship $\beta_1 = a/l \times 100$ between the shorter side and the longer side lengths (Fig. 21). In the case of plans different from the rectangular shape, in addition to the elongated shape of the main body (measured by the parameter β_1

defined above) is necessary to take account of the extent of the deviation: this can be done using parameter β_2. The assignment of a building to the different classes is based on the worst case, in the verification floor, of the conditions imposed by the parameters β_1 and β_2 in the following way:

A:	$\beta_1 \geq 80$	$\beta_2 \leq 10$
B:	$60 \leq \beta_1 < 80$	$10 < \beta_2 \leq 20$
C:	$40 \leq \beta_1 < 60$	$20 < \beta_2 \leq 30$
D:	$\beta_1 < 40$	$\beta_2 > 30$

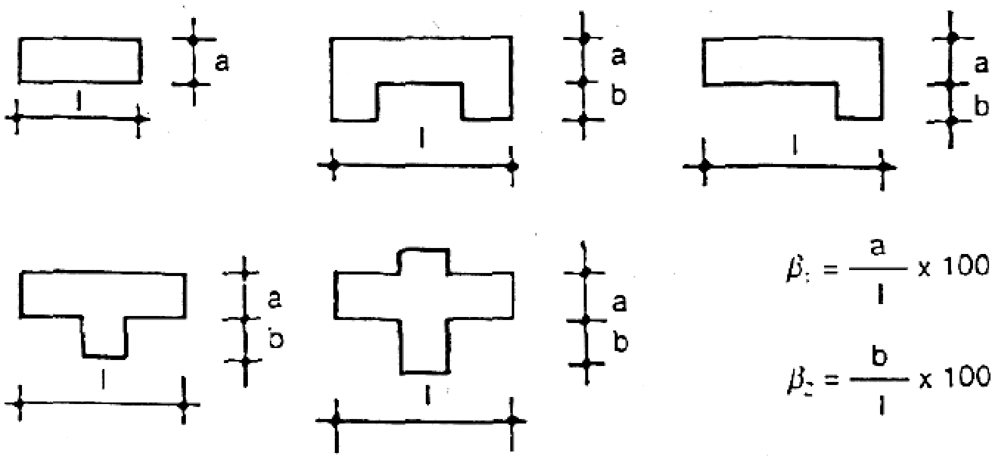


Fig. 21. Determination of factor β for the attribution of the vulnerability class for the configuration in plan parameter.

PARAMETER 7 - CONFIGURTION IN ELEVATION

In the case of masonry buildings, especially the older ones, the main cause of irregularity is the presence of porticos, balconies and roof terraces. The presence of porticos is reported as the percentage ratio between the floor area of the portico (pilotis) and total area of the floor (in the worst conditions). Another element to be considered for the irregularity is the presence of towers of significant mass and height with respect to the remaining part of the building (the ratio between the height of the tower T and the total height of the building H is reported as percentage). The presence of appendages of modest size (chimneys, etc..) it is not taken into account in the assessment of the irregularity. For the evaluation of the variations of mass is considered the ratio ± ΔM/M in which:

ΔM is the mass variation between two consecutive floors;

the sign + means a increase

the sign **-** means a reduction

M is the mass of the lower floor

The case to consider is that most unfavourable.

Variations in percentages lower than 10% may be considered null. Normally, the ratio ± ΔM/M can be replaced by the ratio ± ΔA / A, where A and ΔA are respectively the area of the fllor and its variation.

The four classes are defined as follows:

Class A: **-** Buildings with distribution of masses and resistant elements practically uniform over the whole height; **-** Buildings with mass and resistant elements decreasing with continuity; **-** Buildings with a reduction of the area in plan lower than 10%.

Class B: **-** Buildings with porticos of modest size, affecting not more than 10% of the total area of the floor; **-** Buildings presenting a decrease in the area of the plant greater than 10% and less than or equal to 20%;

- Buildings with towers of height lower than 10% of the total height of the building.

Class C: **-** Buildings with porticos interesting an area greater than 10% and lower than or equal to 20% of the total area of the plan; **-** Buildings with reductions of area greater than

20%; - Buildings with towers having height greater than 10% and less than or equal to 40% of the total height of the building.

Class D: - Buildings with porticos affecting more than 20% of the total area of the floor; - buildings with towers having height of more than 40% of the total height of the building.

PARAMETER 8 – WALLS MAXIMUM INTERAXIS

With this item is accounted the presence of main walls intersected by transverse walls placed at an excessive distance one to each other. The classes are defined as function of the ratio between the interaxis between the transverse walls and the thickness of the main walls.

The classes are defined as follows:

Class A: - Buildings with a ratio interaxis / thickness not exceeding 15

Class B: - Buildings with a ratio interaxis / thickness greater than 15 and not more than 18

Class C: - Buildings with a ratio interaxis / thickness greater than 18 and not more than 25

Class D: - Buildings with a ratio interaxis / thickness exceeding 25.

PARAMETRO 9 - ROOFS

The elements characterizing the influence of the roof on the seismic behavior of a building are essentially two: the type and the weight. The first is taken into account in the definition of the four classes while the latter affects the determination of the weight to be assigned to this parameter. The details required are:

a. the worst this kind of roof present: thrusting, slightly thrusting, not thrusting;

b. the presence or absence of curbs

c. the presence or absence of links

d. the dead load of the roof

e. the perimeter of the roof.

Class A: - Buildings with no thrusting roof provided with curbs and links

Class B: - Buildings with no thrusting roof without curbs or links;- Buildings with slightly thrusting roof provided with curbs and links

Class C: - Buildings with slightly thrusting roof without curbs and links; - Buildings with thrusting roof provided with curbs and links

Class D: - Buildings with thrusting roof provided without curbs and links.

The identification of the type of roof is shown in the following images.

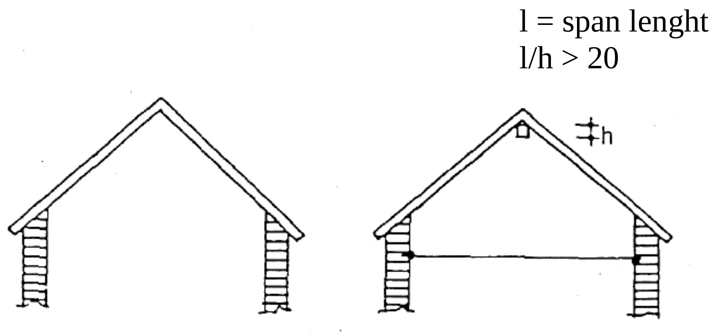


Fig. 22. Thrusting roof – type M.

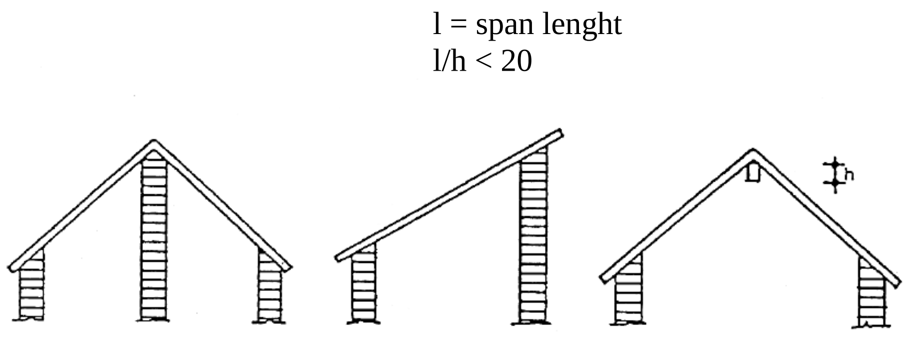


Fig. 23. Slightly thrusting roof – type N

l = span lenght

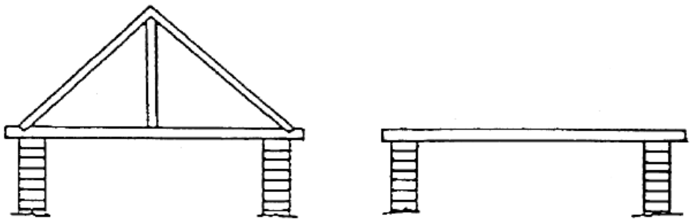


Fig. 24. No-thrusting roof – type O.

PARAMETER 10 – NON STRUCTURAL ELEMENTS

This item takes into account the presence of windows, appendages and projections that may fall causing damage to persons or things. It is a secondary element with respect to the assessment of vulnerability

The classes are defined as follows:

Class A - Buildings without windows, appendages or projections or false ceilings;

Class B - Buildings and windows securely connected to the walls, with chimneys of small size and low weight and with false ceilings well connected. Buildings with balconies forming an integral part with the floors.

Class C - Buildings with external windows or small signs badly connected to the walls and false ceilings of small extension badly connected to of big extension and well connected.

Class D - Buildings presenting: chimneys or other appendages badly constrained to the structure, the parapets badly arranged or other items of significant weight that can fall in presence of an earthquake. Buildings with balconies or other projections (services, etc.), added after the construction of the main building and poorly connected to it. Buildings with false ceilings of great extent and poorly connected.

PARAMETER 11 – CURRENT CONDITIONS

This item takes into account the conservation status of the buildings.

The four classes are defined as follows:

Class A: - Walls in good condition with no visible cracks.

Class B: - Buildings presenting not diffuse capillary cracks, except the cases in which they have been produced by earthquakes.

Class C: - Buildings with medium-sized cracks (crack width: 2-3 mm) or with capillary cracks of seismic origin; Buildings which, while not presenting cracks, are characterized by a state of conservation of the walls such as to determine a significant reduction of strength.

Class D: - Buildings presenting out of plumb walls and/or serious cracks even if not spread; Buildings characterized by severe deterioration of materials; Buildings which, while not presenting cracks, are characterized by a state of conservation of masonry able to cause a serious decrease of resistance.

4.2 GNDT 2nd level vulnerability assessment forms for reinforced concrete buildings (1986 version)

In order to achieve a level of reliability similar to that of masonry buildings, it was decided to adopt as detection tool the GNDT 2nd level form for RC buildings in published in 1986 (Fig 25). This form contains a significantly greater amount of information than the most recent one, published in 1999 and has a composition similar to that of masonry buildings.

Also present in this case 11 evaluation parameters are present, to which is associated a class of vulnerability between A and C for the first 10 and between A and D for the parameter 11, taking into account that the best condition is associated to the class A.

The vulnerability classes are characterized by increasing scores, while individual parameters are weighted by a numerical weight that it would establish the influence within the overall assessment of the vulnerability. The Tab. 5 shows the list of the 11 parameters of vulnerability, the scores assigned to the classes and the weights. The parameters and the scores associated refer to those proposed in the standard GNDT procedure. As in the case of the masonry, the vulnerability index is defined as defined as

$$V = \sum_i c_{vi} p_i$$

Taking values between 0 and 10. The normalized vulnerability index $\overline{V}$ is obtained as

$$\overline{V} = \frac{V}{10} \times 100$$

Depending on the parameters the attribution of vulnerability class may come from simple observations on the building or simplified calculations may be needed.

In the following pages the operations necessary for the identification of the classes of vulnerabilities related to the above parameters therefore are described in detail.

As in the case of masonry buildings, for each parameter the compilation of the forms requires to assign a rating (E, M, B, A) on the quality of information that allowed the attribution of the class.

	PARAMETRO	Class C_{vi} A	B	C	D	Weight p_i
1	Type and organization of the resisting system	0	1	2	-	1,00
2	Quality of the resisting system	0	0,25	0,5	-	1,00
3	Conventional resistance	0	0,5	1	-	1,50
4	Position of the building and foundations	0	0,25	0,5	-	1,00
5	Floors	0	0,25	0,5	-	1,00
6	Configuration in plan	0	0,25	0,5	-	1,00
7	Configuration in elevation	0	0,5	1	-	1,00
8	Connections and critical elements	0	0,25	0,5	-	1,00
9	Low ductility elements	0	0,25	0,5	-	1,00
10	Non-structural elements	0	0,25	0,5	-	1,00
11	Current conditions	0	0,5	1	2	1,00

Table. 4. Vulnerability parameters and related scores and weights

G.N.D.T. - SCHEDA DI VULNERABILITÀ DI 2° LIVELLO (CEMENTO ARMATO)

Codice ISTAT Provincia [ ] Codice ISTAT Comune [ ] Scheda No. [ ]

	PARAMETRI	Classi	Qual. inf.	ELEMENTI DI VALUTAZIONE
1	TIPO ED ORGANIZZAZIONE DEL SISTEMA RESISTENTE (S.R.)	11	22	Pareti di c.a. (cl. A) [1]; Tamp. cons. e telai (cl. A) [2]; Tamp. deb. e telai rig. (cl. B) [3]; Tamp. deb. e telai def. (cl. C) [4]; Telai non tamp. (cl. B o C) [5]
2	QUALITÀ DEL S.R.	12	23	(vedi manuale)
3	RESISTENZA CONVENZIONALE	13	24	Numero di piani **N**; Area tot. cop. A_t (mq); Area A_x (mq); Area A_y (mq); τ_k (t/mq); Alt. media interp. **h** (m); Peso spec. par. p_m (t/mc); Carico perm. sol. p_s (t/mq)
4	POSIZIONE EDIFICIO E FONDAZIONI	14	25	Pend. perc. terr.; Roccia fond. si [1] no [2]; Terr. sc. non sp. fond. si [3] no [4]; Terr. sc. sp. fond. si [5] no [6]; Diff. max di quota Δh (m)
5	ORIZZONTAMENTI	15	26	Piani sfalsati si [1] no [2]; Orizz. rig. e ben coll. [1]; Orizz. def. e ben coll. [2]; Orizz. rig. e mal coll. [3]; Orizz. def. e mal coll. [4]; % or. rig. ben coll.
6	CONFIGURAZIONE PLANIMETRICA	16	27	Rapp. perc. $\beta_1 = a/l$; Rapp. perc. $\beta_3 = e/d$; Rapp. perc. $\beta_4 = \Delta\ d/d$; Rapp. perc. $\beta_5 = c/b$
7	CONFIGURAZIONE IN ELEVAZIONE	17	28	% aumento (+) riduz. (−) di massa; Rapp. perc. **T/H**; Var. in elev. s.r. ∅ [1] 1 [1] 2 [2]; Piano terra port. si [1] no [2]
C8	COLLEGAMENTI ED ELEMENTI CRITICI	18	29	Rapp. perc. $\gamma_1 = s/b$; Rapp. perc. $\gamma_2 = e/b'$ min; Rapp. perc. $\gamma_3 = e/b''$; Rapp. max h/b_{min}; % σ/Rc (approssim.); Colleg. el. pref. si [1] no [2] costr. ordin. [3]; Largh. min. b_{min} (cm)
C9	ELEM. BASSA DUTT.	19	30	Rapp. min. h_{min}/b; Rapp. max h_{medio}/h_{min}
10	EL. NON STRUTT.	20	31	(vedi manuale)
11	STATO DI FATTO	21	32	(vedi manuale)

SCHEMI - RICHIAMI (CEMENTO ARMATO)

Parametro 3. Resistenza convenzionale.

Minimo fra A_x e A_y A (mq) ______

Coefficiente a_o = A/At ______

$q = (A_x + A_y) \cdot h \cdot p_m / At + p_s$ ______

$C = a_o \cdot \tau / (q \cdot N)$ ______ $\alpha = C/(0.4 \cdot R)$ ______

Calcolo di R

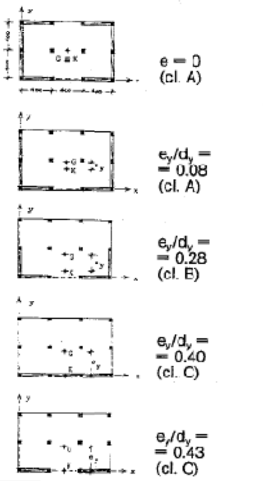
Terreni tipo S_1: R = 2.5 (T < 0.35 sec)

$R = 2.5/(T/0.35)^{2/3}$ (T ⩾ 0.35 sec).

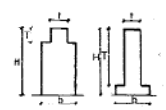
Terreni tipo S_2: R = 2.2 (T < 0.8 sec)

$R = 2.2/(T/0.8)^{2/3}$ (T ⩾ 0.8 sec)

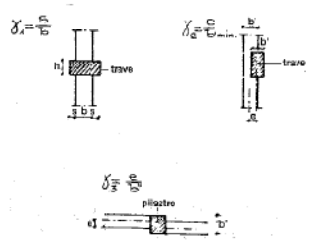
Parametro 6. Configurazione planimetrica.



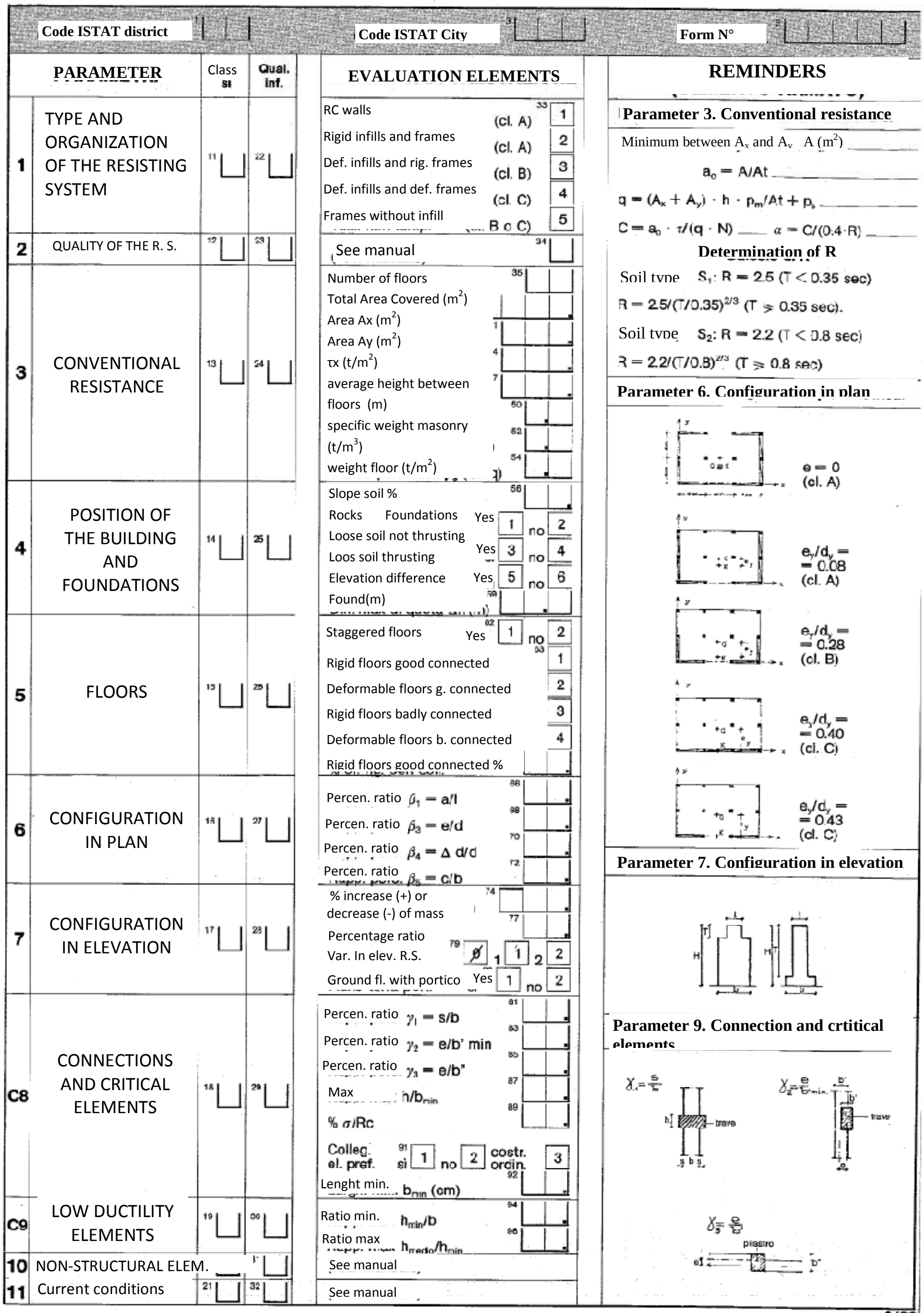
Parametro 7. Configurazione in elevazione.



Parametro C9. Collegamenti ed elementi critici.



G.N.D.T. 2ND LEVEL ASSESMENT FORMS (REINFORCED CONCRETE)



Code ISTAT district | Code ISTAT City | Form N°

	PARAMETER	Class si	Qual. inf.	EVALUATION ELEMENTS
1	TYPE AND ORGANIZATION OF THE RESISTING SYSTEM	11	22	RC walls (cl. A) 1; Rigid infills and frames (cl. A) 2; Def. infills and rig. frames (cl. B) 3; Def. infills and def. frames (cl. C) 4; Frames without infill B o C) 5
2	QUALITY OF THE R. S.	12	23	See manual
3	CONVENTIONAL RESISTANCE	13	24	Number of floors; Total Area Covered (m^2); Area Ax (m^2); Area Ay (m^2); τx (t/m^2); average height between floors (m); specific weight masonry (t/m^3); weight floor (t/m^2)
4	POSITION OF THE BUILDING AND FOUNDATIONS	14	25	Slope soil %; Rocks Foundations Yes 1 no 2; Loose soil not thrusting; Loos soil thrusting Yes 3 no 4; Elevation difference Yes 5 no 6; Found(m)
5	FLOORS	15	26	Staggered floors Yes 1 no 2; Rigid floors good connected 1; Deformable floors g. connected 2; Rigid floors badly connected 3; Deformable floors b. connected 4; Rigid floors good connected %
6	CONFIGURATION IN PLAN	16	27	Percen. ratio $\beta_1 = a/l$; Percen. ratio $\beta_3 = e/d$; Percen. ratio $\beta_4 = \Delta d/d$; Percen. ratio $\beta_5 = c/b$
7	CONFIGURATION IN ELEVATION	17	28	% increase (+) or decrease (-) of mass; Percentage ratio; Var. In elev. R.S. Ø 1 2; Ground fl. with portico Yes 1 no 2
C8	CONNECTIONS AND CRITICAL ELEMENTS	18	29	Percen. ratio $\gamma_1 = s/b$; Percen. ratio $\gamma_2 = e/b'$ min; Percen. ratio $\gamma_3 = e/b''$; Max h/b_{min}; % σ/Rc; Colleg. el. pref. si 1 no 2 costr. ordin. 3; Lenght min. b_{min} (cm)
C9	LOW DUCTILITY ELEMENTS	19	30	Ratio min. h_{min}/b; Ratio max h_{medio}/h_{min}
10	NON-STRUCTURAL ELEM.			See manual
11	Current conditions	21	32	See manual

REMINDERS

Parameter 3. Conventional resistance

Minimum between A_x and A_y A (m^2)

$a_o = A/At$

$q = (A_x + A_y) \cdot h \cdot p_m/At + p_s$

$C = a_o \cdot \tau/(q \cdot N)$ $\alpha = C/(0.4 \cdot R)$

Determination of R

Soil type S_1: $R = 2.5$ ($T < 0.35$ sec)

$R = 2.5/(T/0.35)^{2/3}$ ($T \geq 0.35$ sec).

Soil type S_2: $R = 2.2$ ($T < 0.8$ sec)

$R = 2.2/(T/0.8)^{2/3}$ ($T \geq 0.8$ sec)

Parameter 6. Configuration in plan

Parameter 7. Configuration in elevation

Parameter 9. Connection and crtitical elements

Fig. 25. GNDT 2nd level assessment forms for RC buildings (1986 version).

PARAMETER 1 - TYPE AND ORGANIZATION OF THE RESISTING SYSTEM

The reinforced concrete structure, if framed, responds to the earthquake interacting with masonry infills. The behavior of the three main types is summarized as follows:

1) The construction of type A) is rigid due to the presence of RC walls or consistent masonry infills within the frames; it is assumed a maintaining of the strength capacity during and after the seismic event;

2) The construction of type B) has a rigid-brittle initial behavior followed by the out of use of the rigid elements (walls and panels) and subsequent behavior with good strength and ductility, although with greater deformability, for the presence of seismically designed frames;

3) The construction of type C) has a a rigid-brittle initial behavior, as the previous type, followed by a strong decay of the characteristics of stiffness and resistance.

To identify the main resisting system it is necessary to evaluate (even approximaltely) the resistance offered by the individual resistant elements in the direction defined as the worst. For this purpose, two basic assumptions are made:

a) the cross sections are entirely reacting;

b) each floor can undergo only horizontal translations or rotations around a vertical axis (shear-type deformation).

Under these assumptions the distribution of the forces is proportional to the moments of inertia and shear areas. Since it is assumed that the resistant elements are mainly RC walls of infills within the frames, the flexural deformation can be neglected.

Moreover if it one assumes the shape factors of the sections equal to the unit, it can be concluded that each section is subjected to a force proportional to $A\ \tau \cos^2\alpha / h$ in which A is the area of the cross- section, α is the angle between the reference direction and that of the "strong plane " of the wall, *h* is the height of the element and τ the shear strength assuming these following possible values:

- Masonry satisfying class A requirements $\tau =$ 30÷35 t/m^2
- Masonry satisfying class B requirements $\tau =$ 15÷20 t/m^2
- RC walls (and RC columns) $\tau =$ 150÷250 t/m^2

It can be assumed E = 30.000 τ for masonry and E = 15.000 τ for reinforced concrete. The main resisting system is the one absorbing more than 70% of the horizontal actions. The evaluation of the main resisting system is required for the attribution of the classes described below.

A - Rigid-resisting structure.

Buildings included into the following categories:

1) Buildings with main resisting system constituted by the walls, RC panels or reinforced masonry.

2) Buildings with main resisting system constituted by RC frames and consistent masonry, well connected to the frame, in such a way to satisfy the following requirements:

a) are made of robust elements (solid bricks, blocks with aggregates of concrete expanded clay, natural or artificial squared stone even roughly - such as calcarenite, limestone, etc..) with mortar of good workmanship;

b) the openings have compact shape and do not exceed 30% of the surface of the masonry;

c) the ratio between height and thickness is less than 20;

d) the infills do not have detachments from the frame more than 1 cm;

e) the infills do not protrude, with respect to the external edge of the frame, by more than 20% of the thickness.

Frames composed of beams and columns must surround the masonry whose cross sections have an area greater than 25 b, being b comparable with the thickness of the masonry (in cm.).

B - Rigid-brittle / deformable-resistant structure.

Buildings with main resisting system consisting of masonry infills placed within RC frames having beam / column stiffness ratios exceeding 1.5. The masonry must respect the following requirements (although not satisfying the requirements in A)

a) the openings does not exceed 60% of the total area;

b) the ratio between height and thickness is less than 30;

c) have no detachments from the frame larger than 3 cm;

d) does not protrude, with respect to the outer edge of the frame, of more than 30% of the thickness.

The areas of the cross sections of resisting frames shall not be less than 20 b.

The main resisting system which is obtained not considering masonry fields (bare frames) must meet the following requirements:

a) the beam / column stiffness ratio must be greater than 1.5 with a joint cast in place or organized joint;

b) conventional strength is evaluable in the classes **A** or **B**.

C - Rigid-brittle / deformable-weak structure

In this category are consider of the buildings not included in categories **A** or **B**. Some examples for the identification of the main resisting system are reported in Fig. 25.

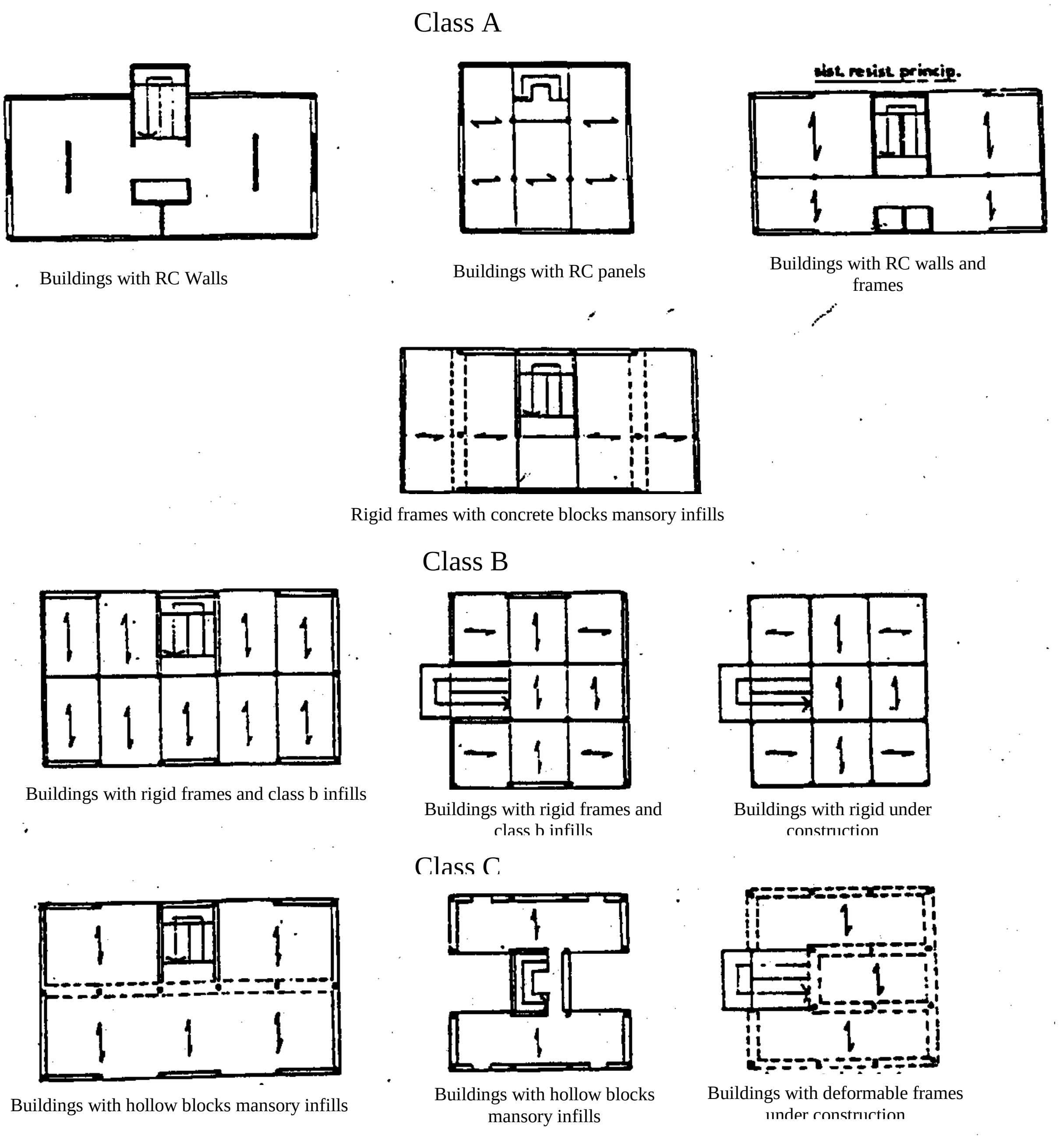


Fig. 26. Sample schemes for the identification of the main resisting system

PARAMETER 2 – QUALITY OF THE RESISTING SYSTEM

The assessment of the quality of the resistant system is made on the basis of the following groups of information:

a) Type and quality of materials used.

b) Characteristics of execution.

c) Design features.

As for the first group, in addition to the direct vision of the materials the knowledge of the age of the building and the establishment of the state of deterioration of the building in general are very useful.

With regard to the second group of information, in addition to the direct establishment, it is important to know the typology of construction methods used in the area (possibly for distinct periods of time) and those most frequently taken by the manufacturer (better if accompanied by information on the choices more frequently adopted by the project manager). The third group of information is relative to the design level, not only ascertainable by direct examination of the drawing, but also indirectly through information on the choices most frequently made by the designer, especially for what concerns the structural details, through information on the design modalities mostly followed in the area (also in this case for distinct periods of time).

<u>Classes</u>

A - Good.

The concrete used (visible in basements, attics, etc.) seems of good consistency, hard to scratch and well executed (with patches limited and sparse). The joints are barely visible and well executed. The rebar have improved adherence (information derived from elements of the project), not in view and not oxidized.

The masonry is made up of compact elements and not degraded, the mortar is not degraded and is not easy to remove.

The information available excludes bad execution and / or procedures or incorrect design choices in the area.

B – Medium.

Buildings that do not fall into classes A or C.

C - Poor.

Generally occur at least two of the following cases:

a) the concrete is of poor quality;

b) the rebars are visible and oxidized and possibly badly disposed;

c) the joints are poor;

d) methods of execution are bad;

e) bad design choices are bad;

f) the masonry walls are classified as poor.

PARAMETER 3 – CONVENTIONAL RESISTANCE

The parameter takes account of a kind of degree of safety with respect to the reference seismic forces, is calculated with the following assumptions:

a) Equivalent static seismic actions.

b) Absence of eccentricity or irregularities in plan.

c) Only the main elements of the resisting system in the most unfavourable direction are considered for the evaluation of strength (in case of absence of infill walls only the column cross-sections are considered, which have to be divided in half if frames do not satisfy the requirements of the level B, for the type of the main structure).

d) The resisting force of each section is conventionally $A \cdot \tau$ in which A is the cross sectional area and τ has been defined before. The reference seismic forces are calculated, for each of the N levels, with the following relationship:

$$F_i = F_h \frac{W_i h_i}{\sum W_i h_i}$$

in which:

W_i is the weight of the floor;

h_i is the height of the floor;

F_h is the resulting seismic action on the building, defind as

$$F_h = \frac{S_e(T) \times W \times \lambda}{g}$$

$S_e(T)$ being the spectral acceleration, relative to the elastic response spectrum associated with the site and conditions of subsoil.

λ is coefficient that takes into account of the distribution of forces over the height assumed equal to 0.85.

The fundamental period T of the structure can be calculated approximately by the expression of Reyleigh

$$T = c_1 H^{3/4}$$

being $c_1 = 0.075$ for RC structures and H the total height.

The coefficient α is defined as the ration between resisting forces and seismic forces

$$\alpha = \frac{A \times \tau}{F_h}$$

A being the minim sum between the cross sectional areas of the columns in the directions *x* and *y*. The classes are attributed as function of the parameter α as follows.

A - $\alpha \geq 1,5$

B - $0,7 \leq \alpha < 1,5$

C - $\alpha < 0,7$

PARAMETER 5 – POSITION OF THE BUILDING AND FOUNDATIONS

The aspects to consider are:

1) Existence (or not) of foundations and their type.

2) Characteristics of the soil.

The difficulties in the assessment of both groups of parameters means that one can limit the investigation to consider: for the first group the existence (or not) of foundations, for the second group, the ascertainable type of soil and its slope. Is added to the second group the presence (or not) of thrusting embankments.

Class A – Buildings with foundations on melted soils, with difference of height of not more than 1.5 m over 10.0 m, or rock soils with various height difference not exceeding 3.0 m over 10.0 m. No thrusting embankments.

Class B - Buildings that cannot be classified in classes A or C.

Class C - Buildings without foundations or with foundations clearly insufficient on any type of soil; Buildings with differences of foundation height greater than 3.0 m over 10.0 m on melted soil or 6.0 m over 10.0 m on the rock; Presence of thrusting embankments.

PARAMETER 5 – FLOORS.

The requirements which must be verified to consider that a floor behaves as a diaphragm are of two types:

a) Slab-type behaviour and high rigidity for planar deformations (so good connection between structural elements);

b) effective connection to the vertical resisting elements.

Classes of floors

A - Rigid and well connected.

Buildings whose floors respect condition a) and c) at least for 70% of the surface.

B - On average rigid and connected.

Buildings that are not classified in A or C.

C – Deformable and poorly connected.

Buildings whose foors do not respect conditions a) and c) or conditions a) and c) are respected for a surface that is lower than 30%.

PARAMETER 6 – CONFIGURATION IN PLAN

The definition of the configuration in plan is related to:

1) Distribution of masses and stiffness.

2) Shape of the plan.

Important information for what concerns point 1) are:

a) the component of the eccentricity between the centre of mass and centre of stiffness, assessed (even approximately) in the verification floor and the direction in which the ratio e/d is maximum (*d* is the length in plan of the building in the considered direction);

b) the retraction Δd of the resisting system, compared to the perimeter of the building in plan, evaluated in the verification floor and in the direction in which the ratio $\Delta d/d$ is maximum;

c) the ratio between the short side and the long side of the plan assessed in the verification floor; the latter takes into account an additional contribution to the eccentricity due mainly to unfavourable distributions of accidental loads.

Important information for what concerns point 2) are:

a) the presence and the shape of the appendages in plan;

b) the size of the appendices.

<u>Classi</u>

A - Regular.

A regular plan that meets all of the following requirements:

1) (for what concerns the distribution of masses and stiffness):

a) The maximum ratio *e/d* is lower than 0.20;

b) At least 70% of the resisting elements follows the perimeter of the plan including the projections infilled with a retraction Δd lower than 0.1 (0.2 for projections not infilled) of the dimension *d*;

c) The ratio between the short side *a* and the long side *l* of the plan rectangle is greater than 0.4.

2) (for what concerns the shape):

For appendages in plan in the minimum ratio between width *c* and protrusion *b* greater than 0.5.

B - Irregular.

Buildings whose verification floor does not meet any of the preceding or following (classes A or C).

C – Very irregular.

A very irregular plan meets one following cases

a) *e/d* is greater than 0.4;

b) more than 70% of the main elements of the resisting system follows the perimeter with a retraction Δd greater than 0.1 (for not infilled projections 0.2) of dimension d;

c) *a/l* is lower than 0.2 and at the same time more than 30% of the elements follows the perimeter with a ratio $\Delta d/d$ greater than 0.1 (for not infilled projections 0.2);

d) There is at least an appendage for which the ratio *c/b* is less than 0.25.

PARAMETER 7 – CONFIGURATION IN ELEVATION

Reference is made to the scheme of a "base" of width *b* and a "tower" of width *t* and height *T*, while the whole building (base + tower) has the height *H* (Fig. 26).

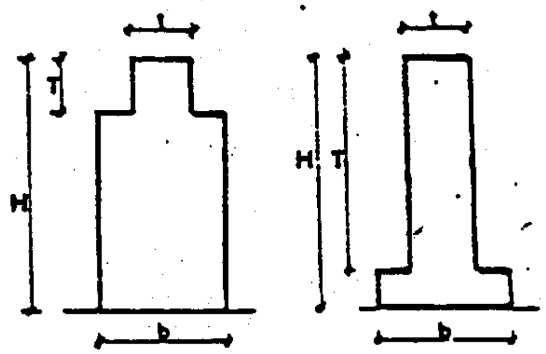


Fig. 27. Sample scheme for the evaluation of configuration in elevation.

Class A

There are no significant variations in the resisting system resistant between two successive floors. There are no significant variations in the distribution of mass in elevation above the verification floor plan and in any case the increases are within 20%.

The ratio *T/H* is less than 0.1 or greater than 0.9.

Class B

Buildings not classified as A or C.

Class C

Buildings with variations in resisting system to 2 classes;

Buildings with variation of 1 class and with mass increase (upward) greater than 20% or with a ratio *T/H* between 0.1 and 0.3 (or between 0.7 and 0.9). Buildings with non-significant variations in the resisting system, but with *T/H* between 0.3 and 0.7 or with mass increase of more than 40%.

Normally, the ratio ± ΔM/M can be replaced by the ratio ± ΔA/A, where A and ΔA are respectively the area of the plan and its variation.

PARAMETER 8 – CONNECTIONS AND CRITICAL ELEMENTS

Connections are defined the areas of connection between the structural elements (beam-column joints, beam-slab joints, foundation-columns joints or walls, joints between structural elements if prefabricated).

Are defined critical all the elements of primary importance for resistance to seismic actions. Are included in this definition almost all connections (central beam-column joints, well-confined joints, almost all beam-floor areas can be excluded); columns; RC walls; RC panels; all elements that have a mean compressive strength greater than 15% of the ultimate one; squat elements.

Classes

A - Good.

Buildings whose connections and critical elements meet all the following requirements:

1) Beam-column Nodes cast in place or prefabricated:

a) the width of the beam is not greater than that of the column plus 20% on each side, or the width of the beam is not greater than that of the column plus a half of the height of the beam on each side;

b) the eccentricity between the axes of the beam and the columns does not exceed 20% of the minimum among the widths of the two elements;

c) the axes of the beams facing the joint have a distance in plan that is more than 30% of the transverse dimension of the column.

2) Joints in prefabricated elements:

a) in the case of simple support, are present restraints avoiding the expulsion of the elements in any direction;

b) are present welding or adhesives or reinforcements such as to classify the joint as organized.

3) For the columns having compression level greater than 15% of the ultimate strength, the minor size is greater than 25 cm.

4) RC walls and panels:

a) the thickness is not less than 12 cm.

b) the ratio between height and thickness is not greater than 25.

B - Medium

Buildings whose connections and critical elements are not classified in the previous of following cases:

C - Poor.

Buildings whose connections and critical elements are classified in one of the following cases:

1) For more than 70% (calculated as the ratio on the elements of the main resisting system) these elements do not meet the requirements of level A.

2) For more than 30% of the elements (with respect to the beam-column joints) refer to one of the following conditions:

a) the depth of the beam is greater than that of the column plus 40% on each side or the total height of the beam on each side;

b) the eccentricity between the axes of the beam and the column exceeds 30% of the minimum among the lengths of the two elements;

c) the axes of the beam facing the node are distant in plan more than 40% of the transverse dimension of the column.

3) The minimum size of the columns having average compression level is greater than 15% of the ultimate strength, is less than 20 cm.

In Fig. 28 some sample schemes for the evaluation of the critical components and connections are reported.

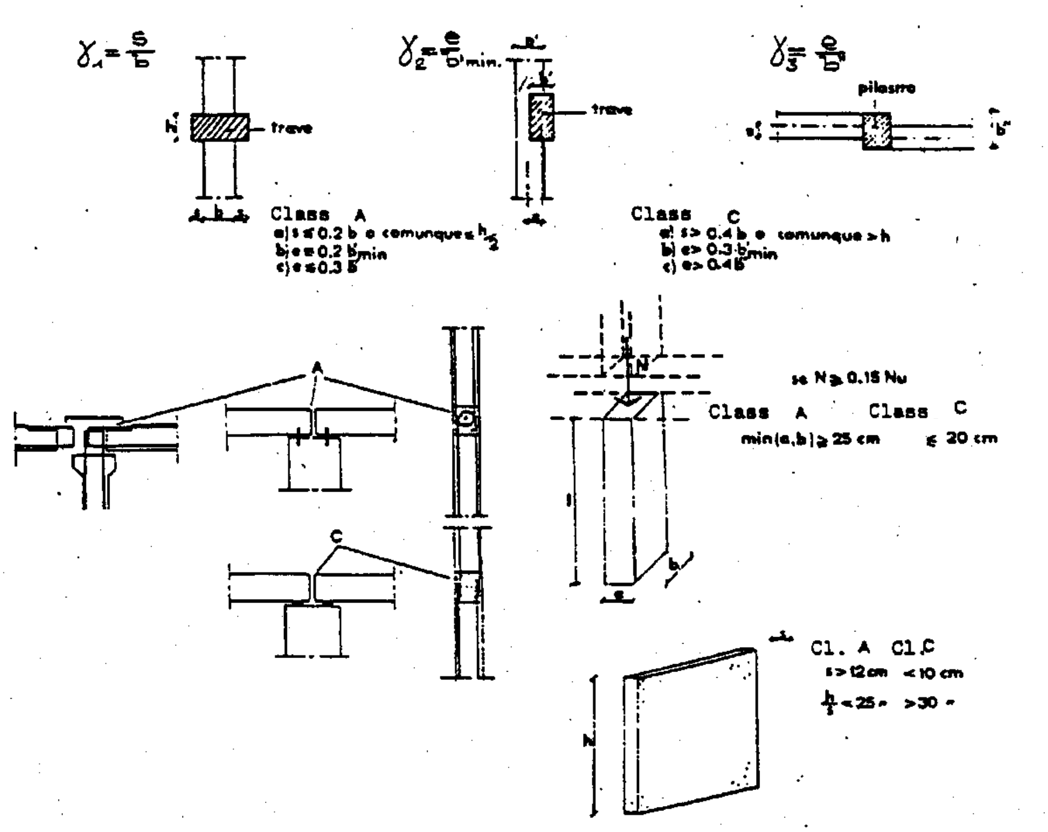


Fig. 28. Sample scheme for the evaluation of the connections and critical elements.

PARAMETER 9 – LOW DUCTILITY ELEMENTS

The parameter takes into account the cases in which the behavior of the building or parts of it is made critical by brittle elements, and / or substantially rigid and relatively with low ductile.

The "identification criteria" are of two types:

a) the net height of the resisting element;

b) the high ductility demand.

The main criterion for the identification is the first.

<u>Classes</u>

A - Absent.

Buildings which are not identifiable within levels B or C.

B - Present with low ductility.

Buildings in which at least only one of the following cases is recognized:

1) The shortest element has height lower than half the height of the other elements.

2) There is at least one element having height lower than 2/3 of the height of the other and a high ductility demand is recognized.

C - Present with very low ductility.

Buildings in which at least only one of the following cases is recognized:

1) The shortest element has height lower than a quarter of the height of the other elements.

2) There is at least one element having with height lower than a half the height of the others and a high ductility demand is recognized.

Some schemes exemplifying the identification of the elements with low ductility are reported in Fig. 29.

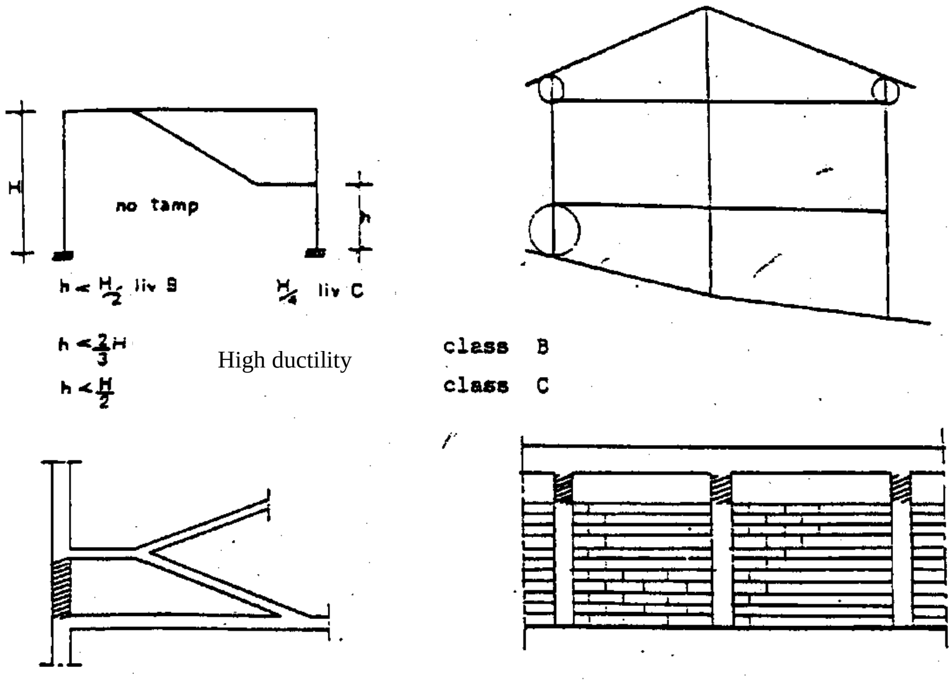


Fig. 29. Sample schemes for the identification of low ductility elements.

PARAMETER 10 – NON STRUCTURAL ELEMENTS

The elements to consider to assess the integrity are (in order of importance):

1) Resisting elements in elevation (columns, walls, infills, beams, slabs). In particular, the elements classified as critical (parameter 9) must be considered

2) Resisting elements in foundation.

3) Non-structural elements (parameter 10)

Class A

Buildings with all elements of type 1 in the first stage (uncracked).

No damage in foundations.

Presence of damage in the elements of type 3, but not affecting the stability under seismic actions.

Class B

Buildings cannot be classified as A or C

Class C

More than 30% of the critical elements of type 1 is in the 2nd stage (cracked).

In the floors are present relevant detachments cracks (more than 5 mm.).

Damage to the foundation established (cracks in the span of the beams, cracks in the connections of the plinths).

Class D

The building should be classified with the maximum possible vulnerability in the following cases: 1) at least a column or RC wall is in the 3rd stage (yielded steel) or beyond; 2) punching cracks are recognized in foundations, poles failures, or similar.

5. CALIBRATION AND DEFINITION OF THE FRAGILITY FUNCIONS

The definition of a relationship between the severity of the earthquake *S* and the damage *D*, through the vulnerability index *V*, is based on the fact that the response of a building, subject to seismic actions of increasing severity is typically characterized by a beginning stage of damaging, a phase of increase of the damage and a rapid decay up to the collapse.

Assuming as index of the severity the parameter $y=a/g$ which identifies the normalized ground acceleration, and as index of the damage parameter *D* between *0* and *1*, which identifies the loss of the economic value, the relationship may be represented by the so-called "fragility functions" (Fig. 28-a).

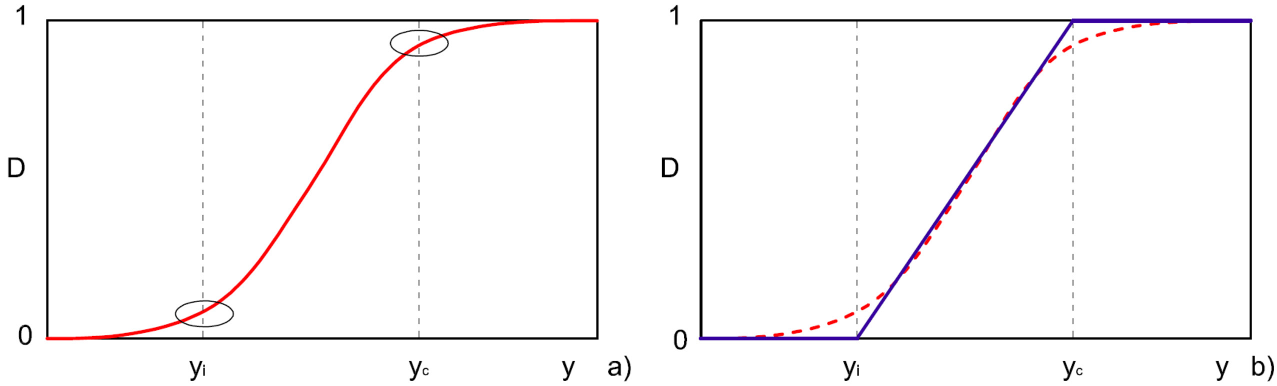


Fig. 30. Fragility functions: a) Fully defined function; b) Trilinear function.

On these curves one can identify the accelerations corresponding to the damage beginning (y_i) and the damage end y_c. For sake of simplicity it was introduced a simplified trilinear fragility function (Fig. 30-b). In this way the problem of establishing the correlation law is reduced to the determination for each vulnerability level of the values y_i and y_c.

The values of acceleration of damage beginning and collapse can be obtained following different strategies. In this case, for masonry buildings, which constitute the majority of the constructions, these were determined through a detailed numerical and experimental analysis on prototype buildings, chosen to be representative of urban centre. For RC buildings in the calculation of the acceleration levels y_i and y_c was performed in a simplified manner.

The analytical expression of the trilinear curves *D(y,V)* as a function of the values y_i and y_c evaluated for each level of vulnerability (fragility curves) is given by:

$$D(y,V) = \begin{cases} d = (y - y_i)/(y_c - y_i) \\ d = 0 \quad \text{per } y < y_i \\ d = 1 \quad \text{per } y > y_c \end{cases}$$

The attribution of the acceleration of early damage and collapse is made through the calibration of analytical laws linking theses values with the normalized vulnerability index. For this aim the expression proposed by Guagenti and Petrini (1989) was chosen and is illustrated in Fig. 31.

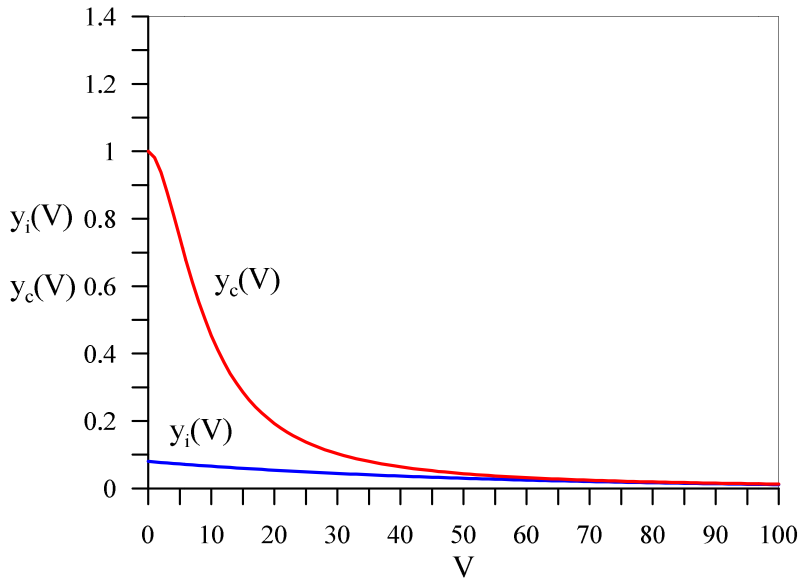


Fig. 31. Diagrams of y(V) relationships (Guagenti e Petrini (1989)).

The analytical expression of the curves reported in Fig. 31 are given by the following equations depending on the parameters α_i, β_i, α_c, β_c and γ.

$$y_i(V) = \alpha_i \, exp[-\beta_i(V)]$$

$$y_c(V) = \alpha_c + \beta_c(V)^{\gamma}$$

The vulnerability – acceleration curves (collapse or initial damage) therefore require the calibration of these 5 parameters that for the case under examination, are calibrated twice, once for buildings with masonry structure, once for buildings with RC structure. The calibration modalities and the results obtained are reported in the following sections.

5.1 Structural identification of the prototype buildings

The calibration of the parameters that govern the fragility curves is a task of great importance for the reliability of the results. For this reason it is necessary to perform a calibration based on experimental investigations aimed at the characterization of advanced structural models characterized by adequate reliability.

With this purpose, for masonry structures, which represent the most conspicuous portion of the urban context, in the logic of a repetition of building types it was proceeded the identification of two prototype buildings to perform experimental investigations aimed to structural identification and the subsequent numerical modelling.

The selected buildings, identified as Building Type A (BT "A") and building type B (BT "B"), are respectively, the future seat of the City Hall of Lampedusa (Fig. 32) and the headquarters of the Marine Protected Area of Lampedusa (Fig. 33). For the two structures are available the original drawings which were checked on site.

Fig. 32. Building Type A. City Hall of Lampedusa*.*

Fig. 33. Building Type A. Seat of the Marine Protected Area of Lampedusa*.*

Fig. 34. Location of the prototype buildings on the historical cartography of Lampedusa

Both structures are very old, the date of construction can be placed around the mid-800, as well as detectable from historical maps (Fig. 34).

For the structural identification of the buildings is was performed the installation of tri-axial accelerometers with the acquisition system “WISENET” in specified points considered to be of fundamental importance for the information detectable.

The Fig. 35 shows an isometric view of Building Type A where are identified the nodes in correspondence of which the accelerometers were paced. In the images above (Figs. 36-38) some accelerometers installed in situ are shown.

In a similar way for the building type B is shown in Fig.39 an isometric view with the identification of the nodes for the acquisition and in Fig. 40 images of the accelerometers installed in situ.

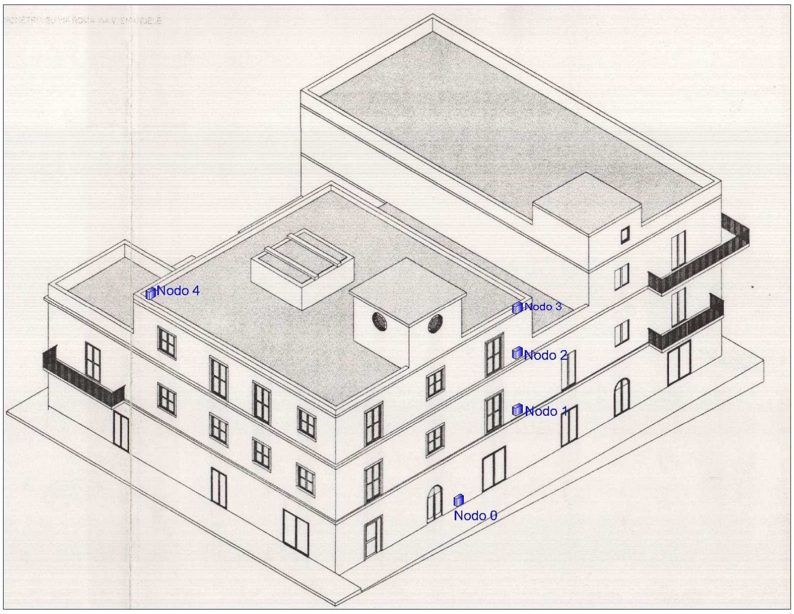


Fig. 35. Position of the acquisition nodes (BT"A").

Fig. 36. Node 0 (BT"A")

Fig. 37. Node 1 (BT"A")

Fig. 38. Nodes 3 e 4 (BT"A")

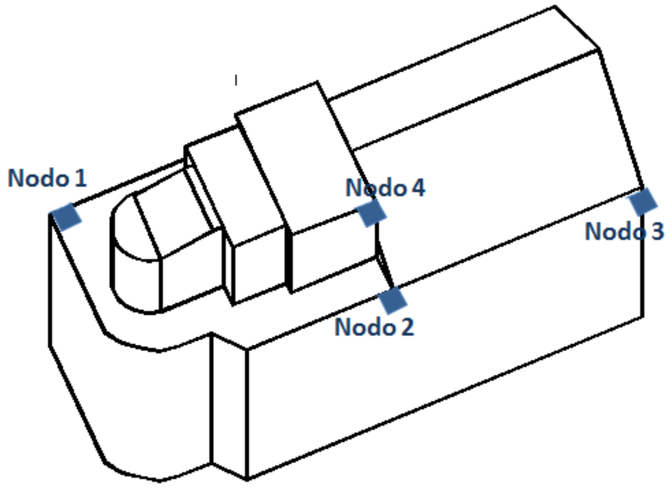


Fig. 39. Acquisition nodes (BT"B").

Fig. 40. Nodes 1 e 4 (BT"B")

For the monitoring of the structures it was chosen to not induce forced vibrations but to detect the response of buildings to environmental noise. The system of accelerometers is connected wireless to a receiver unit that transmits data to a computer installed in situ. The latter is connected to a network connection who processes and stores the data and them sends them to a receiver (University of Palermo - DICAM) where a subsequent phase of post processing and interpretation of the results is carried out.

Fig. 41. Computing unit for the handling of data.

The acquisition system handles the following events automatically:

• Daily Switching on and off with a prefixed frequency of reading

• Local back-up

• Data transfer

• Automatic recovery of the configuration automatic in case of interruption of power.

• Identification specific alarm thresholds

However, the most sensitive and complex phase regarded the post-processing operation which allows to obtain structural information on the building to get a realistic identification.

The analysis of the results in terms of fundamental frequencies detected by the accelerometers is of fundamental importance for the calibration of the structural model to be used for the assessment of the capacity of the buildings.

On the basis of the information experimentally detected for the building type A, a numerical modelling, aimed to the evaluation of the capacity by pushover analysis of the structure, was carried out. The modelling and analysis were carried out by SAP 2000 NL program and are exposed in the following section.

5.2 Prototype building "A", numerical modelling and pushover analysis

The original construction dates back to the mid-800. The building, as it can be seen from its conformation in plan, in elevation and also by a direct observation, was subjected to significant structural changes during the time, extensions and floor raisings which defined the current configuration of building aggregate. The primary structure, constituted by masonry blocks of limestone extracted from local quarries, has a robust composition especially in the perimeter. The thickness of the masonry walls floor is on average of about 80 cm on the ground, of 60 cm at the first elevation, of 35 cm on the second elevation. The oldest part of the building was subjected to recent restoration interventions, which included the replacing of existing floors with mixed clay block – concrete floors. The parts of the building that were added more recently have instead designed been directly with RC floors.

The aggregate building resulting after the transformations that continued over the years can be considered sufficiently representative of the constructions existing in the city centre of Lampedusa. With regard to the vulnerability recognized by the GNDT procedure the building stands at an average value of the normalized vulnerability (***V=36.29***). The main vulnerability characteristic that affect the index is (as will be discussed afterwards) the strong irregularity in elevation, potential cause of formation of soft storey mechanisms. The mechanical characterization of the building has been performed, with regard to the elastic characteristics (Young modulus *E* and shear modulus *G*), by exploiting the results coming from the acquisitions in situ. The strength values, compressive strength f_m and shear strngth τ_0, in the absence of mechanical testing of materials, have been derived using the average values reported for calcarenite masonry from in Table C8A.2.1 of the code in force (DM 14.01.2008). The finally values used are shown in Tab. 5.

f_m	τ_0	E_m	G_m	w
N/cm^2	N/cm^2	N/mm^2	N/mm^2	kN/m^3
190	3,50	1260	420	16

Table 5. Mechanical elastic and strength parameters for masonry (BT “A”).

Structural Model

Structural modelling was performed using the software SAP 2000 NL. A three-dimensional representation of the model is shown in Fig. 42. It was chosen to operate with a "frame-type" schematization of the masonry structure. The walls are modelled as beam/column elements with reference to their centroidal axis. Taking into account that in the building are present RC curbs at any level, it was assumed that the coupling masonry beams were flexurally resistant. These latter were modelled as elastic elements with the same material properties used for masonry.

The presence of concrete slabs allowed to consider the rigid diaphragm constrain. The loads coming from the floors are distributed linearly on the beams at each level. Finally, the connections at the areas of overlap between the walls and the masonry beams, were modelled as rigid elements. In Figs. 43-45 it is shown the structural geometry of the model at any level. In the latter the structural axes are marked in red and the interception point of the mesh closure are marked in blue.

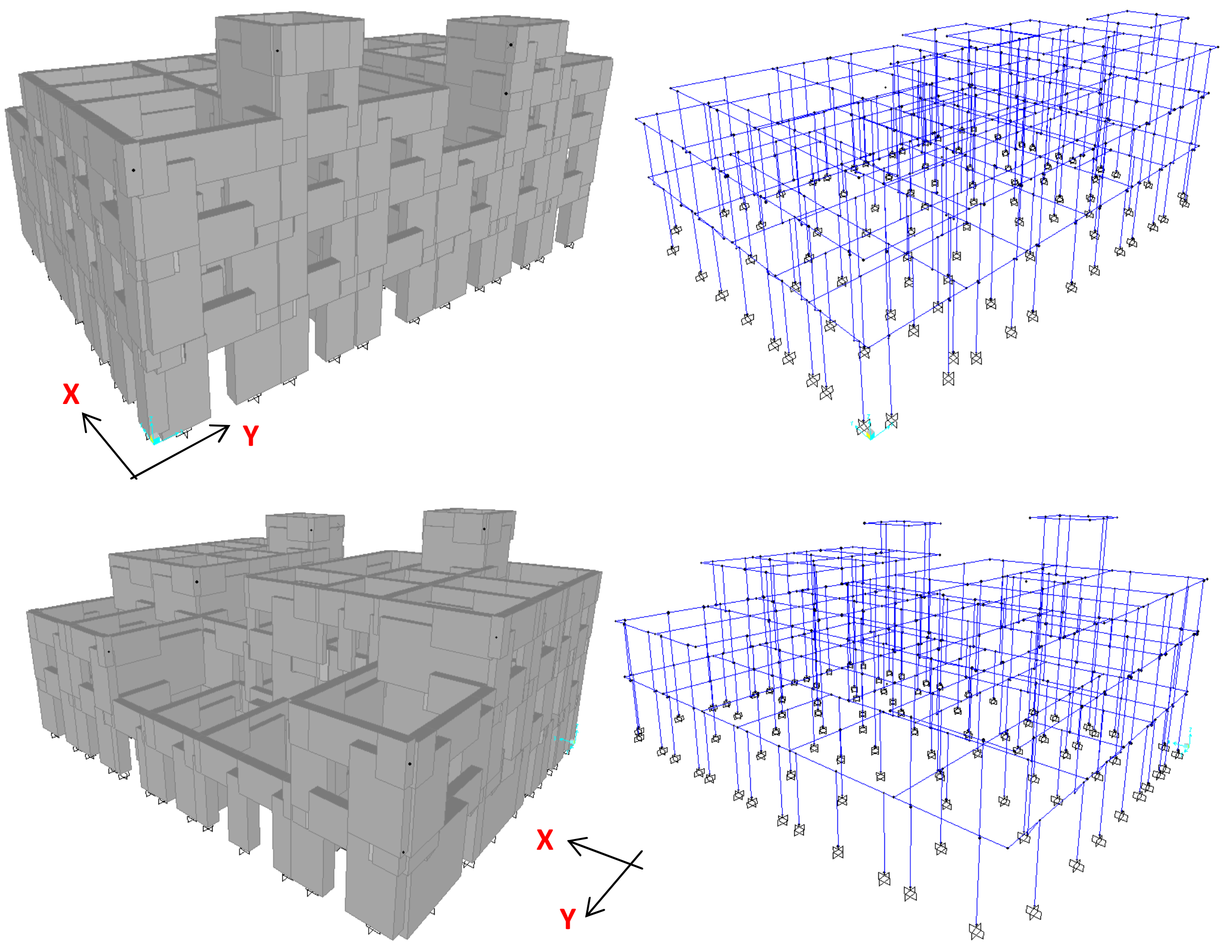


Fig. 42. 3D view of the structural model (BT"A"): Solid and unifilar schemes.

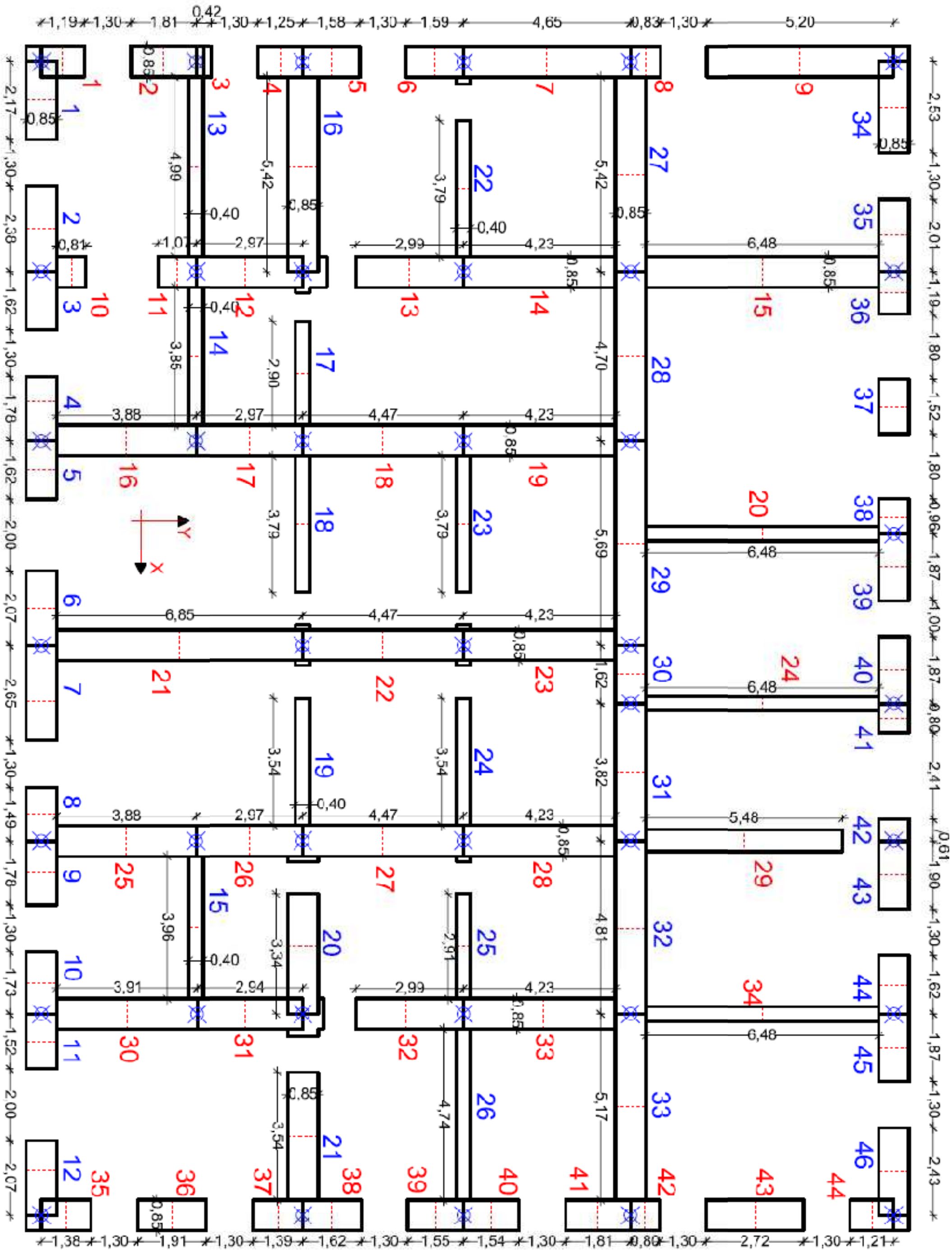


Fig. 43. Structural plan BT"A". Ground Level (+4.15 m).

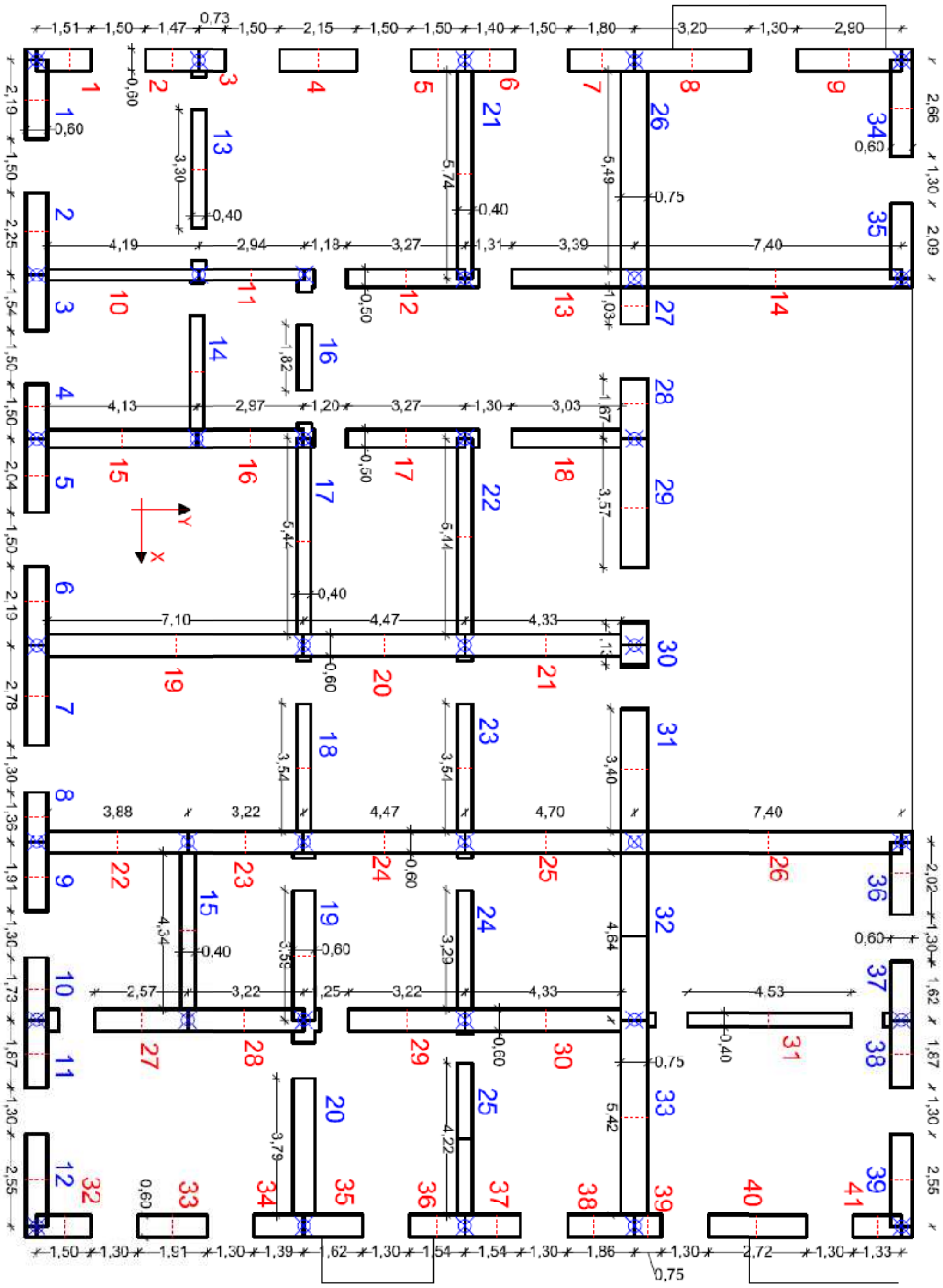


Fig. 44. Structural plan BT"A". First Floor (+8.35).

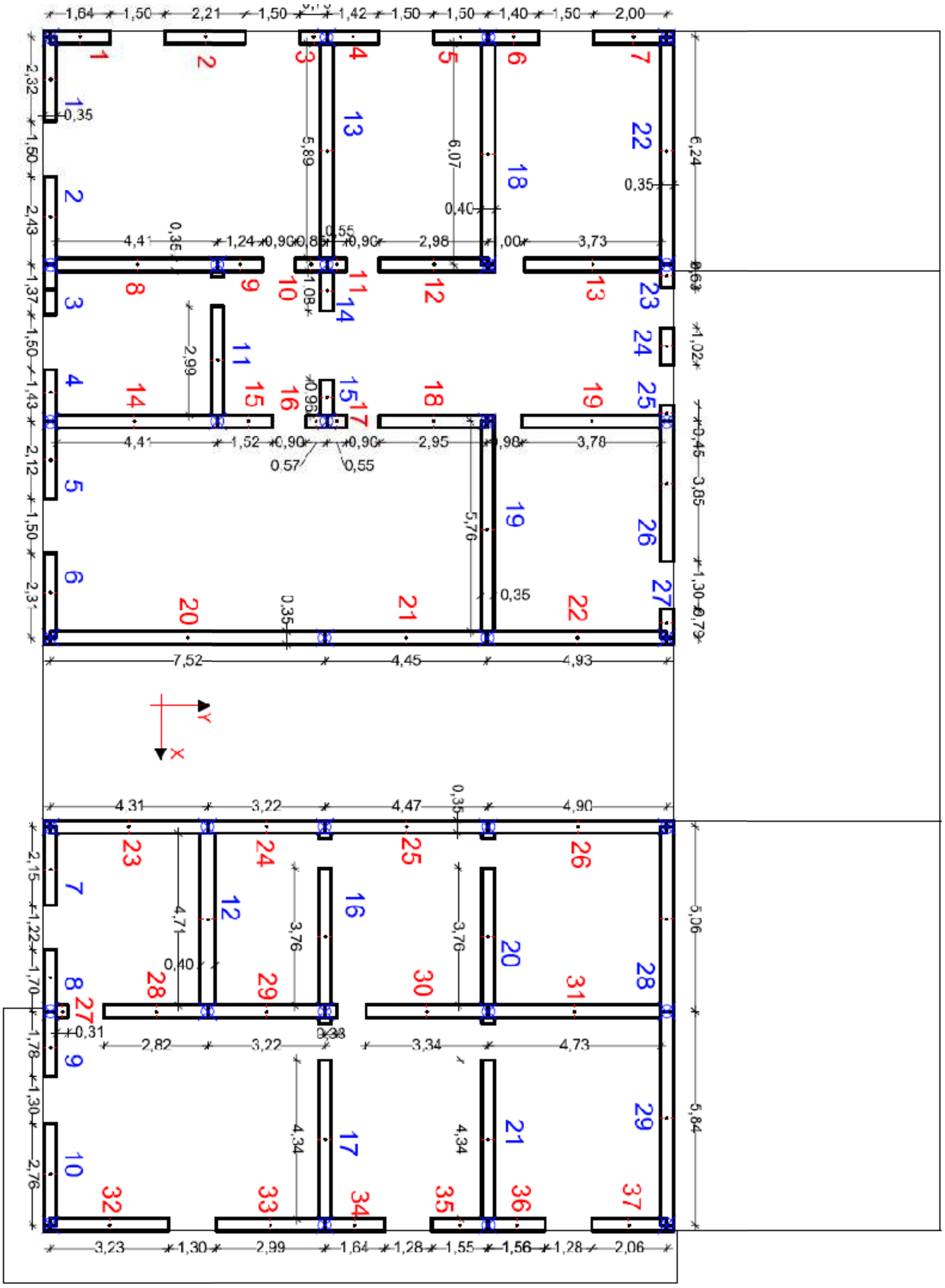


Fig. 45. Structural Plan ET"A". Second Floor (+11.85).

Modelling of mechanical nonlinearities

The introduction of non-linearity in the mechanical model is made through the insertion of shear plastic hinges on the vertical elements. The hinge type is τ–γ (shear-angular sliding) by noting that with the regime of small displacements, the angular sliding is equal to the drift δ/h (Fig. 46-a) if one assumes that the length of the plastic hinge is precisely the entire height. The law is elastic - perfectly plastic (Fig. 46-a) and is characterized by a yielding point in correspondence of the maximum shear stress τ_r calculated according to the expression of Turnšek and Cacovic below reported

$$\tau_r = \tau_{r//} = \frac{1.5\tau_0}{b}\sqrt{1+\frac{\sigma_0}{1.5\tau_0}}$$

being τ_0 the shear strength in the absence of vertical loads as previously defined in Tab. 5, σ_0 the compression stress acting on the wall and b a parameter which takes into account the shape of the wall, varying in the range 1-1.5, and that is assumed to be on average *1.25.*

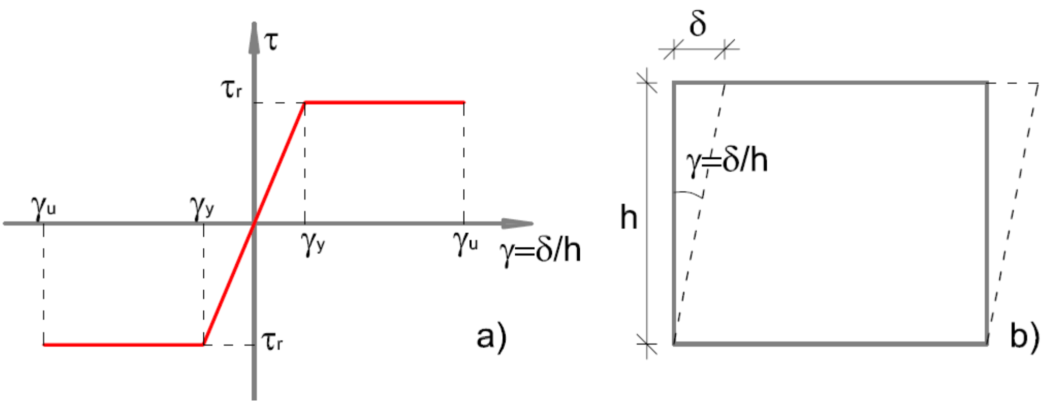


Fig. 46. Stress-Strain Law for shear plastic hinges.

As ultimate drift for the walls it was assumed the value $\gamma_u = \delta_u / h = 0.004$ defined by the DM 14.01.2008 for the life prevention limit state.

The evaluation of τ_r according to the expression of Turnšek and Cacovic is assumed for the walls that are parallel to the direction of the earthquake and will be referred to hereafter $\tau_{r//}$. Since it was necessary to assume a nonlinear model also for the orthogonal walls, taking into account that the crisis of these if dominated by a flexural failure and not a shear failure, it was defined a fictitious shear plastic hinge that is activated in correspondence of the achievement of the shear force associated with the ultimate moments at the ends of the perpendicular wall. Considering an average length l^* for the wall equal to 1 m and the thickness *t*, the shear stress for the orthogonal walls $\tau_{r\perp}$ associated with the flexural mechanism is obtained as

$$\tau_{r\perp} = \frac{2M_u}{l^* t}$$

	W (seismic weight)	N (floor axial load)	σ_0	τ_0	$\tau_{r//}$	$\tau_{r\perp}$
	kN	*kN*	*N/mm²*	*N/mm²*	*N/mm²*	*N/mm²*
Floor 1	7493,00	7493,00	0,191	0,035	0,090	0,030
Floor 2	15440,40	22933,40	0,142	0,035	0,081	0,015
Floor 3	20316,80	43250,20	0,094	0,035	0,070	0,009

Table 6. Reference values for the evaluation of resisting shear stress.

<u>Modal analysis</u>

The modal analysis has detected a significant irregularity in the structural response especially in the *X* direction where the participating mass is distributed on the first two modes at frequencies between 3.5 and 4.10 *Hz*. In direction *Y* the third mode has a mass concentration of 55% at a frequency of about 5 *Hz*. The distribution of the participating masses in the first 12 modes in *X* and *Y* directions is represented in Fig. 47.

The modal shapes associated with the first 3 modes are shown in Fig. 48, and show a significant torsional component in the motion of the building. This condition is justified by the non-regular variation of the plan conformation from one floor to the next that generates significant irregularities in elevation. The details related to the frequencies, periods and participating masses for the first 12 modes are also shown in Tab. 7.

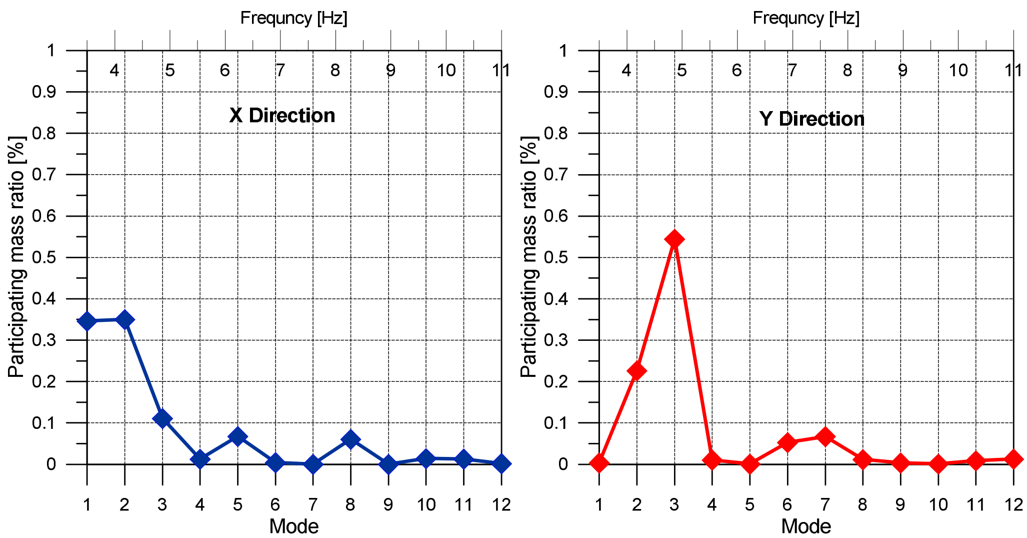
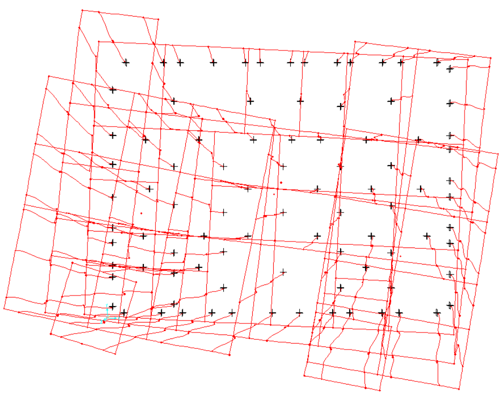
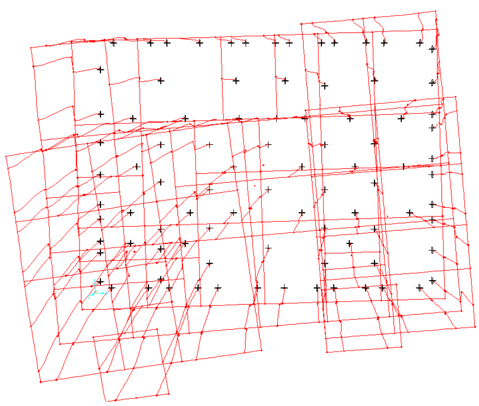


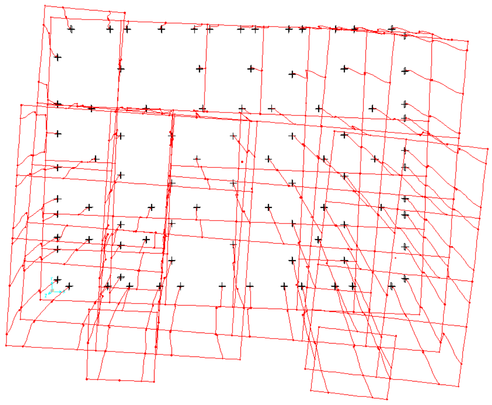
Fig. 47. Participating mass within the first 12 modes in directions X e Y.



Mode 1: T= 0.286 s; f=3.49 Hz



Mode 2: T= 0.243 s; f=4.10 Hz



Mode 3: T= 0.237 s; f=4.21 Hz

Fig. 48. Modal shapes: Modes 1, 2 e 3.

Mode	T (s)	f (Hz)	M% (X)	M%(Y)
1	0,286	3,495	34,7%	0,4%
2	0,244	4,105	35,0%	22,6%
3	0,237	4,211	11,1%	54,4%
4	0,153	6,549	1,3%	1,1%
5	0,127	7,892	6,8%	0,1%
6	0,125	8,020	0,5%	5,3%
7	0,111	8,971	0,0%	6,7%
8	0,108	9,295	6,1%	1,2%
9	0,104	9,576	0,0%	0,4%
10	0,098	10,254	1,4%	0,1%
11	0,096	10,396	1,3%	0,9%
12	0,090	11,127	0,2%	1,3%

Table. 7. Parameters resulting from the modal analysis.

Pushover analysis for the evaluation of early damage and collapse accelerations

The pushover analysis was carried out in order to define the capacity of the building specifically in terms of early damage and collapse accelerations. Given the strategic role of the building the elastic response spectrum was defined considering a nominal reference life V_N*=100* years and a IV class of use C_u*=2.0*. The return period associated with this conditions is T_R=2475 years and the consequent parameters for the spectral characterization are shown in the table below

a_g [g]	F_0	T_c* [s]
0.0747	3.09	0.401

Table. 8. Parameters for spectral characterization.

Finally, taking into account a Class B for the soil and topographical configuration T1, the resulting spectral amplification coefficient is *S=1.2,* associated with T_C*=0.53 s* (spectral period corresponding to the end of the constant acceleration branch). The Fig. 49 shows the reference elastic response spectrum in ADRS format (acceleration - displacement).

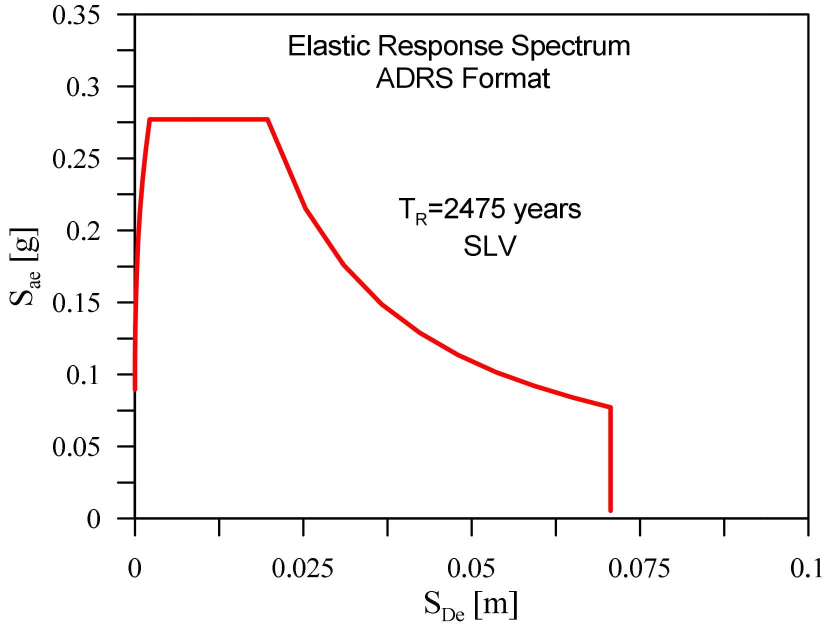


Fig. 49. Reference response speculum in ADRS format .

In order to take into account the behavior of the structure beyond the elastic limit, as well as also suggested by the codes, the pushover analysis was repeated with two force profiles for each direction. The first distribution assumed is a “modal profile”, which is proportional to the product of the predominant eigenvector in the direction considered by the masses of the floors. The second distribution is uniform profile providing forces simply proportional to the masses of the floors. The profiles are normalized with respect to the top value of the reference eigenvector. For the *X* direction of the greater participant mass is concentrated in mode 2, while for the *Y* direction in mode 3. The following table (Tab. 9) summarizes the data for the identification of profiles the shape of the latter.

Direction	X				Normalized force profiles	
Mode	Floor	Φ_1 [m]	m_i [kNs^2m^{-1}]	$\Phi_1 \times m_i$	modal	uniform
2	3	0,012	1073	12,88	1,00	1,00
T [s]	2	0,0105	2039	21,41	1,66	1,90
0,244	1	0,0048	3119	14,97	1,16	2,91
Direction	Y					
Mode	Floor	Φ_1 [m]	m_i [kNs^2m^{-1}]	$\Phi_1 \times m_i$	modal	uniform
3	3	0,011	1073	11,80	1,00	1,00
T [s]	2	0,012	2039	24,47	2,07	1,90
0,237	1	0,0053	3119	16,53	1,40	2,91

Table. 9. Determination of the profiles.

Given that the shear hinges introduced are not sensitive to the axial load variation, the profiles were assigned with a single sign for each direction considered. The response in terms of base shear (*V*) - roof displacement (*d*) obtained by the 4 analyses considered for the multi degrees of freedom system (MDOF) is shown in Fig. 49. The curves are linked to those of the equivalent single degree of freedom system (SDOF) (Fig. 51) through the following relationships

$$V^* = \frac{V}{\Gamma_1};\ d^* = \frac{d}{\Gamma_1}$$

where Γ is the modal participation factor for the predominantly mode in the direction considered. The identification of the properties of SDOF is performed associating a bilinear equivalent curve. In this way the mass, the stiffness and the period of the equivalent SDOF can be calculated as:

$$m^* = \sum_{i=1}^{n} \Phi_{i1} m_i \,; \quad k^* = \frac{F_y^*}{d_y^*} \,; \quad T^* = 2\pi \sqrt{\frac{m^*}{k^*}}$$

The values are reported in Tab.10.

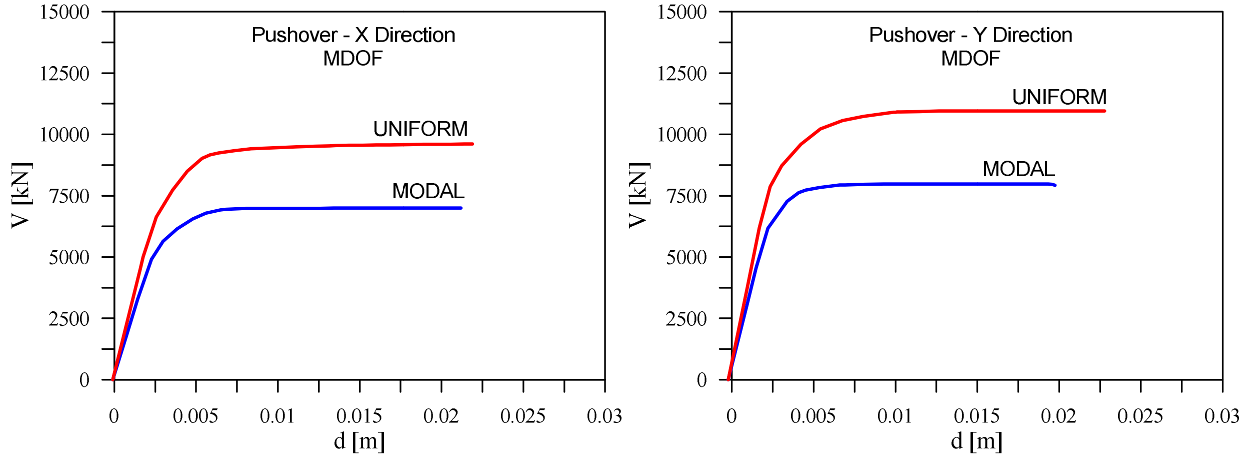


Fig. 50. Capacity curves of the MDOF systems.

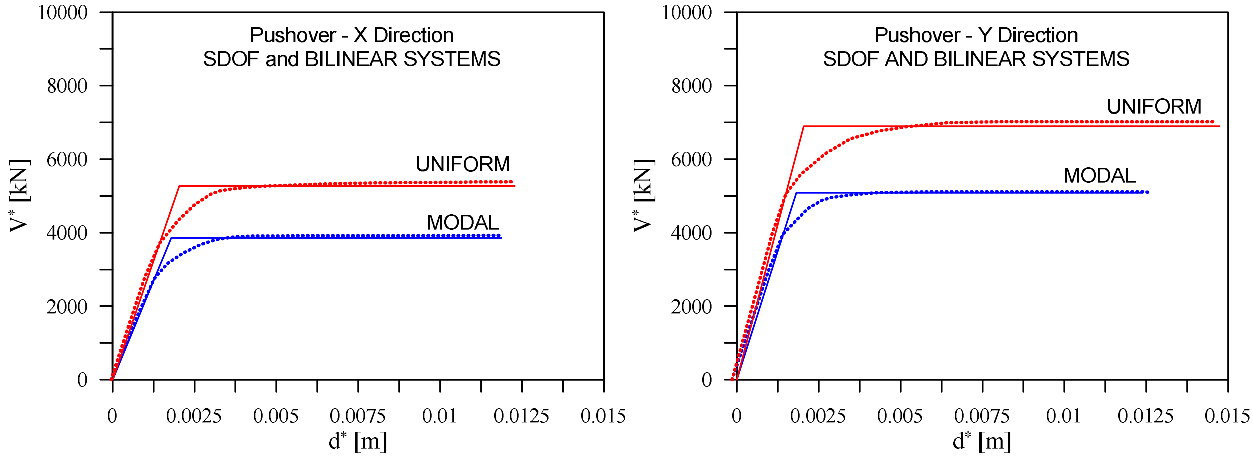


Fig. 51. Capacity curves of the SDOF systems and bilinear equivalent curves.

	Dir X MOD	**Dir X UNI**	**Dir Y MOD**	**Dir Y UNI**
k* [kN-m]	2158105,45	2587950,43	2808123,13	3374419,02
m* [kNs²/m]	3891,67	3891,67	5295,45	5295,45
T* [s]	0,267	0,24	0,27	0,25
***F^*_y* [kN]**	3862,10	5267,68	5089,76	6900,31
***d^*_y* [m]**	0,00179	0,0020	0,0018	0,0020
d_u* [m]	0,0119	0,0123	0,0124	0,0147
Γ [kNs²]	1,78		1,56	

Table. 10. Properties of the bilinear equivalent SDOF systems.

Once determined the capacity curves, it was firstly verified the capacity of the structure to support the request associated to the earthquake with the reference spectrum for the 4 conditions considered. This test gives also an idea about the reliability of the vulnerability index detected by the GNDT procedure and for better comprehension can be performed in the ADSR plane overlapping the constant ductility non nonlinear demand spectrum and the bilinear curve of the SDOF (taking care in this care to normalize the ordinates by the mass *m**). In this way it is possible to evaluate for each SDOF the yield acceleration S_{ay} and the acceleration that would be required to an indefinitely elastic system having the same period *T**, respectively, as:

$$S_{ay} = \frac{F_y^*}{m^*}; \quad S_{ae} = S_{ae}(T^*) = \frac{F_e^*}{m^*}$$

the reduction factor *q** is thus evaluable as:

$$q^* = \frac{S_{ae}}{S_{ay}}$$

Verifying that for all the SDOFs $T^* < T_c$, the requested ductility for all of them (each one characterized by *T** and *q**) can be calculated by the following expression (Miranda and Bertero (1993))

$$\mu_r = (q^* - 1)\frac{T_c}{T^*} + 1 \qquad (T^* < T_C)$$

The acceleration and displacement components for the non-linear spectrum having the constant ductility μ_r , are obtained by the expressions (Vidic et al. (1994))

$$S_a = \frac{S_{ae}}{q(\mu_r, T)}$$

$$S_d = \frac{\mu_r}{q(\mu_r, T)} S_{de} = \frac{\mu_r}{q(\mu_r, T)} \frac{T^2}{4\pi^2} S_{ae} = \mu_r \frac{T^2}{4\pi^2} S_a$$

Since the ductility has to remain constant, in the previous expressions only μ_r is fixed, while the factor *q* varies with the period *T.*

For the 4 cases considered the superposition of the demand spectra of and capacity curves has led to the results shown in Fig. 52-53. It can be noted that the displacements associated with the capacity curves are always greater than the request displacement identified by the performance point. This evidence, however, appears to be consistent with the mid-low value of vulnerability (*V=36.29*) calculated for the building. In Figs. 54-55 are reported the most critical failure mechanisms detected through the pushover analysis for the directions *X* and *Y*. The profiles who have determined the most critical conditions, are for both the directions, the modal profiles, which require a larger capacity for the higher floors. The collapse mechanisms detected, are substantially localized at the first elevation, and due to the significant variation in lateral stiffness and resistance that occurs from the ground floor to the next. In these conditions the resistant capacity and overall deformation of the building are comparable to that owned by the single floor.

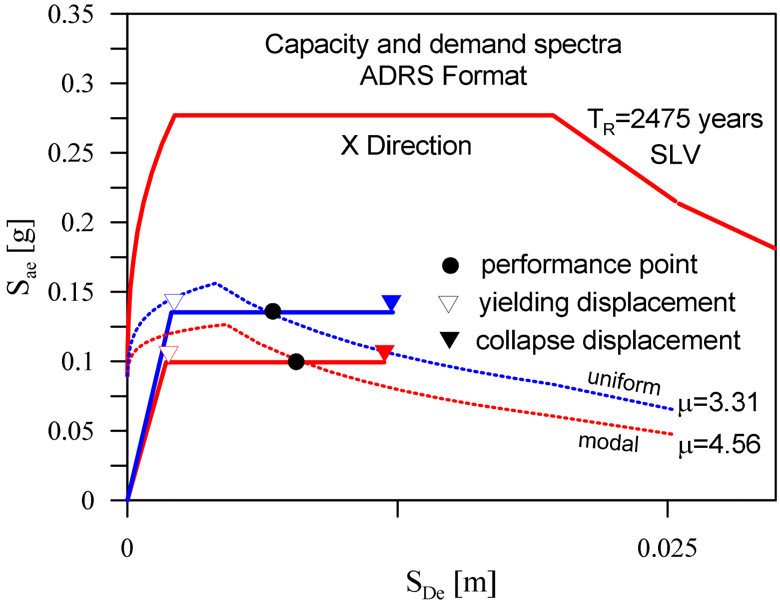


Fig. 52. Capacity and demand spectra in AD format. X Direction.

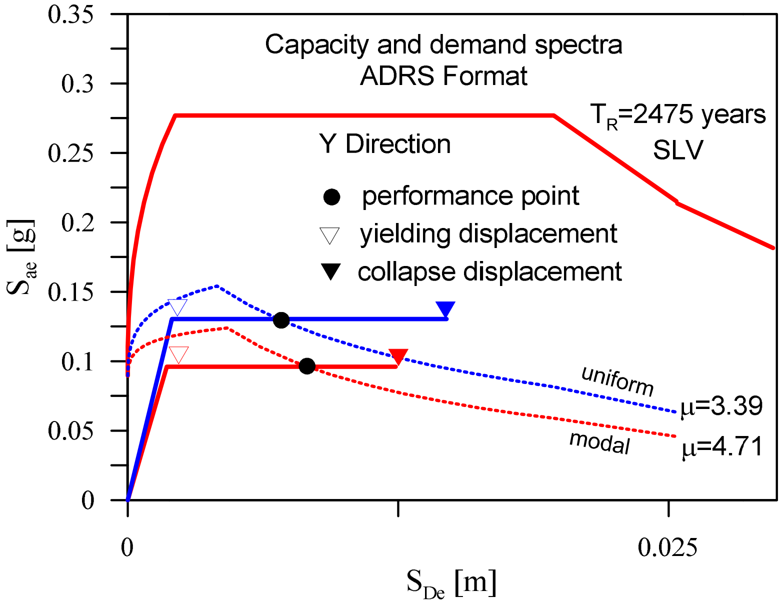


Fig. 53. Capacity and demand spectra in AD format. Y Direction.

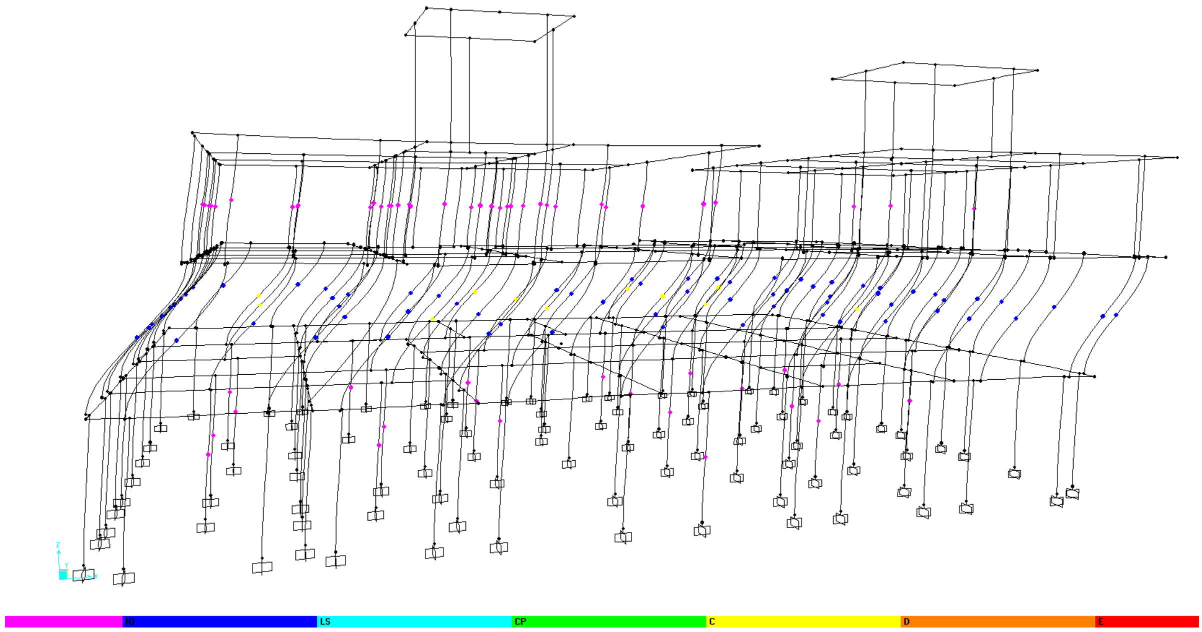


Fig. 54. Collapse mechanism for X direction.

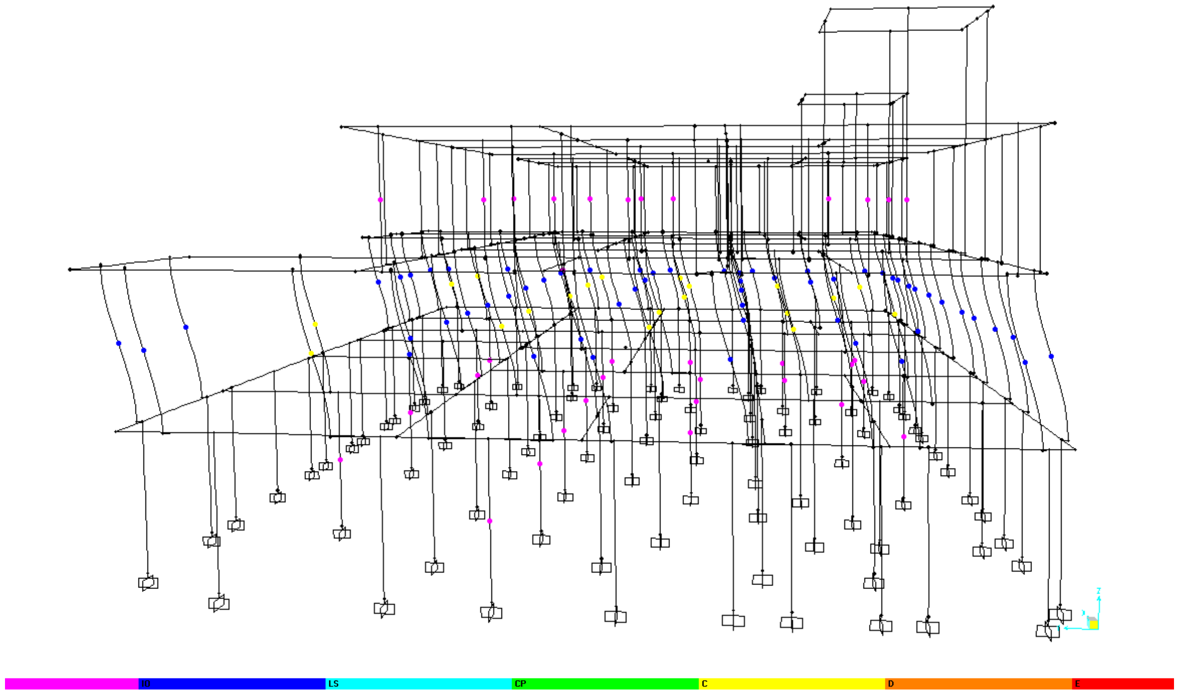


Fig. 55. Collapse mechanism for Y direction.

In order to define the fragility functions previously defined, it is necessary to calculate the peak ground accelerations (PGA) corresponding to the beginning of the damage (PGA_i) and collapse (PGA_c). The first condition was associated to the yielding displacement, the second to the collapse displacement.

The previous expression by Miranda and Bertero in the case of $T^*<T_c$ can be rewritten as

$$d^*_{r,max} = \frac{d_e^*}{q^*}\left[(q^*-1)\frac{T_c}{T^*}+1\right] \qquad T^*<T_C$$

which expresses, for a system characterized by q^* and T^*, the relationship between the inelastic displacement demand $d^*_{r,max}$ and the displacement that would be required for the ideal indefinitely elastic and d^*_e.

By fixing the reduction factor $q^*=\tilde{q}^*$ (reduction factor recalculated as function of the actually available ductility), the period T^* and each time by replacing the limit values of the yielding displacement d_y^* and ultimate displacement obtained d_u^*, it is possible to calculate the displacement d_e^* associated to an elastic response spectrum characterized by a different PGA value as

$$d^*_{e,u} = S_{De,u}(T^*) = \frac{d_u^*\tilde{q}^*}{\left[(\tilde{q}^*-1)\frac{T_c}{T^*}+1\right]}\,;\; d^*_{e,y} = S_{De,u}(T^*) = \frac{d_y^*\tilde{q}^*}{\left[(\tilde{q}^*-1)\frac{T_c}{T^*}+1\right]}$$

The reduction factor $\tilde{q}^*$ is recalculated as function of the actually available ductility by means of the expression

$$\tilde{q}^* = 1+(\mu_d-1)\frac{T^*}{T_C} \qquad (T^*<T_C)$$

once assigned the following values for the ductility corresponding to the early damage and collapse.

$$\mu_{d,y} = 1; \qquad \mu_{d,u} = \frac{d_u^*}{d_y^*}$$

the associated spectral acceleration can be thus calculated as

$$S_{ae,u}(T^*) = \frac{4\pi^2}{T^{*2}} S_{De,u}(T^*); \quad S_{ae,y}(T^*) = \frac{4\pi^2}{T^{*2}} S_{De,y}(T^*)$$

Since in this case $T_B \leq T^* \leq T_C$, the expression of the response spectrum is

$$S_{ae}(T^*) = PGA \times S \times F_0$$

Substituting the values $S_{ae,u}(T^*)$ and $S_{ae,y}(T^*)$ the PGA values are obtained. Recalling the position *y=a/g* one obtains

$$y_i = PGA_i = \frac{S_{ae,y}(T^*)}{S \times F_0}; \quad y_c = PGA_c = \frac{S_{ae,c}(T^*)}{S \times F_0}$$

The reference PGA_c and PGA_i values calculated for the different load profiles considered are reported in Tab. 11 within the other parameters necessary for their determination.

Collapse PGA (PGA_c)							
	d_u^*	$\tilde{q}^*$	T_c	T*	d^*_{eu}	Se(T*)	$y_c=PGA_c$ [g]
DIR X mod	0,0120	3,84	0,529	0,267	0,0069	3,85	**0,106**
DIR X uni	0,0123	3,28	0,529	0,240	0,0067	4,57	**0,126**
DIR Y mod	0,0124	3,98	0,529	0,270	0,0072	3,90	**0,107**
DIR Y uni	0,0147	3,93	0,529	0,250	0,0080	5,07	**0,140**
Early damage PGA (PGA_i)							
	d_y^*	$\tilde{q}^*$	T_c	T*	d^*_{ey}	Se(T*)	$y_i=PGA_i$ [g]
DIR X mod	0,0018	1,0	0,529	0,267	0,0018	0,99	**0,0272**
DIR X uni	0,0020	1,0	0,529	0,240	0,0020	1,39	**0,0383**
DIR Y mod	0,0018	1,0	0,529	0,270	0,0018	0,98	**0,0270**
DIR Y uni	0,0020	1,0	0,529	0,250	0,0020	1,29	**0,0355**

Table. 11. PGA_c and PGA_i for the calculated for the considered analyses.

5.3 Calibration of the fragility functions for masonry buildings

The analyses previously carried out allowed to determine the reference critical accelerations for the building, characterized by a vulnerability index ***V=36.29***. In particular, reference was made to minimum values respectively for accelerations of early damage and collapse which are shown below

$$y_i = 0{,}0270\ g\ ;\ \ y_c = 0{,}106\ g$$

These values are used to calibrate the coefficients governing the *y(V)* relationships according to the model by Guagenti and Petrini (1989) in order to provide expressions suitable for the buildings of the city centre of Lampedusa (Fig. 56). The values of the coefficients α_i, β_i, α_c, β_c e γ which allowed to achieve the best correspondence between the collapse and early damage accelerations and the vulnerability, obtainable by the analytical expressions are given in Tab. 12.

α_i	0,0578	α_c	1,9371	γ	2,423
β_i	0,0210	β_c	0,00123		

Tab. 12. Parameters calibrating the y(V) relationships *(masonry).*

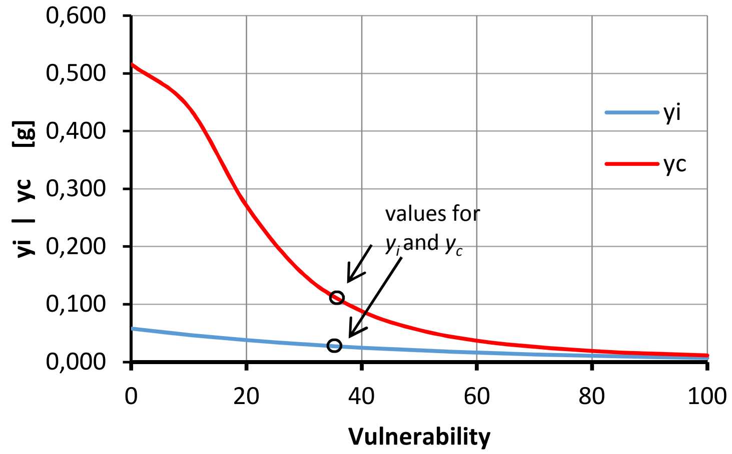


Fig. 56. Calibration of the *y(V)* relationships *for the masonry buildings of the city centre of Lampedusa*

Consequently, the fragility functions for masonry buildings of the city centre of Lampedusa are univocally identified at the different vulnerability indexes (Fig. 57). The latter, once known the vulnerability index of a building, allow to determine the level of damage that this will undergo as function of the severity of the earthquake, identified by the peak ground acceleration. This tool is particularly useful since it allows to make damage estimations on buildings for gives scenarios of seismic intensity and can be used for organizational purposes for the emergency management.

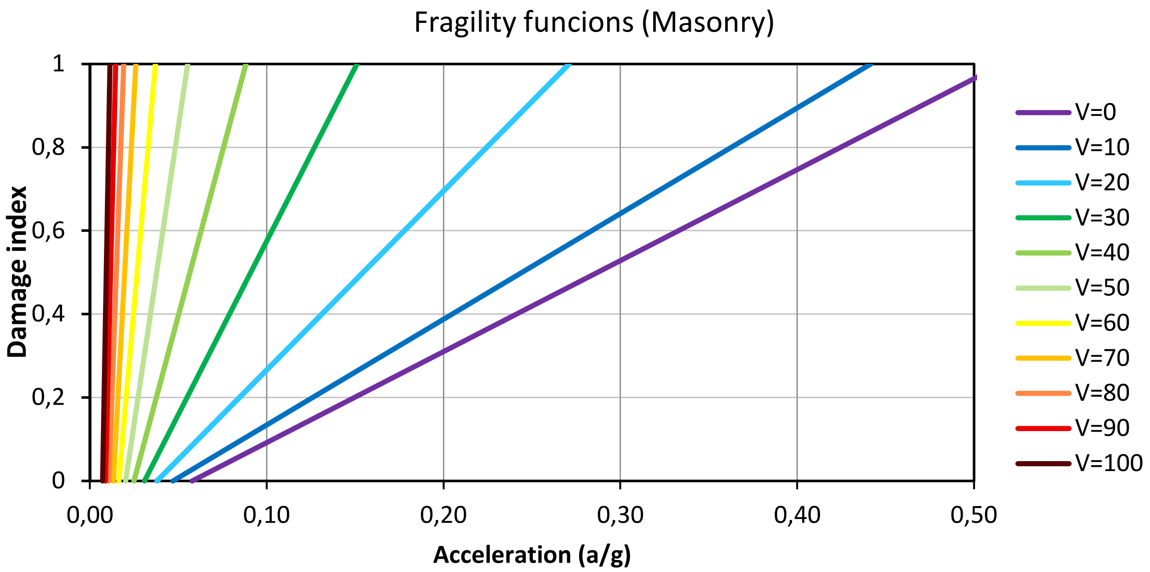


Fig. 57. Fragility functions for the masonry buildings of the city centre of Lampedusa.

5.4 Calibration of the fragility functions for RC buildings

Since no experimental data were available for RC building, the calibration of the *y(V)* curves and the corresponding fragility functions has been carried out using a simplified method (Dolce et al. (2004) for the calculation of the accelerations of collapse and start damage.

Although this method is based on simplified assumptions, it allows to consider a significantly higher number of samples of buildings than those that can be actually investigated experimentally.

The PGA value of the earthquake producing the collapse of the structure can be obtained by calculating the corresponding spectral acceleration.

Considering for the value of spectral acceleration linear static values, the relationship between S_a and *PGA* adopted by the model adopted, is:

$$S_a = PGA \times \alpha_{PM} \times \alpha_{AD} \times \alpha_{DT} \times \left(\frac{1}{\alpha_{DUT}} \right)$$

where:

α_{PM} is a reductive coefficient that is function of the modal participation factor for the fundamental mode, which is assumed to be equal to 1 for one-story buildings in and 0.8 for multi-storey buildings;

α_{AD} is the spectral amplification and corresponds to the value F_0 given by DM 14.01.2008.

α_{DT} is a coefficient taking into account the dissipation capacity of the building. For RC buildings is equal to 1 or to 0.8, respectively in the case where the contribution infill is or is not directly put into account in the resistance of the structure.

α_{DUT} is equivalent to the reduction factor which reduces the intensity of the seismic action as a function of structural ductility. For RC buildings it is possible to assume a prudential value between 2 and 3 depending on the structural regularity. For the evaluation of PGA_i (early damage) this value is assumed equal to 1.

The PGA value associated with the collapse conditions or initial damage is then obtained as

$$PGA = \frac{S_a}{\alpha_{PM} \times \alpha_{AD} \times \alpha_{DT} \times \left(\frac{1}{\alpha_{DUT}} \right)}$$

determining the value of S_a as the ratio between the resisting base shear V_R calculated by simplified rules and its weight W , also calculated by approximation:

$$S_a = \frac{V_R}{W}$$

This operation has been performed for all the reinforced concrete buildings of the urban centre of Lampedusa, which had previously accounted for the assessment of the vulnerability according to the GNDT procedure. 24 buildings have been considered, representing the 90% of the RC existing buildings. Tab. 14 shows the values calculated for PGA_c and PGA_i for the buildings in question.

The values are reported on the vulnerability - intensity diagram representing the distribution of collapse and early damage accelerations at the different vulnerabilities (Fig. 58). Through a best fitting of the points it is possible to calibrate the functions $y_i(V)$ and $y_c(V)$ for reinforced concrete buildings of the urban centre of Lampedusa. The values of the calibration coefficients obtained are reported in Tab. 13.

α_i	0,270	α_c	1,637	γ	2,2087
β_i	0,0280	β_c	0,000904		

Table. 13. Parameters for calibration of y(V) functions (reinforced concrete).

The associated fragility function are reported in Fig. 59.

Code Building	N° Floors	Base shear V_R [kN]	Weight W [kN]	S_a	α_{PM}	α_{AD}	α_{DT}	$\alpha_{DUT\,(c)}$	$\alpha_{DUT\,(i)}$	PGA_c	PGA_i	V
33	1	135	218,71	0,617	1	2,88	1	2,5	1	0,536	0,214	13
42	1	60	142,96	0,420	1	2,88	1	2,5	1	0,364	0,146	18
44d	3	337	1084,89	0,311	0,8	2,88	1	3	1	0,404	0,135	40
51a	2	90	254,64	0,353	0,8	2,88	1	3	1	0,460	0,153	20
59	2	120	446,93	0,268	0,8	2,88	1	3	1	0,350	0,117	28
60	1	180	340,75	0,528	1	2,88	1	2,5	1	0,459	0,183	15
67	3	240	766,32	0,313	0,8	2,88	1	3	1	0,408	0,136	20
87a	3	144	480,2	0,300	0,8	2,88	1	3	1	0,390	0,130	25
95a	2	60	203,61	0,295	0,8	2,88	1	3	1	0,384	0,128	10
100	1	210	393,57	0,534	1	2,88	1	2,5	1	0,463	0,185	8
214	3	480	2108	0,228	0,8	2,88	1	3	1	0,296	0,099	33
219	2	90	318,64	0,282	0,8	2,88	1	2	1	0,245	0,123	50
223	2	120	385,68	0,311	0,8	2,88	1	2,5	1	0,338	0,135	28
227	2	240	401,01	0,598	0,8	2,88	1	2,5	1	0,649	0,260	8
229	2	165	271,96	0,607	0,8	2,88	1	2,5	1	0,658	0,263	10
230	2	225	616,8	0,365	0,8	2,88	1	2,5	1	0,396	0,158	30
244	2	135	354,12	0,381	0,8	2,88	1	2,5	1	0,414	0,165	18
245a	4	180	625,73	0,288	0,8	2,88	1	2,5	1	0,312	0,125	25
245B	2	225	569,05	0,395	0,8	2,88	1	2,5	1	0,429	0,172	33
256	3	210	872,91	0,241	0,8	2,88	1	2,5	1	0,261	0,104	38
272	2	96	246,65	0,389	0,8	2,88	1	2,5	1	0,422	0,169	20
274	2	90	153,47	0,586	0,8	2,88	1	2,5	1	0,636	0,255	5
296	3	120	797,3	0,151	0,8	2,88	1	2,5	1	0,163	0,065	38
310	3	135	474,05	0,285	0,8	2,88	1	2,5	1	0,309	0,124	20

Table. 14. Values of PGA_c and PGA_i for the buildings considered

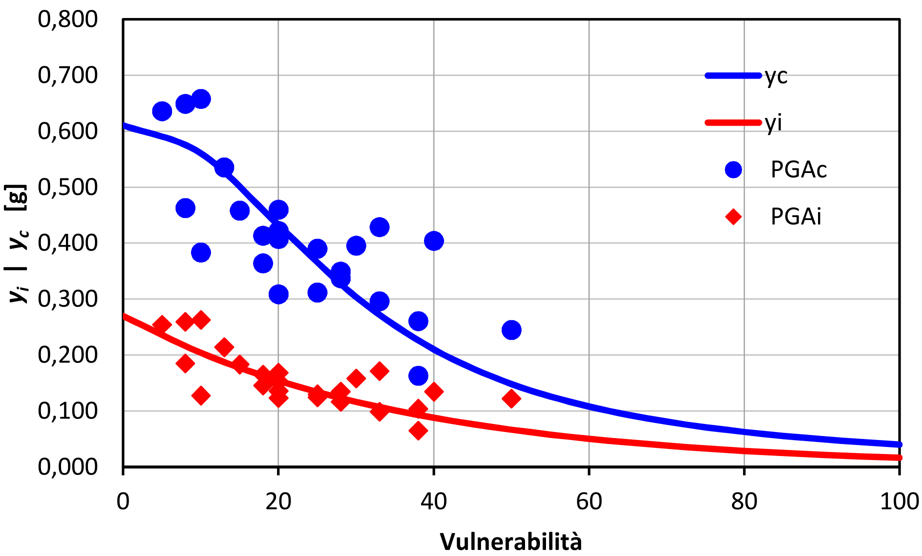


Fig. 58. Distribution of PGA_c and PGA_i values and y(V) functions calibrated for the RC buildings of the urban centre of Lampedusa

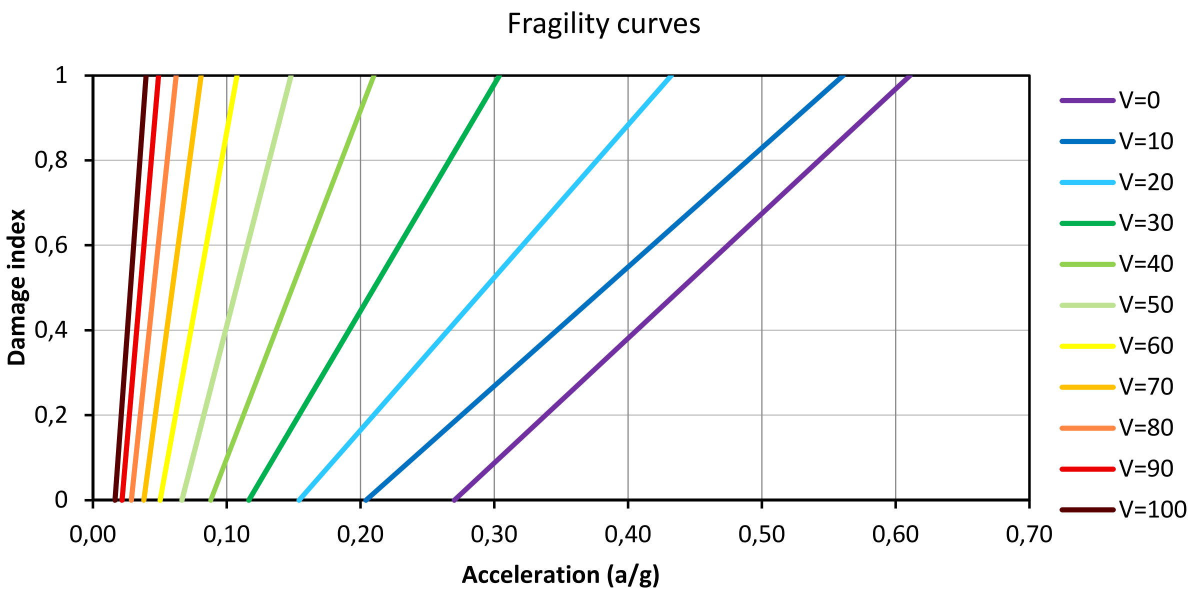


Fig. 59. Fragility functions for the RC buildings of the city centre of Lampedusa.

6. VULNERABILITY OF THE URBAN CENTRE OF LAMPEDUSA: CONCLUSIONS

In this section are illustrated and discussed the results of the assessment of the seismic vulnerability of the centre of the island of Lampedusa, obtained according to the procedures described in previous chapters. The operations involved an area comprising almost all of the buildings in the city centre. The Fig. 60 shows the extension of the area investigated.

As previously highlighted the prevailing structural typology is masonry (of calcarenite or concrete blocks). The majority of masonry buildings are configured as aggregate of buildings and therefore the assessment of the vulnerability regarded entire blocks in which the primary structure is shared by the component buildings. The following table summarizes the data of the investigation campaign carried out, involving a number of 288 individual buildings or building aggregates.

Total buildings	Masonry buildings	Reinforced concrete buildings
288	264	24
	91,7%	**8,3%**

Table 15. Quantitative data on the buildings analysed.

A first statistical output, relative to buildings in masonry, is observable in Fig. 61 in which is represented the probabilistic distribution of the normalized vulnerability index. It is clear that the vulnerability is settled to low-mid levels. The average normalized vulnerability index is 25.30, while the maximum does not exceed 50. The distribution, however, shows a wide variance.

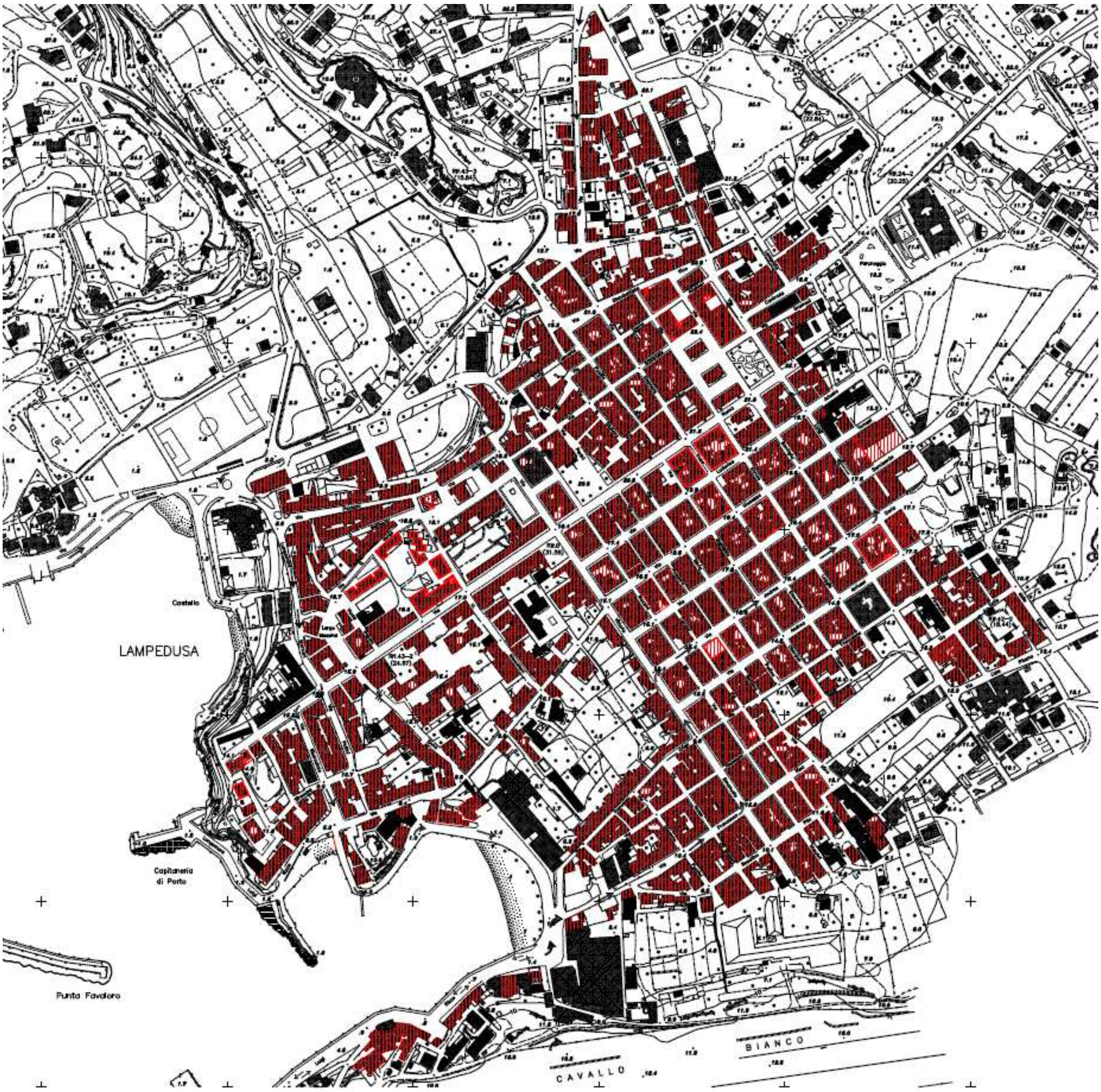


Fig. 60. Building aggregates involved in the assessment of the vulnerability.

The overall overview that has emerged is in agreement with the predictions made in the opening chapters, in which a good general condition was evidenced (construction details executed properly, presence of curbs and rigid floors, limited height). The elements of major criticality detected regard essentially the presence of aggregates building with strong irregularities in elevation. These buildings reached in fact the highest levels of vulnerability among those found reaching values between 35 and 45.

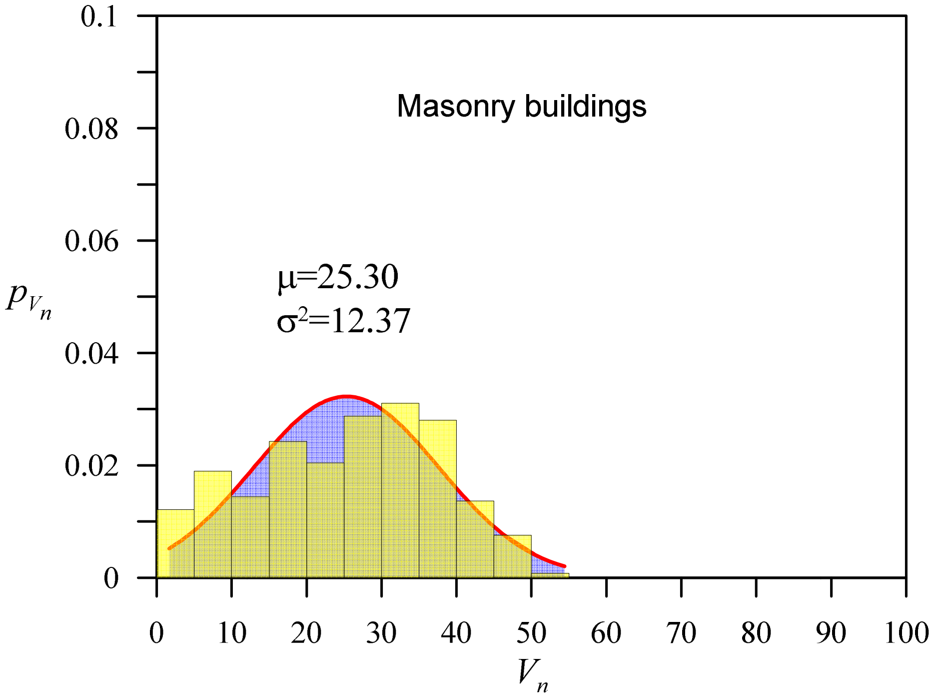


Fig. 61. Probabilistic distribution of the vulnerability index for masonry buildings.

The following charts (Fig. 62-63) report the percentage amount found for the classes of each of the 11 parameters considered in the assessment forms.

Subsequently, making use of fragility curves defined in the previous chapter, it was provided a quantitative prediction of the possible damage scenarios for the buildings of the city centre associated to earthquakes of different severity. In particular the diagrams in Fig. 64 show the estimation of the percentage of the buildings involved at different levels of damage to earthquakes having return periods of 475, 975 and 2475 years. It can be observed that, due to the relatively low vulnerability detected, the damage scenario is quite limited for the first two levels of intensity considered for the earthquake. More than a half of the buildings have damage indexes comprised between 5 and 20%.

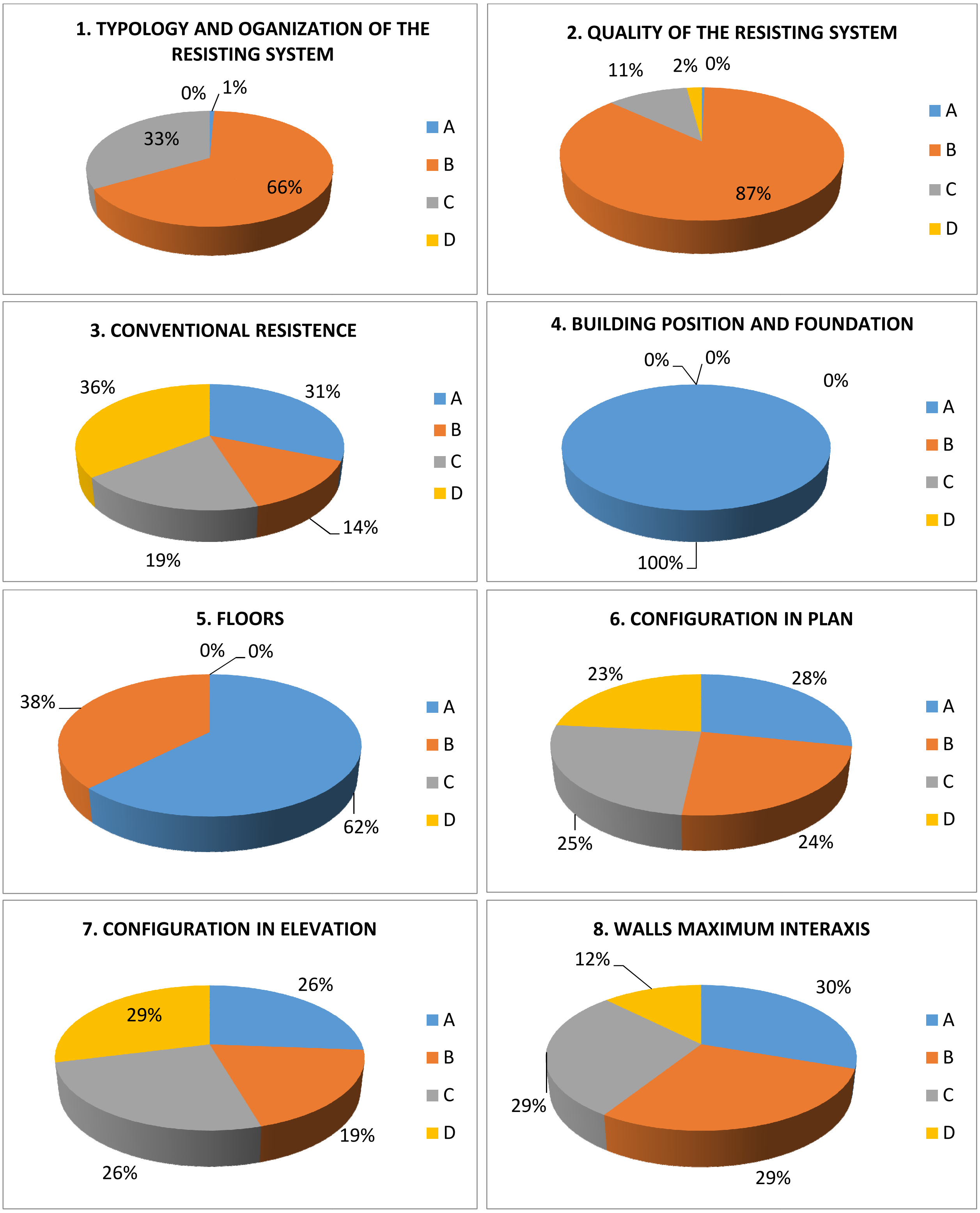


Fig. 62. Percentage amount found for the classes of each of the 11 parameters considered in the assessment forms of masonry buildings (parameters 1-8).

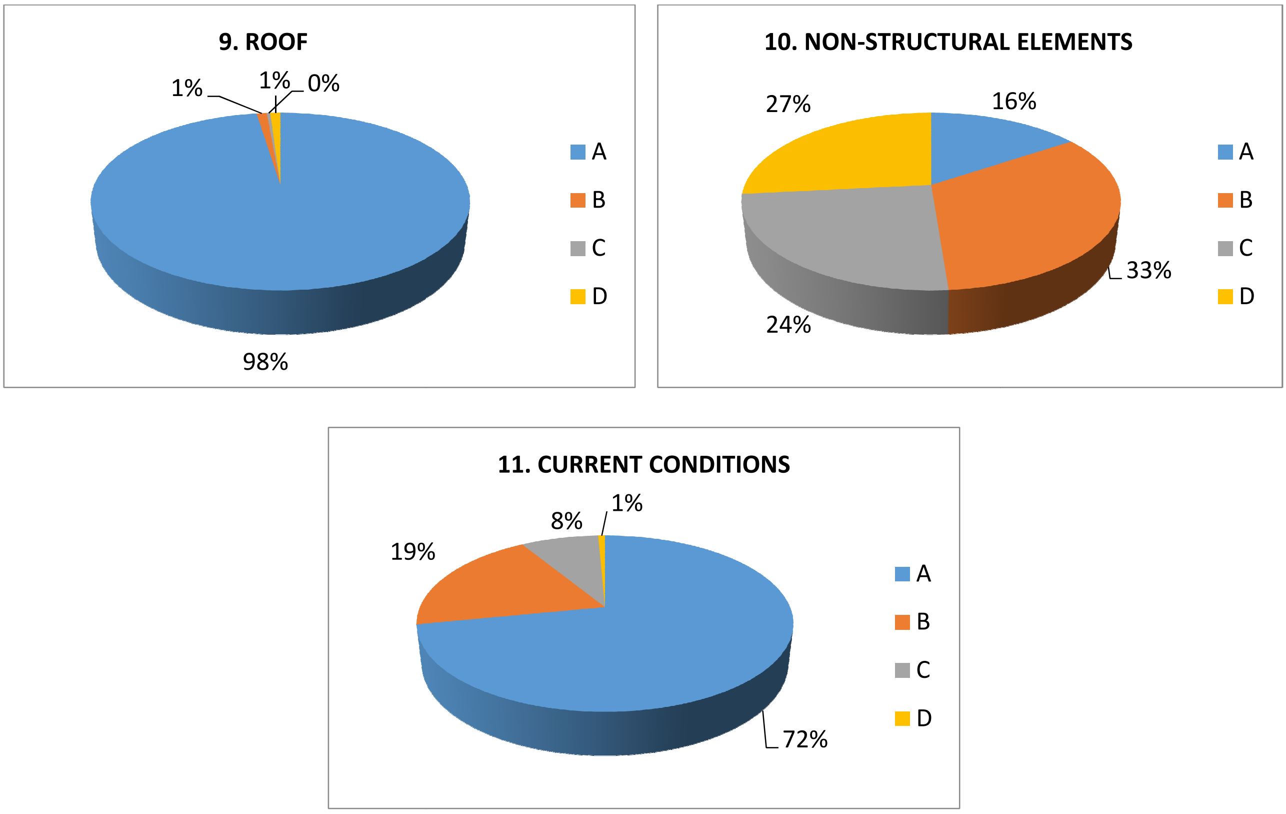
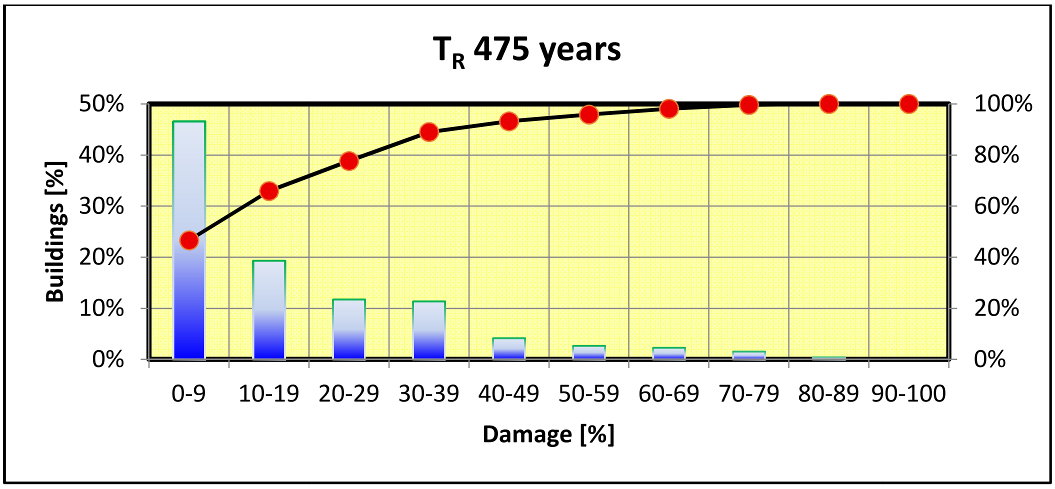
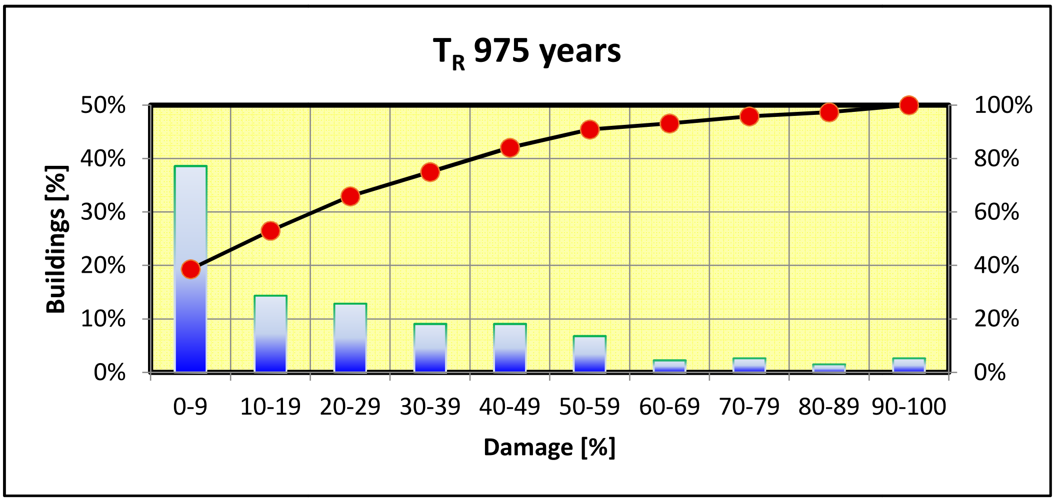


Fig. 63. Percentage amount found for the classes of each of the 11 parameters considered in the assessment forms of masonry buildings (parameters 9-11).

The percentages of damage is more widespread for the more severe case considered (T_R = 2475 years). A further analysis was performed with regard to the distribution of values of early damage and collapse PGA (Figs. 65-66) detected through the use of the analytical expressions calibrated in the previous chapter. Regarding PGA_c is observed that about 60% of the building reach a value in the range 0.10-0.20 g, while the achievement of PGA_c is mostly concentrated on accelerations in the order of 0030-0035 g.





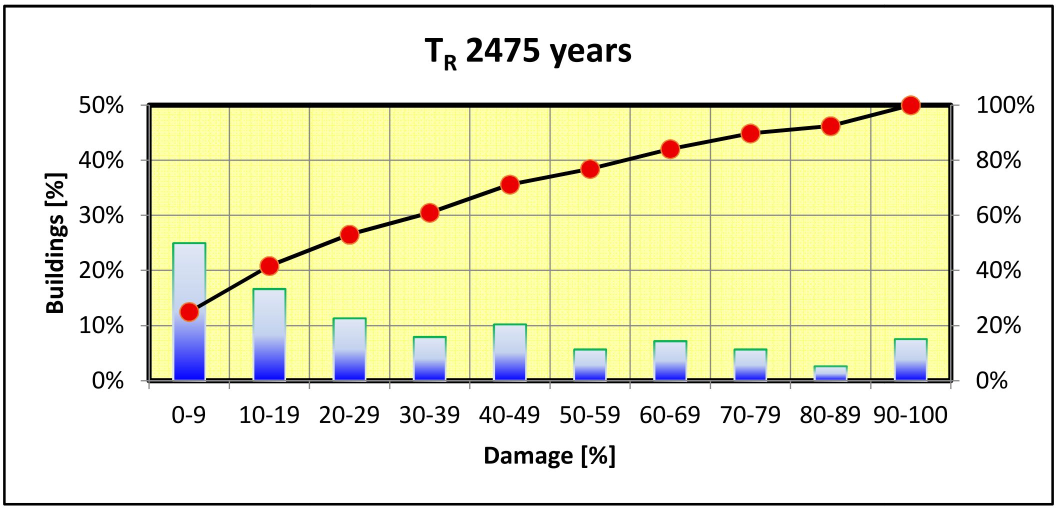


Fig. 64. Percentage of buildings at the different damage levels for the return periods 475, 975 e 2475 years and cumulative distribution.

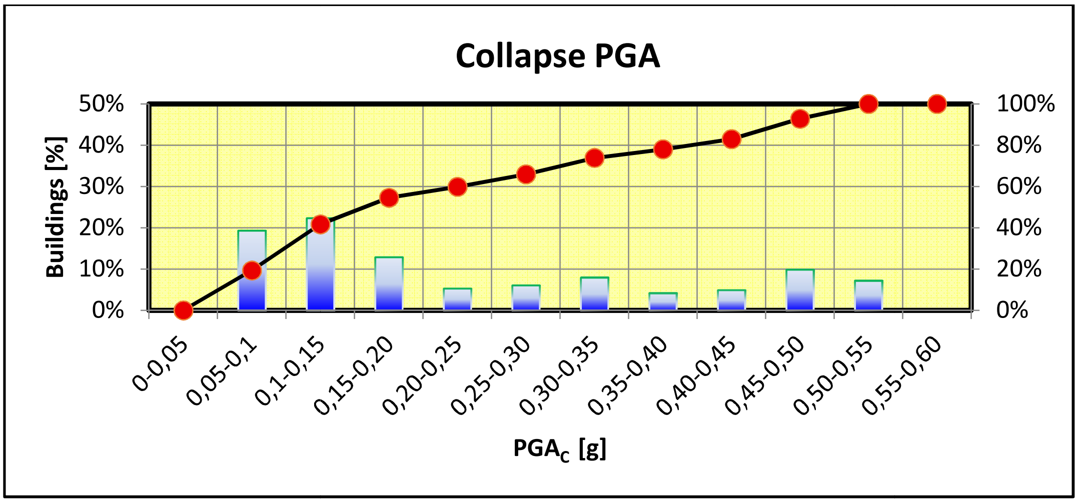


Fig. 65. Percentage distribution of collapse PGA for masonry buildings and cumulative distribution.

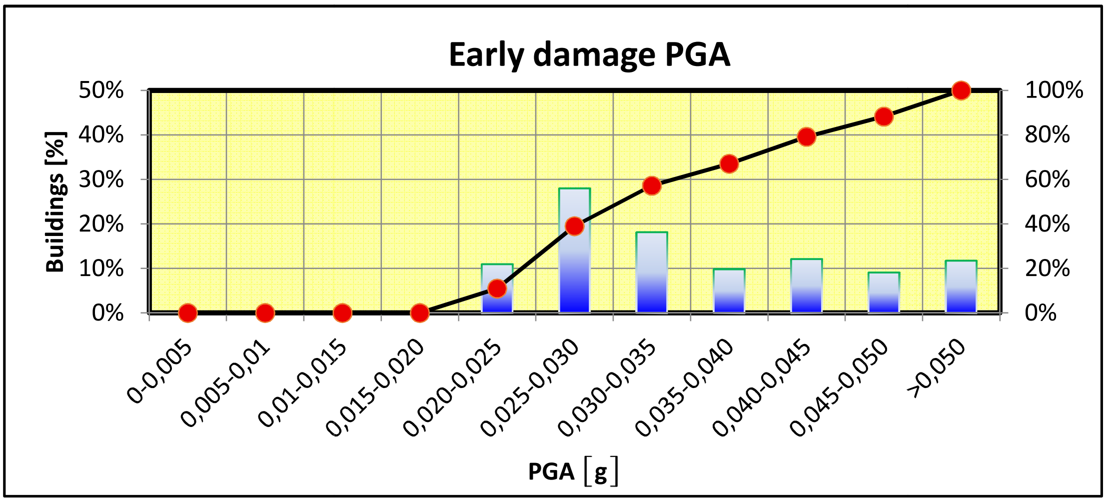


Fig. 66. Percentage distribution of early damage PGA for masonry buildings and cumulative distribution.

A probabilistic distribution similar to that detected for masonry was obtained by examining the results obtained by the vulnerability assessment of the RC buildings (Fig. 67). The average vulnerability index is equal to 24.60, while also this case were not detected values of vulnerability exceeding 55.

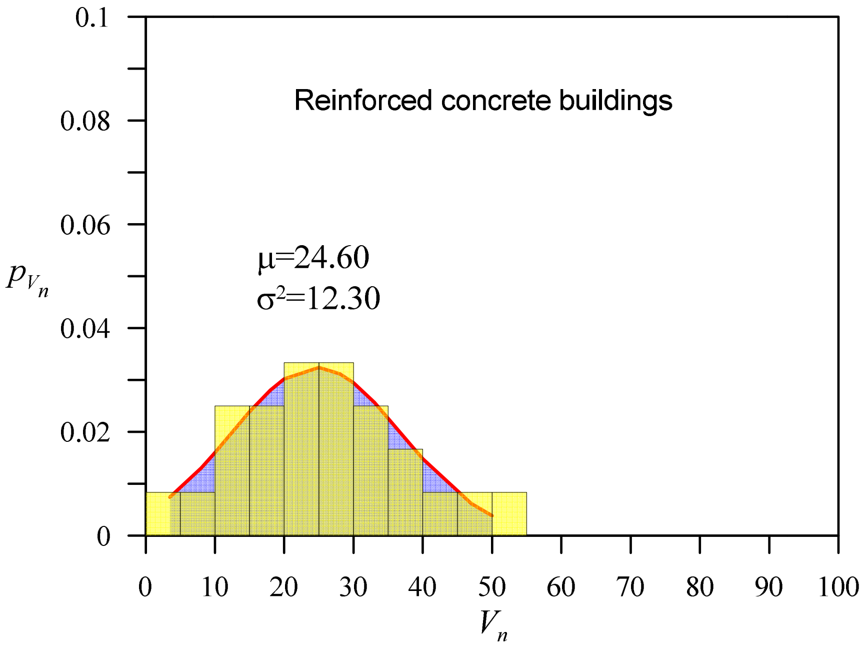


Fig. 67. Probabilistic distribution of the vulnerability index for RC buildings.

The low-mid levels of vulnerability characterizing the reinforced concrete buildings are consistent with the investigations carried out in situ, in which it was highlighted a prevalence of low rise buildings (1 or 2 floors) almost regular in plan and elevation. On the other hand it appears clear that, most of the RC buildings were not seismically designed and in some cases present structural elements with low ductility. The percentage distributions of the classes related to each of the parameters examined in the assessment forms are shown in the following graphs (Figs. 68-69). Also in the current case it was performed an analysis on the distribution of the early damage and collapse PGA values (Figs. 70-71). Regarding to the collapse PGA a large variability of the results is observed. However, it can be stated that the 75% of the buildings have PCA_c value between 0.40 and 0.60 *g*. The early damage PGA values appear less scattered, reaching accelerations in the order of 0015-0025 *g*.

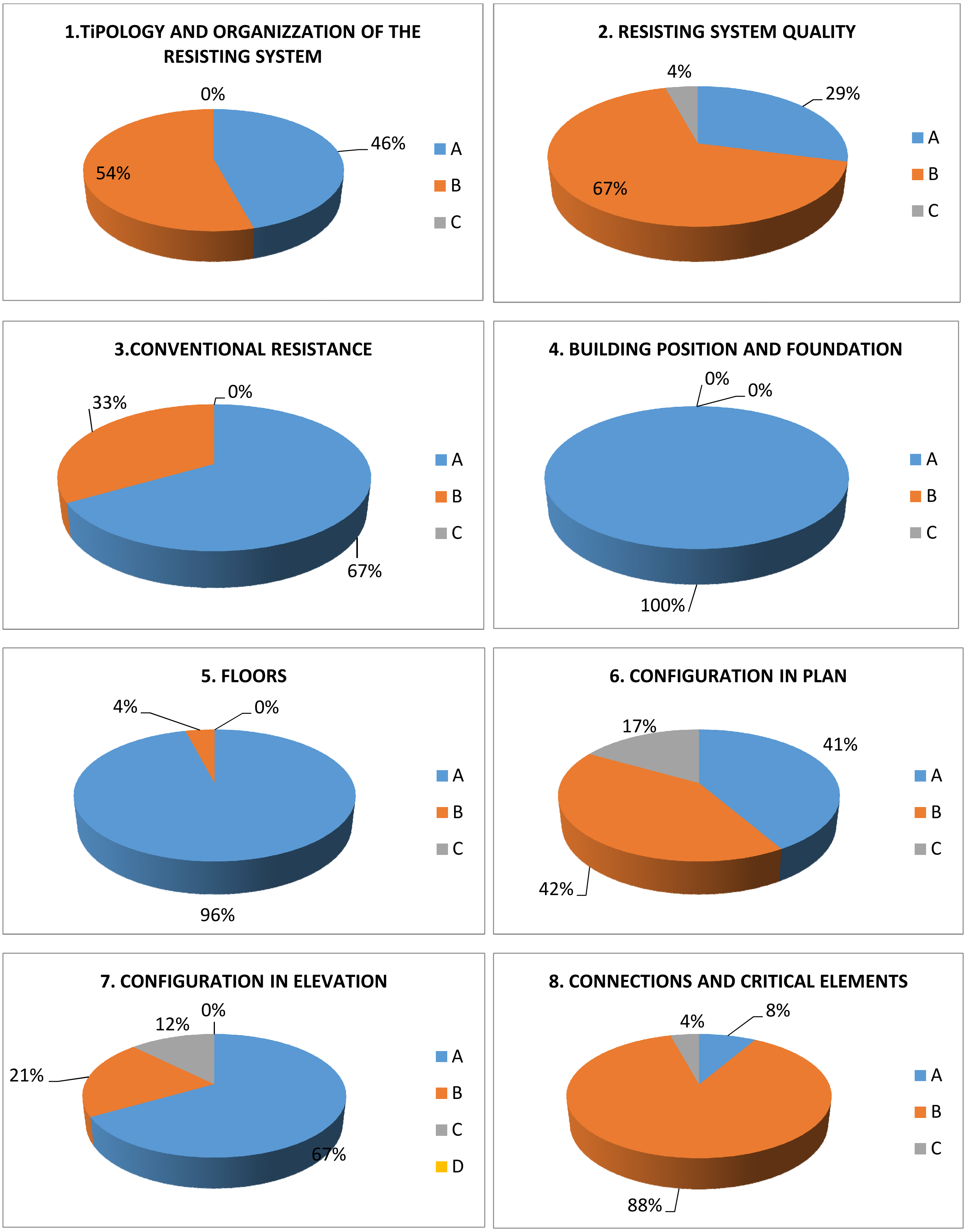


Fig. 68. Percentage amount found for the classes of each of the 11 parameters considered in the assessment forms of RC buildings (parameters 1-8).

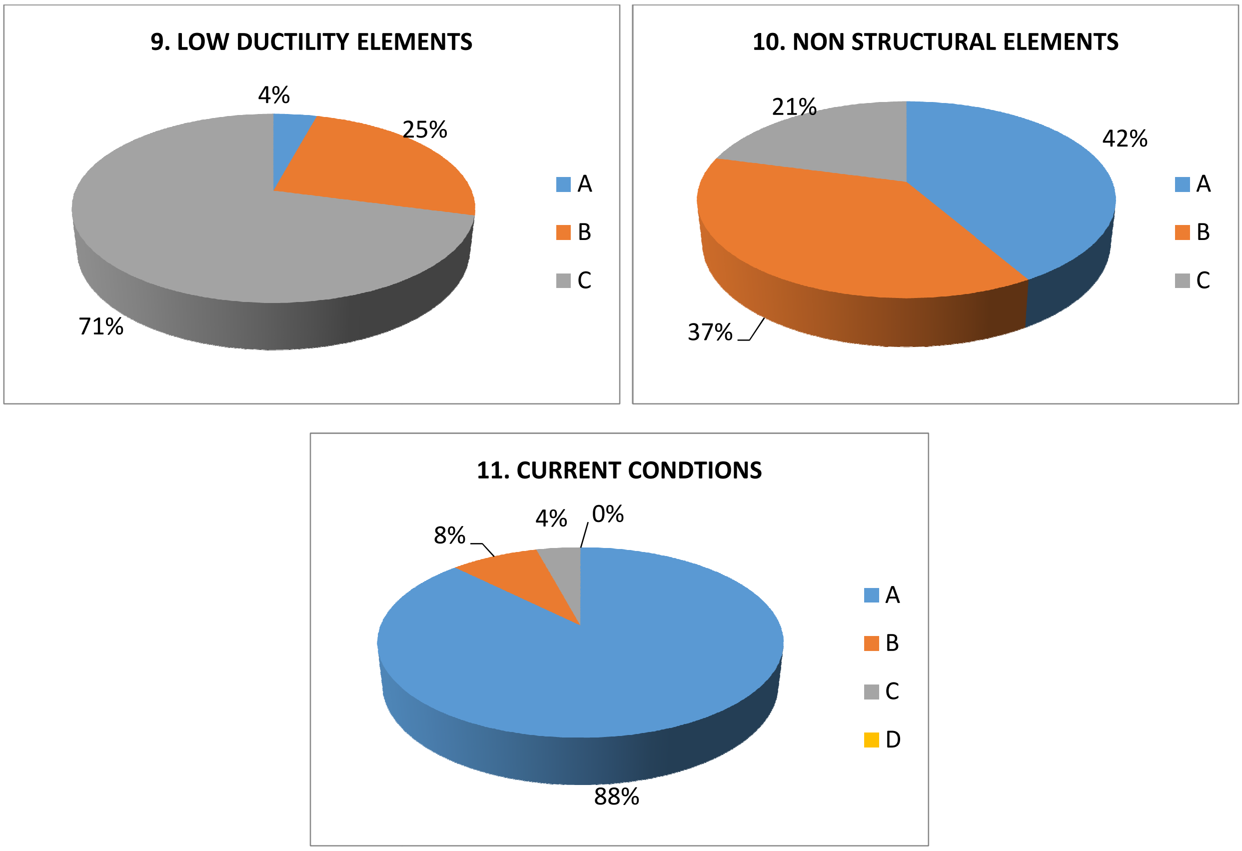


Fig. 69. Percentage amount found for the classes of each of the 11 parameters considered in the assessment forms of RC buildings (parameters 9-11).

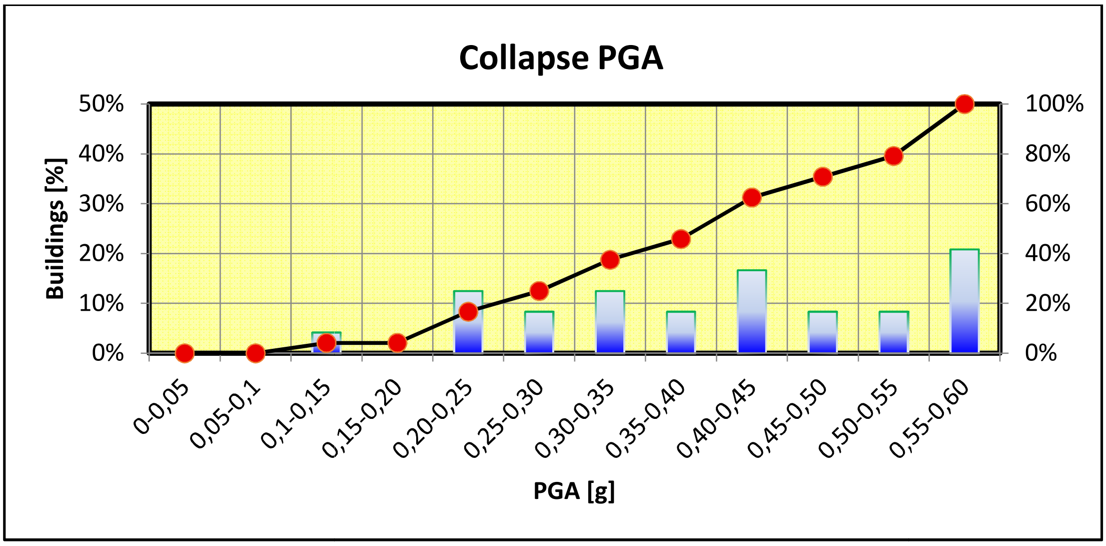


Fig. 70. Percentage distribution of collapse PGA for RC buildings and cumulative distribution.

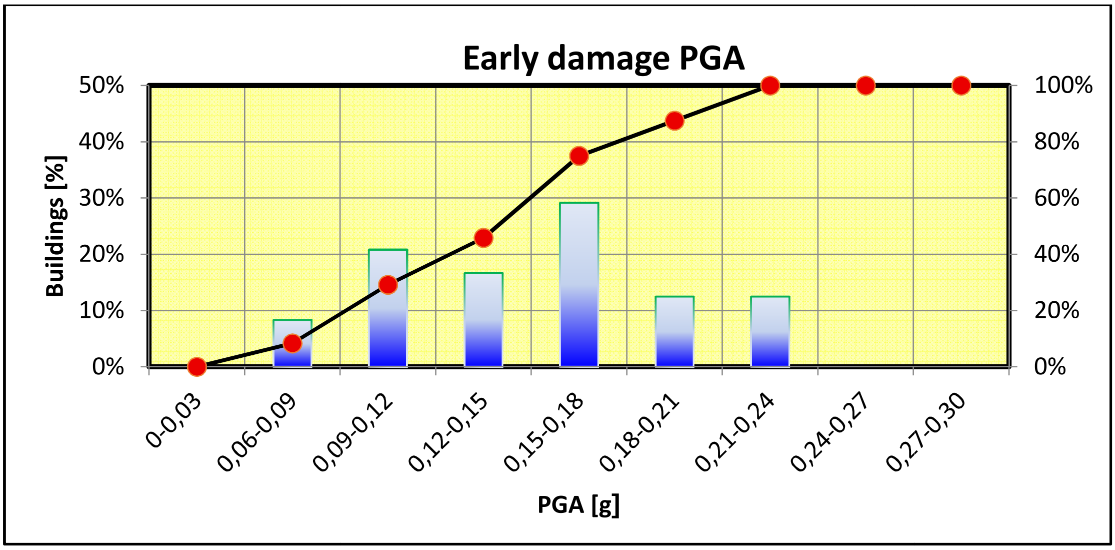


Fig. 71. Percentage distribution of early damage PGA for RC buildings and cumulative distribution.

As a final output of the research work, three maps are presented. The first one (Fig. 72) represents the map of the vulnerability index, obtained for the city centre of Lampedusa including both masonry and RC buildings. The map has a reference chromatic scale of the normalized vulnerability index going from to cooler colours (blue - green) associated with low vulnerability to warm colours (red-orange) associated with high vulnerability. Other 2 maps follow (Fig. 73-74) presenting a chromatic scale of the PGA_c and PGA_i values detected for the building aggregates. In these two maps the warmest colours are associated with the lowest values of acceleration, representative of the most critical condition. In accordance with the purposes stated for this research work these latest outputs constitute a particularly useful tool in the planning of emergency actions, providing a clear and unambiguous representation of the distribution of the most critical areas of the urban centre. It appears evident from the maps as the areas characterized by a greater vulnerability are those that refer to the oldest urban disposition, that was also the most subject to further transformations during the time. The peripheral areas, consisting of new or newer buildings, resulted instead less vulnerable, consistently with the expectations coming from the initial assessments.

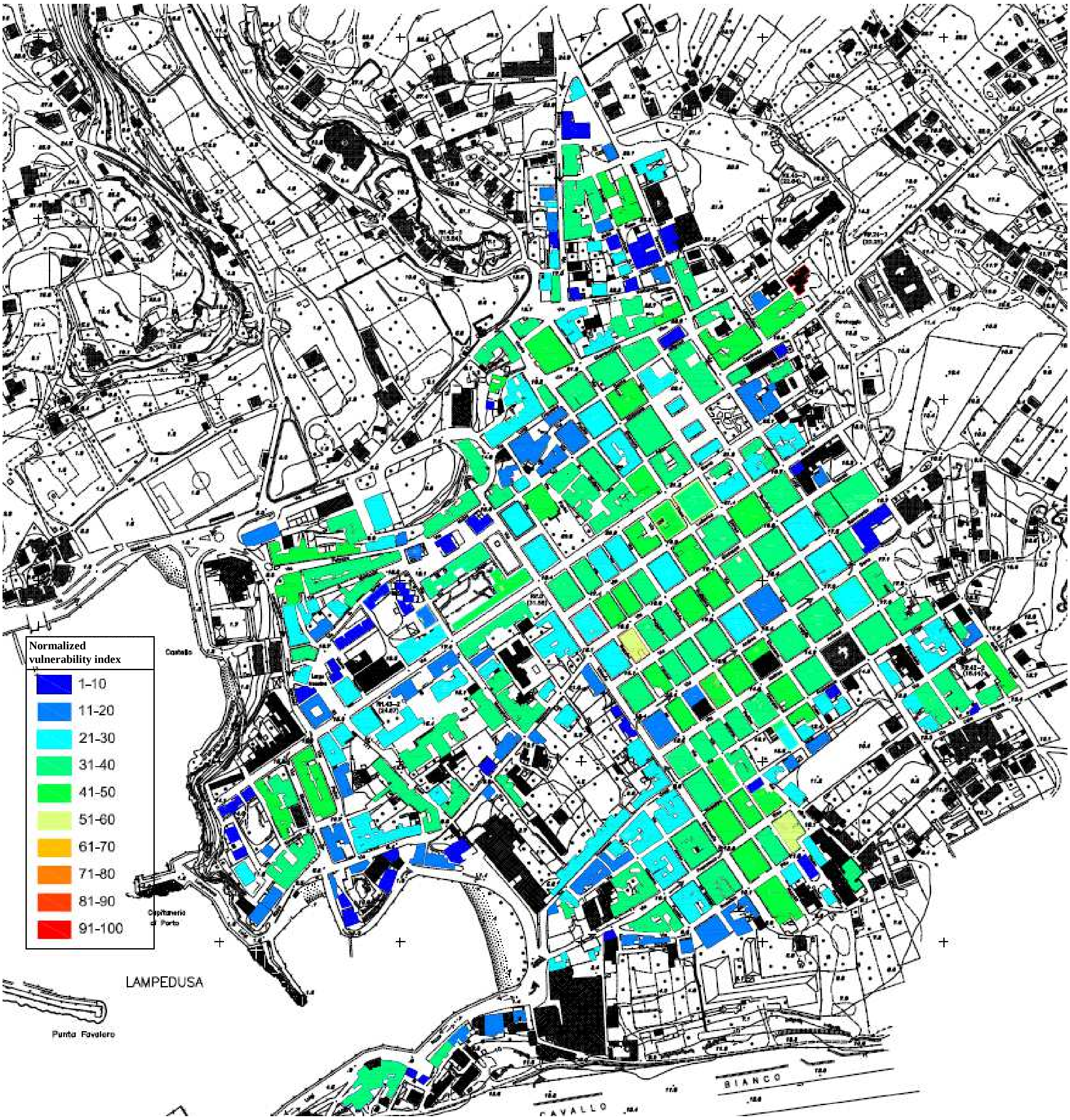


Fig. 72. Vulnerability map for the city centre of Lampedusa.

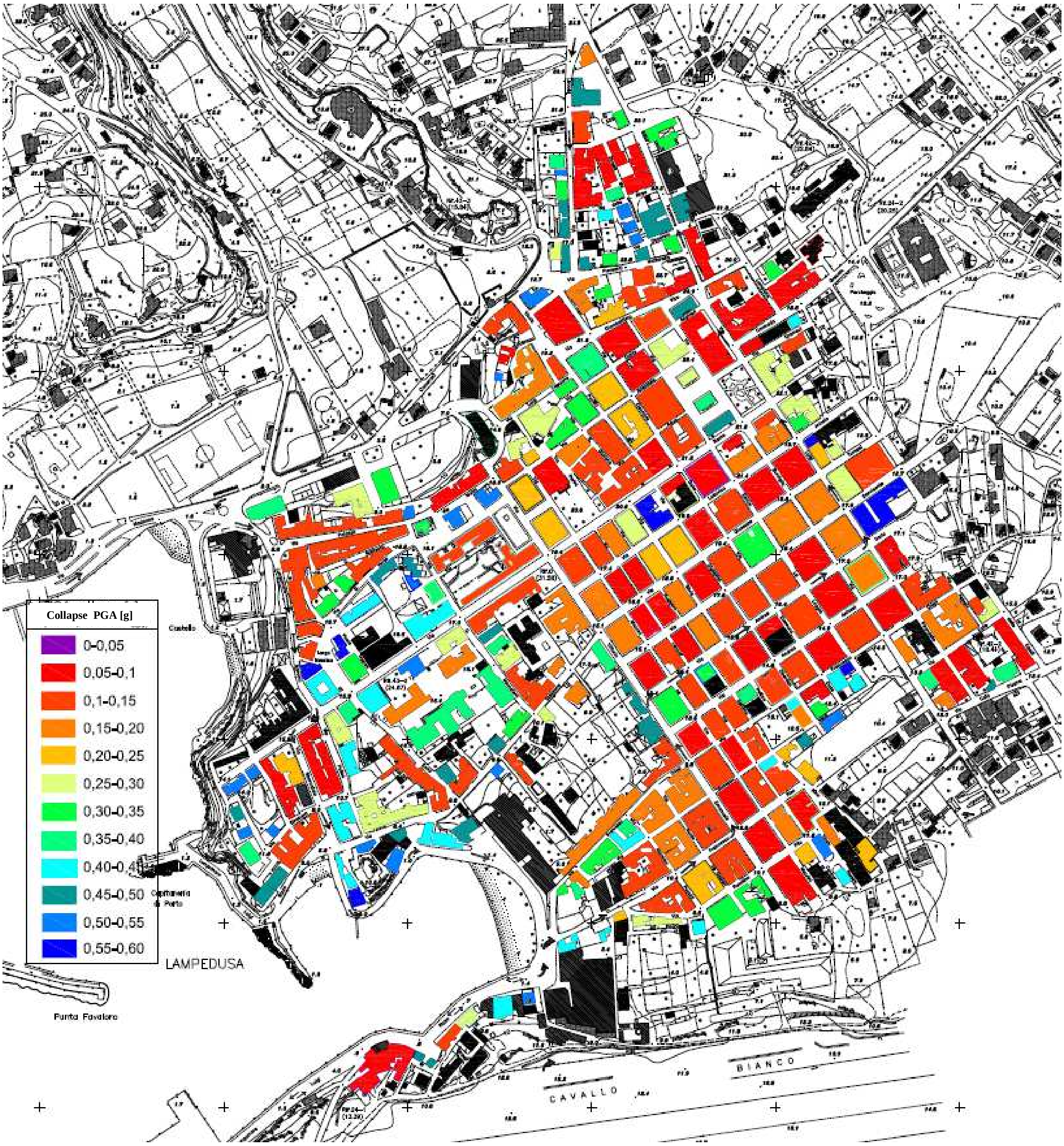


Fig. 73. Collapse PGA map for the city centre of Lampedusa.

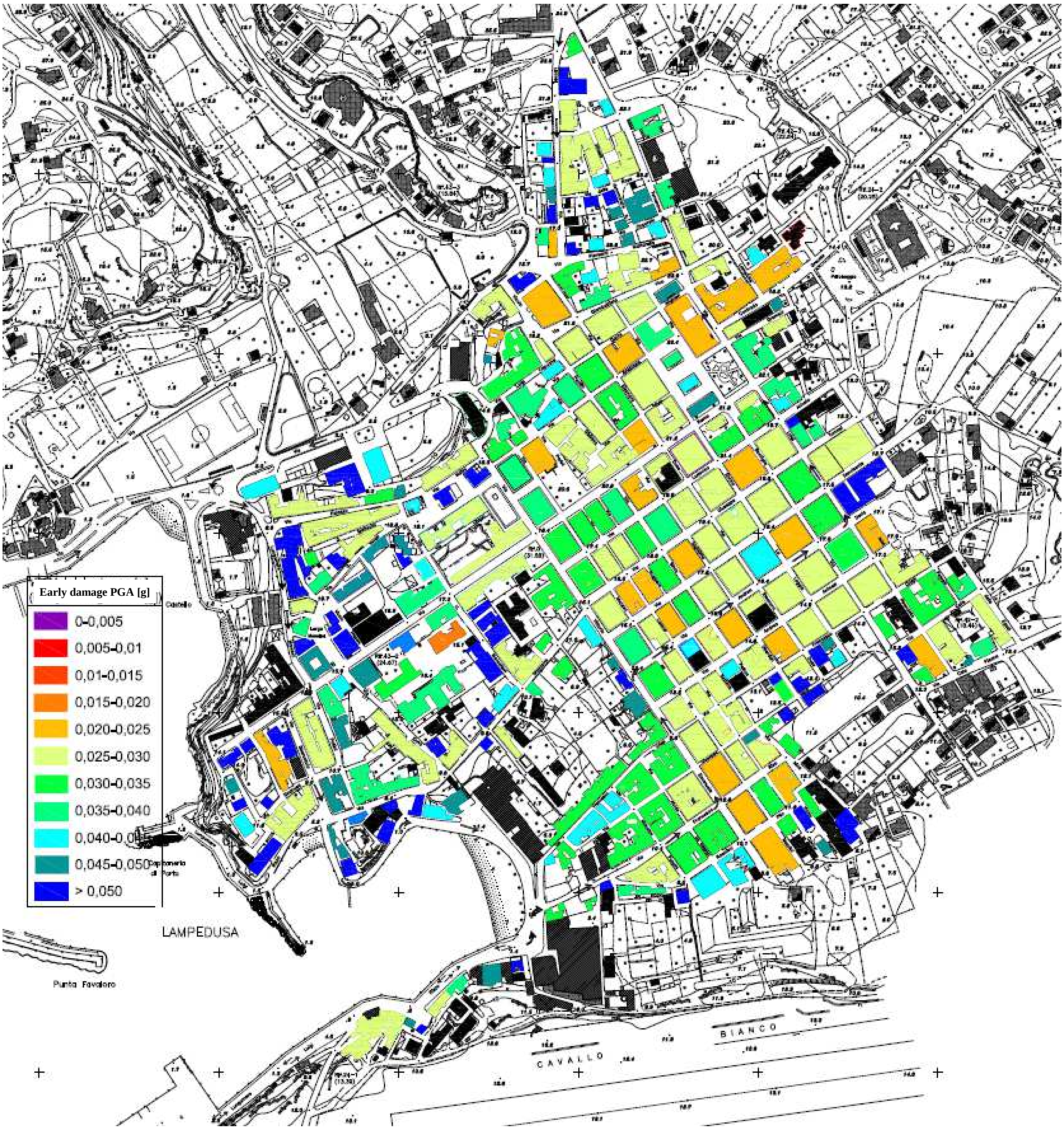


Fig. 74. Early damage PGA map for the city centre of Lampedusa.

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