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SEISMIC VULNERABILITY ASSESSMENT OF CLASSES OF REINFORCED CONCRETE STRUCTURES

Settore Scientifico Disciplinare

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To my Family

To the Life that will come!

Seismic vulnerability assessment of classes of reinforced concrete structures

Valutazione della vulnerabilità sismica di classi di edifici in c.a.

La presente tesi è cofinanziata con il sostegno della Commissione Europea, Fondo Sociale Europeo e della Regione Calabria.

L'autore è il solo responsabile di questa tesi e la Commissione Europea e la Regione Calabria declinano ogni responsabilità sull'uso che potrà essere fatto delle informazioni in essa contenute

Abstract

The modern seismic design is based on the concept to meet different performance levels, for each of which the structure should not exceed the predetermined degrees of damage.

The analysis of elastic-linear benefit from the simplicity of use and theoretical understanding, but are not able to predict the inelastic deformation capacity offered by a structure, for that reason they are unsuitable for a modern seismic design (based on the concept of performance), where the non-linear behavior and the conditions close to the collapse are investigated.

To achieve an accurate and realistic prediction of the seismic response of a structure is necessary to have analytical tools that allow to figure out the nonlinear behavior and its evolution over time.

The IDA, the Incremental Dynamic Analysis (designed by Prof. D. Vamvatsikos - foreign tutor's writer - and Professor C. A. Cornell), addresses the need to want to investigate the dynamic behavior of a structure at various levels of seismic intensity. Given an accelerogram, different dynamic analyzes on the same structure but with a seismic increasingly scaled input are performed, up to the collapse of the structure or until a predetermined level of deformation or displacement takes place.

The incremental dynamic analyzes are clearly preferable like nonlinear analysis, because only with the previous is possible to grasp the dynamic behavior of the structure resulting in potential savings in terms of actions to be pursued.

This is the concept on which is based the study carried out during this period: the evaluation of seismic vulnerability, especially of reinforced concrete structures, using the analysis above described.

Very significant and useful for the purpose of the research was the period spent abroad, during which a probabilistic and a statistical technique to assess losses caused by earthquakes of entire urban areas was developed.

The used approach is "multi-level", for classes of buildings that represent the building types that are in the examined area.

The starting point was the observation of an area inside the City Hall of Zografou, the district within which the NTUA (National Technic University of Athens) is located, by detecting some significant features of 305 surveyed buildings (such as number of floors, irregularities in height and in plant, year of construction). Each of these characteristics has been considered as discriminatory for the belonging of the particular building to a specific group. Homogeneous groups were then treated with techniques of statistical type, including the Clustering method, by which the number of the models (12 models) is resulted much lower than the number of the buildings analyzed, representative of the structures present in the whole area examined.

Taking as a reference the legislation in force at time of the construction of each model to designing it (making choices about the statistical characteristics of the materials used), the results related to static analysis and IDA, have been considered for the assessment of seismic losses the whole area they represent.

The approach based on "damage factor" compared to other models for which are known seismic losses, led to further evaluation in terms of statistical dispersion of results.

The steps are repeatable, with the necessary precautions, in other areas, and they give the opportunity to describe the seismic fragility of the heritage of entire cities. The results are useful to provide valuable information to organizations such as the Civil Protection and / or insurance agencies.

Abstract

La moderna progettazione antisismica è basata sul concetto di soddisfare diversi livelli di prestazione, per ognuno dei quali la struttura non deve superare dei prestabiliti gradi di danneggiamento.

Le analisi elastiche-lineari godono della semplicità di utilizzo e comprensione teorica, ma non sono in grado di prevedere la capacità di deformazione inelastica offerta da una struttura, per tale motivo risultano inadatte per una moderna progettazione antisismica (basata sul concetto prestazionale), dove si vogliono indagare i comportamenti non-lineari e le condizioni prossime al collasso.

Per ottenere una previsione accurata e realistica della risposta sismica di una struttura è necessario disporre di strumenti di analisi che permettano di coglierne il comportamento non lineare e la sua evoluzione nel tempo.

L'IDA, l'Incremental Dynamic Analysis (ideata dal Prof. D. Vamvatsikos - Tutor estero della scrivente – e dal Prof. C. A. Cornell), nasce dalla necessità di voler indagare il comportamento dinamico di una struttura a diversi gradi di intensità sismica. Per fare ciò, dato un accelerogramma, si svolgono diverse analisi dinamiche sulla stessa struttura ma con un input sismico di volta in volta scalato in maniera crescente, fino a raggiungere il collasso della struttura o un prefissato livello di deformazione o spostamento.

Le analisi dinamiche incrementali sono sicuramente da preferire come analisi di tipo non lineare, in quanto solo con queste si riesce a cogliere il comportamento dinamico della struttura con conseguente potenziale risparmio in termini di interventi da effettuare.

Proprio quest'ultimo è stato il concetto su cui si è basato lo studio svolto durante questo percorso: la valutazione della vulnerabilità sismica, in particolar modo delle strutture in c.a., utilizzando le analisi sopra specificate.

Molto significativo e utile ai fini della ricerca è stato il periodo trascorso all'estero, durante il quale si è messa a punto una tecnica probabilistica e di tipo statistico per valutare le perdite derivanti da fenomeni sismici di intere aree urbane.

L'approccio utilizzato è stato di tipo "multi-livello" su classi di edifici rappresentative di tipologie edilizie effettivamente presenti sul territorio.

Si è partiti dall'osservazione di un'area interna al Municipio di Zografou, il distretto entro il quale ricade la NTUA (National Technic University of Athens), rilevando alcune significative proprietà dei 305 edifici oggetto dell'indagine (quali numero dei piani, irregolarità in altezza e in pianta, epoca della costruzione). Ciascuna di queste caratteristiche è stata presa in considerazione come discriminante per l'appartenenza del particolare edificio ad uno specifico gruppo. I gruppi omogenei sono poi stati trattati con tecniche di tipo statistico, tra le quali il metodo Clustering, grazie al quale si sono ottenuti i modelli in numero nettamente inferiore agli edifici analizzati (12 modelli), rappresentativi delle strutture presenti nell'intera area esaminata. Tenendo come riferimento la legislazione vigente al tempo della costruzione di ciascun modello e progettando quest'ultimo in sua conformità (operando scelte di tipo statistico sulla scelta delle caratteristiche dei materiali utilizzati), i risultati relativi ad analisi statiche non lineari e analisi di tipo IDA, sono state considerate alla base della valutazione delle perdite sismiche dell'intera area che rappresentano.

L'approccio basato sul "fattore di danno" rispetto ad ulteriori modelli per i quali sono note le perdite sismiche, ha condotto ad un'ulteriore valutazione statistica in termini di dispersione dei risultati.

I passaggi adoperati sono ripetibili, con i dovuti accorgimenti, in altre zone, con la possibilità di descrivere la fragilità sismica del patrimonio di intere città e i cui risultati

sono utili a fornire valide informazioni ad enti quali la protezione Civile e/o agenzie assicurative.

Contents

Introduction

The present work has the final objectives of performing in probabilistic terms a seismic vulnerability evaluation of classes of reinforced concrete existing buildings, which were designed and built before or after the application of modern seismic codes.

An evaluation procedure that is based on a survey carried out through the typological census of part of the estate to a quite pushed level of detail (to satisfactorily characterize building types in the area under investigation), is presented.

A simplified method for seismic vulnerability assessment allows the repeatable transition from general to particular urban contexts. The investigated structures (like the matching results), are representative samples of buildings that rise in specific area of cities.

After the practical data collection (in absence of appropriate buildings census), the meaningful properties in seismic terms of constructions are processed through clustering and tidying procedures.

The derived models (whose number is very lower than the initial number of buildings taken into account), are “re-designed” using the seismic code in force at the time of construction.

Selection and initial dimensioning, design to the particular code, modeling, inelastic SPO analysis of the obtained models are based on the results of Zeris et al. [2005].

To estimate IDA curves from the results of SPO for each models, the tool SPO2IDA has been used.

In the single building vulnerability study, get to a detail level that permits the evaluation of seismic behavior of various factors (possible collapse mechanism, different characteristics of the construction materials, etc.), is possible.

In case in which the building is representative of a class, it represents the 'average' (the model) of a identified type in terms of macro parameters morphological / structural (plan dimensions, number of floors, period of construction, etc.) which describes the variation of all the buildings of a class.

In this work, the above-mentioned method for seismic vulnerability assessment is applied to Zografou area, one of the suburb in the eastern part of Athens agglomeration, where there is also the Campus of the National Technical University of Athens (NTUA).

Seismic risk

1.1 Introduction

The earthquake is a dangerous event that has often resulted in the destruction of or damage to property and / or leading to a significant loss of life.

This is surely one of the damaging events generated by natural forces, it is the most feared by man because of the large number of casualties it causes, in particular, from the statistics of natural disasters and man-show that is actually the major cause of loss of lives.

Certainly, the scale of a natural disaster depends not only from the elements, but also by factors of human relevance, such as the construction techniques or the quality of the preventive measures in the affected region.

For this reason, to determine the impact that future earthquakes could have on the buildings in a given region is referred to the evaluation of the "seismic risk" which requires separate analysis of three basic components:

- the "hazard",
- the "vulnerability" and the
- '"exposure"

whose convolution defines risk.

The earthquake risk in a certain timeframe, is the provision of social and economic losses expected as a result of the occurrence of an earthquake estimated for the reference area during this time interval.

Following this approach the dangerousness (or "hazard"), expresses the probability of occurrence of a physical process or event that can cause loss of life and property; the vulnerability expresses the quantity of resources to be lost in relation to the event; exposure represents the value of the resources at risk. Defined as the risk is understandable that the occurrence of a catastrophic event in the desert, for example, carries a risk close to zero since the property at risk are almost zero (exposure).

In the case of buildings, the seismic vulnerability of a building is its susceptibility to being damaged by an earthquake and it can be expressed "*by all the probability of achieving a series of damage levels up to the collapse, evaluated according to the seismic intensity and its occurrence*"(Augusti and Ciampoli, 1999).

Therefore the vulnerability of a building should be defined by a probabilistic relationship between intensity and level of damage, in operational terms, a vulnerability analysis has to assess the damage caused by earthquakes of various intensities.

Defined these three terms, establish whether the study is performed as a preventive measure (risk analysis) or for emergency management (scenario analysis) is necessary. The choice between risk analysis and scenario analysis depends on the purpose of the study; established the goal, for the vulnerability study, also the approach changes: it is probabilistic for risk analysis, deterministic for scenario analysis.

1.2 Hazard, exposure and vulnerability risk: definition and interrelations

The risk is defined, in general, as the probability that as a result of a certain event, a given functional system (a person or a community, a building or a complex of buildings, a settlement or a region), in the course of an assigned period of time (a year, the nominal life of the system), suffers damage (mechanical, functional), and derive from these losses to a community (those in the system, the inhabitants of a region or a nation, a class) regarding certain resources (human lives, health, standards, economic goods, cultural values).

The risk can be expressed as the convolution of the ***hazard***, ***exposure*** and ***vulnerability***.

The seismic risk, in particular, represents the probability that a structure (a functional system) exceeds a predetermined limit state (damage) due to an earthquake (event) during an assigned time period. This definition is the transposition to the field of earthquake engineering, of the more general concept of reliability of a system. Therefore the seismic risk is the complement to one of the reliability of the structural system in the observation period.

As for the damage, it is necessary to differentiate the damage to people and damage to structures. To reduce the risk within reasonable limits, it should be subject to at least two different design conditions:

- *Damage Limit State*: structures must be designed to withstand in elastic field, stresses induced by the event whose intensity corresponds, with reference to the characteristics of the area in question, for a return period of the order of nominal life of the structure (in the case of earthquakes is assumed in general for normal buildings for housing, a return period of 50 years);

- *Ultimate Limit State*: the structures have to have sufficient reserves of strength, over the elastic limit, to tolerate without collapsing the actions of an event of such intensity as to suggest extremely unlikely the occurrence of an event of greater intensity. The event which has to be considered in this second design condition is therefore characterized by a return period of 475 years.

The first condition is especially directed to limit the damage to the buildings, while the second condition makes clear reference to the Safety of Life.

1.3 PBEE Methodology[1]

PBEE attempts to address performances primarily at the system level in terms of risk of collapse, fatalities, repair costs, and post-earthquake loss of function.

Initial efforts to frame and standardize PBEE methodologies produced SEAOC's Vision 2000 report (1995) and FEMA 273 (1997), a product of the ATC-33 project. The authors of these documents frame PBEE as a methodology to assure combinations of desired system performance at various levels of seismic excitation. The system-performance states of Vision 2000 include fully operational, operational, life safety, and near collapse.

[1] "An Overview of PEER's Performance-Based Earthquake Engineering Methodology" – K. A .Porter, Ninth International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP9) July 6-9, 2003, San Francisco.

Levels of excitation include frequent (43-year return period), occasional (72-year), rare (475-year) and very rare (949-year) events. These reflect Poisson-arrival events with 50% exceedance probability in 30 years, 50% in 50 years, 10% in 50 years, and 10% in 100 years, respectively. The designer and owner consult to select an appropriate combination of performance and excitation levels to use as design criteria, such as those suggested in Fig. 1.1.

FEMA 273 expresses design objectives using a similar framework, although with slightly different performance descriptions and levels of seismic excitation. Each global performance level is detailed in terms of the performance of individual elements. A design is believed to satisfy its global objectives if structural analysis indicates that the member forces or deformations imposed on each element do not exceed predefined limits.

Performance is binary and largely deterministic: if the member force or deformation does not exceed the limit, it passes; otherwise, it fails.

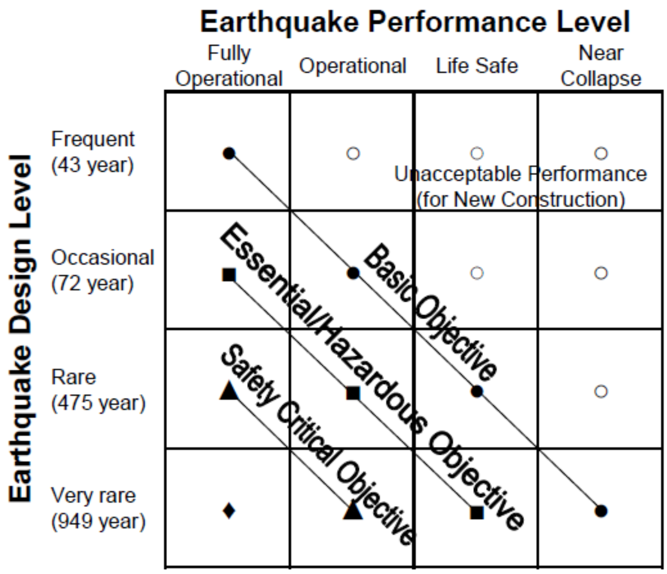


Fig. 1.1: Vision 2000 recommended seismic performance objectives for buildins (after SEAOC, 1995)

If the acceptance criteria are met, the design is believed to assure the performance objective, although without a quantified probability.

Other important pioneering PBEE efforts include ATC-32 (1996a), ATC-40 (1996b), and FEMA 356 (2000).

1.3.1 PEER approach

The *Pacific Earthquake Engineering Research (PEER) Center*, based at the University of California, Berkeley, is one of three federally funded earthquake engineering research centers.

A central feature of PEER's approach is that its principal outputs are system-level performance measures: probabilistic estimates of repair costs, casualties, and loss-of-use duration ("dollars, deaths, and downtime.").

The objective of the methodology is to estimate the frequency with which a particular performance metric will exceed various levels for a given design at a given location.

These can be used to create probability distributions of the performance measures during any planning period of interest. From the frequency and probability distributions can be extracted simple point performance metrics that are meaningful to facility stakeholders, such as an upper-bound economic loss during the owner-investor's planning period.

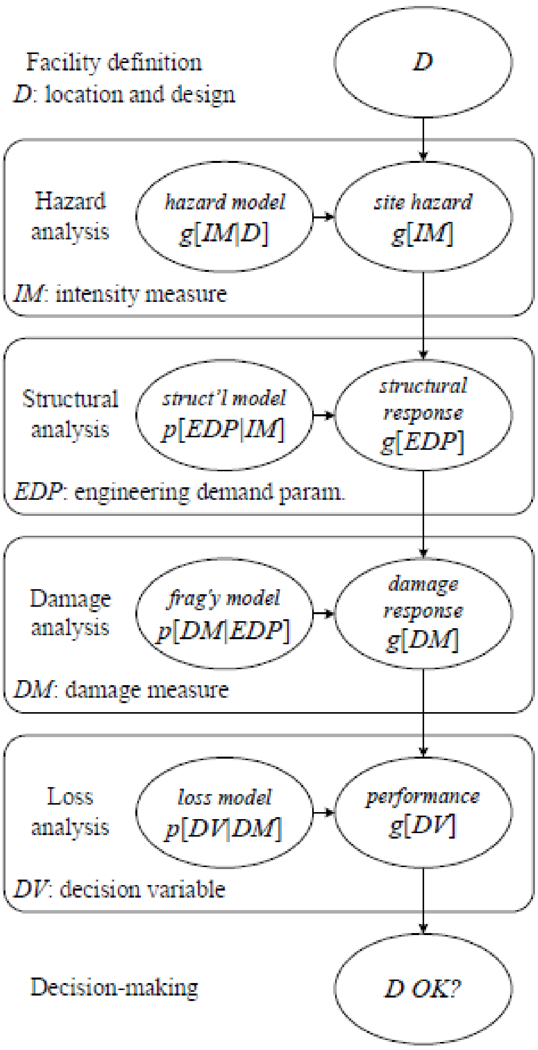


Fig. 1.2: PEER analysis Methodology

PEER's PBEE approach involves four stages (Fig. 1.2):

- hazard analysis,
- structural analysis,
- damage analysis,
- loss analysis.

1.3.1.1 Hazard Analysis

In the hazard analysis, one considers the seismic environment (nearby faults, their magnitude-frequency recurrence rates, mechanism, site distance, site conditions, etc.) and evaluates the seismic hazard at the facility considering the facility location and its structural, architectural, and other features (jointly denoted by design, D), to produce the seismic hazard, g[IMgD].

The hazard curve describes the annual frequency with which seismic excitation is estimated to exceed various levels. Excitation is parameterized via an intensity measure (*IM*) such as *Sa*(*T*1), the damped elastic spectral acceleration at the small-amplitude fundamental period of the structure. In our analyses to date, the hazard analysis includes the selection of a number of ground-motion time histories whose *IM* values match three hazard level of interest, namely, 10%, 5%, and 2% exceedance probability in 50 years.

PEER researchers have used *Sa* so far in our analyses, and have established procedures to select design ground motions consistent with the site hazard (e.g., Somerville and Collins, 2002). We will also test nine alternative *IM*s (see Bray, 2002, for a list) that might estimate performance with less uncertainty. We will test each *IM* for conditioning on magnitude, distance, and possibly other parameters that might relate to performance. (These are the efficiency and sufficiency tests described by Luco and Cornell, 2001). Most of the candidate *IM*s are scalars; some are vectors (e.g., Pandit et al., 2002). Some are more relevant to excitation of structures (e.g., Cordova et al., 2001), while some focus on ground failure (Kramer and Mitchell, 2002).

1.3.1.2 Structural Analysis

In the structural analysis, the engineer creates a structural model of the facility in order to estimate the uncertain structural response, measured in terms of a vector of engineering demand parameters (EDP), conditioned on seismic excitation and design (p[EDP|IM,D]).

EDPs can include internal member forces or local or global deformations, including ground failure (a preliminary list is provided in Porter, 2002). The structural analysis might take the form of a series of nonlinear time-history structural analyses. The structural model need not be deterministic some PEER analyses have included uncertainty in the mass, damping, and force-deformation characteristics of the model

1.3.1.3 Damage Analysis

EDP is then input to a set of fragility functions that model the probability of various levels of physical damage (expressed via damage measures, or DM), conditioned on structural response and design, p[DM|EDP,D].

Physical damage is described at a detailed level, defined relative to particular repair efforts required to restore the component to its undamaged state. Fragility functions currently in use give the probability of various levels of damage to individual beams, columns, nonstructural partitions, or pieces of laboratory equipment, as functions of various internal member forces, story drift, etc. They are compiled from laboratory or field experience. For example, we have compiled a library of destructive tests of reinforced concrete columns (Eberhard et al., 2001). The result of the damage analysis is a probabilistic vector of *DM*.

1.3.1.4 Loss Analysis

The last stage in the analysis is the probabilistic estimation of performance (parameterized via various decision variables, DV), conditioned on damage and design p[DV|DM,D].

Decision variables measure the seismic performance of the facility in terms of greatest interest to stakeholders, whether in dollars, deaths, downtime, or other metrics. Our loss models for repair cost draw upon well-established principles of construction cost estimation. Our model for fatalities, currently in development, draws upon empirical data gathered by Seligson and Shoaf (2002) and theoretical considerations elaborated by Yeo and Cornell (2002). Later research will address injuries.

Note that location aspects of D are relevant to many DVs such as repair cost.

1.3.1.5 Decision Making

The analysis produces estimates of the frequency with which various levels of DV are exceeded. These frequencies can be used to inform a variety of risk-management decisions.

If one performs such an analysis for an existing or proposed facility, one can determine

whether it is safe enough or has satisfactorily low future earthquake repair costs. If one re-analyzes the same facility under redesigned or retrofitted conditions, one can assess the efficacy of the redesigned facility to meet performance objectives, or weigh the reduced future losses against the upfront costs to assess the cost-effectiveness of the redesign or retrofit. For example, if one refers to the reduction in the present value of future losses as benefit (B) then the expected benefit during time T of a retrofit measure that changes the design of a facility from D to D' can be calculated as

$$E[B|T,D,D'] = T\int DV g[DV|D]dDV - T\int DV g[DV|D']dDV$$

Vulnerability assessment methods

2.1 Introduction

A qualitative definition of seismic vulnerability, that can be widely accepted, is as follows: *the proneness of some category of elements at risk to undergo adverse effects inflicted by potential earthquakes*. This kind of definition, which is definitely vague, requires of course considerable refinements in order to become an operational tool for various purposes, like estimate of seismic risk, development of earthquake scenarios, or development of strategies of risk mitigation. The concept of vulnerability pertains to a system of basic concepts involved in risk analysis.

Vulnerability assessment of existing buildings is an issue of major importance to the territory like Italy and Greece, where much of the built heritage was not erected according seismic criteria. The study of this problem is important for the determination of the safety level of these structures after a seismic event, in order to carry out studies of scenario by identifying buildings at greater risk on the territory and to plan interventions useful to the restoration of security, and also to address first post earthquake aid towards the most vulnerable areas.

2.2 Vulnerability Evaluation

We could distinguish three types of seismic vulnerability:

- *Direct vulnerability*, which determines the propensity of a single physical element or complex to suffer damages due to an earthquake;
- *Generated vulnerability*, which it is defined according to the crisis that is induced by the collapse of a single or complex physical element;
- Delayed vulnerability, which specifies the effects that occur in the later stages to the earthquake and during the first emergency.

This work refers to the first kind of vulnerability that relates directly to one side the seismic action, and on the other hand, the damage that it causes the physical system.

The first issue to be dealt is the choice of the parameter that can identify these variables.

As far as concerned the seismic action, we could consider different possibilities, like the *macroseismic intensity* that represents a very useful parameter because of its direct correlation of the intensity scale with the earthquake damages. However, this choice is not so convenient in terms of structural damage assessment, because it is difficult connect it whit the spectral values, that allow to define the risk. The use of *spectral quantities* is more advisable, because these offer also the possibility to evaluate the damage in a structural analysis since they have a clear mechanical meaning.

The damage, instead, is generally expressed by economic cost or indexes.
In the first case, the cost is expressed as a cost necessary for the recovery of the construction and in general this cost is related to the value of new construction; in

the latter, indexes can be used in qualitatively or quantitatively ways, but they always require a standardized scale and subsequent correlation to the economic value, in sense that in every case the total damage of the building is necessary to express through a single indicator that is easily convertible in economic terms.

In the last three decades, various methodologies for estimating the vulnerability have been developed and their classification task is far from simple.

The methodological paths that we could follow are varied, and the choice of one or the other depends primarily on the size of the sample analyzed, as well as the availability of information or by the relative difficulty of finding its, and by the objectives of the analysis of vulnerability that we are running and the disposable income and time that we want.

A first essential distinction, therefore, should be made according to the size of the sample for which we want to assess the vulnerability; in theory, in fact, it is possible to evaluate the vulnerability of a single building, as well as a class of buildings shared the same typical features, or by widening the area of investigation, a neighborhood, a city, a land area even wider, etc.. Of course the basic information that are essential to the performance of the analysis will vary from case to case, as well as, necessarily, the investigation methods and the reliability of results that it is hoped to achieve have to be vary.

2.3 Vulnerability procedure paths

The various methods for vulnerability assessment that have been proposed in the past for use in loss estimation can be divided into two main categories: empirical or analytical, both of which can be used in hybrid methods[2] (see Fig. 2.1).

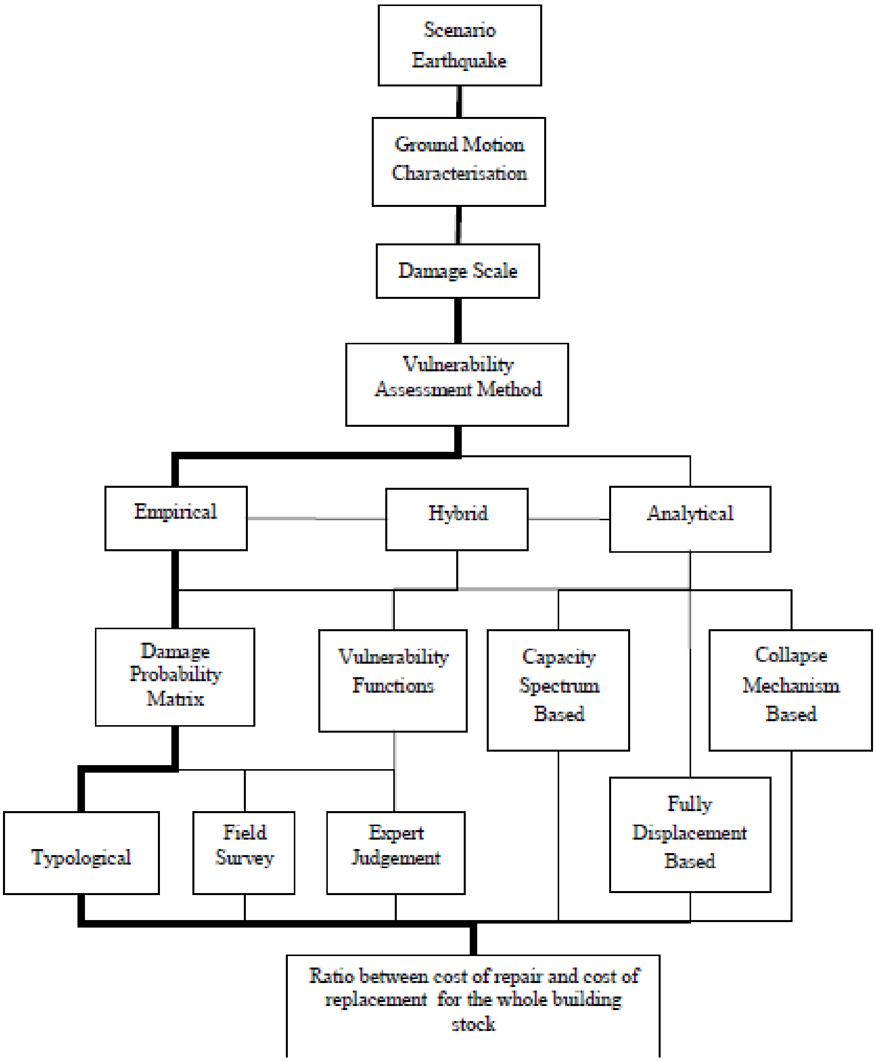


Fig. 2.1: The components of seismic risk assessment and choices for the vulnerability assessment procedure (the bold path shows a traditional assessment method

Calvi et al. presented some of the most important methods of vulnerability assessment that over time have been adopted, highlighting each the methodology.

2.4 Individual and class vulnerability

The attention in this study is focused on vulnerability assessment of classes of buildings, so to pass from the particular to the general, the estimation of vulnerability of a single building is discussed, and then the eventual determination of the relative fragility curves, by framing the study on an analysis of type reliability front, is presented.

Traditionally, the study of the seismic behavior of a individual building is not considered as a vulnerability analysis, although it may provide a measure of the damage that a structure may suffer as a result of the seismic phenomenon. The study of the detail of the individual case requires input information on the mechanical and geometrical characteristics of the building and a very accurate computational cost such as to justify its use only in those cases in which a sufficiently approximate estimate of the degree of construction safety is desired, as for buildings of strategic importance or intrinsic historical/monumental value.

2.4.1 Individual Building Vulnerability

The study of the vulnerability of each building involves, theoretically, the degree of damage estimated expected for each level of seismic intensity. The conceptually clearer way, and also the most complete, to perform this estimate is to build the fragility curves for the particular structural investigated system. In general, a fragility curve of a building represents, on the basis of the variation of seismic intensity, the probability that the building reaches a particular limit state. In mathematical terms this is expressed by the function of conditional probability P

[SL|I] in which SL|I expresses the attainment of a limit state (that of predetermined damage thresholds) for the value of seismic intensity I, which may be represented by the PGA, PGV, spectral acceleration etc. depending on the purposes of the considered case. For each building, of course, building more fragility curves is possible, each corresponding to a predetermined limit state. An example of fragility curves constructed on the basis of the peak ground acceleration (PGA), as the parameter of seismic intensity, is shown in Fig. 2.2, where two fragility curves obtained for the same structural system are represented simultaneously and they are corresponding, each, to achieve a different limit state (Light Damage, Life Safety).

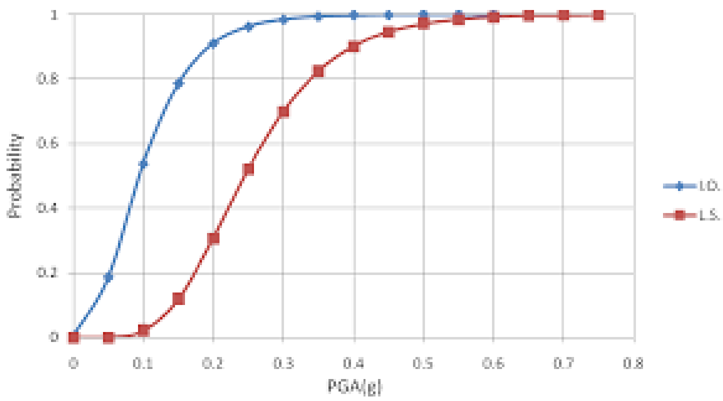


Fig. 2.2: Building fragility curve

By definition, the study of vulnerability of buildings provides to give the estimate of the degree of expected damage in probabilistic measure, in relation to type of analyzed structure and considered seismic intensity.

It should be underlined that the analysis of a single building is different from the analysis of a building that represents other constructions.

In the study of the vulnerability of the individual building, push to a level of detail that evaluates the influence of various factors on the seismic behavior is possible (particular failure mechanisms, characteristics of building materials, etc.). In case of representative of a class building, instead, it constitutes an *average* building of a typology of constructions identified in terms of macroelements (kind of structure, shape, plan dimension, number of floors, construction period, etc., defines the description of a category). The influence on the seismic response of the macroscopic parameters in an analytical approach is difficult to take into account, so for one category, the fragility curves are built *empirically*, ie by statistically treating the observed data on the degree of damage sustained as a result of earthquakes for buildings referable to the same typological class.

The single building study can be addressed through a mechanic/analytical approach. When the damage degree dependent of the seismic input (then the fragility) is evaluated by numeric seismic simulation, the fragility curves are analytically obtained. The computational and modelling cost is very high, so the analytical fragility curves construction is legitimized only for very important building cases (strategic or monumental importance) or for scientific research purpose.

2.5 Damage Probability Matrix

The Damage probability Matrix (DPM) are matrix generated to buildings categories and they express the probability that a certain level of damage for each seismic intensity occurring. Theoretically, therefore, they can be constructed by referring to a generic scale of damage, whether expressed in terms of costs (eg as the ratio of the cost of repair on the cost of reconstruction), both in phenomenological terms, that is, on the basis of a qualitative evaluation of the different degree of damage that buildings can have.

For example, the MSK-76 scale (Medvedev, 1977) is the first model of Damage Probability Matrix (also if it is not complete). In this scale three classes of different construction typology are presented: unit A defines stones construction, bricks building are in unit B, and reinforced construction are in unit C. The seismic intensity is based through the damage scenarios that there are in the territory: the damage level has six degrees (Table 2.1).

Table 2.1: MSK76 damage levels

Damage	Description
0	No damage
1	Slight Damage: thin cracks and fall of small pieces of plaster
2	Average Damage: small cracks in walls, fall of substantial portions of plaster, cracks in chimneys and part of its falls
3	High Damage: formation of large cracks in the walls, chimneys fall
4	Destruction: gaps between the walls, the possible collapse of portions of buildings, separate parts of the building are out, collapse of interior walls
5	Total Damage: total collapse of the building

The MSK scale has twelve seismic intensity levels: the first four are associated with phenomenological aspects that concern the soil (without damages on the constructions) and MSK scale repeats the content of MCS scale in terms of seismic intensity. From the fifth to the tenth degree, instead, the earthquake intensity is associated with the entity of the structure damages, through rates and parts of damaged constructions.

The MSK scale has some limits: it does not take in account of the particular and modern construction typologies and the data are very coarse: not every damage levels are considered.

2.6 Building Class Approach

The building class approach studies scenario investigations. it allows to assess the vulnerability of the built in a given area in relation to any seismic event.

The typological classification of the buildings is based on surveys conducted specifically in the area of study and reflects in more faithful way the actual characteristics of the investigated building heritage. This implies a greater investigative cost, especially at the beginning of the territorial survey, but it allows to have more reliable results because the used structural models for the analysis are calibrated to the constructive classes that exist in the area.

The evaluation of the seismic capacity of classes of buildings takes place in expeditious manner and with relatively simplified calculation models ('*typologica*l' models). For each identified building class in the area, then, one or more detail models are constructed. They constitute means of verification and calibration of the 'typological' models and they are connection to the assessment of the vulnerability in relation to the considered seismic input. Thus, the required information in input are mainly aimed at the construction of representative models of buildings: information about geometric and constructive characteristics that affect the seismic behavior are gathered.

The geographical area in which to report the study can not be too large (citywide surveys) unless the structures are not of considerable homogeneity in the territory or the researcher has sufficient resources to allow more measurements in large areas. The input data can be obtained by integrating different cognitive factors such as field surveys executed or specially made for other purposes but from which we are able to get enough information from those required, knowledge of the characteristics of strength and deformation of building materials actually used, all sorts of information derived from interviews, consulting projects, regulations etc., and the recurring design characteristics at the time of construction.

The statistical analysis of the information collected is used to define the typological classes recurring, differentiated by morphology geometrical / structural, period of construction, number of floors, height between floors, etc..

For each detected class, then, a building "sample" is extracted: it has to be studied in greater detail and the characteristics that define the seismic behavior has to be punctually specified.

Several seismic analysis can be conducted on the representative building of each class and the seismic capacity will be studied in probabilistic terms so to obtain the class vulnerability.

Seismic Analysis

3.1 Introduction

The great scientific evolution that has taken place in recent years allows to design "safe" structures, able to satisfy the performance requirements. Nevertheless, most of the existing building is seismically vulnerable. Almost all of the buildings was built in a period when the economy was led by a "boom" building and knowledge of structures, materials and activities was very limited and not sufficiently supported by adequate normative bases in technical terms. Therefore, the problem related to the structural safety is actual and it affects both the scientific community and public administrations. Existing buildings have degradation and age problems, as well as being designed according to standards, design practices and structural engineering concepts very different from those that are currently understood and accepted. Following a seismic event, knowing the level of safety of these structures is important to carry out scenario studies (identifying riskier buildings on the territory and planning useful interventions to the restoration of security), and to direct emergency aid to the most vulnerable areas.

The modern seismic design is based on the concept to satisfy the various performance levels, for each of which the structure should not exceed the predetermined degree of damage. For this kind of design a nonlinear analysis is essential. Designers need a more extensive theoretical knowledge (especially when they have to perform dynamic analysis), involving more computational effort than the linear static analysis. The linear elastic analyzes are quite simple to perform, but they are not able to predict the capacity of inelastic deformation that the structure offers, so they are unsuitable to a modern seismic design (based on the performance concept), where the nonlinear and near-collapse behavior should be investigated.

3.2 Nonlinear Static Analysis

In order to obtain an accurate and realistic prediction of the seismic response of a structure, have analysis tools that allow to capture the nonlinear behavior and its time evolution is necessary. The nonlinear dynamic analysis in step is undoubtedly the most comprehensive and effective tool (assuming that the structural model accurately reproduces the real system): the response of the structure is determined by step-by-step integration of the equations of motion of a system with nonlinear multi degrees of freedom (MDOF).

The following points hinder a widespread use in professional practice:

- the choice of involved parameters is delicate and significantly influence the results of the same;
- to obtain statistically reliable results, numerous analyzes that use a discrete set of accelerograms (appropriately selected and not easily defined) must be conducted;
- the accuracy of the analysis runs counter to the simplicity and speed of execution;
- the results interpretation is complex and expensive.

An attractive alternative is to work non-linear static analysis procedures that, while maintaining remarkable ease of use and interpretation of the typical linear static analysis, allows more realistic and reliable estimate of the structural response in nonlinear field. Their application is becoming more common both to design and to verify the structures.

3.2.1 Pushover Analysis

The *pushover analysis* is a type of analysis that can evaluate the nonlinear seismic behavior of the building in different directions of seismic motion.

It consists in applying to the building gravity loads and a system of horizontal forces that by maintaining unchanged the relative proportions between the forces themselves, are all scaled so as to increase monotonically the horizontal displacement of a control point of the structure (usually a point on top of the building), until the achievement of the ultimate limit state.

The pushover analysis can be performed by applying to the structure a forces system or a displacements system.

Any increase in the loads, the structural strength is revalued and the stiffness matrix is updated in accordance with the achievement of the convergence continuing to limit state of damage or default to the collapse of the structure.

The analysis is repeated by changing the forces distribution on the height and direction of the forces, considering various angular scans for the earthquake. The resistant elements are considered to be elasto - plastic, with a limited ductility and the limit rotations at yield and collapse are measured, according to legislative indications.

In a nonlinear analysis, the keywords are: “*demand*”, “*capacity*” and “*performance*”.

The *demand* is the measure of seismic ground motion, the effects of the soil on the structure or on the structural elements. It can be defined by the response spectra.

The *capacity* is the ability of the structure and its structural elements to resist to the corresponding seismic demand. For the structure, it can then be represented by a curve that defines the global behavior, using a function of the structural response shear -displacement (*pushover* curve).

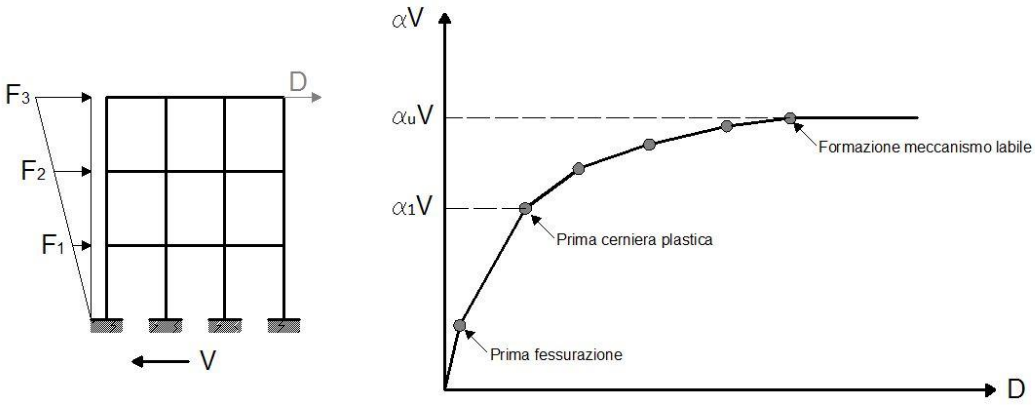


Fig. 3.1: Pushover curve

The *performance* represents the degree to which the *capacity* absorbs the *demand*. In other words, it indicates the real performance expected from the structure and it is obtained by the intersection of the capacity curve and the demand curve (Fig. 3.2).

The final goal of the analysis is to check the position of the intersection point (*performance point - PP*) compared to a point that defines the limit state design. The structure has to have the ability to resist to the seismic demand so that the performance is compatible with the project objectives.

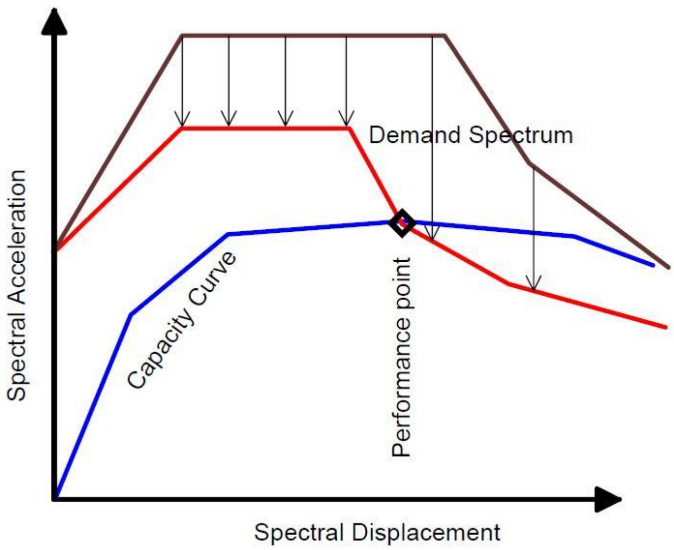


Fig. 3.2: Performance Point

The response analysis of the structure is connected to that of a system with single degree of freedom (SDOF), equivalent to the starting structure. The linear static methods allow to identify the maximum displacement of SDOF system and then also the response of the structure (point performance) subject to a seismic event featured by its response acceleration spectrum.

The Fig. 3.1 shows the evolution of the structural response to the growing intensity of the vector of the applied equivalent static forces. We may notice that when the forces grow, the number of plastic hinges which is formed subsequently to the achievement of the elastic threshold (first hinge formation), increases up to the achievement of a configuration corresponding to exhaustion of the capacity of an element.

Summarizing, then, the force - displacement system curve typical of a ductile behavior, which reaches the collapse, is characterized by the following phases:

1. The (not yet damaged by the seismic action) structural system is subjected to the action of equivalent seismic forces that have a defined distribution. In this first phase, the applied forces system produces elastic system displacement but not plastic deformation.
2. The intensity of applied forces system increases until to determine a first section plasticization, when a yielding limit is reached.
3. The structure is pushed further, leading to progressive formation of other plastic hinges and further global damage. At the beginning of the 'i-th step of charging, the structure is mechanically modified than its initial configuration because some plastic hinges are present. This modified structure can also be subjected to the action of an updated system of equivalent seismic forces. The loading step will end when the multiplier of the increasing horizontal forces will cause a new plasticity.
4. The analysis ends in correspondence of the activation of a collapse mechanism determined by the achievement of a plasticization degree that generates lability in the structure (global collapse) or the attainment of limit conditions.

The analysis is applicable for new design buildings and for existing buildings. In the first case, the pushover analysis can be supported by a linear analysis based on the factor q, in function of the value of the effectively available overstrength ratio α_u/α_1.

In the latter, after the materials and the reinforcemet characterization, thanks to the the pushover analysis is possible to seismically verify the building, which can be analyzed in the actual state and in the state resulting after the application of reinforcements to assess the achievement of complete seismic upgrading or only to evaluate an improvement compared to their previous state. To the lateral forces system is given the task of reproducing the effects that result on the structure as a result of an earthquake, so the choice of the distribution of forces adopted makes it more or less valid the whole analysis.

The shape of all the profiles of the lateral loads reported in the design codes is fixed and it does not vary during the analysis. This is one of the main limitations of the non-linear static procedures: the real distribution of inertial forces changes continuously on a building during an earthquake, due both to the higher vibration modes than that for the structural degradation. To obviate these drawbacks, non-linear static procedures that consider the presence and interaction of different vibration modes of the structure (multimodal interaction) have been developed. Furthermore, in the more advanced methods, the simultaneous variability of the distribution of the lateral forces which grow with the multiplier of the loads (loads adaptivity) is considered. By the simultaneous presence of multimodality and adaptivity, nonlinear static analysis results tend to become closer to the nonlinear dynamic analysis results. In this way the accuracy of the solution improves. Antoniou S. and Pinho [2004] proposed advanced procedures called FAP (Force-based Adaptive Pushover) and DAP (Displacement-based Adaptive Pushover).

The reliability of the results obtained from the use of algorithms FAP and DAP has been extensively tested on plan structures, where both methods give good results. For spacial structures, the validation is still experimental, especially as it regards the structures with strong irregularities, in which the dynamic behavior is extremely different from regular structures: torsional modes can dominate over those translational. In regular structures, the *mass center* (MC) coincides with the *rigidity center* (RC) and the validated classic pushover analysis for flat frame produced encouraging results, and also it applies to the spatial structures that are regular in plan and in height. When instead irregular structures are investigated (MC different from RC), serious torsional stiffness problems born. The predictions are incorrect, especially as regards the rotations of plane. The dynamic behavior of a spatial structure may be more complicated also because the translational vibration modes are coupled to the torsional modes. In these cases, a static analysis with difficulty captures the dynamic effects of the structure.

3.3 Nonlinear Dynamic Analysis

The design and seismic check of the buildings starts in most cases with analytical methods in which actions of the earthquake are represented in the form of response spectra, but some situations require fully dynamic analysis and the actions of the earthquake have to be represented in the form of time-history of acceleration. These situations include the safety design of critical structures, highly irregular buildings, or isolated structures designed for a high ductility degree. For such projects, the simulation of structural response conducted using one scaled elastic response spectrum of the structure factor is not appropriate, but a series of accelerograms adapted to the dynamic analysis is considered opportune.

The nonlinear dynamic analysis based on the use of accelerograms consist in to calculate the seismic response of the structure by means of direct integration (step-by-step) of the motion equations, using a nonlinear structure model. It has the purpose to assess the dynamic behavior of the structure in the non-linear range, allowing direct comparison between demand ductility and available ductility at each step of load, as well as to verify the integrity of the structural elements in relation to possible plastic behaviors.

Unlike of the static analysis, nonlinear dynamic analysis does not require the prior definition of global seismic demand. In fact, the global shift demand is estimated during the modal analysis, which provides only to estimate the peak response (through methods of combining static SRSS and CQC); the dynamic analysis allows to *accurately* calculate the maximum seismic response. Against these advantages in terms of accuracy, there is the need to define the nonlinear behavior of the structure in a more comprehensive way, including even the most accurate description of the *cyclic behavior*. Furthermore, the response is very sensitive to the input data, the accelerograms have to be defined in a proper way and the computational effort is high. Despite these limited complications, the nonlinear dynamic analysis is certainly the method for more accurate calculation, since it allows to know the time evolution of various parameters of the structural response (displacements, strains, strength and stresses).

This analysis has gained increasing importance also because of the need to apply it to structures with base seismic isolation, which are protected by devices whose behavior has strong nonlinearity that do not follow standardized models.

3.3.1 Incremental Dynamic Analysis (IDA)

The Incremental Dynamic Analysis (IDA)[3] is a structural analysis method that offers a complete prevision of the seismic demand, using nonlinear dynamic analyzes and subjecting the building to different sets of accelerogram. It addresses the need to want to investigate the dynamic behavior of a structure to different levels of seismic intensity. To do that, given an accelerogram, different dynamic analyzes are

[3] Vamvatsikos D., Cornell C.A. : "*Direct estimation of the seismic demand and capacity of MDOF systems through Incremental Dynamic Analysis of an SDOF Approximation*". ASCE Journal of Structural Engineering, 131(4): 589-599, 2005

conducted on the same structure but with a different seismic input, each time the input is scaled in ascending order, up to the collapse of the structure or a fixed level of deformation or displacement (Fig. 3.3).

This type of analysis provides additional advantages over a single dynamic analysis: it allows to observe the evolution of the structural behavior of the building with increasing of the seismic forcing.

In particular, it is possible to understand

- how the structure reaches the crisis,
- what kind of crisis is in place,
- where there are structural weaknesses,
- where the first plastic hinges are formed,
- what is the elastic behavior and post-yield,
- how the answer dynamic varies moving from linear to nonlinear behavior.

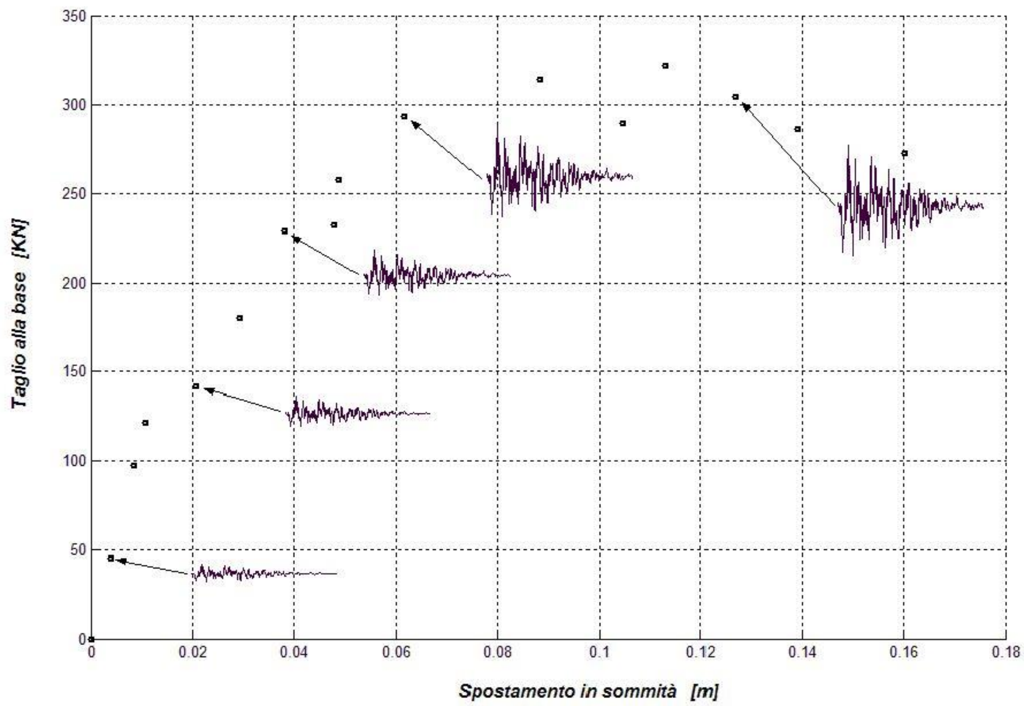


Fig. 3.3: IDA analysis results:

The efficiency of IDA is also confirmed by FEMA, that indicates it as the primary tool to determine the overall capacity of collapse of a structure.

3.3.1.1 Methodology

IDA was presented by D. Vamvatsikos and CA Cornell [2002]. It provides to subject the structure model to a series of accelerograms, each scaled by multiple levels of intensity.

In this way one or more response curves parameterized with the level of intensity are obtained.

The IDA is a widely applied method and it includes:

- the response or request range compared with the range of potential levels of ground motion;
- a better understanding of the structural implications due to earthquakes with levels of intensity of the more or less rare movement of the soil;

- a better understanding of the change in the nature of the structural response to the increase of the level of the earthquake;
- the evaluation of the dynamic capacity of the whole structure;
- the opportunity to compare the behavior of the structure subject to various earthquakes.

The first step is to define all necessary terms, including the choice of the accelerograms referred to the soil (usually spectrum compatible accelerograms are those used).

Given the accelerogram unscaled a_1, which varies as a function of time the terms that need to be introduced are[4]:

- The ***SCALE FACTOR (SF)*** of a scaled accelerogram, a_λ , is the non-negative scalar $\lambda \in [0,+\infty)$ that produces a_λ when multiplicatively applied to the unscaled (natural) acceleration time-history a_1.
- A ***MONOTONIC SCALABLE GROUND MOTION INTENSITY MEASURE*** (or simply intensity measure, ***IM***) of a scaled accelerogram, a_λ , is a non-negative scalar IM $\in [0,+\infty)$ that constitutes a function, $IM = f_{a1} (\lambda)$, that depends on the unscaled accelerogram, a_1, and is monotonically increasing with the scale factor, λ.
- ***DAMAGE MEASURE (DM)*** or STRUCTURAL STATE VARIABLE is a non-negative scalar $DM \in [0,+\infty]$ that characterizes the additional response of the structural model due to a prescribed seismic loading.
- A ***SINGLE-RECORD IDA STUDY*** is a dynamic analysis study of a given structural model parameterized by the scale factor of the given ground motion time history.

The single record IDA study can not fully capture the behavior of the structure.

[4] Incremental Dynamic Analysis Dimitrios Vamvatsikos and C.Allin Cornell

The IDA can depend on chosen accelerogram, then search for more accelerograms to better represent the response of the structure is sufficient.

- A ***MULTI-RECORD IDA STUDY*** is a collection of single-record IDA studies of the same structural model, under different accelerograms.
- An ***IDA CURVE SET*** is a collection of IDA curves of the same structural model under different accelerograms, that are all parameterized on the same *IMs* and *DM.*

Defining the validity of DM obtained by scaling the used accelerograms is very important.

The value of DM is obtainable by the average of the DM obtained by the earthquakes which have been scaled with the same level of IM. There is the need to figure out if this method of action is correct: the answer depends on the kind of structure, on IM and on DM. It is correct for short periods (1 second), for DM as maximum displacement interstory, with IM as the first way of the period of the acceleration spectral and for a general class of earthquakes (moderate or large magnitude) except where IM is defined by the PGA (Peak Ground Acceleration)[5].

About the number of accelerograms to be used, the authors propose a number ranging between 10 and 20 for mid-rise buildings, as the results relating to the seismic demand were sufficiently precise during the tests.

IDA behavior can greatly change depending on several factors:

- numerical convergence
- algorithm choice
- interpolation problem
- DM and IM summarization

[5] Peak ground acceleration (PGA) is equal to the maximum ground acceleration that occurred during earthquake shaking at a location. PGA is equal to the amplitude of the largest absolute acceleration recorded on an accelerogram at a site during a particular earthquake.

- sensitivity in the size of earthquakes

IDA offers the possibility to handle a large amount of data for numerous analyzes giving helpful conclusions. The recorded earthquakes, the number of tests for each earthquake, the interpolation results, the approximations are some of the issues that make a difference on the accuracy of the final results. The method is designed to foster a compromise between speed and accuracy.

3.3.1.2 IDA curves properties

IDA curves can be realized in two or more dimensions depending on the number of IM chosen.

Conventionally, the IM variable is shown on the x-axis and the variable DM is reported on the y-axis. Some examples of these curves are shown in Fig. 3.4 in which are shown 4 different structural behaviors of a framed steel structure of 5 floors.

The response is very variable, although common features are detected, including the initial portion, characterized by a $S_a \leq 0.2g$, almost identical, which ends with the entry into the plastic range of the first element. The slope *IM/DM* of this section takes the name of "elastic stiffness" and it is an intrinsic characteristic of the structure.

The four different curves end for different values of *IM*. In the curve "*a*", after reaching the condition of first yielding, it leads to a significant degradation of the structure with increasing displacement for small variations of IM. The curves "*a*", "*b*", "*d*" end with a plateau which indicates the attainment of the condition of dynamic instability (defined in analogy to the static instability) and the possible collapse of the structure.

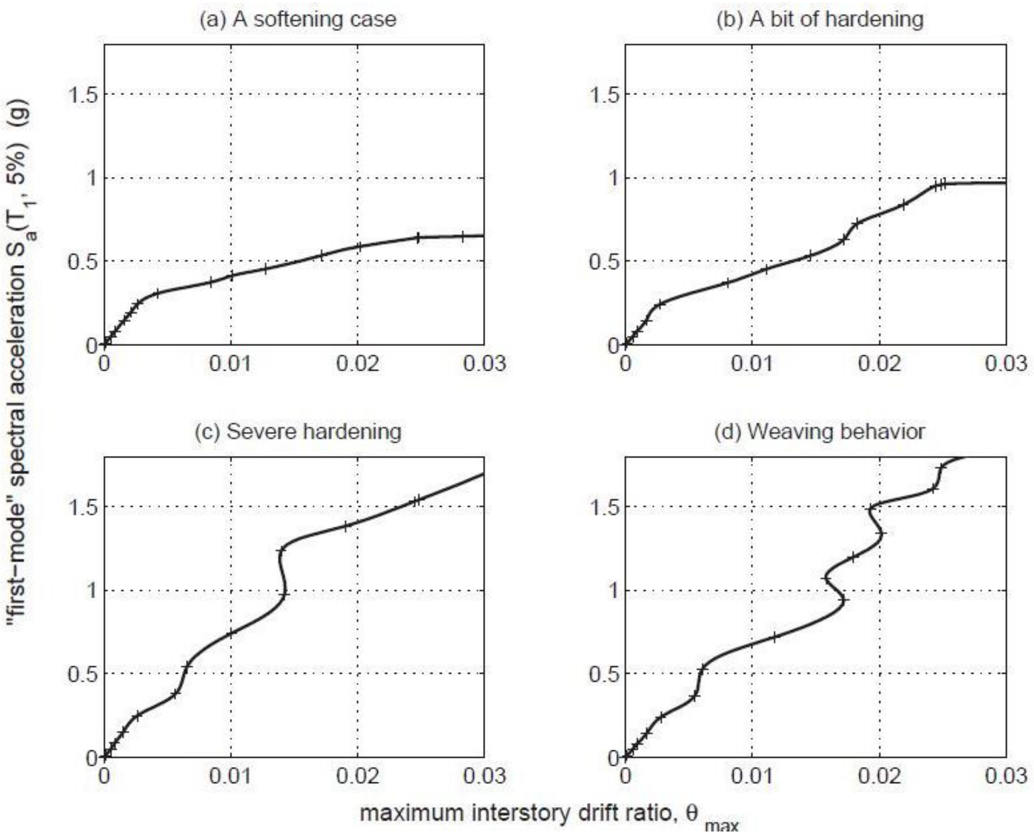


Fig. 3.4: Examples of IDA curves

The behavior of the curves "c" and "d" shows a non monotonic of the measure of damage, parts in which despite the stress increases, DM is reduced. This phenomenon is produced by the occurrence of high losses induced by plastic deformation of some structural elements.

In other words, a strong initial seismic shake produces the yielding of the structural elements present in a plane can happen, which acts as a dissipator, cutting off part of the energy induced by seismic and preserving the other floors from the remaining part of the earthquake.

An extreme example of hardening is also represented by the phenomenon of "structural resurrection" (Fig. 3.5). In fact it can happen that the response exhibits a collapse (normally represented by the non-convergence of the numerical DM) for a given IM, while for higher values is found a high damage, but finite.

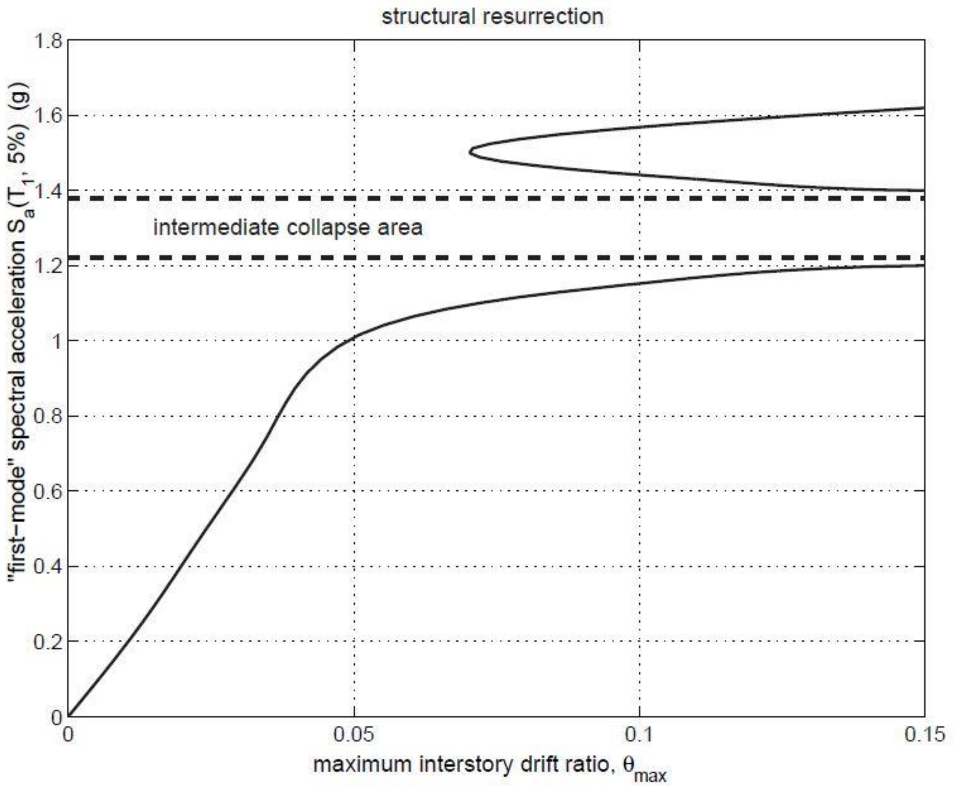


Fig. 3.5: Example of the phenomenon of structural resurrection

3.3.2 Achievement of performance levels according to FEMA

Performance levels or *limit states* are the important "ingredients" in the *Performance Based Earthquake Engineering* (*PBEE*), and the IDA curves contain the necessary information to determine them.

FEMA (*Federal Emergency Management Agency*) put special attention to the different levels (that representing indications, valueless prescriptive) developed in order to provide efficient tools in the rehabilitation of buildings damaged by earthquakes to the designers, in order to determine the damage of structural elements.

The project was carried out through the collaboration of various agencies, such as the *Building Seismic Safety Council* (*BSSC*) and the *American Society of Civil Engineering* (*ASCE*) and it is characterized for its innovative approach to "performance-based", ie it is focuses on usability and on the damage of the structures rather than on the strength of the elements.

To understand the analysis, briefly describe the fundamental concepts introduced by FEMA is required.

First, four levels of building performance are defined:

1. *Operational Level* (*OL*);
2. *Immediate Occupancy level* (*IO*);
3. *Life Safety Level* (*LS*);
4. *Collapse Prevention Level* (*CP*).

These levels represent the discrete points on the ideal continue line describing the behavior of the structure, which then are easily identifiable in the IDA curves.

Each level of response of the building is defined according to a performance level of the structure and of a level of performance of the non-structural components.

The association of a level of performance for the building and a certain intensity of seismic activity is a "*rehabilitation goal*".

Any combination can be considered by the designer, but the one described in the indications of FEMA is the *Basic Safety Objective* (*BSO*).

The latter is based on the following assumptions:

- the building has to satisfy the *Life Safety Building Performance* level for an earthquake of type *BSE-1*;
- the building has to satisfy the *Collapse Prevention Building Performance* level for an earthquake of type *BSE-2*.

IDA curves are an excellent tool to determine the properties of strength and ductility of the structure, and they make it easy to highlight the achievement of different performance levels. However, the problems relating to the non-monotonicity of the curves IM / DM are limits, being constituted by very precise values of DM that can be reached several times during the incremental dynamic analysis. Particular caution is recommended in assigning the levels.

To overcome this problem, the following criteria are available:

- ***DM- based rule***:

  The *DM-based rule* is based on the assertion *DM≥CDM* for which the limit has been exceeded. Normally, this is the criterion used. A graphical representation of this criterion are presented in *Fig. 3.6*.

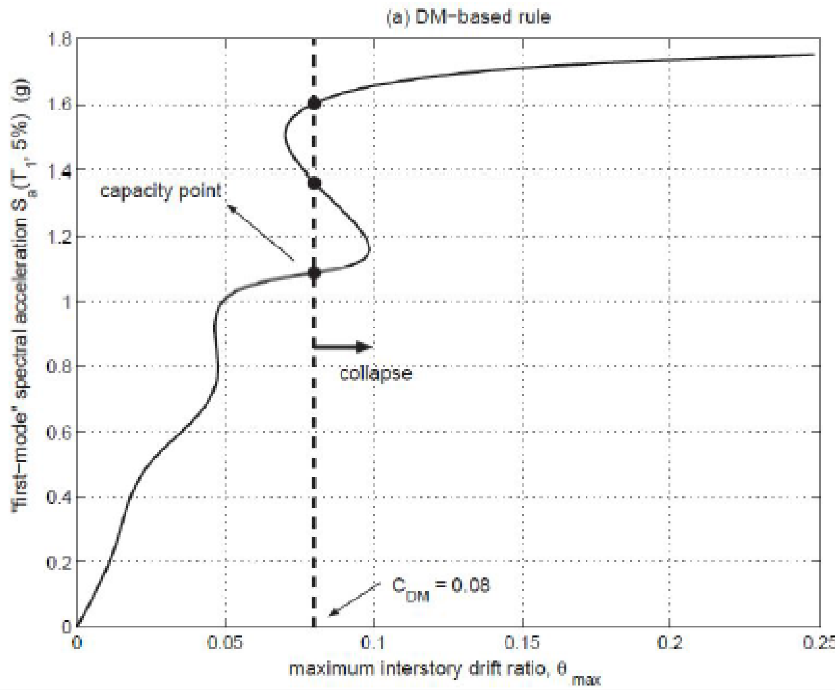


Fig. 3.6: DM-based rule

This method is the most in favor of safety method. Many authors suggest, in fact, to make reference to the first intersection between the IDA curve and the and the straight limit line. Methods based on this criterion have the obvious limitation of not being able to accurately identify the structural collapse, but they have the advantage of being easily implementable.

Two examples of this criterion that are present in the directions FEMA are: the maximum ratio interstory / height and maximum plastic rotations.

- ***IM- based rule***:

These methods born from the need to identify more accurately the collapse of the building and in the case of *IM* monotonous, the collapse can be expressed with the condition IM≥ CIM. The quality of this method, shown in *Fig. 3.7*, is that it generates a single condition of collapse, even if it is impossible to define a value of *CIM* valid for all curves IDA.

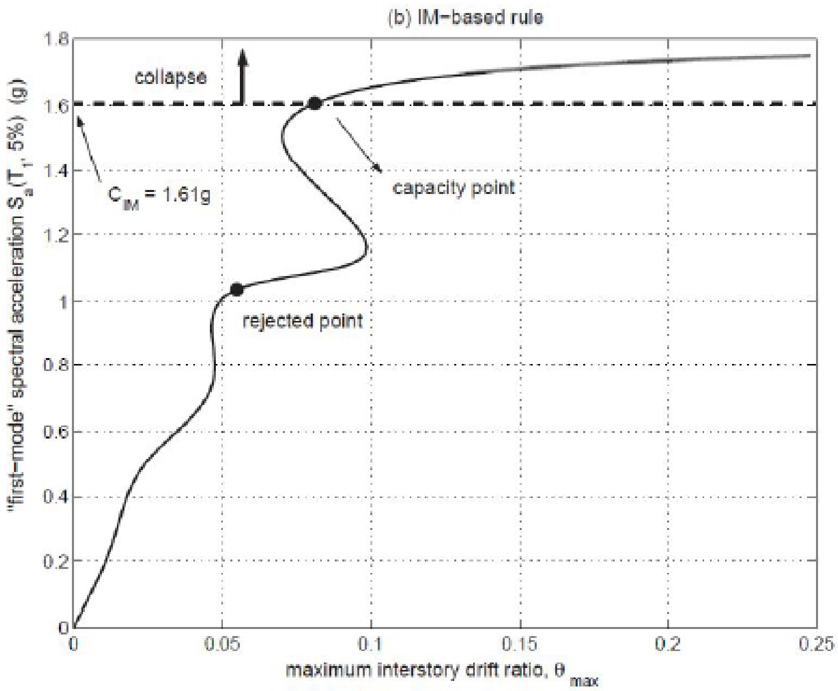


Fig. 3.7: IM-based rule

In addition, in this case, the FEMA directions are usable for the design of steel frames, in which it identifies the capacity of the structure as the last point of the curve IDA with a slope equal to 20% of the elastic.

The major limitation of this method is represented by the non-monotonicity of the curves IDA already previously treated.

3.3.3 SPO2IDA tool[6]

The Incremental Dynamic Analysis is a computer-intensive procedure that has been incorporated in modern codes and offers through demand and capacity prediction capability, in regions ranging from elasticity to global dynamic instability, by using a series of nonlinear dynamic analysis under suitable multi-scaled ground motion records.

Professional practice favors simplified methods, mostly using single-degree-of-freedom (SDOF) models that approximate the behavior of multi-degree-of-freedom (MDOF) system, by matching its Static Pushover (SPO) curve, coupled with empirical equations derived for such oscillators to rapidly obtain a measure of the seismic demand. These procedure use oscillators with bilinear backbones that only allow for elastic perfectly-plastic behavior, and occasionally positive or negative post-yelding stiffness.

The SPO2IDA software makes available empirical relations for full quadrilinear backbones and, when suitable applied, it provides the ability to accurately approximate the full IDA and investigates the connection between the curve SPO of the structure and its seismic behavior.

If the SPO of the MDOF system is plotted on $\theta_{\max}$ versus S_a *(T1, 5%)* axes, where the total base shear is divided by the total mass and scaled to match the elastic part of IDA by an appropriate factor (that is equal to one for SDOF system), and by plotting SPO curve versus median IDA curve on the same graph, it is observed that both curves

[6] "*Direct estimation of the seismic demand and capacity of MDOF systems through Incremental Dynamic Analysis of an SDOF approximation*" – D. Vamvatsikos, C.A. Cornell, M. Asce

are composed of the same number of corresponding and distinguishable segments (Fig. 3.8)

The elastic segment of the SPO coincides by design with the elastic IDA region, having the same elastic stiffness, while the yelding and hardening of the SPO forces the median IDA to approximately follow the familiar equal displacement rule for moderate period structures by maintaining the same slope as in elastic region.

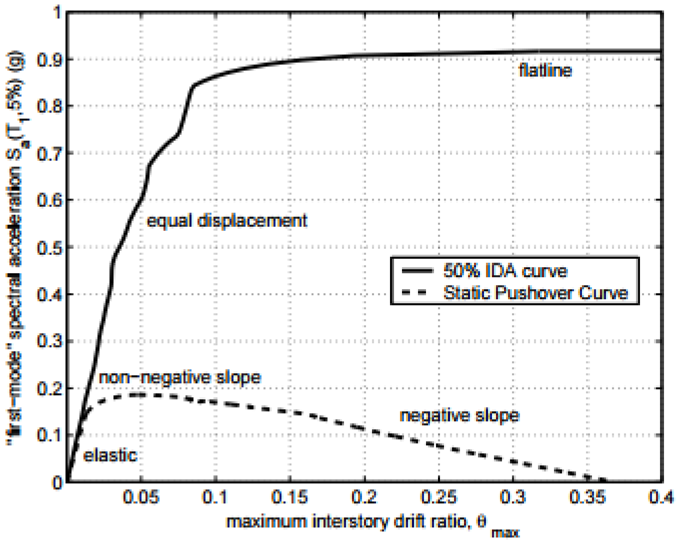


Fig. 3.8: The median IDA compared against the SPO generated by an inverted triangle load pattern

Past the peak, the SPO's negative stiffness appears as a characteristic flattening of the IDA, the flatline, that eventually signals global collapse when the SPO curve reaches zero strength. This apparent qualitative connection of SPO and the IDA drives the research effort to provide a simple procedure that will use the (relatively easy to obtain) SPO plus some empirical quantitative rules to estimate the fractile IDAs for a given structure, providing the IDA curves at a fraction of the IDA computation.

Based on the established principle of using SDOF oscillators to approximate MDOF systems, the SPO2IDA connection has been investigated for simple oscillators. The SDOF systems studied were short, moderate and long periods with moderately pinching hysteresis and 5% viscous damping, while they featured backbones ranging from simple bilinear to complex quadrilinear with an elastic, hardening and negative-stiffness segment plus a final residual plateau that terminated with a drop to zero strength.

The oscillators were analyzed through IDA and resulting curves were summarizated into their 16%, 50% and 84% fractile IDA curves which were in turn fitted by flexible parametric equations.

Having compiled the results into the SPO2IDA tool, available online, an engineer-user is able to effortlessly get an accurate estimate of the performance of virtually any oscillator without having to perform the costly analyses, almost instantaneously recreating the fractile IDAs in normalized coordinates of $R = S_a(T_1,5\%)/S_a^y(T_1,5\%)$ (where $S_a^y(T_1,5\%)$ is the $S_a(T_1,5\%)$ -value versus ductility μ.

Large Scale Modelling

4.1 Introduction

This chapter describes the issues related to large-scale modeling for vulnerability assessment of classes of buildings. Since the description of the methodology in the abstract may not be very clear, we have chosen to detail the basic steps of the procedure with an example on the evaluation of the vulnerability of the built in reinforced concrete made for the Zogafou area.

4.2 Focus Area

In order to make a credible assessment of the vulnerability, developing a typological classification of the building heritage that is thorough and well organized is necessary.

In Italy, as in Greece and in other European and Mediterranean country, statistical agencies as the *Istat* provide data as the year and type of construction, number of inhabitants and other information that are not seismically important. Wanting to get more detail on built, refer to different sources of knowledge, such as field surveys, or consultations of project work, or even interviews with designers or local workers is necessary. Thinking of using surveys to sample the entire housing stock is unrealistic. Turn our attention to areas with strong homogeneity of the construction, which can be identified on the basis of urban studies at the regional scale first, and then to municipal scale, is better: in this way the dispersions in the results of the sampling are avoided. This task is easier if just one type of structural systems is taken into account. Focusing on areas where there are reinforced concrete buildings, the field of investigation with respect to the structural system is restricted. This corresponds, in fact, to identify the real test areas, in which to make a finding based on typological surveys ad hoc, makes sense.

4.2.1 Zografou Area

In this study, Zografou Area resulted suitable to conduct the vulnerability assessment as the constructive features are corresponding to what is illustrated in the previous paragraph.

4.2.1.1 Location and historical notes about Zografou

Zografou is an inner suburb of Athens, located about 4 km East of Athens City centre. Towards the East the Municipality extends to the forested Hymettus Mountain. The built-up area of Zografou is continuous with that of Athens.

Fig. 4.1: Location of Zografou Area within the Region

After the departure of the Ottomans from the area in the 1830s, the zone came into the ownership of Ioannis Koniaris, mayor of Athens from 1851–1854, and Leonidas Vournazos.

In 1902, Eleni Vournazos, widow of Leonidas, sells 1,250 stremma of the Kouponia/Goudi area to Ioannis Zografos (d. 1927), a Member of Parliament for the Nationalist Party and university professor. Dividing it into plots, he sold them for installments of 112 drachma per month. The first houses were erected in 1919. Within ten years, 100 had been built. At this time, the foundations of the Church of St. Theraponta were erected.

In 1929, the area, now known as Zografou, was split from the city of Athens and became an independent community. It was elevated to a municipality in 1947.[5] its first president being Sotirios Zografos, the son of Ioannis. In 1935, the area of Kouponia (now Ano Ilisia) was incorporated into the community.

In 2011 (year of the latest population census), in Zografou there were 71,026 inhabitants.

4.2.1.2 Housing Schemes

As regards the constructive point of view, Zografou is a recent area, in which the most of building is represented by reinforced concrete structures.

An aggregate of 503 representative r.c. buildings is detected (Fig. 3) in several ways.

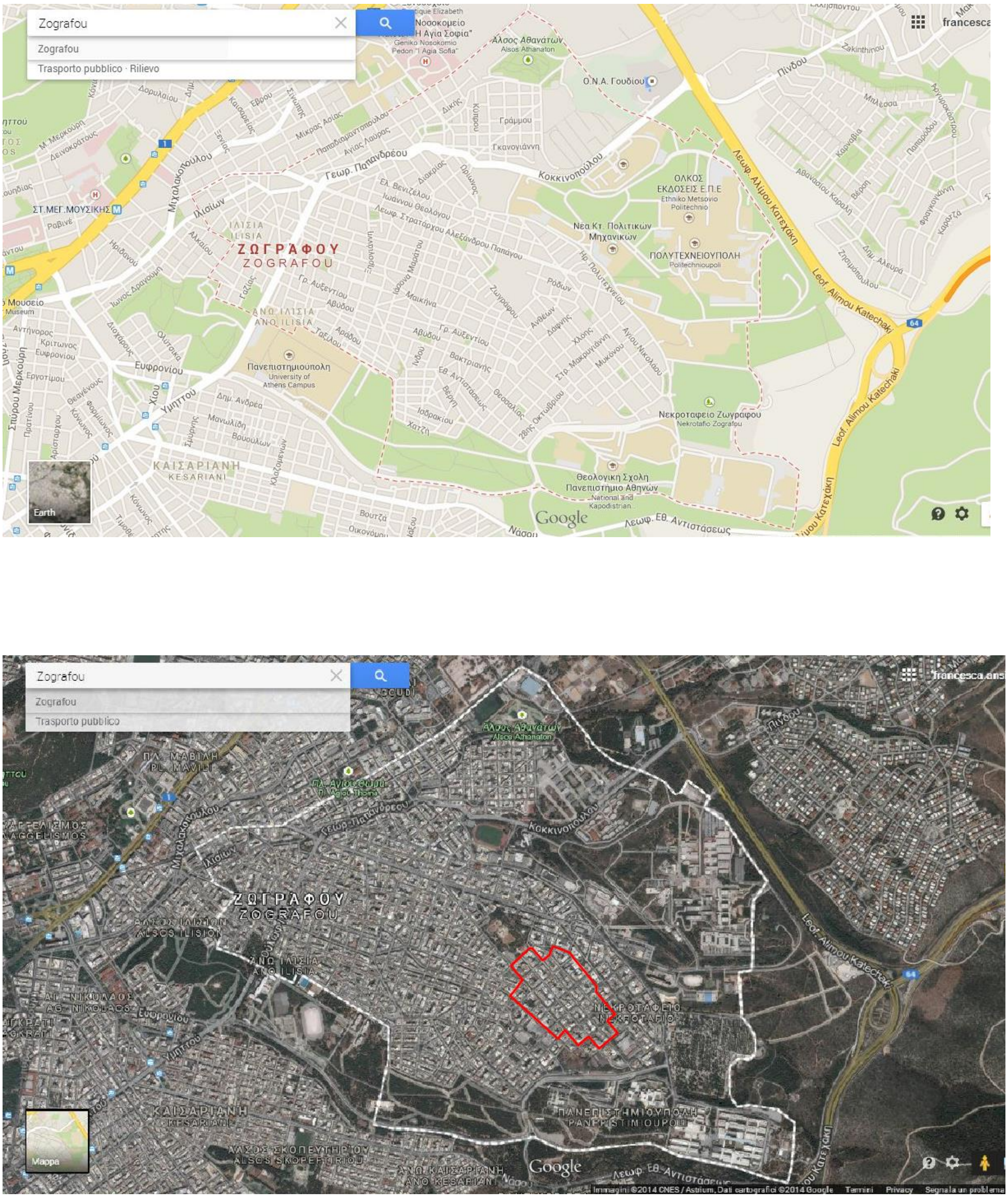


Fig. 4.2: Zografou Area in white outline (503 detected buildings in red outline)

A photographic surveys and visual investigation were conducted, a gathering cadastral information was carried out, professional advices were taken into account to obtain knowledge framework.

For each building, features like:

- number of floor,
- year of construction,
- kind of plan shape,
- area,
- presence of open ground floor and
- presence of setback

were treated statistically.

4.2.1.3 Statistical information

Table 4.1: Number of floor

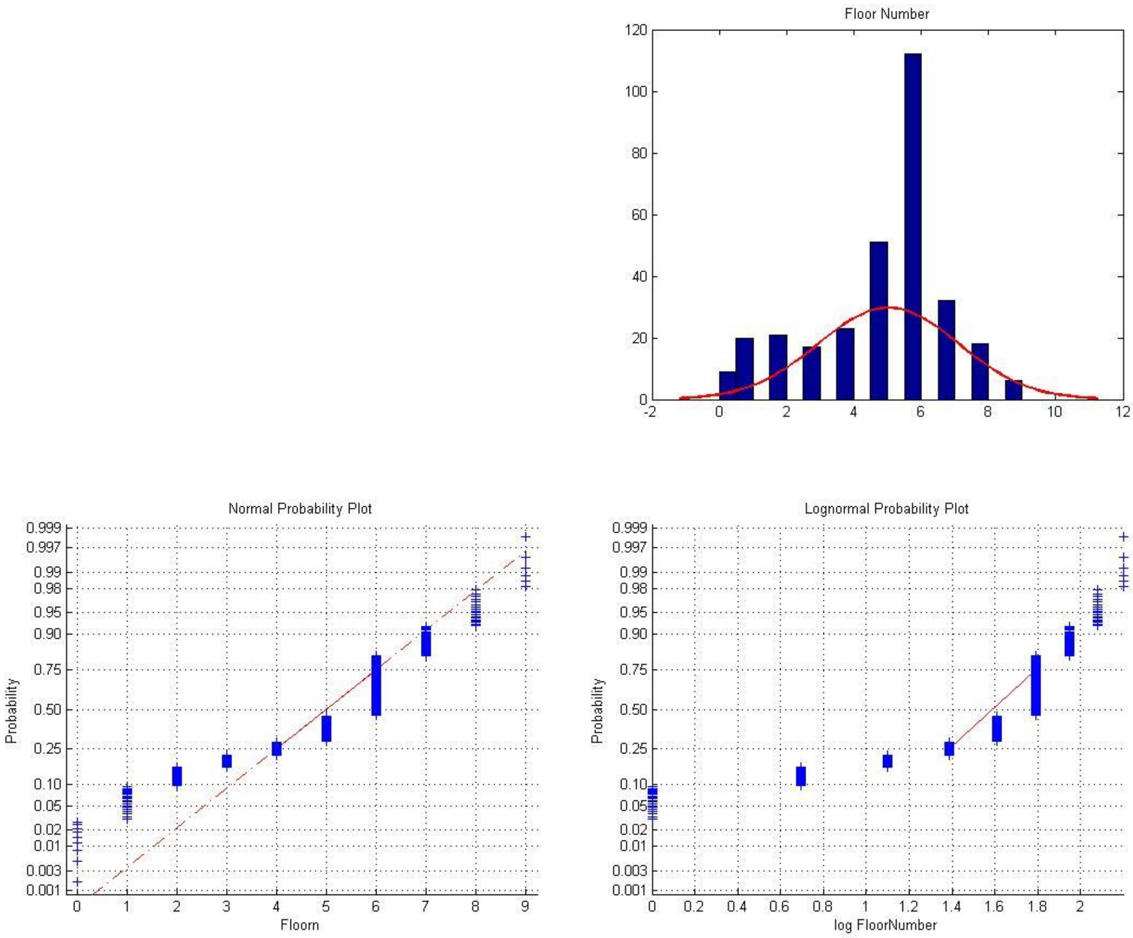
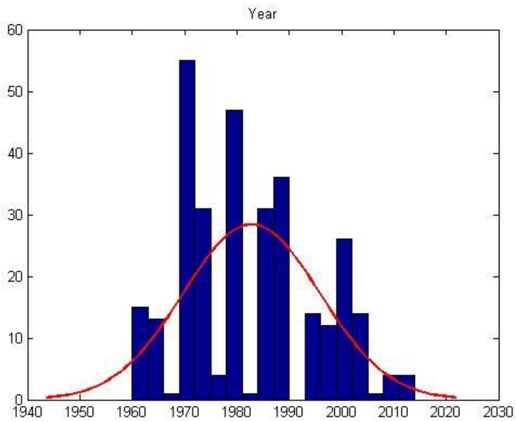
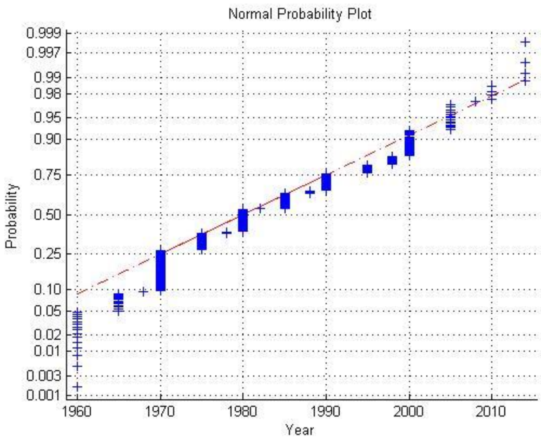


Table 4.2:Year of Construction





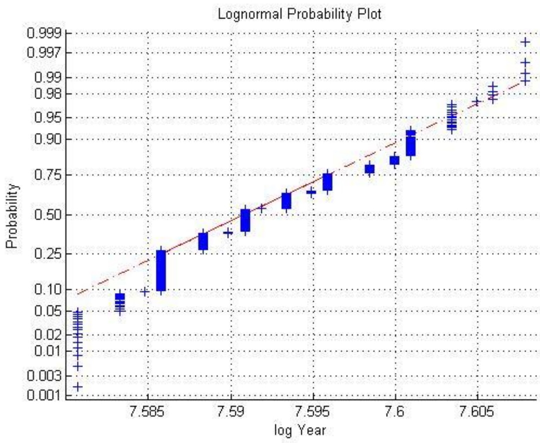


Table 4.3: Plan Area

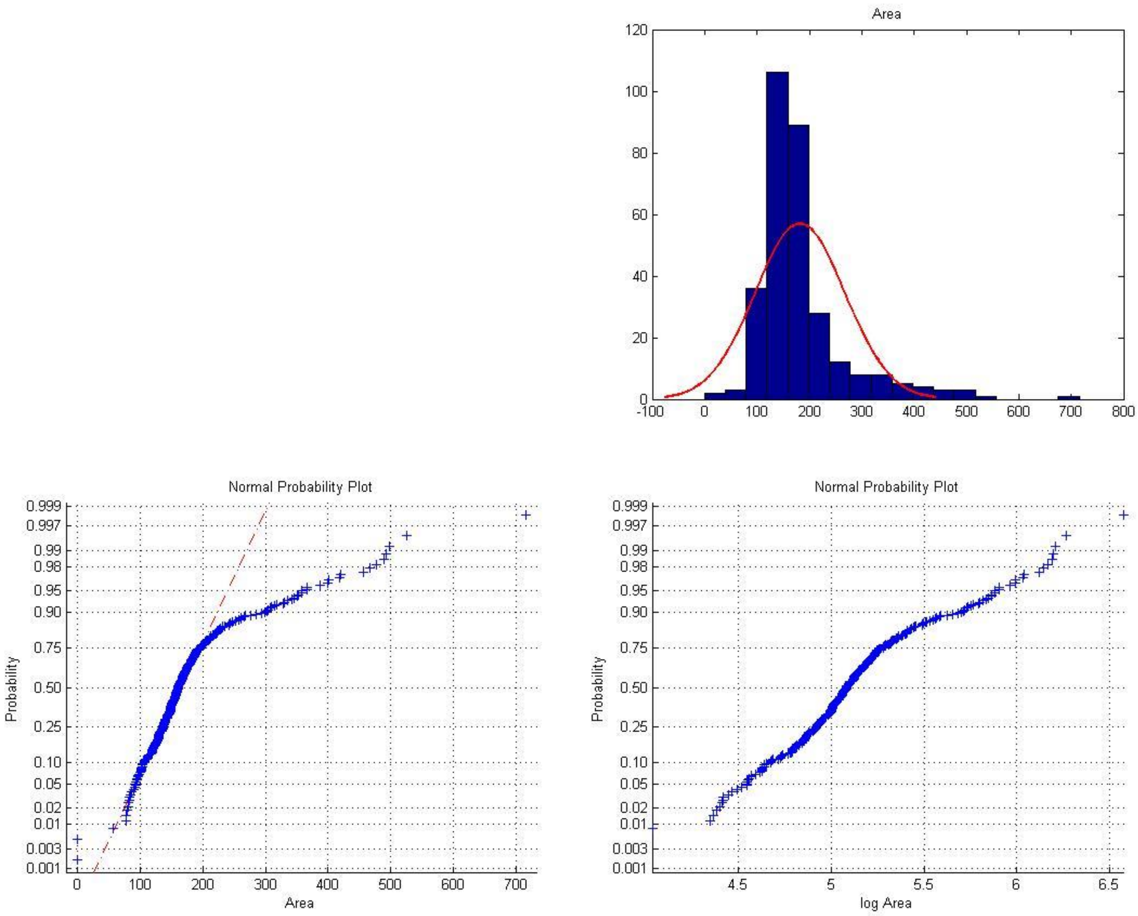


Table 4.4: Correlation coefficient

			Correlation coefficient					
	Mean	std	Area	Floor number	Open ground floor	Setback	Shape	Year
Area	182,00	86,0756		0,1133	-0,1984	0,1507	0,1352	-0,1322
Floor number	5,03	2,0723	0,1133		0,4615	-0,0032	-0,0431	0,5557
Open ground floor	0,26	0,4369	-0,1984	0,4615		0,0233	0,006	0,5629
Setback	0,06	0,2832	0,1507	-0,0032	0,0233		0,0482	-0,0274
Shape	0,35	0,4776	0,1352	-0,0431	0,006	0,0482		-0,0668
Year	1982,80	13,0337	-0,1322	0,5557	0,5629	-0,0274	-0,0668	

4.3 Definition of building characteristics

Take into consideration the different characteristics that distinguish the buildings is very important in order to obtain useful information. Thanks to them, we can arrive at models that reflect reality.

From the year of construction of the building (identified through research at the local land registry and through information of experts in the construction industry), depends the building codes that has be used.

Quantity as the number of floors, the presence of pilotis and setback are immediately definable, already during the survey operations.

Irregularities in plan and height (if the setback is present) are additional parameters that define the seismic behavior of buildings.

4.3.1 Greek seismic code[7]

Existing RC buildings in Greece, engineered for seismic resistance, have envolved through three generations of seismic design codes, namely RD59 [1959], MOD 84 [1984] and EAK [2000], the former two being based on the allowable stress design method while the latter, currently in its second revision, being based on ULS design.

Reinforced concrete buildings built in Greece as well as other countries, constructed up to the 1980s, represent a significant portion of whole building estate that have been designed either in absence of specific previsions for seismic loading (before the 1950s) or with past generations of seismic codes, not in line with modern ductile design or prescribed seismic performance philosophy. In Greece, these buildings have been designed either with the first seismic code in effect, RD59 [1959], or subsequently, the 1984 Interim Modifications to RD59 [MOD84, 1984], following a series of major damaging earthquakes between 1978 and 1981. A reliable assessment, therefore, of these structures' structural behaviour and vulnerability under earthquake excitation, accounting for the particularities of the stock (e.g., vertical irregularity, material quality, infills and so on) is a subject of significant social and economic importance. A reliable knowledge of key performance parameters such as the form of failure and the available *q* factor and ductility capacity provides useful planning information for their retrofit and/or strengthening.

[7] *"Evaluation of the seismic performance of existing RC buildings: I. Suggested methodology"* and *"Evaluation of the seismic performance of existing RC buildings: II. A case study for regular and irregular buildings*"– C. Repapis, E. Vintzileou, C. Zeris, Journal of Earthquake Engineering

In order to assess these parameters a methodology was developed, based on inelastic static pushover (SPO) analysis, following an initial design and a series of failure Limit Criteria (LC) evaluations in order to establish the limiting deformability of the structure.

Such as in [7], the buildings can be classified according to the generation of Greek seismic Codes in the following four categories:

Buildings constructed in the 60s (Group 60). These structures have been designed according to RD59 [1959] following allowable stress procedures and simplified structural analysis models. Allowable stresses due to combined flexural / axial loads ranged between 5 to 8 MPa for the concrete (grade C12) and 140 MPa for the (smooth) steel reinforcement (grade S220), with a 20% increase for design under the seismic load combinations. Nominal values of dead and live loads were specified in the Greek Loadings Code [LC45, 1945] still in effect today. Structural elements possess no critical region reinforcement for confinement and no capacity design provisions were used in their design. A special check was carried out for perimeter columns and beams, while interior beams were usually designed for gravity loads only. Seismic design was based on a three-zone classification system, with the seismic base shear coefficients ε being 4%, 6% or 8% of the structural weight, for seismic zones I, II or III, respectively, for stiff soil.
Buildings of this period are characterized by dense and regular column spacing, relatively short bay sizes (3.0 to 4.0 m) and the absence of shear walls. Perimeter frames are infilled with unreinforced masonry walls 0.25 m thick, of good quality workmanship, with window and door openings usually in the same positions at each floor. Interior masonry partitions 0.10 m thick are used in the interior plan of the structure in an irregular pattern, depending on current use (or change of use); as a consequence, these are only considered as mass but not part of the lateral resisting system. Apart from openings, large window openings in the perimeter infill layout may also be encountered primarily at the ground, but also in any of the upper floors, either intentionally or when the use of the building changed from residential to commercial during its lifetime. The cross-section dimensions of columns are relatively narrow, reflecting the tendency of early designs to be economic in concrete usage since it was in situ mixed and manually conveyed and placed, and because of the relatively

low level of seismic actions. As another consequence of this, concrete exhibits wide scattering in its mechanical properties.

Buildings constructed in the 70s (Group 70). These structures have also been designed according to RD59 [1959], but more elaborate structural analysis models were adopted. Concrete grade becomes C16 while S400 steel is specified, with an allowable design stress of 240 MPa. Column spacing is regular but the bay sizes are increased to 5.0 or 6.0 m. Reinforced concrete shear wall cores were introduced in the 70s at the elevator shaft (typically 0.20 m thick). Partial infill irregularity is more frequently encountered at the ground floors. As before, structural elements possess no critical region reinforcement nor were there any capacity design provisions used in their design.

Buildings constructed in the 80s (Group 80). These structures have been designed according to the 1984 Interim Modification of RD59 [MOD84, 1984], which were introduced following the 1978 Thessaloniki and 1981 Athens earthquakes. Although the seismic base shear coefficients did not change, entire frame models (including shear walls) and triangular seismic load distribution substituted for earlier simplified models.

Use of multiple closed stirrups with reduced tie spacing at the end member critical regions, edge member reinforcement in shear walls, shear reversal design in beams (through controlling the allowable shear stress) and a form of joint capacity design using service flexural resistance levels were introduced. The building geometry remains same as in the Group 70 with the spans increasing to 7.00 m; often an open first storey (pilotis) was intentionally specified in which the use of infill walls is completely avoided for commercial development or parking space. Perimeter shear walls with an elevator core were typically used and concrete member dimensions generally become wider.

Buildings constructed in the 90s (Group 90). These structures have been designed primarily after 1995, with the adoption of the Greek Earthquake Resistant Design Code [EAK, 2000] and the Greek Code for Design of Concrete Works [EKOS, 2000]. Both are Ultimate Limit State (ULS) design codes, encompassing the majority of the currently established requirements for ductile response introduced in contemporary seismic provisions (among others, EC8 [2003]). These modern seismic codes introduce the use of inelastic design response spectra, the behaviour factor *q*, more stringent detailing for local ductility and confinement, capacity design, weak beam strong column behaviour and penalties for irregularity and plan torsion. Structures of this generation exhibit long spans, with or without an open first storey (with a penalty), provisions for adequate shear walls and large member dimensions.

4.3.2 Regularity definition

4.3.2.1 Regular buildings in plan

Area and shape of construction have been obtained thanks to the vision of the Zografou land registry files.

A building can be thought as a "regular" construction when the configuration is compact in plan, approximately symmetrical with respect to two orthogonal directions, in relation to the distribution of masses and stiffnesses[8].

Operationally, if *Hin* and *Lin* are the minimum internal dimensions to the outline of the building, and *Hout* and *Lout* is the maximum outside dimensions of the outline of the building (see Fig. 4.3), the regularity is defined by the following control:

$$H_{in} < 0.25 H_{out} \cup L_{in} < 0.25 L_{out}$$

[8] This definition of regularity in plan is also established in NTC08 (Italian code), § 7.2.2.

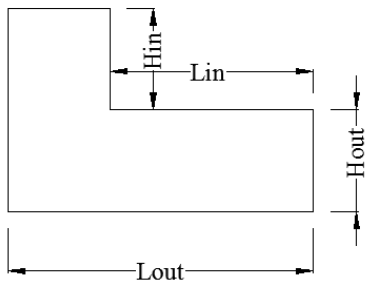


Fig. 4.3: Hypothetical shape of the building

In this way, for rectangular shape $H_{in} = 0$, $L_{in} = 0$, and the building results regular in plan.

4.3.2.2 Regular buildings in height

The presence of setback determines irregularities in height.

4.4 Data mining process

Data Mining is the extracting knowledge process from database, by the application of algorithms that identify the associations that are not immediately recognizable among the information and make them visible. In other words, with the name data mining we mean the application of one or more techniques that allow the exploration of large amounts of data, with the aim of identifying the most significant information and to make them available and directly usable as part of the decision-making. The extraction of knowledge, that is of significant information is made by identification of associations, patterns, or repeated sequences, hidden in the data.

The data mining algorithms have been developed to exploit the wealth of information contained in large collections of data that we have available. Often the data are in the heterogeneous, redundant, unstructured form. In this context, being able to exploit the potential wealth of information that we have available is a huge advantage. Powerful and flexible tools are necessary: the large amount of data and their heterogeneous nature makes inadequate traditional tools. These are divided into two types: *statistical analysis tools* and instruments typical of querying databases such *data retrieval*. As it regards the first, difficulties arise from the fact that they hardly operate on large amounts of data as they require sampling operations with consequent loss of information. The data retrieval is, in fact, a tool for querying databases and it consists of formulating a query. The system seeks, inside the database, all cases which meet the requirements in the query, all the data that have the required characteristics, then providing the answer. The identification of hidden associations can therefore only proceed by trial. While the use of data retrieval tools allows to have precise answers to any specific questions, data mining answers more general questions. This second approach allows to bring out the existing data associations without requiring the formulation of hypotheses a priori. The algorithm will put in evidence the characteristics, which occur repeatedly in the data. It is therefore an exploratory approach and not, as in data retrieval, verificativo. In this way we can discover relationships that not only were hidden and unknown, but that we would never even speculated could exist.

The data mining tools arise from integration of various fields of research as statistics, pattern recognition, or the machine learning, and have been developed independently from databases, in order to operate on raw data.

The used techniques are different and, consequently, also the algorithms that implement them. The choice depends primarily on the objective to be achieved and the type of data to analyze. The most used techniques are:

- *Clustering*
- *Neural Networks*
- *Decision trees*
- *Identifying Associations*

Clustering techniques and the use of *neural networks* allow to make segmentation operations on the data, that is, to identify homogeneous groups, or types, which have regularities in them and are able to characterize and differentiate them from other groups. *Neural networks* and *decision trees* allow to carry out the classification, to make use of the knowledge gained in training to classify new objects or predict new events. The techniques of *analysis of the associations* allow the identification of the rules in the concomitant occurrence of two or more events.

4.5 The knowledge extraction process

Regardless of the specific application, a process of knowledge extraction runs through some phases which can be schematically in:

- Goal definition
- Identification of sources of data
- Abstraction and data acquisition
- Pre-processing
- Data mining
- Interpretation and evaluation of results
- Representation of results

4.5.1 Cluster Analysis

In this work the methodology of data mining was used by adopting the technique of cluster analysis for the classification of buildings. The cluster analysis, sometimes translated as analysis of bunch (vine fruits) is a technique born in the 60s and 70s, aimed to identifying groups of data within a known population.

The cluster analysis is based on simple procedures and easily automated, it uses heuristics tecniques and rests on a rather elementary mathematics. On the other hand, precisely because of its simplicity has favored the spread between the researchers of natural science, and the readability of its results, the high heuristic potential and the availability of several analysis tools automatically make it a valuable tool and merits consideration.

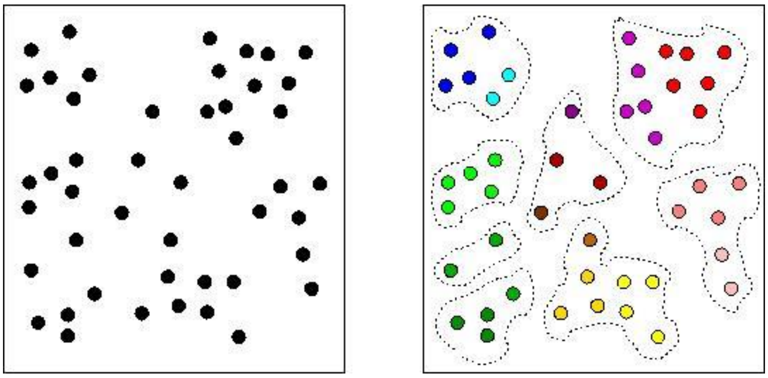


Fig. 4.4: Example of distinction in groups

In the context of a discriminant analysis, or a procedure of automatic classification in general, can make sense to ask if some variables, suppose in a number *q*, are not redundant, that is, if you do not add any information useful to the classification with respect to all the remaining *p-q*. Paradoxically, the elimination of variables that individually have a low index of separation can be a bad idea. In contrast, it is possible that a variable with a high information content for classification is unnecessary when used with other variables.

4.5.1.1 Distribution Methods

The objective of this class of algorithms is the division of the available data into n

subsets or clusters $C_1,..,C_n$ therefore such that

$$C_1 \cup ... \cup C_n = \{d_i / 1 \leq i \leq n\}$$

$$C_j \cap C_k = 0 \qquad j \neq k$$

so that the elements of each subset are the most compact as possible.

Main representative of the category of distribution methods is the algorithm ***Kmeans***, that is the most known and used. It uses as objective function to minimize the sum of squares of the distances between the points and the sample mean of the cluster that they belong to. Mathematically:

$$S_w \equiv \sum_{j=1}^{n} \sum_{i=1}^{Nj} (d_{ji} - m_j)(d_{ji} - m_j)$$

The number of possible configurations of the n clusters on the *N* data proves to be equal to

$$\frac{1}{n!}\sum_{j=1}^{n}(-1)^{n-j}\binom{n}{j}j^{N}$$

If *x* is the vector of *N* length that preserves the codes associated with the clusters of membership of each data. The *K-means* method, starting from an initial assignment x_0 and scanning the data one by one, at each step calculates averages and function objective, and assigns the observation to the cluster for which the new assessment of the objective function is minimal. The process stops when x remains unchanged for *N* consecutive cycles.

This algorithm is good at every step, but is not necessarily the optimal solution looked for. It is advisable to repeat the procedure with different initial configurations. It takes into consideration, however, that the objective function suffers from some limitations, and provides poor results for clusters do not sufficiently compact and separate, or having very different cardinality.

4.5.1.2 Hierarchical Methods

The goal of these algorithms is the organization of data in a hierarchical structure, which includes observations very similar in small clusters at lower levels, and more observations basically connected in larger clusters and generic at the highest levels, until you get to set of all data.

Formally, a sequence of *h* partition of cardinality strictly increasing of *N* data is obtained. If n_h is the cardinality of the it^h partition. Then:

$$1 = n_1 < \ldots < n_h \le N$$

In other words, the first partition of the sequence is represented by only one set C_1 including all observations; the second partition provides $n_2 > 2$ disjoint subsets and complementary to C_1, and so on, until the last partition, which is known does not necessarily involve the fragmentation of data in *N* singletons or degenerate clusters.

The methods of hierarchical analysis are divided into two major categories: the *divisive procedures* (procedures used in this work), and the *agglomerative or associative procedure*. In the first the new cluster are obtained by division of clusters belonging to the previous level. At the start there is a single cluster with all individuals, at the end there are many clusters as there are individuals.

The divisive build a tree diagram, dendrogram, which gives a picture of the relationships between objects.

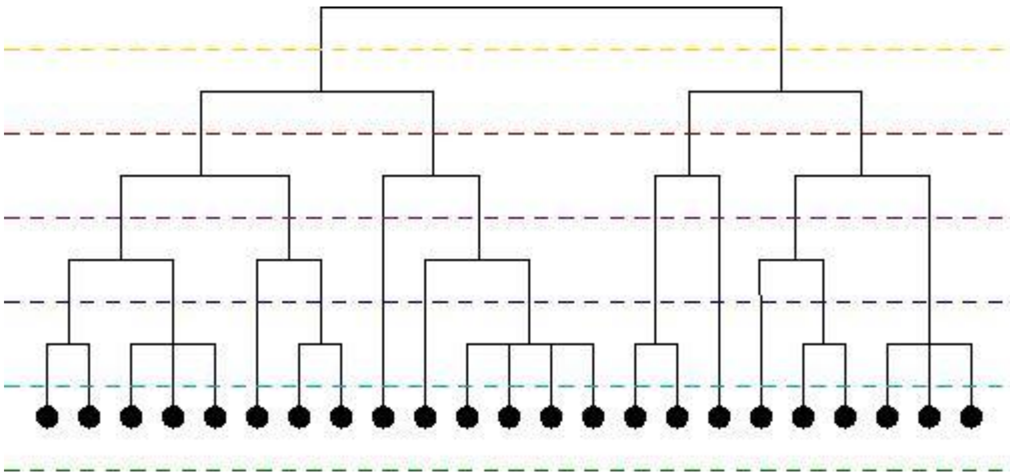


Fig. 4.5: An example of dendrogram

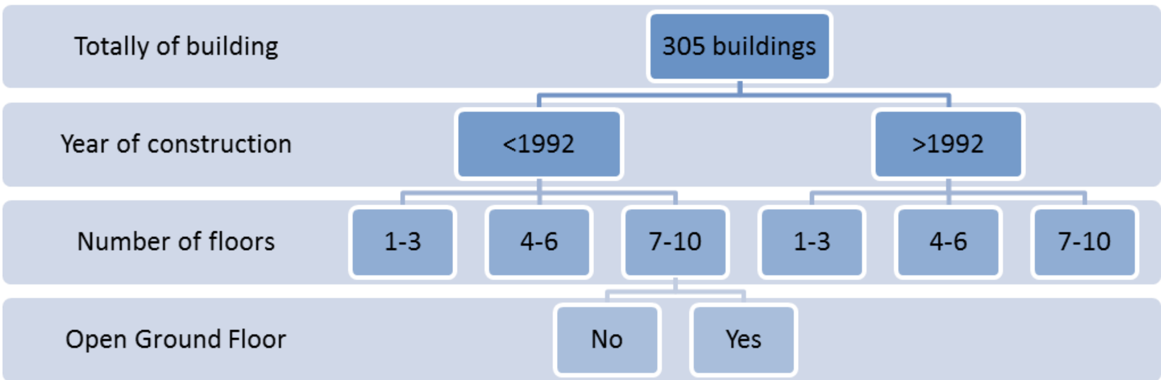
4.5.2 Clustering Models

4.5.2.1 First step: hierarchical by hand

It's important to control the level and the way that defines the groups.

Group the objects into a binary, hierarchical cluster tree and rational manner

Table 4.5: Dendrogram after hierarchical "by hand" procedure



So, the obtained group are 7:

- ***Group 1*** has 46 buildings (15% of total), 1-3 number of floors, and the construction period is before 1992.
- ***Group 2*** has 84 buildings (27% of total), 4-6 number of floors, and the construction period is before 1992.
- *Group 3* has 104 buildings (34% of total), 7-10 number of floors, and the construction period is before 1992. This group is dividend in two:
    - ***Group 3_0*** has 75 buildings (25% of total) and absence of pilotis;
    - ***Group 3_1*** has 29 buildings (9% of total) and presence of pilotis;
- ***Group 4*** has only one building (0.3% of total), 1-3 number of floors, and the construction period is after 1992.

- ***Group 5*** has 7 buildings (2.3% of total), 4-6 number of floors, and the construction period is after 1992.
- ***Group 6*** has 63 buildings (21% of total), 7-10 number of floors, and the construction period is after 1992.

4.5.2.2 Second Step: Clustering

As it has already been written in the previous paragraphs, *cluster analysis* (also called *segmentation analysis* or *taxonomy analysis)*, creates groups, or clusters, of data. Clusters are formed in such a way that objects in the same cluster are very similar and objects in different clusters are very distinct. Measures of similarity depend on the application.

K-means clustering is a partitioning method that creates a single level of clusters.
Kmeans treats each observation in your data as an object having a location in space. It finds a partition in which objects within each cluster are as close to each other as possible, and as far from objects in other clusters as possible.

Each cluster in the partition is defined by its member object and its centroid, or center. The centroid for each cluster is the point to which the sum of the distance from all objects in that cluster is minimized. Kmeans computes cluster centroids differently for each distance measure, to minimizethe sum with respect to the measure that you specif.

Kmeans uses an iterative algorithm that minimizes the sum of distances from each object to its cluster centroid, over all clusters. This algorithm moves objects between clusters until the sum cannot be decrease further. The result is a set of clusters that are as compact and well-separated as possible. You can control the details of the minimization using several optional input parameters to kmeans, including ones for the initial values of the cluster centroids, and for the maximum number of iterations.

To get an idea of how well-separated the resulting clusters are, you can make a *silhouette plot* using the cluster indices output from kmeans. The *silhouette plot* displays a measure of how close each point in one cluster is to points in the neighboring clusters. This measure ranges from +1, indicating points that are very distant from neighboring clusters, through 0, indicating points that are not distinctly in one cluster or another, to -1, indicating points that are probably assigned to the wrong cluster. Silhouette returns these values in its first output.

After various checks on the total sum of the distances of the centroids, the best solution has given the following Silhouette Plots:

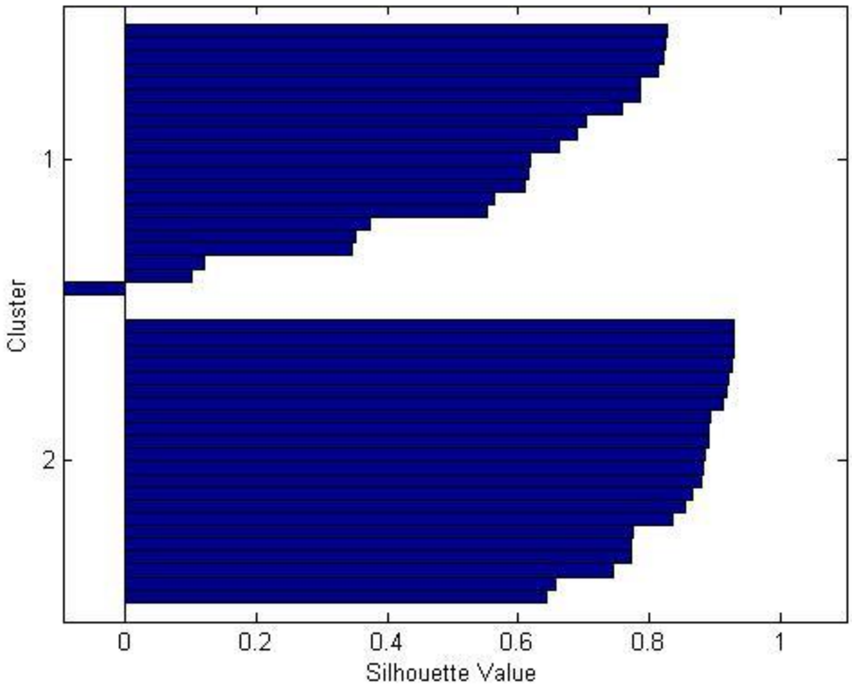


Fig. 4.6: Silhouette plot for Group 1, ***2 clusters***

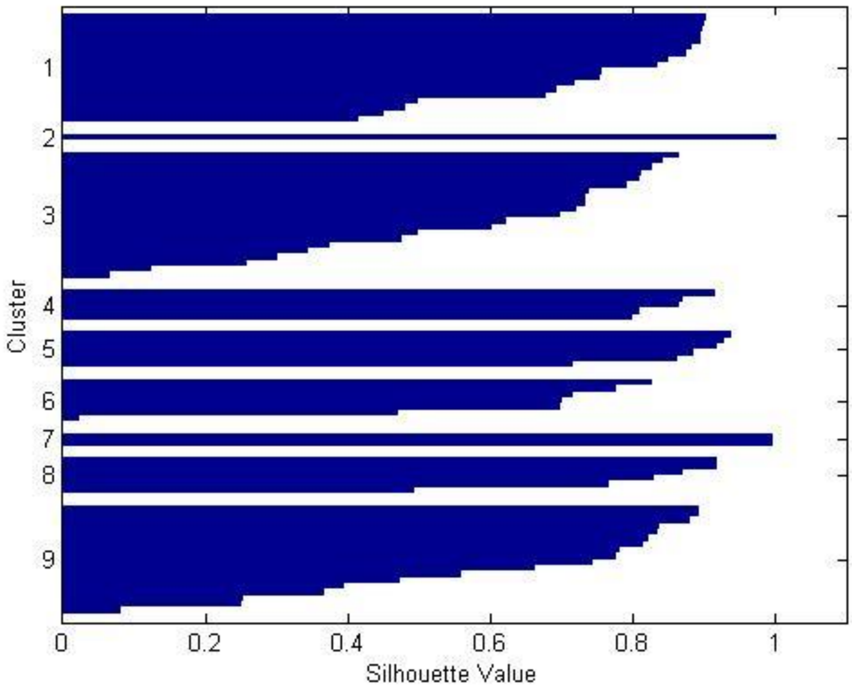


*Fig. 4.7: Silhouette plot for Group 2, **9 clusters***

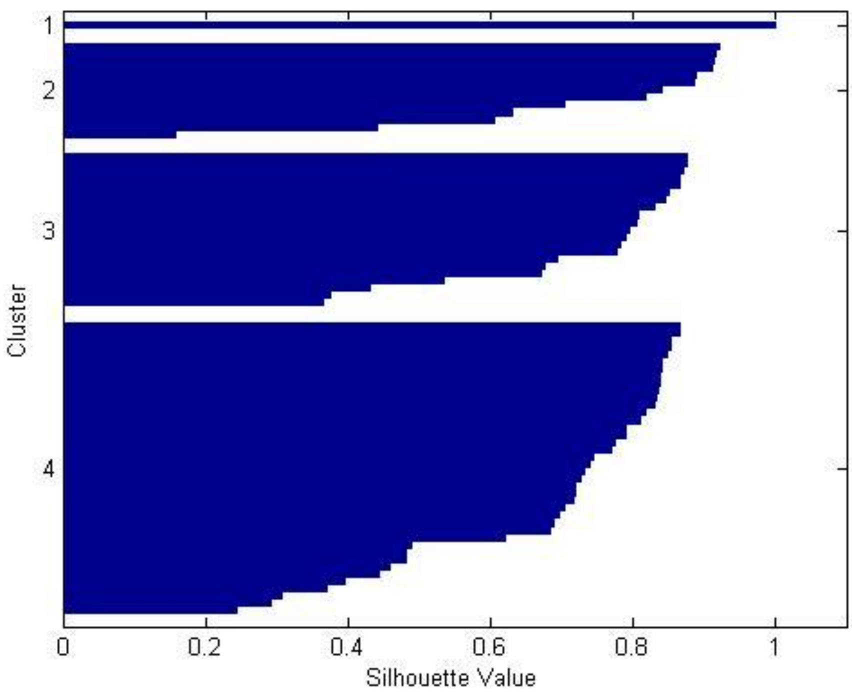


*Fig. 4.8: Silhouette plot for Group 3_0, **4 clusters***

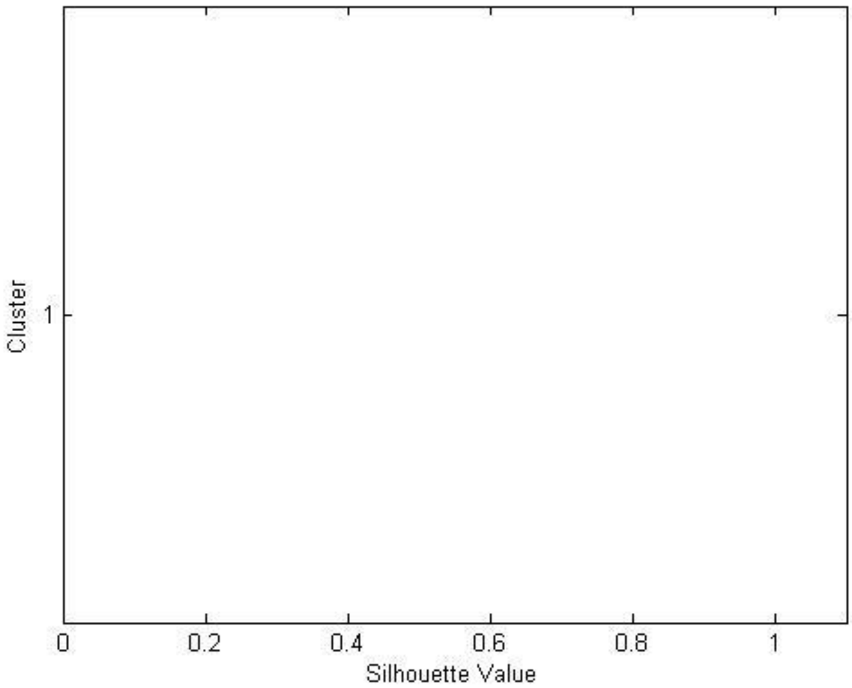


*Fig. 4.9: Silhouette plot for Group 3_1, **0 clusters***

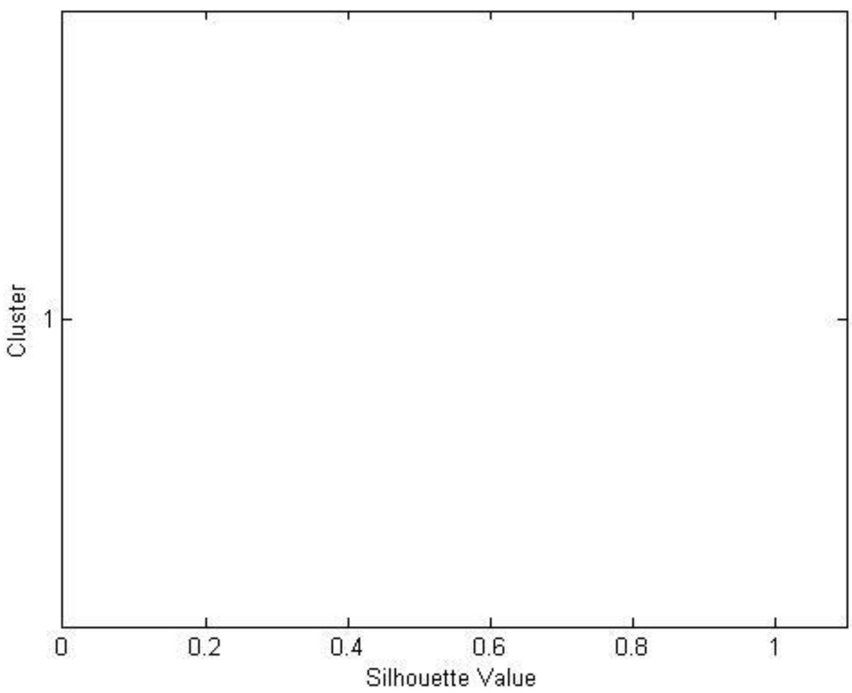


*Fig. 4.10: Silhouette plot for Group 4, **0 clusters***

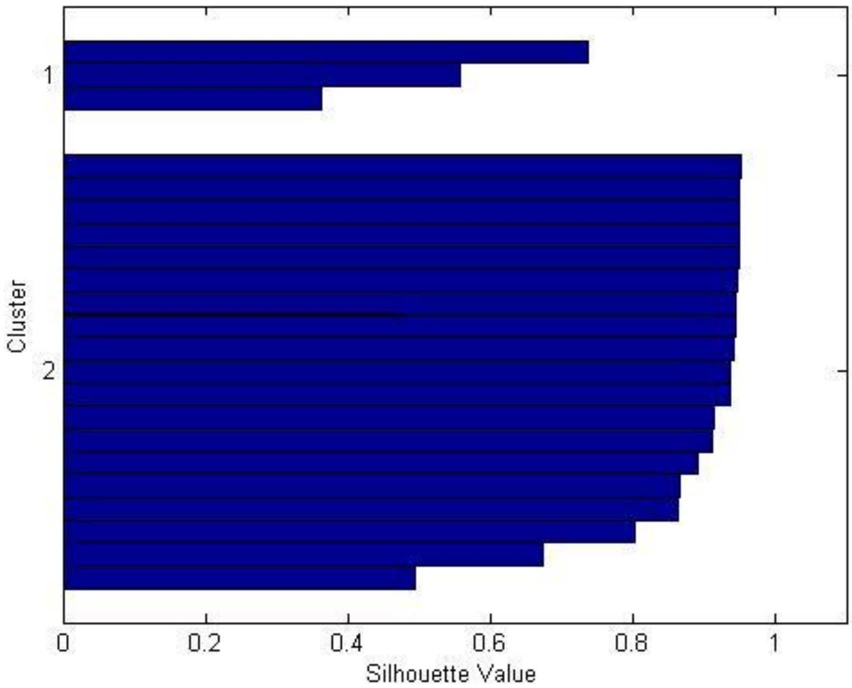


*Fig. 4.11: Silhouette plot for Group 5, **1 cluster***

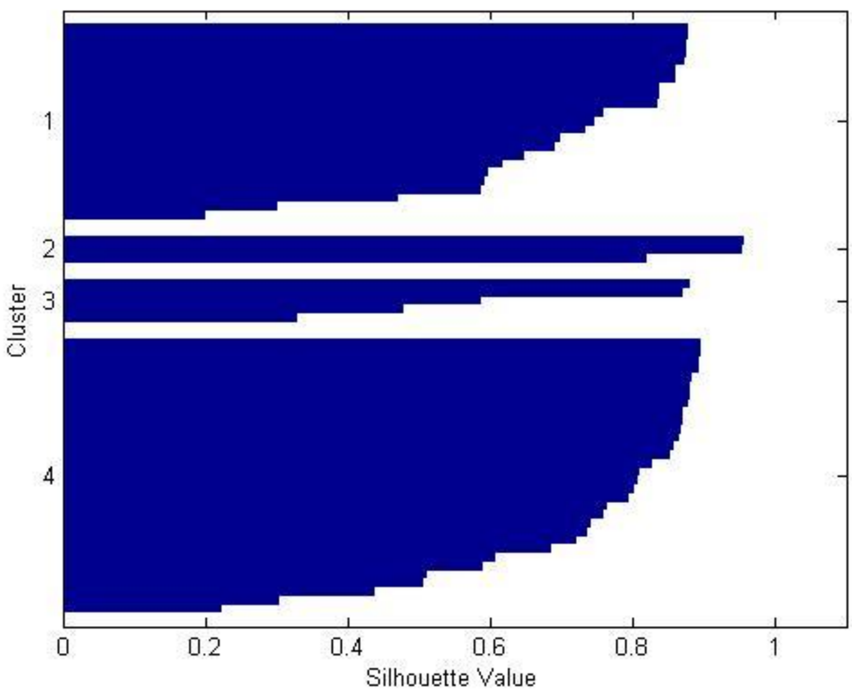


*Fig. 4.12: Silhouette plot for Group 6, **4 clusters***

So, the obtained clusters are, after k-means procedure, 21 in total. Table 4.6 shows the reults:

Table 4.6: Clustering results

Floor n	Year	Area	Setback	Max Ratio	Open Groundfloor		
2,26087	1969,348	110,1295	0,043478	0,198157	0	*Centroid 1*	*Group 1*
2,347826	1970	175,6061	0,086957	0,200278	0	*Centroid 2*	
5,285714	1973,571	242,016	0	0,059491	0	*Centroid 1*	*Group 2*
6	1965	715,604	0	0,574409	0	*Centroid 2*	
5,666667	1975,143	166,8733	0,095238	0,140635	0	*Centroid 3*	
5,6	1977	324,1154	0	0,092239	0	*Centroid 4*	
5,666667	1979,778	202,3093	0,055556	0,157969	0	*Centroid 5*	
4,833333	1972,5	137,06	0	0,064276	0	*Centroid 6*	
4,166667	1972,167	92,28133	0	0	0	*Centroid 7*	
6	1972,5	399,9035	0,5	0,193258	0	*Centroid 8*	
5,333333	1976,667	484,0717	0,333333	0,375571	0	*Centroid 9*	
7,075	1981,95	146,2613	0,05	0,143632	0	*Centroid 1*	*Group 3_0*
7,095238	1979,81	206,4382	0,047619	0,163554	0	*Centroid 2*	
7,076923	1976,923	331,6429	0,076923	0,286931	0	*Centroid 3*	
7	1980	497,926	1	0,602784	0	*Centroid 4*	
7,206897	1984,345	163,0444	0,068966	0,17686	1	*Centroid 1*	*Group 3_1*
-	-	-	-	-	-	*Centroid 1*	*Group 4*
5,857143	1999,714	196,7519	0	0,170053	0,142857	*Centroid 1*	*Group 5*
7,9375	2000,531	160,2857	0,09375	0,115107	0,71875	*Centroid 1*	*Group 6*
8,333333	1997,667	401,427	0	0,17407	0	*Centroid 2*	
8,347826	2001,304	116,1861	0,043478	0,123026	0,826087	*Centroid 3*	
8,6	1997,2	251,065	0	0	0,8	*Centroid 4*	

4.5.2.3 Third step: Clusters Optimization

The clustering results can be optimized: some of them can be joined together because some characteristics are similar. For example, the first two clusters of *Group 1*, can lead to only one cluster, that we name *Model 1*. The same procedure is adopted for other clusters (see Table 4.7).

Table 4.7: Obtained Models

Floor n	year	Area	setback	max Ratio	Open groundfloor				Floor n	year	Area	setback	maxRatio	Open groundfloor
2,2608696	1969,3478	110,12952	0,0434783	0,1981566	0	*Centroid 1*	Group 1	Model1	2,304348	1969,674	142,8678	0,065217	0,1992171	0
2,3478261	1970	175,60609	0,0869565	0,2002776	0	*Centroid 2*								
5,2857143	1973,5714	242,016	0	0,0594912	0	*Centroid 1*	Group 2	Model 2	4,971429	1973,81	198,8682	0	0,0540016	0
5,6	1977	324,1154	0	0,0922392	0	*Centroid 4*								
4,8333333	1972,5	137,06	0	0,064276	0	*Centroid 6*								
4,1666667	1972,1667	92,281333	0	0	0	*Centroid 7*								
5,6666667	1979,7778	202,30928	0,0555556	0,1579687	0	*Centroid 5*		Model 3	5,666667	1977,46	184,5913	0,075397	0,149302	0
5,6666667	1975,1429	166,87329	0,0952381	0,1406353	0	*Centroid 3*								
6	1972,5	399,9035	0,5	0,1932583	0	*Centroid 8*		Model 4	6	1972,5	399,9035	0,5	0,1932583	0
6	1965	715,604	0	0,5744088	0	*Centroid 2*		Model 5	5,666667	1970,833	599,8378	0,166667	0,47499	0
5,3333333	1976,6667	484,07167	0,3333333	0,3755713	0	*Centroid 9*								
7,075	1981,95	146,26125	0,05	0,1436324	0	*Centroid 1*	Group 3_0	Model 6	7,085119	1980,88	176,3497	0,04881	0,1535934	0
7,0952381	1979,8095	206,43824	0,047619	0,1635544	0	*Centroid 2*								
7,0769231	1976,9231	331,64292	0,0769231	0,2869309	0	*Centroid 3*		Model 7	7,076923	1976,923	331,6429	0,076923	0,2869309	0
7	1980	497,926	1	0,6027836	0	*Centroid 4*		Model 8	7	1980	497,926	1	0,6027836	0
7,2068966	1984,3448	163,04441	0,0689655	0,1768596	1	*Centroid 1*	Group 3_1	Model 9	7,206897	1984,345	163,0444	0,068966	0,1768596	1
						Centroid 1	Group 4							
5,8571429	1999,7143	196,75186	0	0,1700529	0,142857143	*Centroid 1*	Group 5	Model 10	5,857143	1999,714	196,7519	0	0,1700529	0,142857143
7,9375	2000,5313	160,28566	0,09375	0,1151067	0,71875	*Centroid 1*	Group 6	Model 11	8,295109	1999,679	175,8456	0,045743	0,0793774	0,781612319
8,3333333	1997,6667	401,427	0	0,1740696	0	*Centroid 2*		Model 12	8,333333	1997,667	401,427	0	0,1740696	0
8,3478261	2001,3043	116,18613	0,0434783	0,1230255	0,826086957	*Centroid 3*								
8,6	1997,2	251,065	0	0	0,8	*Centroid 4*								

4.5.2.4 Obtained Models

At the end of this manual union, the model are 12, like shown in **Errore. L'origine riferimento non è stata trovata.**

Table 4.8: Table of structural Models

Model N.	Floors	Year (new/old)	Open Groundfloor (soft 1st floor)	Irregularity Ratio	Setback Ratio
1	2	old	no	0,20	0,06
2	5	old	no	0,05	0,00
3	6	old	no	0,15	0,07
4	6	old	no	0,20	0,50
5	6	old	no	0,48	0,17
6	7	old	no	0,15	0,05
7	7	old	no	0,29	0,08
8	7	old	no	0,60	1,00
9	7	old	yes	0,18	0,07
10	6	new	no	0,17	0,00
11	8	new	no	0,08	0,04
12	8	new	no	0,17	0,00

Structural Design and Modelling of the structures

5.1 Introduction

The design of structures was carried out taking into account the Greek regulations in force at the time of construction. The models obtained by statistical procedures were standardized to those proposed in the precious study by Prof. Zeris.

After performing for each model the pushover analysis, through the use of SPO2IDA, results in terms of incremental analysis have been obtained.

5.2 Features of the Models

Taking into account the Seismic Code in force during the period of construction of each Models obtained through the statistical procedure, in the following Table (Table 5.1) shows the geometrical features.

Table 5.1: Table of structural Models according the seismic code

Model N.	Floors	Area	Year (new/old)	Seismic Design Code	Open Groundfloor (soft 1st floor)	Irregularity Ratio	Setback Ratio
1	2	142,8678	1970	RD59	no	0,20	0,06
2	5	198,8682	1974	RD59	no	0,05	0,00
3	6	184,5913	1977	RD59	no	0,15	0,07
4	6	399,9035	1972	RD59	no	0,20	0,50
5	6	599,8378	1970	RD59	no	0,48	0,17
6	7	176,3497	1980	RD59	no	0,15	0,05
7	7	331,6429	1977	RD59	no	0,29	0,08
8	7	497,926	1980	RD59	no	0,60	1,00
9	7	163,0444	1984	RD59	yes	0,18	0,07
10	6	196,7519	2000	EAK	no	0,17	0,00
11	8	175,8456	2000	EAK	yes	0,08	0,04
12	8	401,427	2000	EAK	no	0,17	0,00

In the "*Evaluation of the seismic performance of existing RC buildings: II. A case study for regular and irregular buildings*"[9], there are the descriptions of structural groups of buildings according to the generation of Greek seismic Codes. The Authors depict the buildings in the following four categories:

- ***Buildings constructed in the 60s (Group 60).*** These structures have been designed according to RD59 [1959] following allowable stress procedures and simplified structural analysis models. Allowable stresses due to combined flexural / axial loads ranged between 5 to 8 MPa for the concrete (grade C12) and 140 MPa for the (smooth) steel reinforcement (grade S220), with a 20% increase for

[9] Repapis C., Zeris C., Vintzileou E.

design under the seismic load combinations. Nominal values of dead and live loads were specified in the Greek Loadings Code [LC45, 1945] still in effect today. Structural elements possess no critical region reinforcement for confinement and no capacity design provisions were used in their design. A special check was carried out for perimeter columns and beams, while interior beams were usually designed for gravity loads only. Seismic design was based on a three-zone classification system, with the seismic base shear coefficients ε being 4%, 6% or 8% of the structural weight, for seismic zones I, II or III, respectively, for stiff soil.

Buildings of this period are characterized by dense and regular column spacing, relatively short bay sizes (3.0 to 4.0 m) and the absence of shear walls. Perimeter frames are infilled with unreinforced masonry walls 0.25 m thick, of good quality workmanship, with window and door openings usually in the same positions at each floor. Interior masonry partitions 0.10 m thick are used in the interior plan of the structure in an irregular pattern, depending on current use (or change of use); as a consequence, these are only considered as mass but not part of the lateral resisting system. Apart from openings, large window openings in the perimeter infill layout may also be encountered primarily at the ground, but also in any of the upper floors, either intentionally or when the use of the building changed from residential to commercial during its lifetime. The cross-section dimensions of columns are relatively narrow, reflecting the tendency of early designs to be economic in concrete usage since it was in situ mixed and manually conveyed and placed, and because of the relatively low level of seismic actions. As another consequence of this, concrete exhibits wide scattering in its mechanical properties.

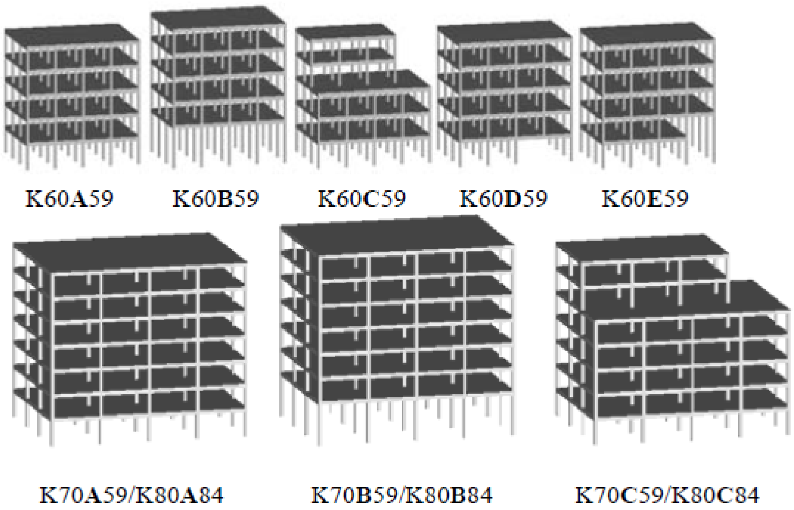
- ***Buildings constructed in the 70s (Group 70).*** These structures have also been designed according to RD59 [1959], but more elaborate structural analysis models were adopted. Concrete grade becomes C16 while S400 steel is specified, with an allowable design stress of 240 MPa. Column spacing is regular but the bay sizes are increased to 5.0 or 6.0 m. Reinforced concrete shear wall cores were introduced in the 70s at the elevator shaft (typically 0.20 m thick). Partial infill irregularity is more frequently encountered at the ground floors. As before, structural elements possess no critical region reinforcement nor were there any capacity design provisions used in their design.
- ***Buildings constructed in the 80s (Group 80).*** These structures have been designed according to the 1984 Interim Modification of RD59 [MOD84, 1984], which were introduced following the 1978 Thessaloniki and 1981 Athens earthquakes. Although the seismic base shear coefficients did not change, entire frame models (including shear walls) and triangular seismic load distribution substituted for earlier simplified models. Use of multiple closed stirrups with reduced tie spacing at the end member critical regions, edge member reinforcement in shear walls, shear reversal design in beams (through controlling the allowable shear stress) and a form of joint capacity design using service flexural resistance levels were introduced. The building geometry remains same as in the Group 70 with the spans increasing to 7.00 m; often an open first storey (pilotis) was intentionally specified in which the use of infill walls is completely avoided for commercial development or parking space. Perimeter shear walls with an elevator core were typically used and concrete member dimensions generally become wider.

- ***Buildings constructed in the 90s (Group 90).*** These structures have been designed primarily after 1995, with the adoption of the Greek Earthquake Resistant Design Code [EAK, 2000] and the Greek Code for Design of Concrete Works [EKOS, 2000]. Both are Ultimate Limit State (ULS) design codes, encompassing the majority of the currently established requirements for ductile response introduced in contemporary seismic provisions (among others, EC8 [2003]). These modern seismic codes introduce the use of inelastic design response spectra, the behaviour factor q, more stringent detailing for local ductility and confinement, capacity design, weak beam strong column behaviour and penalties for irregularity and plan torsion. Structures of this generation exhibit long spans, with or without an open first storey (with a penalty), provisions for adequate shear walls and large member dimensions.

The twelve Models obtained after statistical procedure, are in Group70, Group80 and Group90.

5.3 Buildings form and irregularity

The Models have been classified according to [2], with reference to the Fig. 5.1.



K	Bare buildin
T1 – T8	Infilled buildi
60/70/80	Period of constru (frame geomet
A	Regular buildi
B	Taller 1st stor
C	One recessed ba both directions a upper two stor
D	Discontinuous co
E	Discontinuous b at the ground st
59/84/ EAK	Earthquake resi design code
II/III	Seismic zone I

Fig. 5.1: Buildings considered and their notation

In order to examine the influence of exterior infills in the building inventory, both the bare frame structures as well as structures with fully or partially unreinforced masonry infilled perimeter frames are analysed, in all cases considered. Infill panels in all cases are 0.25 m thick, following the conventional construction of double leaf exterior walls. Single leaf interior partition walls are not considered as taking part to the lateral resisting system but are included in the mass of the building. Eight different arrangements of the perimeter infill panels are identified in [2] (Fig. 5.2):

- T1: Fully infilled perimeter frames (a)
- Infilled perimeter frames having an open ground storey (pilotis) (b)
- T3: Partially infilled perimeter frames (Fig. 2c)
- T4: Infilled perimeter frames leaving open an intermediate (3rd) storey (d)
- T5: Infilled perimeter frames leaving open the two lower storeys (e)
- T6: Infilled perimeter frames with infill panel height in the first storey equal to 67% the storey height (f)
- T7: Infilled perimeter frames with infill panel height in the first storey equal to 50% the storey height (f)

- T8: Infilled perimeter frames with infill panel height in the first storey equal to 33% the storey height (f).

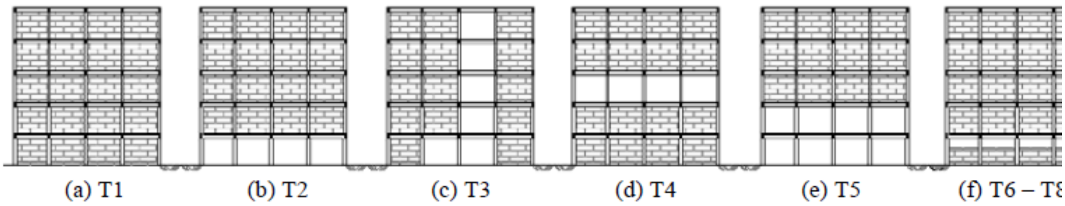


Fig. 5.2: Distribution of masonry infilled bays considered

5.4 Structural Design and Modelling of the Structures

In [2], Buildings are designed for seismicity zones I, II or III in stiff soil [RD59, 1959], with the exception of two frames in zone III. The design loads considered for all the frames are described in the detailed comparative study of K60A59 and K60AEAK in Repapis et al. [2005]. All K60 buildings make use of B160 concrete (28 days mean cube compressive strength of 16 MPa) and StI smooth (namely S220 per EC2 [2002]) longitudinal and transverse reinforcement. Detailing practices adopted at the time of construction (use or not of bent up bars in beams, improper anchorage of the bottom bars at interior joints *etc*.) are considered in modelling, as described in Repapis et al. [2005].

Group 60. These are five storeys high, with a storey height of 3.00 m and regular 3.50 m bay sizes in both directions and 12 cm thick slabs. All K60x59 buildings in seismic zone I have 350x350 [mm] columns at the base, gradually reduced to 250x250 [mm] from the third floor to the roof; exception to this are buildings K60B59, having ground column dimensions 400x400 [mm] and

K60D59, in which column dimensions each side of the discontinuity vary from 450x450 [mm] at the ground floor to 300x300 [mm] at the roof. With the exception of K60D59, column longitudinal reinforcement ranges between 1.0% and 2.5%, while transverse 8 mm diameter ties are used, spaced from 250 [mm] at the lower storeys to 400 mm at the roof. Beams are kept to dimensions 200/500 [mm] in all cases and are lightly reinforced (about 0.4 % steel ratio). The beams supporting the discontinuous column in K60D59 are 300/600 [mm] with 1.7% reinforcement ratio, also partly due to the additional design requirement of increasing the discontinuous column vertical load by ($1+2\varepsilon$) [RD59, 1959]. Minimum stirrups 8mm in diameter at 300 [mm] are used throughout, since more than 50% of the design shear is resisted by the bent up longitudinal bars. Column dimensions and the longitudinal reinforcement ratio are slightly increased for the irregular buildings in seismicity zone II, while transverse stirrup of same diameter are used, spaced from 150mm at the ground floor to 400mm at the roof. Beams remain similar in cross section and are again relatively lightly reinforced.

Groups 70 and 80. The Group 70 and 80 buildings are seven storeys high, with a storey height of 3.00 m and 6.00 m bay sizes in both directions. All K70 and K80 buildings are designed with C16 concrete, S400 (ribbed) longitudinal and S220 (smooth) transverse reinforcement. Buildings K70x59 and K80x84 in zone I have column dimensions ranging between 600x600 [mm] – interior - and 900x250 [mm] – exterior – at the ground floor, to 300x300 [mm] and 350x250 [mm] at the roof, with a longitudinal reinforcement ratio from 1% to 3%. In Group 80, a minimum amount of 8 mm diameter stirrups at 100 mm is introduced by MOD84 [1984], close to the actual stirrup requirement of Group 70 buildings in their ground floor. Buildings of Group 70 and 80 typically have an open shear wall core at the stairwell, U shaped in cross section, 2.0 m wide in each direction and 200 mm thick. In Group 70, edge member columns in this wall are reinforced with 4Φ12 longitudinal bars while the panel is reinforced with Φ8/200 bars each way and each face. Edge member columns of the wall in Group 80 are reinforced with 6Φ18 at their edge (twice as much in the corners), with the web reinforcement remaining unaltered. Perimeter beams remain 250/500 [mm] while interior beams increase to 300/600 [mm]. Furthermore, due to the increased bay size, beam longitudinal reinforcement ratios are also increased, while the slab thickness becomes 16 cm. The transverse reinforcement of the beams remains low (8 mm diameter stirrups at 300 mm), while for the Group 80 buildings, the column minimum applies (8 mm diameter stirrups at 100 mm).

Group 90. For comparison reasons, two buildings of similar geometric forms are designed according to currently enforced EAK [2000] and EKOS [2000], using, however, the same reinforcement grade as Groups 70/80: i) a five-storey frame with regular 3.50 m bay sizes (K60AEAK) and ii) a seven-storey frame with 6.00 m bay sizes (K80AEAKnw). Both are assumed to be located in the same seismicity as their existing counterparts (currently zone II), with a peak effective ground acceleration of 0.16g. For building K60A59, column dimensions range from 400x400 [mm] in the three lower storeys to 350x350 [mm] above and 200/500 [mm] beams. Building K80AEAKnw columns range from 600x600 [mm] interior and 900x250 [mm] exterior at the ground storey to 300x300 [mm] and 350x250 [mm] at the roof, respectively. Perimeter beams are 250/500 [mm], while interior beams increase to 250 or 300/600 [mm]. In this Group, no bent up bars are used and all the shear forces are resisted by transverse reinforcement alone. The transverse steel ratio is therefore considerably increased in this Group while the maximum stirrup spacing at the critical regions is now 100 mm in all members.

Always in [2], all buildings are modelled as plane frames in series using an extended version of program Drain-2DX [Prakash et al. 1993], as discussed in Repapis et al. [2005]. For the estimation of inelastic moment-curvature characteristics the average yield and ultimate tensile strength is equal to 310 MPa and 420 MPa, for S220 steels and 430 MPa and 630 MPa, for S400 steels, respectively, as obtained from experimental tests. Shear walls are modelled using line elements while infills are modelled by equivalent diagonal struts resisting compression only, with an infill strength of 2.5 MPa. For frame K60A59, a range of infill properties was investigated.

5.4.1 Results of Modelling

For the entire building inventory considered in [2], SPO analyses were performed with both uniform and triangular distributions of lateral load. Representative results for a typical group of irregular buildings analysed are shown inFig. 5.3.

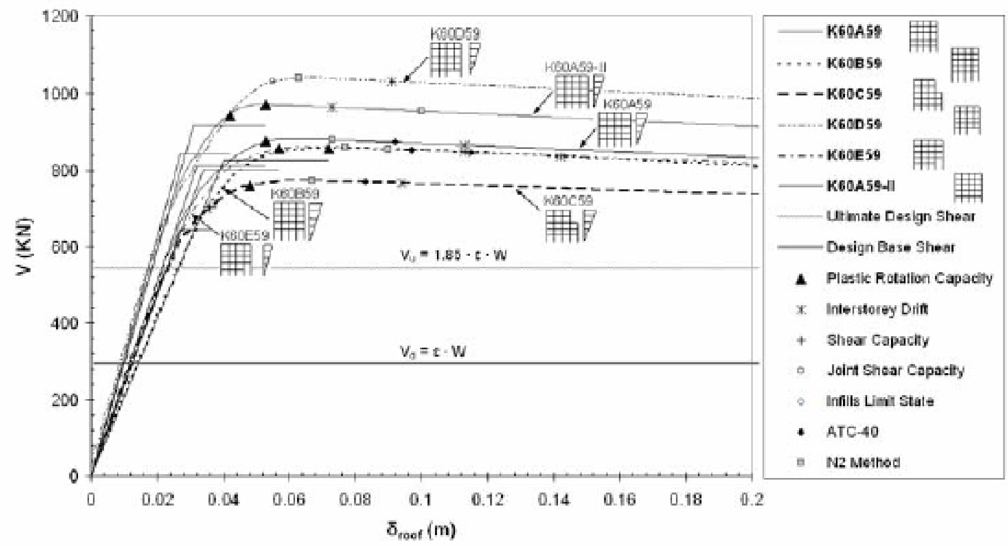


Fig. 5.3: Inelastic SPO characteristics of irregular buildings of the 60s within seismic zones I and II, in [2]

In Table 5.2 the following data are summarised for the entire building set: The first fundamental mode period of vibration of each building *T*, the estimated maximum base shear resistance *Vmax*, the overstrength *Ω*, the global ductility *μ* and the ductility of the equivalent system *μ'* (both used for the evaluation of the behaviour factor *q*, as described in Repapis et al. [2005]), the ultimate roof displacement *δu*, the ultimate displacement of the equivalent system *δu'*, the target performance displacement demands *δATC-40* and *δN2* according to the Capacity

Spectrum Method [ATC-40, 1996] and the N2 [Fajfar, 1999] methods and the LC that controlled failure. Furthermore, for the infilled frames within the group, the assumed compressive strength of the masonry infill walls *fm* is also tabulated. Out of the two distributions considered, values are given only for the triangular profile of lateral forces, since this profile represents the distribution closer to the lateral load profile under first mode response and is adopted in modern seismic provisions.

Table 5.2: Results from pushover analyses in [2]

Building	f_m [MPa]	T [sec]	V_{max} [KN]	Ω	μ	μ'	q	δ_u [m]	δ_u' [m]	δ_{ATC} [m]	δ_{N2} [m]	Limit Criter.
K60A59	–	0.84	876.6	1.61	1.63	2.13	2.03	0.053	0.068	0.092	0.073	θ_{pl}
T160A59	2.5	0.44	2159.3	3.97	1.63	4.61	2.87	0.041	0.120	0.032	0.034	θ_{pl}
T260A59	2.5	0.51	1332.1	2.45	1.76	3.29	2.38	0.030	0.056	0.050	0.039	θ_{pl}
T360A59	2.5	0.52	1616.2	2.97	1.57	3.35	2.42	0.039	0.084	0.046	0.042	θ_{pl}
T460A59	2.5	0.59	980.5	1.80	1.62	2.29	1.97	0.028	0.039	0.056	0.040	θ_{pl}
T560A59	2.5	0.65	988.3	1.82	1.79	2.62	2.23	0.034	0.050	0.081	0.049	θ_{pl}
T660A59	2.5	0.45	971.6	1.79	1.27	1.66	1.52	0.012	0.015	0.035	0.036	Shear
T760A59	2.5	0.47	1003.2	1.85	1.28	1.71	1.56	0.013	0.017	0.040	0.036	Shear
T860A59	2.5	0.49	1067.8	1.96	1.28	1.78	1.60	0.015	0.021	0.043	0.038	Shear
T160A59	1.5	0.51	1892.6	3.48	2.14	5.85	3.32	0.063	0.176	0.045	0.042	Infills
T260A59	1.5	0.56	1256.1	2.31	1.70	2.99	2.32	0.036	0.063	0.056	0.047	θ_{pl}
T360A59	1.5	0.58	1487.4	2.74	1.98	4.24	2.97	0.057	0.123	0.054	0.049	θ_{pl}
T460A59	1.5	0.63	980.3	1.80	1.55	2.17	1.92	0.032	0.043	0.061	0.047	θ_{pl}
T560A59	1.5	0.67	987.9	1.82	1.91	2.84	2.39	0.041	0.060	0.080	0.053	θ_{pl}
T160A59	0.5	0.65	1242.7	2.29	2.04	3.74	2.91	0.063	0.116	0.065	0.058	θ_{pl}
T260A59	0.5	0.67	1105.3	2.03	1.91	3.11	2.56	0.055	0.090	0.071	0.059	θ_{pl}
T360A59	0.5	0.69	1131.5	2.08	2.06	3.49	2.84	0.065	0.112	0.071	0.061	θ_{pl}
T460A59	0.5	0.72	974.2	1.79	1.51	2.09	1.92	0.041	0.056	0.073	0.060	θ_{pl}
T560A59	0.5	0.73	975.3	1.79	1.89	2.77	2.42	0.052	0.077	0.081	0.063	θ_{pl}
T760A59	0.5	0.66	1166.8	2.15	1.85	3.14	2.56	0.055	0.091	0.067	0.058	Shear
K60B59	–	0.97	858.7	1.58	1.80	2.37	2.34	0.072	0.095	0.115	0.090	θ_{pl}
T160B59	2.5	0.56	2168.1	3.99	1.80	5.31	3.37	0.062	0.186	0.047	0.050	θ_{pl}
T260B59	2.5	0.71	1017.1	1.87	2.05	3.17	2.67	0.043	0.066	0.091	0.054	θ_{pl}
T360B59	2.5	0.66	1558.9	2.87	1.83	3.99	3.07	0.060	0.132	0.061	0.057	θ_{pl}
T460B59	2.5	0.67	1033.2	1.90	1.56	2.28	2.02	0.034	0.050	0.071	0.051	θ_{pl}
T560B59	2.5	0.83	891.2	1.64	2.14	2.99	2.74	0.054	0.075	0.113	0.065	θ_{pl}
K60C59	–	0.72	705.0	1.57	1.33	1.62	1.55	0.036	0.044	0.083	0.067	Shear
T160C59	2.5	0.39	1845.2	4.11	1.45	4.03	2.66	0.036	0.101	0.038	0.027	Shear
T260C59	2.5	0.45	1049.5	2.34	1.29	2.06	1.77	0.018	0.029	0.046	0.037	Shear
T360C59	2.5	0.46	1504.1	3.35	1.52	3.57	2.48	0.040	0.096	0.039	0.038	θ_{pl}
T460C59	2.5	0.51	734.1	1.63	1.29	1.59	1.48	0.016	0.021	0.050	0.037	Shear
T560C59	2.5	0.57	930.0	2.07	1.37	2.04	1.80	0.025	0.037	0.062	0.046	Shear
K60D59	–	0.76	945.0	1.74	1.56	2.13	1.98	0.042	0.058	0.074	0.063	θ_{pl}
T160D59	2.5	0.42	2442.4	4.49	1.63	5.19	3.06	0.043	0.135	0.029	0.030	θ_{pl}
T260D59	2.5	0.47	1995.9	3.67	1.78	4.83	2.94	0.042	0.116	0.039	0.035	θ_{pl}
T360D59	2.5	0.50	1962.8	3.61	1.60	4.11	2.69	0.042	0.107	0.036	0.040	θ_{pl}
T460D59	2.5	0.55	1145.0	2.11	1.44	2.22	1.89	0.025	0.040	0.046	0.038	θ_{pl}
T560D59	2.5	0.57	1457.9	2.68	1.51	2.89	2.29	0.035	0.067	–	0.043	θ_{pl}
K60E59	–	0.87	856.9	1.62	1.69	2.24	2.14	0.057	0.076	0.097	0.077	θ_{pl}
T160E59	2.5	0.45	2092.9	3.96	1.63	4.60	2.86	0.042	0.120	0.034	0.032	θ_{pl}
T260E59	2.5	0.53	1244.9	2.36	1.79	3.26	2.40	0.03	0.055	0.054	0.040	θ_{pl}
T360E59	2.5	0.53	1593.8	3.02	1.58	3.43	2.48	0.041	0.089	0.046	0.044	θ_{pl}
T460E59	2.5	0.59	969.6	1.84	1.55	2.20	1.91	0.026	0.037	0.057	0.041	θ_{pl}
T560E59	2.5	0.68	951.3	1.8	2.08	3.12	2.59	0.042	0.062	0.088	0.052	θ_{pl}
K70A59	–	1.17	2156.8	1.15	1.19	1.23	1.23	0.055	0.057	0.149	0.128	Shear
T170A59	2.5	0.67	4402.8	2.35	1.39	2.30	2.04	0.046	0.076	0.070	0.070	θ_{pl}
T270A59	2.5	0.69	2870.4	1.53	1.24	1.46	1.41	0.026	0.031	0.075	0.070	Shear
T370A59	2.5	0.76	3634.9	1.93	1.31	1.82	1.73	0.045	0.062	0.083	0.079	θ_{pl}
T470A59	2.5	0.74	1610.2	0.86	1.23	1.21	1.20	0.014	0.013	–	0.066	Shear
T570A59	2.5	0.76	2590.4	1.38	1.25	1.40	1.37	0.027	0.031	0.089	0.078	Shear
K70B59	–	1.33	2430	1.30	1.42	1.58	1.58	0.092	0.103	0.190	0.156	θ_{pl}
T170B59	2.5	0.75	3764.8	2.01	1.32	1.89	1.78	0.044	0.064	0.085	0.083	Shear
T270B59	2.5	0.83	2800.4	1.49	1.24	1.44	1.42	0.037	0.043	0.104	0.092	θ_{pl}
T370B59	2.5	0.87	3133	1.67	1.25	1.55	1.52	0.047	0.059	0.104	0.098	θ_{pl}

Table 5.2: Results from pushover analyses in [2] (continued from previous page)

Building	f_m [MPa]	T [sec]	V [KN]	Ω	μ	μ'	q	δ_u [m]	δ_u' [m]	δ_{ATC} [m]	δ_{N2} [m]	Limit Criter.
T470B59	2.5	0.83	1607.9	0.86	1.24	1.22	1.21	0.017	0.016	0.093	0.090	Shear
T570B59	2.5	0.94	2520.1	1.34	1.22	1.34	1.33	0.040	0.044	0.127	0.100	θ_{pl}
K70C59	–	1.01	1800.0	1.10	1.19	1.21	1.21	0.043	0.044	0.135	0.116	Shear
T170C59	2.5	0.59	4630.9	2.83	1.43	2.81	2.27	0.052	0.101	0.065	0.065	θ_{pl}
T270C59	2.5	0.61	2870.0	1.76	1.25	1.60	1.51	0.027	0.034	0.069	0.066	Shear
T370C59	2.5	0.66	3837.5	2.35	1.38	2.28	2.01	0.052	0.087	0.077	0.075	θ_{pl}
T470C59	2.5	0.66	1679.3	1.03	1.25	1.26	1.24	0.015	0.015	–	0.062	Shear
T570C59	2.5	0.68	2940.0	1.79	1.26	1.64	1.56	0.034	0.044	0.081	0.074	Shear
K70A59nw	–	1.38	2436.4	1.30	1.40	1.55	1.55	0.089	0.098	0.170	0.147	θ_{pl}
T170A59nw	2.5	0.72	4109.3	2.19	1.35	2.09	1.92	0.045	0.069	0.075	0.076	θ_{pl}
T270A59nw	2.5	0.76	2870.4	1.53	1.23	1.44	1.41	0.029	0.033	0.081	0.076	Shear
T370A59nw	2.5	0.82	3348.3	1.79	1.28	1.67	1.62	0.045	0.059	0.089	0.087	θ_{pl}
T470A59nw	2.5	0.83	2710.3	1.44	1.21	1.37	1.35	0.032	0.036	0.085	0.085	Shear
T570A59nw	2.5	0.90	2800.4	1.49	1.23	1.42	1.41	0.035	0.040	0.098	0.087	θ_{pl}
K80A84	–	1.17	2252.6	1.20	1.11	1.15	1.15	0.050	0.052	0.107	0.122	Shear
T180A84	2.5	0.67	3797.1	2.02	1.25	1.77	1.65	0.033	0.048	0.057	0.069	Shear
T280A84	2.5	0.69	2450.4	1.31	1.26	1.38	1.34	0.019	0.021	0.060	0.070	Shear
T380A84	2.5	0.76	3415.5	1.81	1.24	1.62	1.56	0.037	0.049	0.068	0.079	Shear
T480A84	2.5	0.74	1540.2	0.82	1.19	1.18	1.17	0.013	0.013	0.063	0.075	Shear
T580A84	2.5	0.76	2380.3	1.27	1.21	1.30	1.28	0.022	0.023	0.066	0.076	Shear
K80B84	–	1.33	2480.0	1.32	1.24	1.35	1.35	0.070	0.077	0.127	0.146	Shear
T180B84	2.5	0.75	4516.0	2.40	1.42	2.42	2.19	0.055	0.094	0.069	0.083	Shear
T280B84	2.5	0.83	4116.6	2.19	1.48	2.37	2.23	0.061	0.097	0.077	0.089	Shear
T380B84	2.5	0.87	3963.6	2.11	1.42	2.18	2.09	0.062	0.096	0.083	0.096	Shear
T480B84	2.5	0.83	1599.8	0.86	1.20	1.18	1.18	0.016	0.015	0.075	0.088	Shear
T580B84	2.5	0.94	2590.4	1.38	1.29	1.45	1.44	0.036	0.041	0.084	0.095	Shear
K80C84	–	1.01	2250.0	1.38	1.13	1.23	1.23	0.048	0.053	0.097	0.110	Shear
T180C84	2.5	0.59	3829.2	2.34	1.23	1.92	1.73	0.035	0.054	0.054	0.066	Shear
T280C84	2.5	0.61	2480.3	1.52	1.27	1.50	1.43	0.020	0.024	0.056	0.066	Shear
T380C84	2.5	0.66	3355.9	2.05	1.23	1.74	1.63	0.037	0.054	0.063	0.075	Shear
T480C84	2.5	0.66	1610.0	0.98	1.21	1.21	1.19	0.014	0.014	0.059	0.070	Shear
T580C84	2.5	0.68	2450.0	1.50	1.23	1.43	1.39	0.024	0.027	0.062	0.072	Shear
K80A84nw†	–	1.38	3265.9	1.73	1.61	2.21	2.21	0.140	0.192	0.128	0.146	θ_{pl}
K80A84nw	–	1.38	2927.2	1.56	1.27	1.52	1.52	0.098	0.117	0.128	0.146	joint
T180A84nw	2.5	0.72	5443.9	2.90	1.40	2.78	2.42	0.069	0.136	0.064	0.077	θ_{pl}
T280A84nw	2.5	0.76	4549.3	2.42	1.38	2.34	2.13	0.057	0.096	0.068	0.078	θ_{pl}
T380A84nw	2.5	0.82	4600.0	2.45	1.35	2.28	2.14	0.070	0.119	0.077	0.088	θ_{pl}
T480A84nw	2.5	0.83	3699.9	1.97	1.29	1.81	1.74	0.048	0.067	0.072	0.085	θ_{pl}
T580A84nw	2.5	0.90	3967.3	2.12	1.37	2.08	2.02	0.060	0.089	0.076	0.088	θ_{pl}
K60A59-II	–	0.76	973.5	1.19	1.72	1.87	1.77	0.053	0.058	0.283	0.100	θ_{pl}
K60D59-II	–	0.74	1120	1.37	1.59	1.86	1.75	0.049	0.058	0.128	0.097	θ_{pl}
K60E59-II	–	0.79	963.1	1.22	1.68	1.85	1.76	0.055	0.061	0.296	0.106	θ_{pl}
K70A59nw-III†	–	1.31	3595.5	0.95	1.28	1.28	1.28	0.112	0.112	0.407	0.272	θ_{pl}
K80A84nw-III†	–	1.27	5299.3	1.40	1.34	1.53	1.53	0.164	0.187	0.233	0.264	θ_{pl}
K70A59nw-III	–	1.31	3453.1	0.92	1.21	1.21	1.21	0.102	0.102	0.407	0.272	joint
K80A84nw-III	–	1.27	4121.7	1.10	1.1	1.11	1.11	0.106	0.107	0.233	0.264	joint
K60AEAK	–	0.63	1607.7	1.22	7.63	9.11	5.44	0.272	0.328	0.041	0.052	Drift
T160AEAK	2.5	0.4	3283.7	2.41	2.14	4.16	2.71	0.067	0.129	0.020	0.026	Infill
T260AEAK	2.5	0.45	2340.1	1.72	2.61	3.92	2.62	0.065	0.098	0.023	0.031	θ_{pl}
T360AEAK	2.5	0.45	2617.3	1.92	2.1	3.33	2.38	0.064	0.103	0.026	0.032	Infill
T460AEAK	2.5	0.45	2781.3	2.05	3.05	5.47	3.15	0.101	0.181	0.024	0.029	Infill
T560AEAK	2.5	0.53	1730.7	1.27	5.84	7.18	3.87	0.141	0.172	0.029	0.038	θ_{pl}
K80AEAKnw	–	1.28	3873.5	1.56	2.73	3.80	3.80	0.25	0.350	0.105	0.130	θ_{pl}
T180AEAKnw	2.5	0.7	6589.7	1.78	1.33	1.76	1.66	0.081	0.107	0.056	0.074	Infill

Table 5.2: Results from pushover analyses in 2 (continued from previous page)

Building	f_m [MPa]	T [sec]	V [KN]	Ω	μ	μ'	q	δ_u [m]	δ_u' [m]	δ_{ATC} [m]	δ_{N2} [m]	Limit Criter.
T280AEAKnw	2.5	0.73	5889.5	1.64	1.44	1.85	1.74	0.079	0.102	0.059	0.076	Infill
T380AEAKnw	2.5	0.79	5762.1	1.69	1.31	1.66	1.61	0.085	0.108	0.067	0.084	Infill
T480AEAKnw	2.5	0.79	5880.1	1.73	1.31	1.69	1.63	0.083	0.106	0.064	0.081	Infill
T580AEAKnw	2.5	0.86	5111.3	1.58	1.57	2.01	1.93	0.086	0.110	0.070	0.084	Infill

5.5 Adaptation of the Models

The twelve Models obtained after the statistical procedures have been adapted to the results of Modeling of [2], in order to obtain the SPO curves for each Model that represent the whole Zografou Area.

Operationally, the steps to obtain the Adapted Model are the following:

1. From the number of floors, heigh is obtained;
2. Through the formula $T_1 = 0.075H^{0.75}$, *T_1* is obtained;
3. Number of floors and Area permit to obtain the Weight;
4. Year of construction, presence of Opengroundfloor, presence of Setback, and Irregularity, establish the reference structure of [2]

Design period according to NEAK / EAK (post 1994) for rectangular plan buildings made of reinforced concrete

$$T = 0.09 \cdot \frac{H}{\sqrt{L}} \cdot \sqrt{\frac{H}{H + \rho \cdot L}}$$

H = height of building

L = Length of building in the direction of interest (x or y)

ρ = sum of the area of shear walls divided by the sum of the area of shear walls and columns together.

In Fig. 5.4 is shown the procedure to calculate the ***seismic zone coefficient ξ*** (in Greek Code $\Phi_d(T)$ is equivalent to ξ):

ΣΕΙΣΜΙΚΕΣ ΔΡΑΣΕΙΣ ΣΧΕΔΙΑΣΜΟΥ

2

Περιοχή Περιόδων **Εξίσωση**

$0 \leq T < T_1$: $\Phi_d(T) = \gamma_I \cdot A \cdot \left[1 + \frac{T}{T_1}\left(\frac{\eta \cdot \theta \cdot \beta_0}{q} - 1\right)\right]$ .... (2.1.α)

$T_1 \leq T \leq T_2$: $\Phi_d(T) = \gamma_I \cdot A \cdot \frac{\eta \cdot \theta \cdot \beta_0}{q}$ .... (2.1.β)

$T_2 < T$: $\Phi_d(T) = \gamma_I \cdot A \cdot \frac{\eta \cdot \theta \cdot \beta_0}{q} \cdot \left(\frac{T_2}{T}\right)^{2/3}$ .... (2.1.γ)

όπου:

$A = \alpha \cdot g$ μέγιστη οριζόντια σεισμική επιτάχυνση του εδάφους *(παρ. 2.3.3)*,

g επιτάχυνση της βαρύτητας,

γ_I συντελεστής σπουδαιότητας του κτιρίου *(παρ. 2.3.4)*,

q συντελεστής συμπεριφοράς της κατασκευής *(παρ. 2.3.5)*,

η διορθωτικός συντελεστής για ποσοστό απόσβεσης $\neq 5\%$,

θ συντελεστής επιρροής της θεμελίωσης *(παρ. 2.3.7)*,

T_1 και T_2 χαρακτηριστικές περίοδοι του φάσματος *(Πίνακας 2.4)*,

$\beta_0 = 2.5$ συντελεστής φασματικής ενίσχυσης και

Α, Β, Γ, Δ κατηγορία εδάφους *(παρ. 2.3.6)*.

Fig. 5.4: Extract from the Greek legislation – 05EAK

In this study:

soil B T1 = 0,15

T2 = 0,6

γ1 = 1

Zografou A = 0,16g 1,568

η = 5%

θ = 1

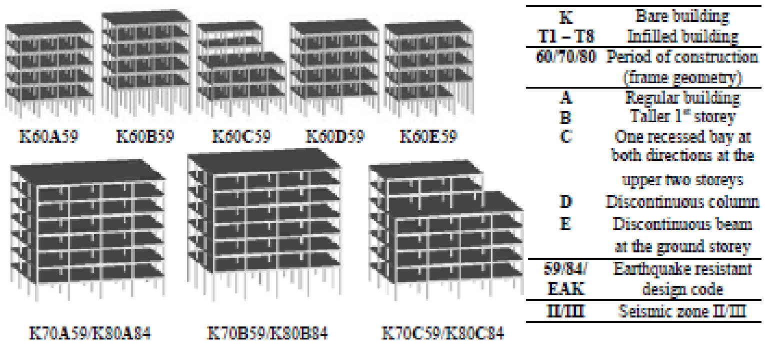
β0 = 2,5

q = 3,5

5.5.1 Adapted Model 1

Model 1:	buildings before 1995 Numbers of floor: 1-3	46 buildings (15% on Tot)	2 Clusters of similar characteristics 1 model

Year	70
Floor N.	2
Area	143
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 1:	**K60A59**
	T160A59

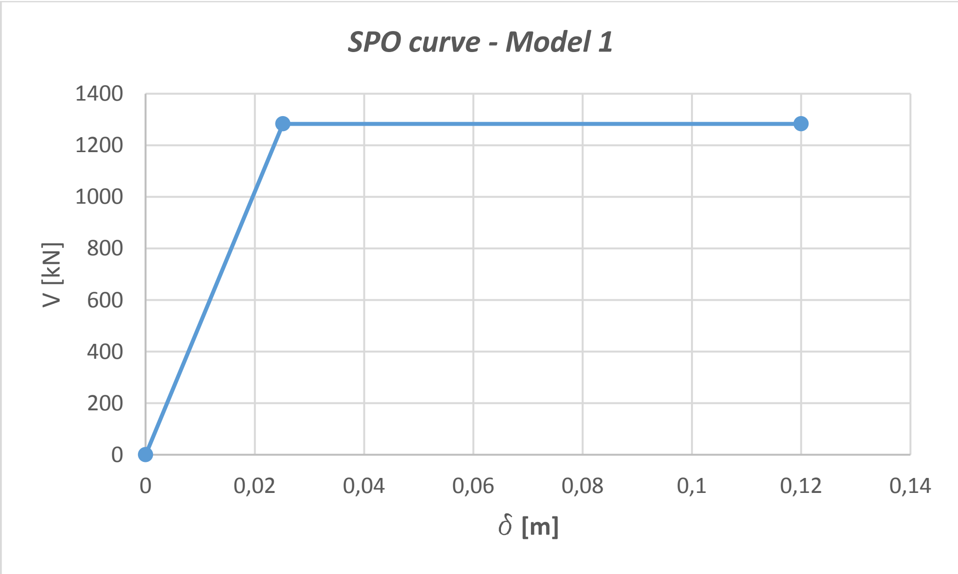


Htot		6	*m*
T1	(0,075H^0,75)	0,287524	*sec*
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,845238	
W		4377,6	*kN*

*T1 means fully infilled perimeter frames

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T160A59	2,5	0,44	2159,3	**3,97**	1,63	4,61	2,87	**0,041**	**0,12**	0,032	0,034	θpl

Γ	C_0 in FEMA 356	1,2
δyMDOF	(δuMDOF/μMDOF)	0,025153
Vmax	(ΩξβW)	**1282,741**
Cy	(Vmax/W)	0,293024
δyESDOF	δyMDOF/Γ	**0,020961**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,536541



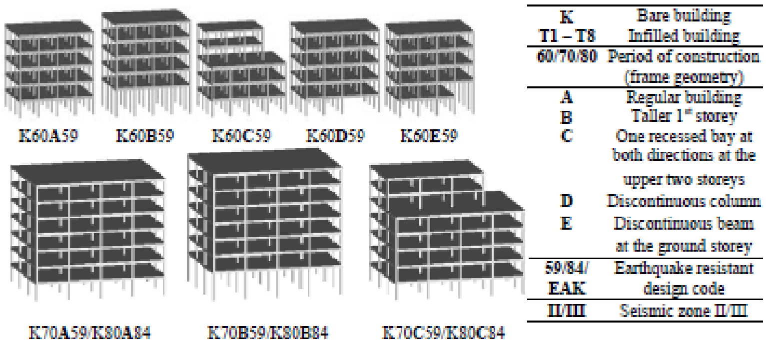
5.5.2 Adapted Model 2

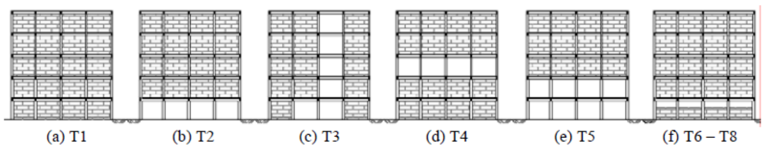
Model 2:	buildings before 1995 Numbers of floor: 4-6	84 buildings (27% on Tot)	9 Clusters of similar characteristics 4 models

Model 2 **4 clusters**	Model3 2 clusters	Model 4 1 cluster	Model5 2 cluster

32 buildings
10,49%

Year	1974
Floor N.	5
Area	199
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 2:	**K70A59**
	T170A59

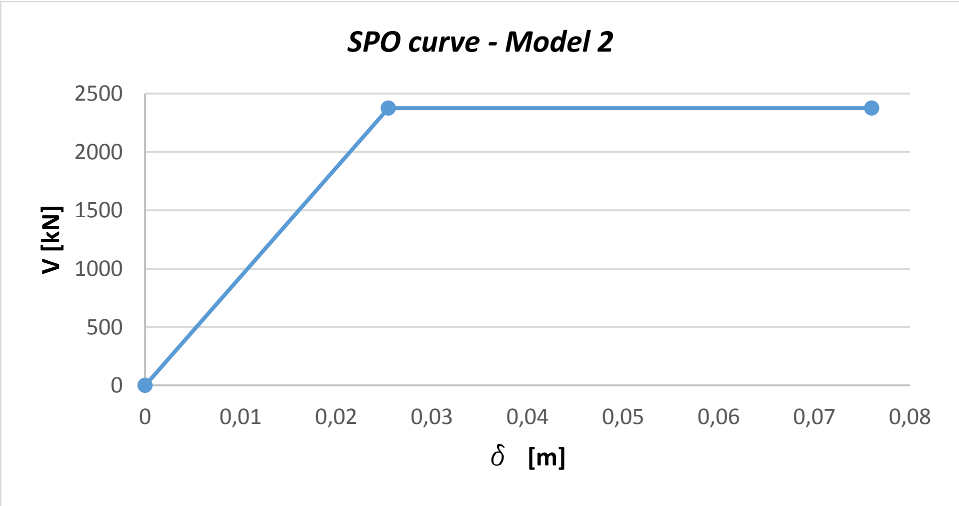




Htot		15	m
T1	(0,075H^0,75)	0,571649	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		15514,88	kN

Buildings	fm [Mpa]	T [sec]	Vmax **[kN]**	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC **[m]**	δN2 **[m]**	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**2375,421**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994



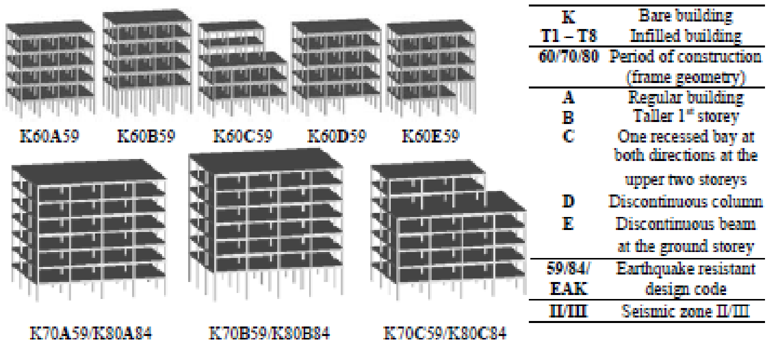
5.5.3 Adapted Model 3

Model 3:	buildings before 1995 Numbers of floor: 4-6	84 buildings (27% on Tot)	9 Clusters of similar characteristics 4 models

Model 2	**Model3**	Model 4	Model5
4 clusters	2 clusters	1 cluster	2 cluster

27 buildings
8,85%

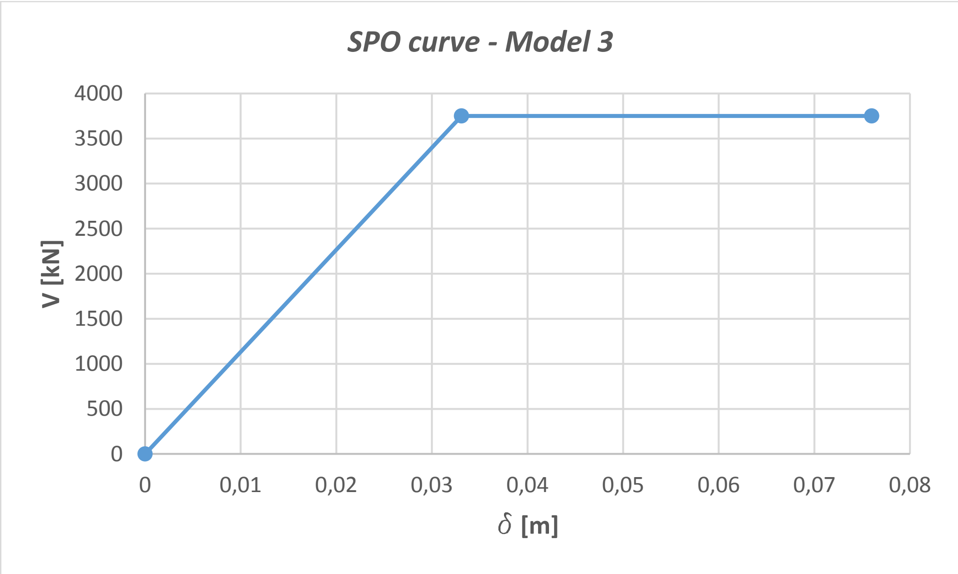
Year	1977
Floor N.	6
Area	185
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 3:	**K70A59**
	T170A59



Htot		18	m
T1	(0,075H^0,75)	0,655414	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		24495,75	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**3750,448**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994

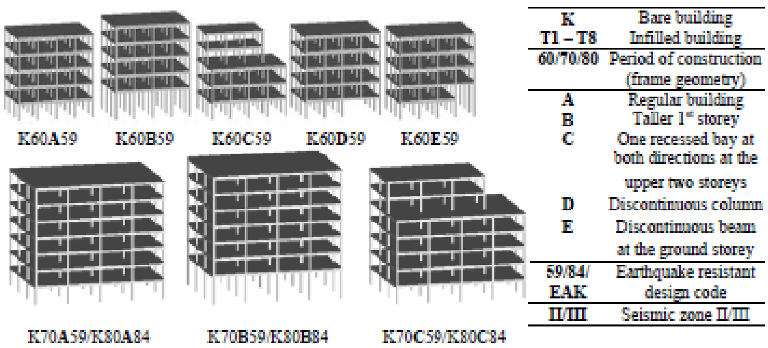


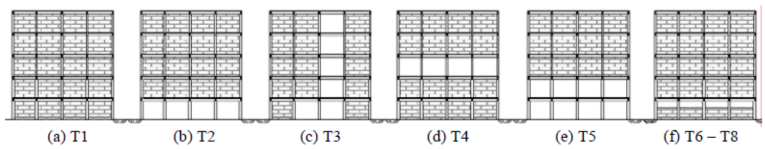
5.5.4 Adapted Model 4

Model 4:	buildings before 1995 Numbers of floor: 4-6	84 buildings (27% on Tot)	9 Clusters of similar characteristic 4 models

Model 2	Model3	**Model 4**	Model5
4 clusters	2 clusters	1 cluster	2 cluster
		6 buildings 2%	

Year	1972
Floor N.	6
Area	400
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 4:	**K70A59**
	T170A59

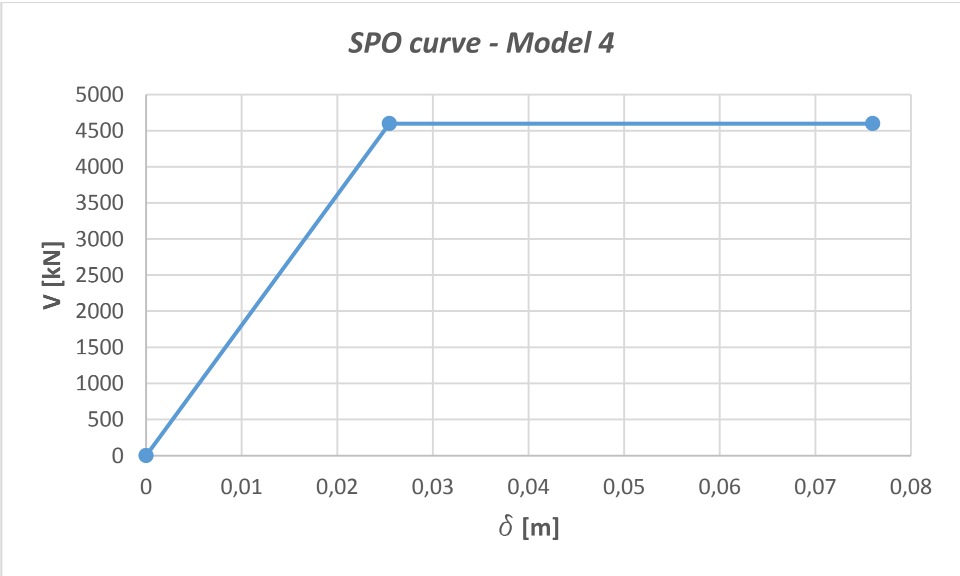




Htot		18	m
T1	(0,075H^0,75)	0,655414	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		30036,15	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**4598,717**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994

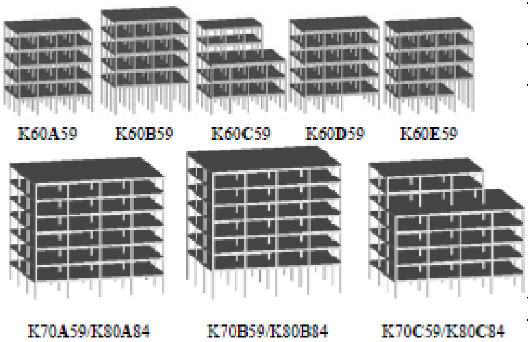


5.5.5 Adapted Model 5

Model 5:	buildings before 1995 Numbers of floor: 4-6	84 buildings (27% on Tot)	9 Clusters of similar characteristic 4 models

Model 2	Model3	Model 4	**Model5**
4 clusters	2 clusters	1 cluster	2 cluster
			19 buildings
			6,20%

Year	70
Floor N.	6
Area	600
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 5:	**K70..59**
	T170A59

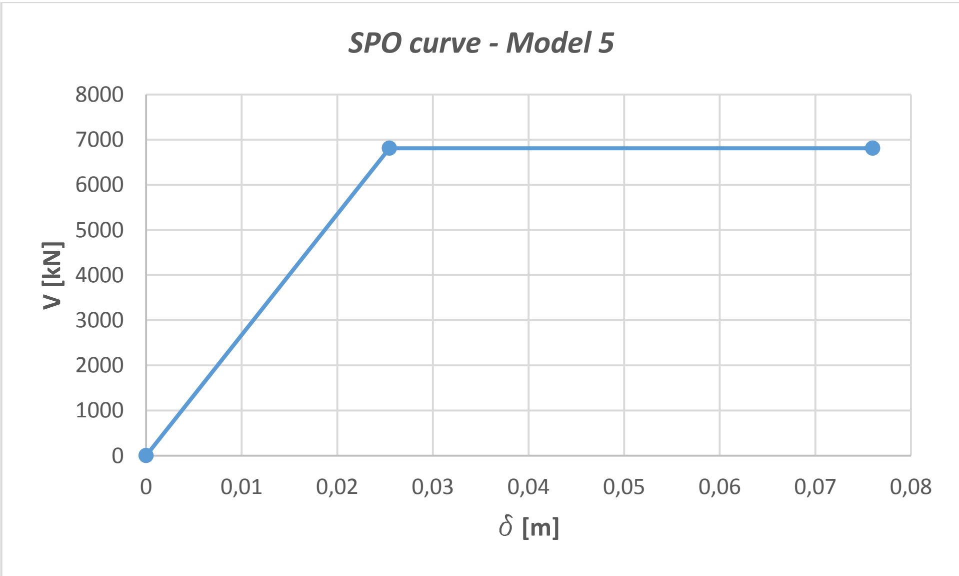


K	Bare building
T1 – T8	Infilled building
60/70/80	Period of construction (frame geometry)
A	Regular building
B	Taller 1st storey
C	One recessed bay at both directions at the upper two storeys
D	Discontinuous column
E	Discontinuous beam at the ground storey
59/84/ EAK	Earthquake resistant design code
II/III	Seismic zone II/III

Htot		18	m
T1	(0,075H^0,75)	0,655414	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		44485,65	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**6811,023**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994



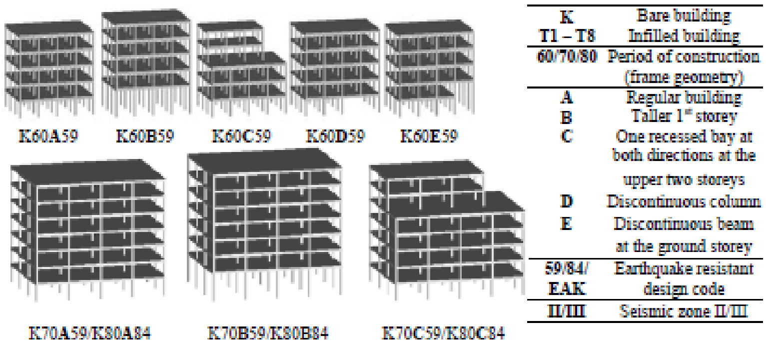
5.5.6 Adapted Model 6

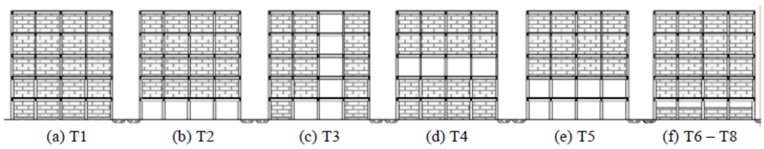
Model 6:	buildings before 1995 Numbers of floor: 7-10	75 buildings (25% on Tot)	4 Clusters of similar characteristics 3 models

Model 6 2 clusters	Model7 1 cluster	Model 8 1 cluster

14 buildings
4,60%

Year	1980
Floor N.	7
Area	176
OGF	No
Setback	No
Regularit y	A
Code	RD59
Model 6:	**K70A59**
	T170A59

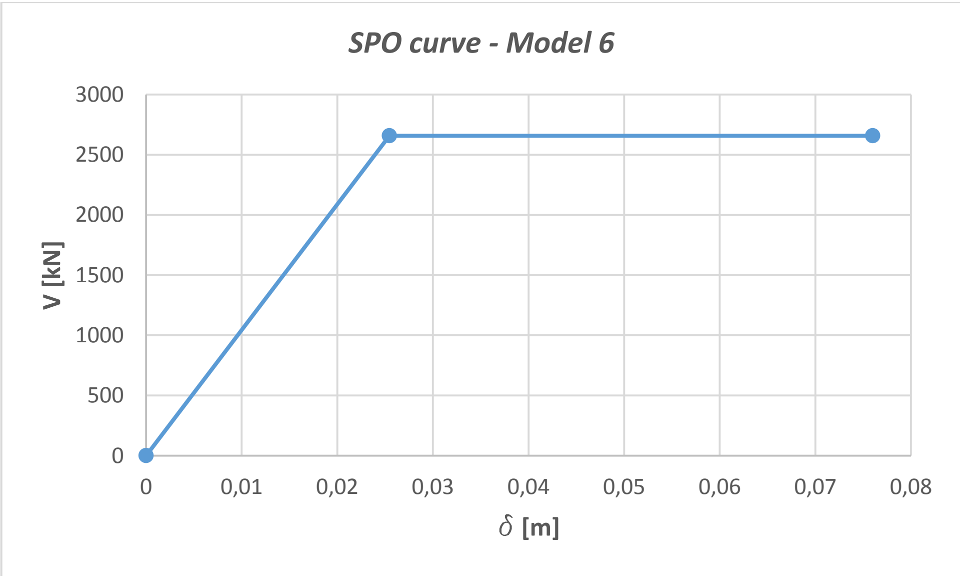




Htot		21	m
T1	(0,075H^0,75)	0,735742	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		17359,65	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**2657,868**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994



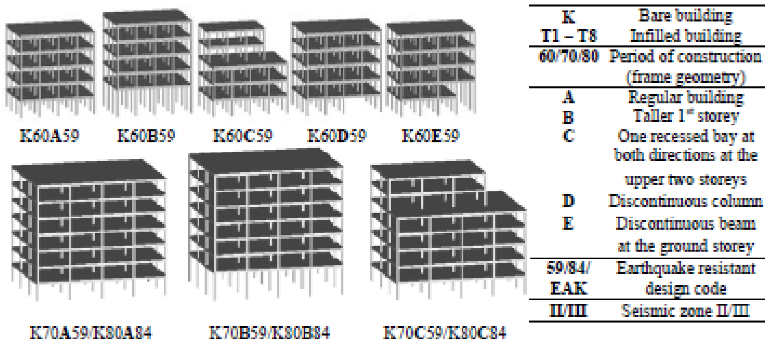
5.5.7 Adapted Model 7

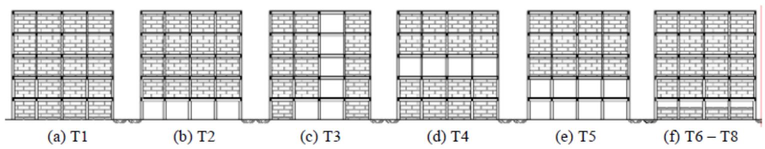
Model 7:	buildings before 1995 Numbers of floor: 7-10	75 buildings (25% on Tot)	4 Clusters of similar characteristics 3 models

Model 6	**Model7**	Model 8
2 clusters	1 cluster	1 cluster

21 buildings
6,90%

Year	1976
Floor N.	7
Area	332
OGF	No
Setback	No
Regularity	A
Code	RD59
Model 7:	**K70..59**
	T170A59

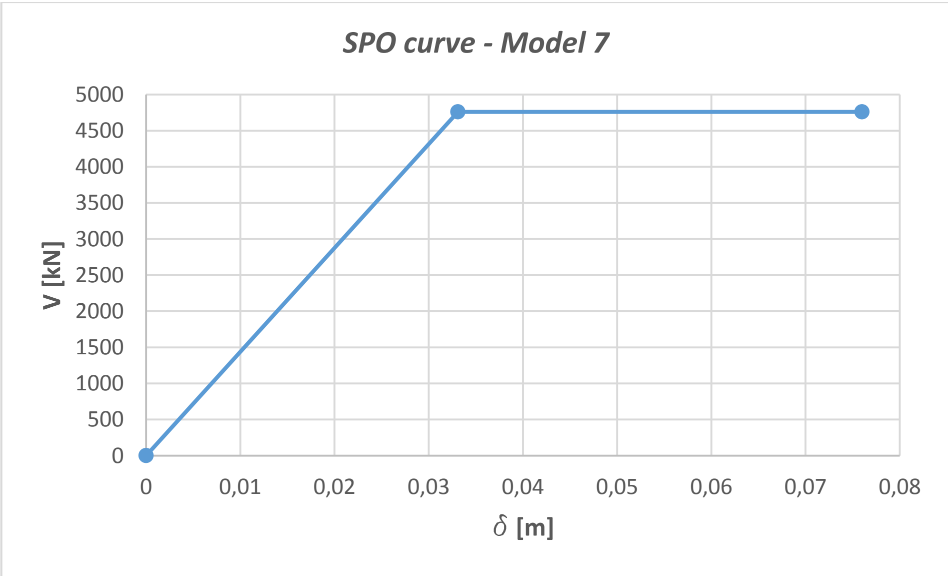




Htot		21	m
T1	(0,075H^0,75)	0,735742	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		31090,5	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170A59	2,5	0,67	4402,8	**2,35**	1,39	2,3	2,04	**0,046**	**0,076**	0,07	0,07	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,033094
Vmax	(ΩξβW)	**4760,144**
Cy	(Vmax/W)	0,153106
δyESDOF	δyMDOF/Γ	**0,025457**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,817994

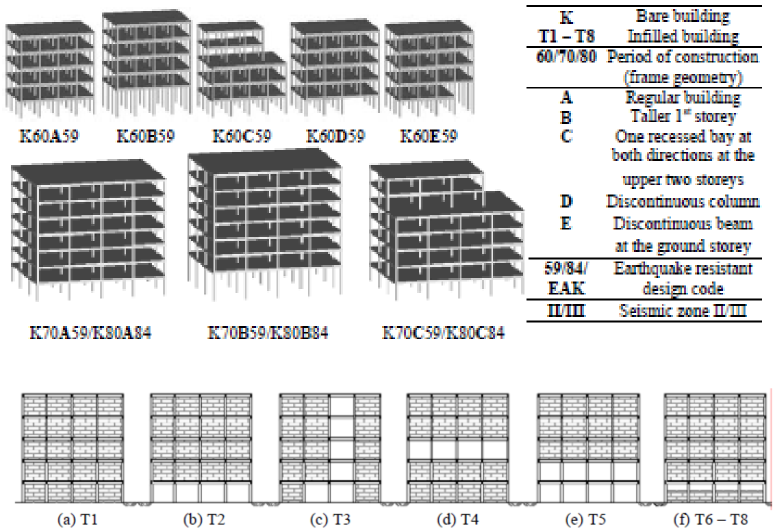


5.5.8 Adapted Model 8

Model 8:	buildings before 1995 Numbers of floor: 7-10	75 buildings (25% on Tot)	4 Clusters of similar characteristic 3 models

Model 6 2 clusters	Model7 1 clusters	**Model 8** 1 cluster
		40 buildings 13,11%

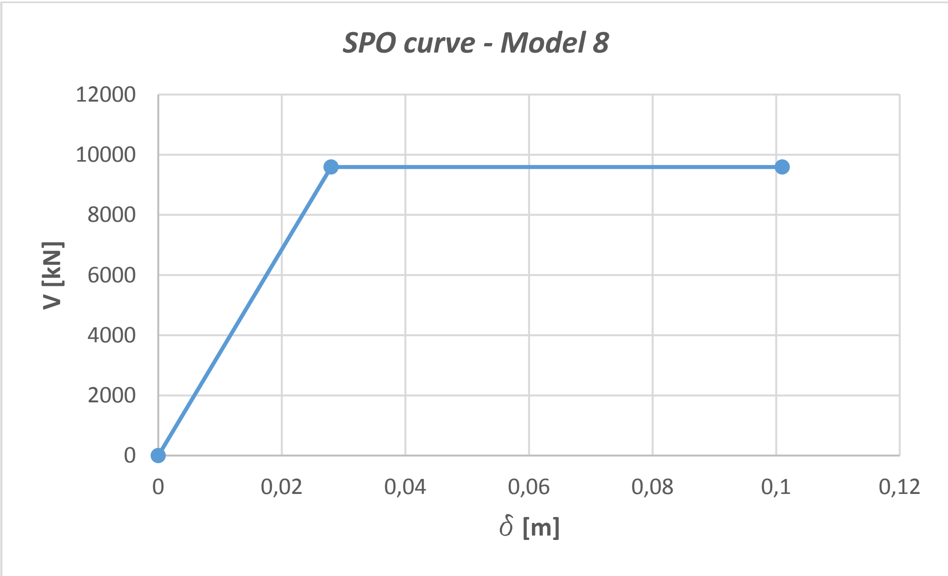
Year	1980
Floor N.	7
Area	498
OGF	No
Setback	Yes
Regularity	C
Code	RD59
Model 8:	**K70C..59**
	T170C59



Htot		21	m
T1	(0,075H^0,75)	0,735742	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		52018,75	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	µ	µ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T170C59	2,5	0,59	4630,9	**2,83**	1,43	2,81	2,27	**0,052**	**0,101**	0,065	0,065	θpl

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,036364
Vmax	(ΩξβW)	**9591,154**
Cy	(Vmax/W)	0,184379
δyESDOF	δyMDOF/Γ	**0,027972**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,781363

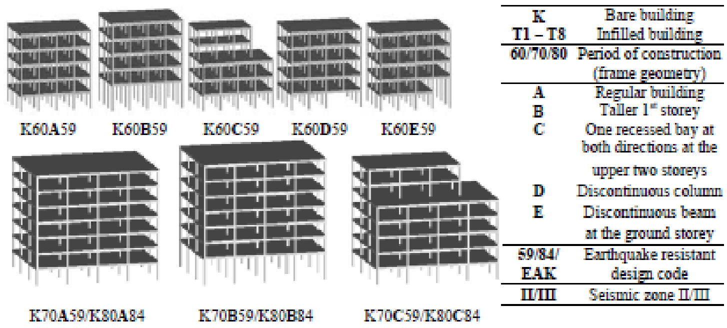


5.5.9 Adapted Model 9

Model 9:	buildings before 1995 Numbers of floor: 7-10	29 buildings (9% on Tot)	1 Cluster 1 model

1 building
0,33%

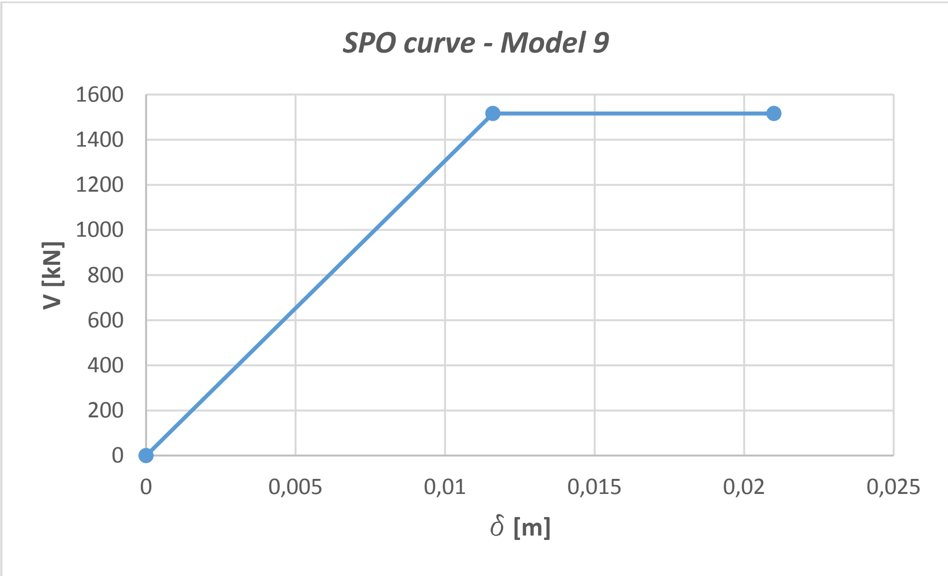
Year	1984
Floor N.	7
Area	163
OGF	Yes
Setback	No
Regularity	A
Code	RD59 MOD 84
Model 9:	**T280A84**
	T280A84



Htot		21	m
T1	(0,075H^0,75)	0,735742	sec
ξ	seismic zone coeff.	4%	
β	(fu/fe)	1,628788	
W		17762,33	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T280A84	2,5	0,69	2450,4	**1,31**	1,26	1,38	1,34	**0,019**	**0,021**	0,06	0,07	Shear

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,015079
Vmax	(ΩξβW)	**1515,988**
Cy	(Vmax/W)	0,085348
δyESDOF	δyMDOF/Γ	**0,0116**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	0,739552

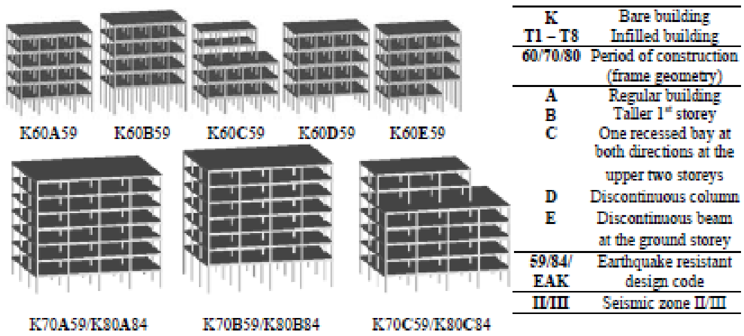


5.5.10 Adapted Model 10

Model 10:	buildings after 1995 Numbers of floor: 4-6	7 buildings (2,3% on Tot)	1 Cluster 1 model

7 buildings 2,29%

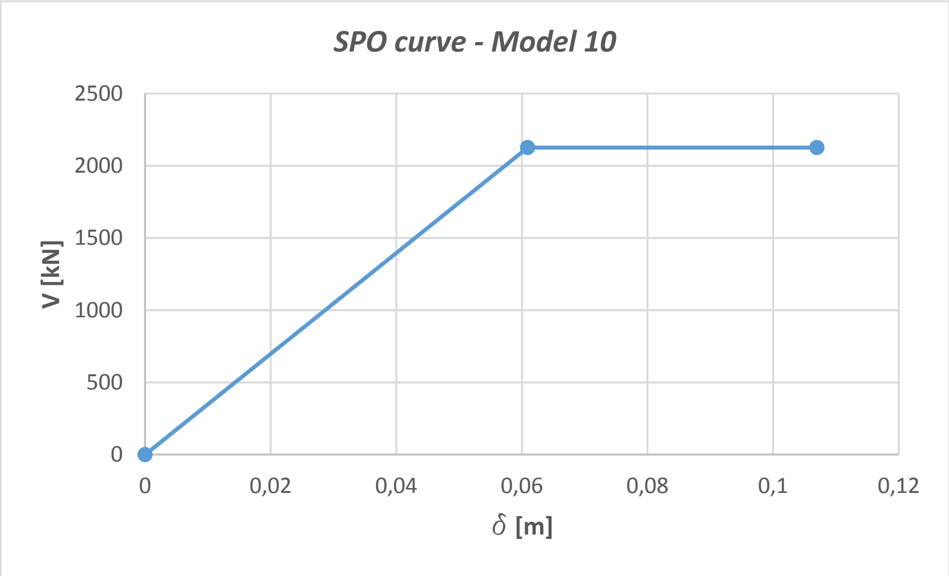
Year	2000
Floor N.	6
Area	197
OGF	No
Setback	No
Regularity	A
Code	EAK
Model 10:	**K..AEAK**
	T180AEAK



Htot		18	m
T1	(0,075H^0,75)	0,655414	sec
ξ	seismic zone coeff.	5%	
β	(fu/fe)	1,23625	
W		17251,65	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T180AEAK	2,5	0,7	6589,7	**1,78**	1,33	1,76	1,66	**0,081**	**0,107**	0,056	0,074	Infill

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,060902
Vmax	(ΩξβW)	**2125,91**
Cy	(Vmax/W)	0,123229
δyESDOF	δyMDOF/Γ	**0,046848**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	1,2369



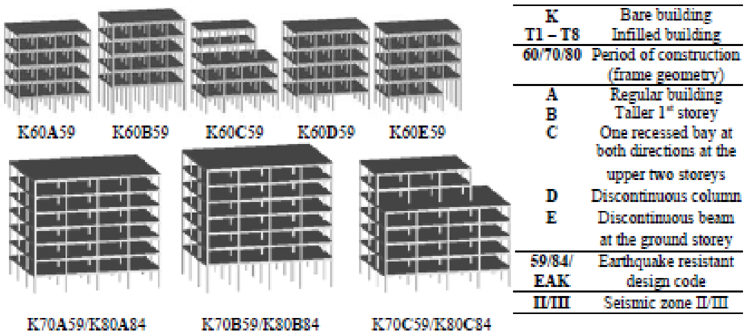
5.5.11 Adapted Model 11

Model 11:	buildings after 1995 Numbers of floor: 7-10	63 buildings (21% on Tot)	4 Clusters 2 models

Model 11 3 clusters	Model 12 1 cluster

60 buildings
19,67%

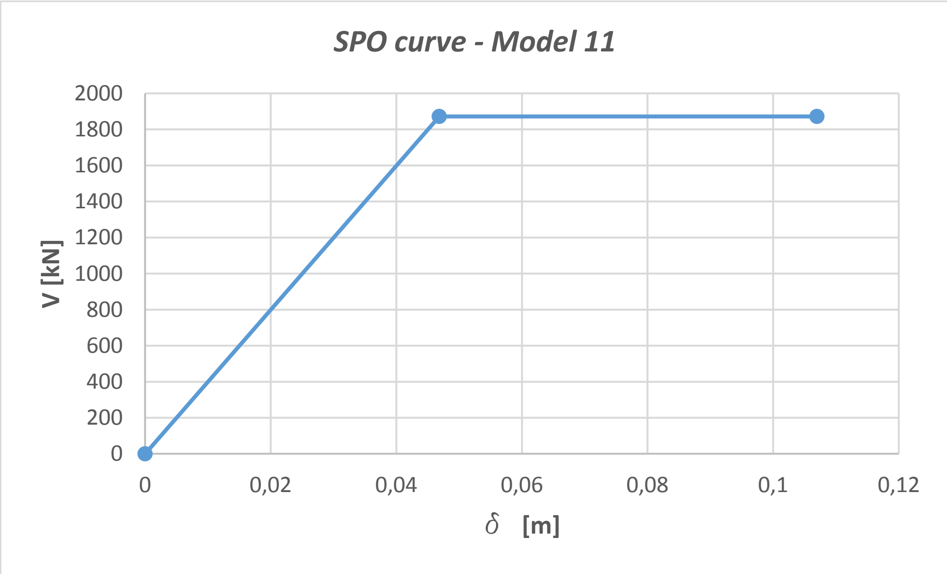
Year	2000
Floor N.	8
Area	196
OGF	Yes
Setback	No
Regularity	A
Code	EAK
Model 11:	**K..AEAK**
	T180AEAK



Htot		18	m
T1	(0,075H^0,75)	0,655414	sec
ξ	seismic zone coeff.	5%	
β	(fu/fe)	1,23625	
W		16108,2	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	μ	μ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T180AEAK	2,5	0,7	6589,7	**1,78**	1,33	1,76	1,66	**0,081**	**0,107**	0,056	0,074	Infill

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,060902
Vmax	(ΩξβW)	**1871,48**
Cy	(Vmax/W)	0,116182
δyESDOF	δyMDOF/Γ	**0,046848**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	1,273863

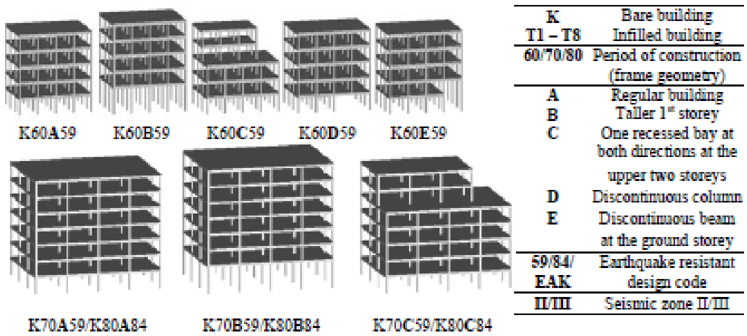


5.5.12 Adapted Model 12

Model 12:	buildings after 1995 Numbers of floor: 7-10	63 buildings (21% on Tot)	4 Clusters

Model 11 3 clusters	**Model 12** 1 cluster
	3 buildings
	0,98%

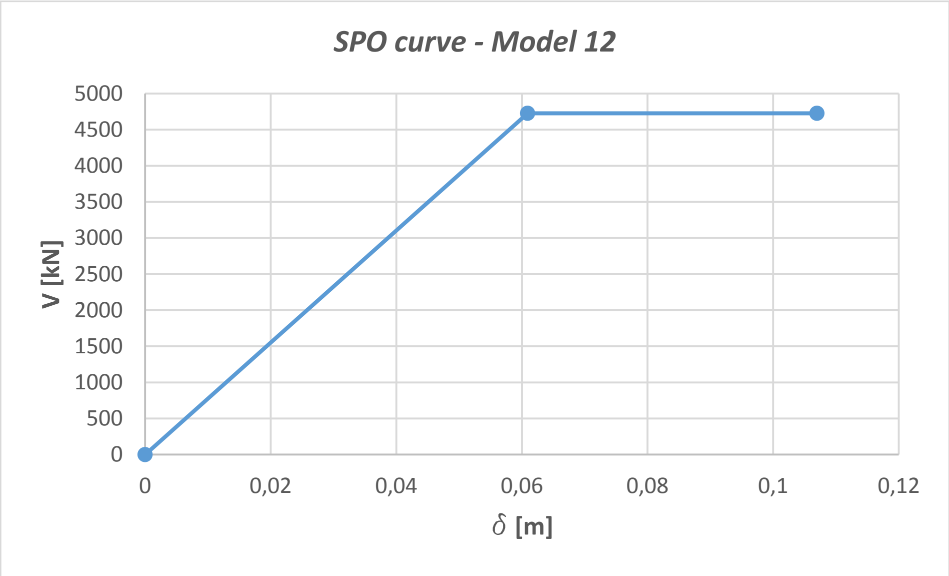
Year	1998
Floor N.	8
Area	401
OGF	No
Setback	No
Regularity	A
Code	EAK
Model 12:	**K..AEAK**
	T180AEAK



Htot		24	m
T1	(0,075H^0,75)	0,813242	sec
ξ	seismic zone coeff.	5%	
β	(fu/fe)	1,23625	
W		46966	kN

Buildings	fm [Mpa]	T [sec]	Vmax [kN]	**Ω**	µ	µ'	q	**δu [m]**	**δu' [m]**	δuATC [m]	δN2 [m]	Limit Criter.
T180AEAK	2,5	0,7	6589,7	**1,78**	1,33	1,76	1,66	**0,081**	**0,107**	0,056	0,074	Infill

Γ	C_0 in FEMA 356	1,3
δyMDOF	(δuMDOF/μMDOF)	0,060902
Vmax	(ΩξβW)	**4725,549**
Cy	(Vmax/W)	0,100616
δyESDOF	δyMDOF/Γ	**0,046848**
T (actual)	(2π(δESDOF/(Cy g))^0,5)	1,368854

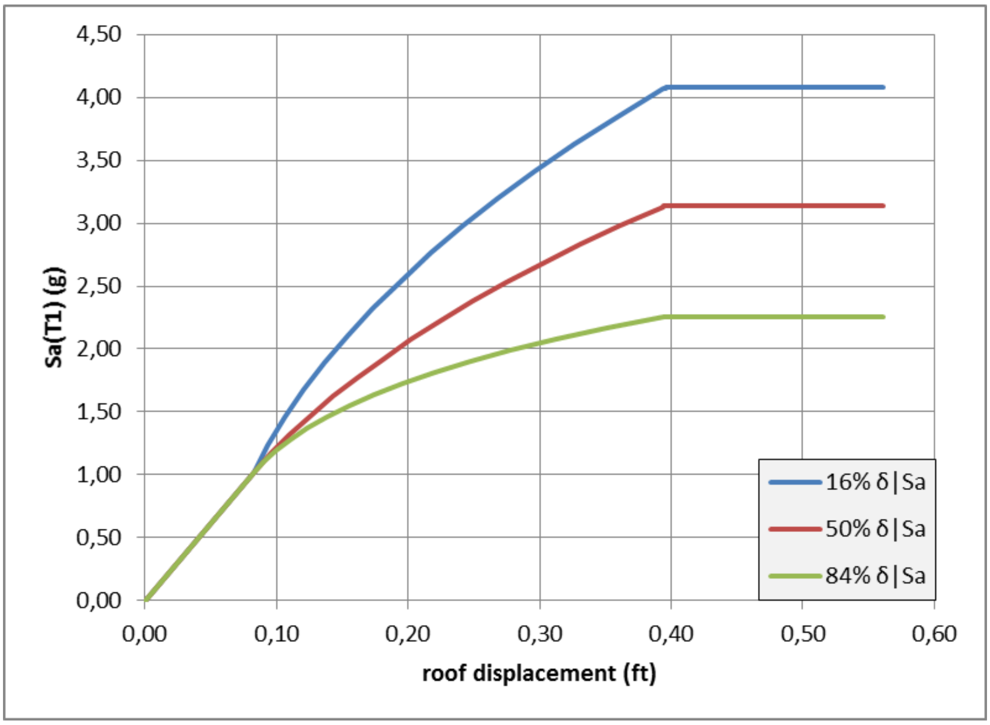


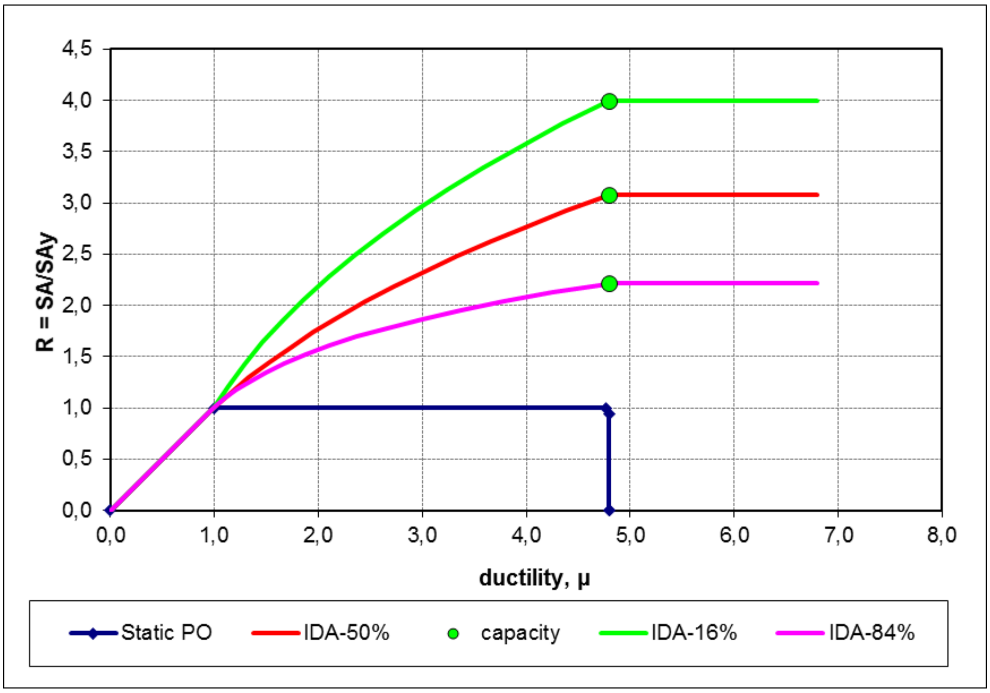
Results

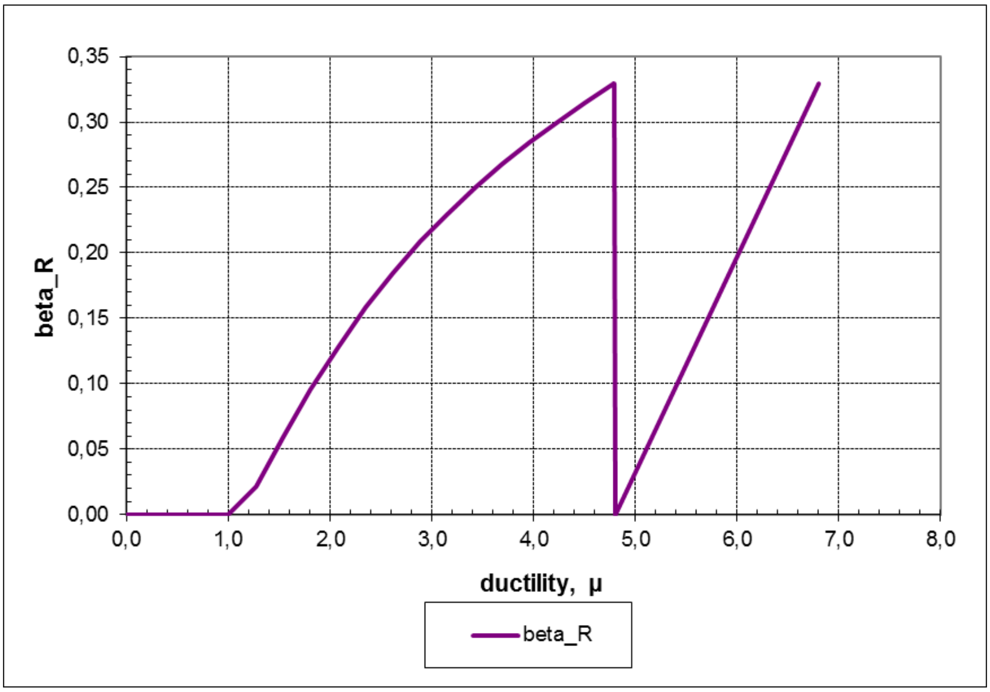
6.1 Introduction

Through the SPO2IDA tool, IDA informations have been obtained from SPO curves presented in the previous chapter.

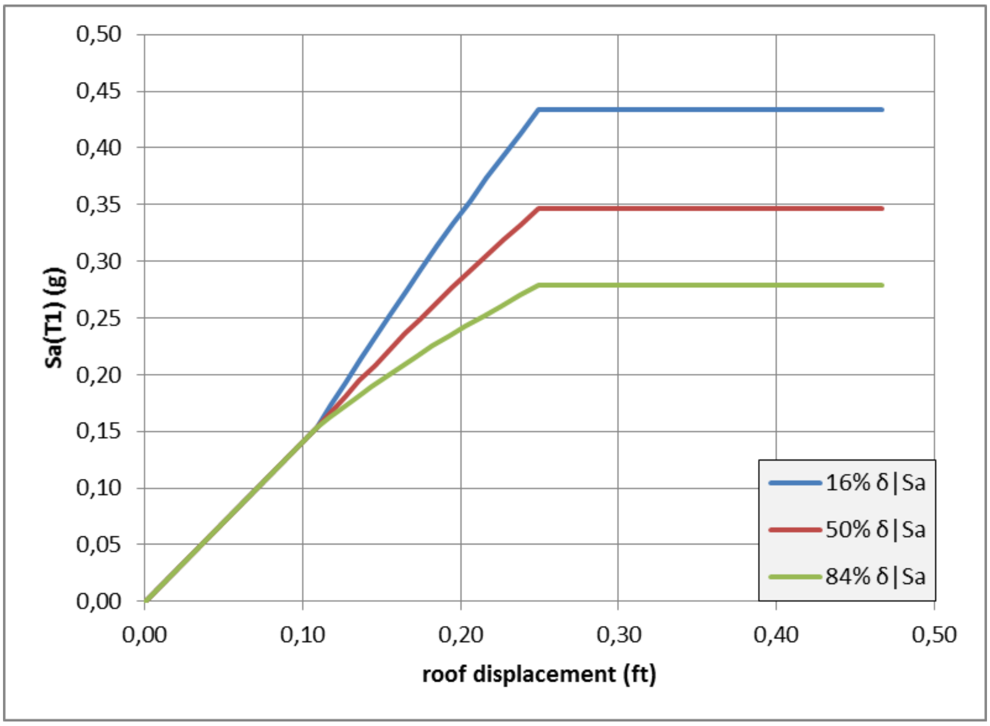
6.2 Model 1



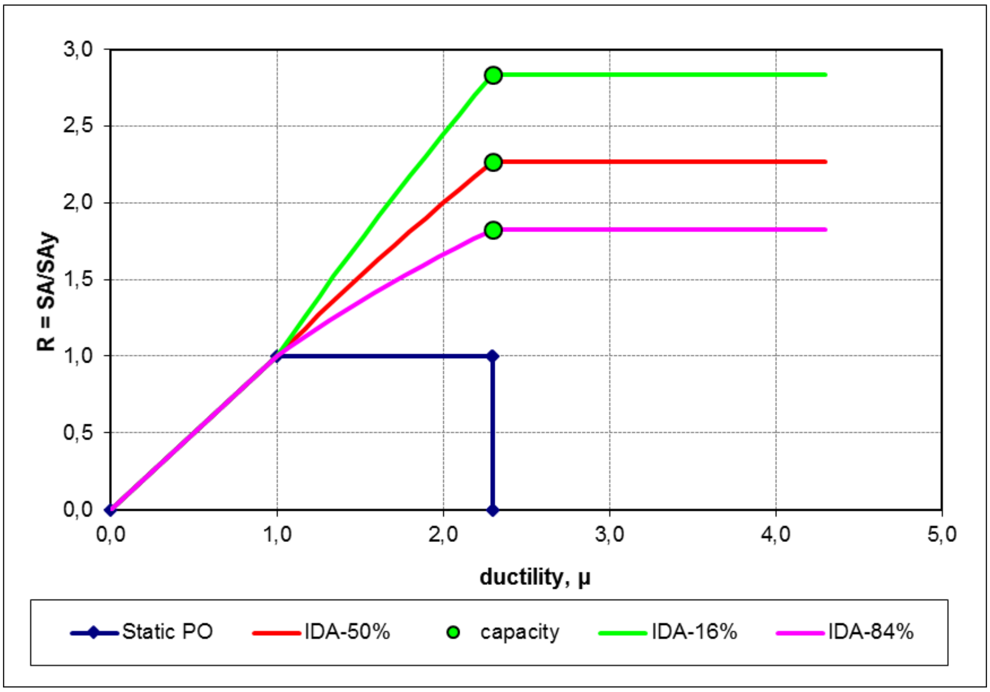

4,5
4,0
3,5
3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
6,0
7,0
8,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%


0,35
0,30
0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
6,0
7,0
8,0
ductility, μ
beta_R

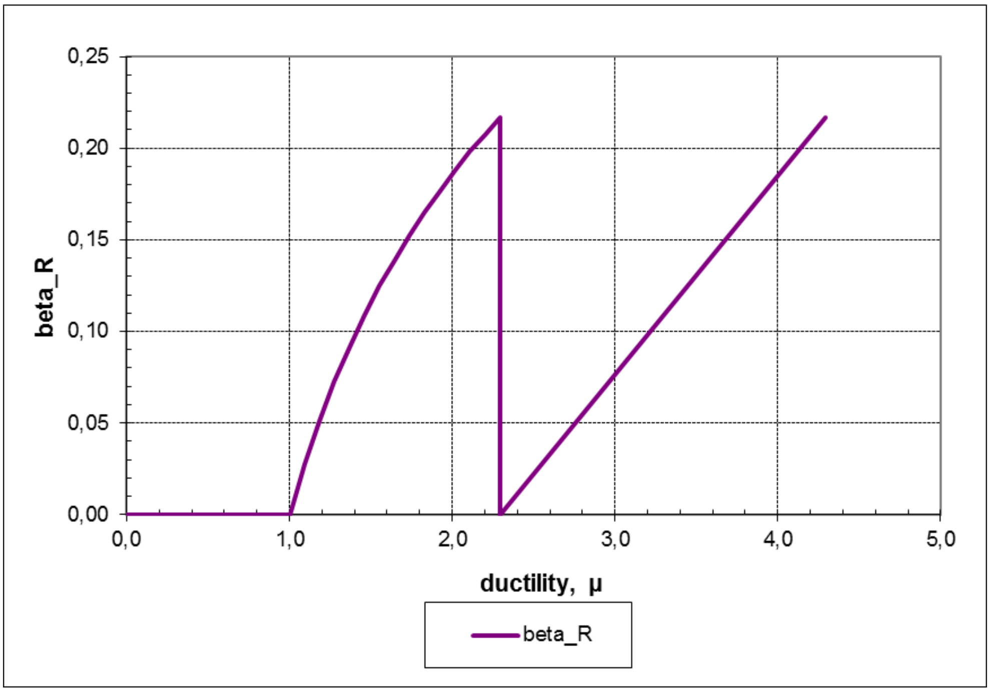
6.3 Model 2



Sa(T1) (g)
0,50
0,45
0,40
0,35
0,30
0,25
0,20
0,15
0,10
0,05
0,00
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

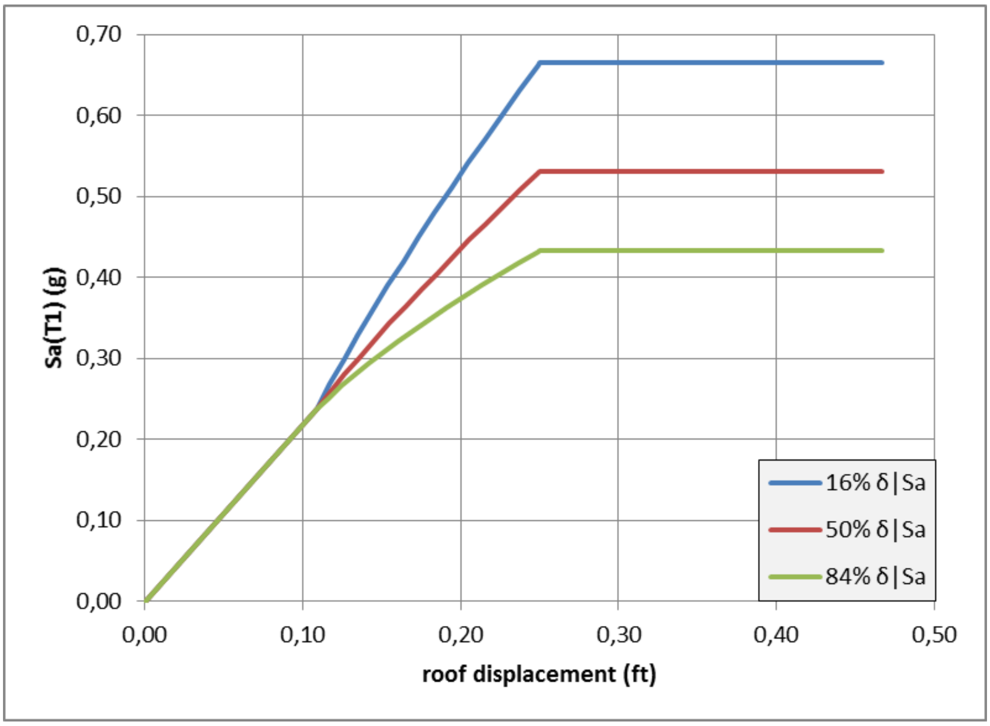


R = SA/SAy
3,0
2,5
2,0
1,5
1,0
0,5
0,0
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

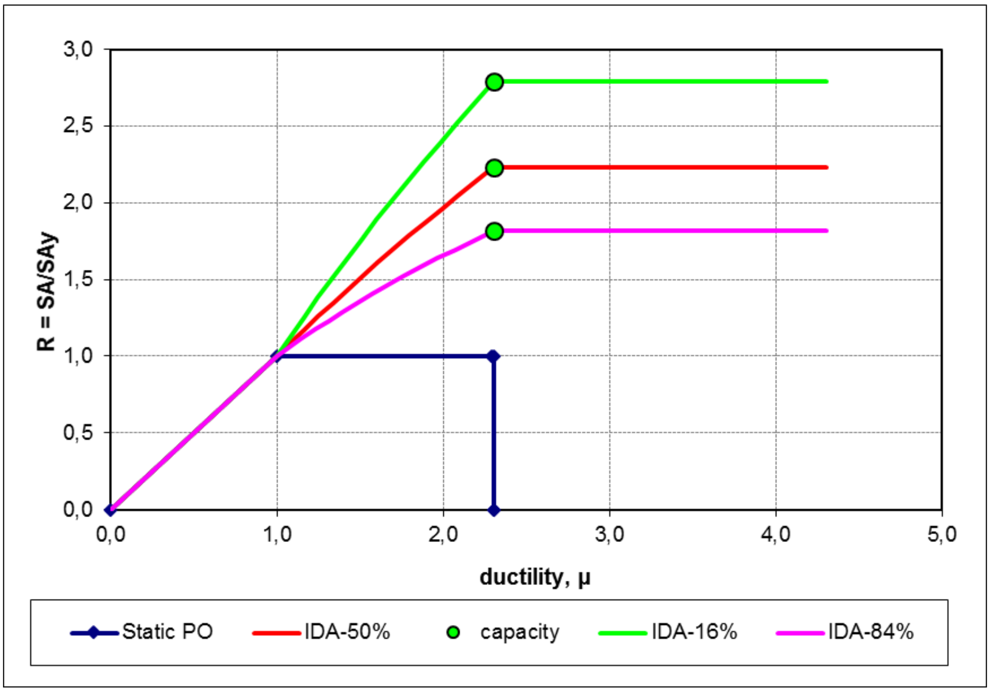


0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

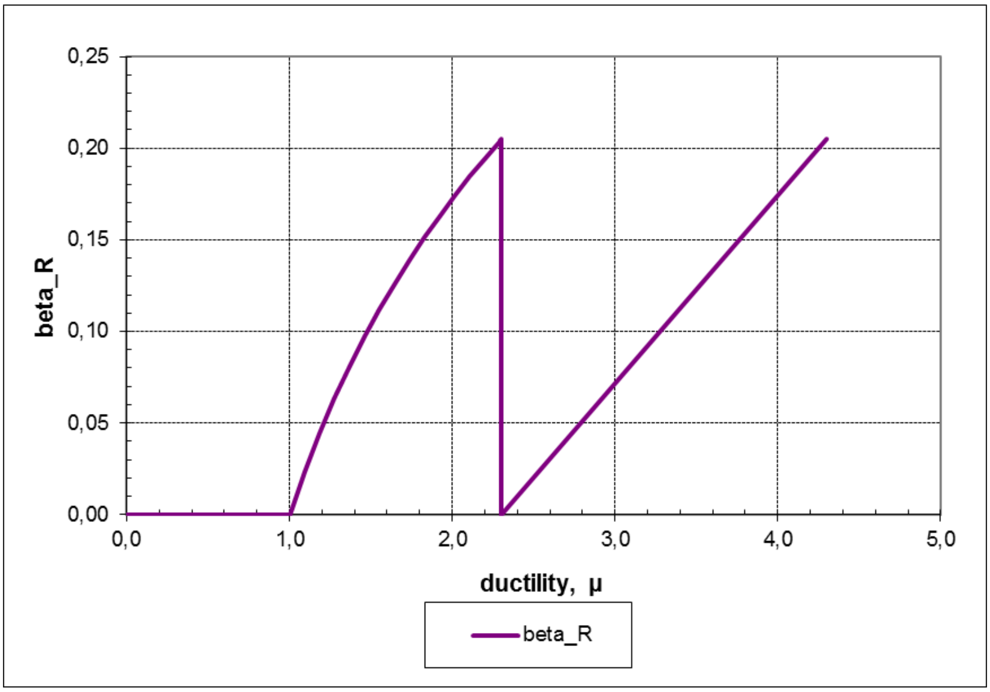
6.4 Model 3



0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

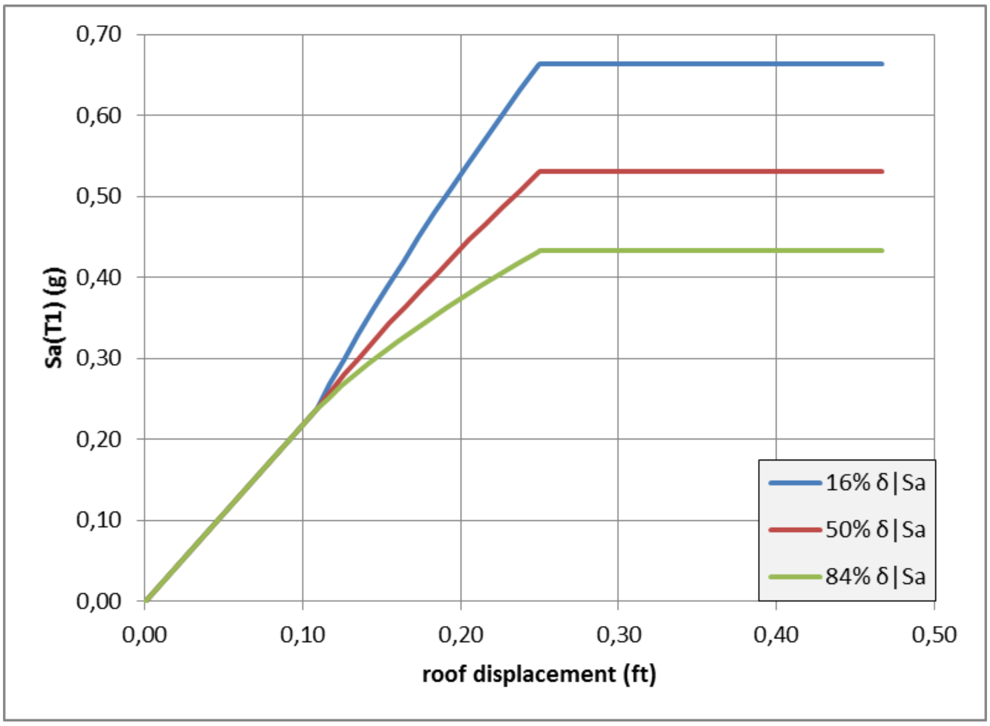


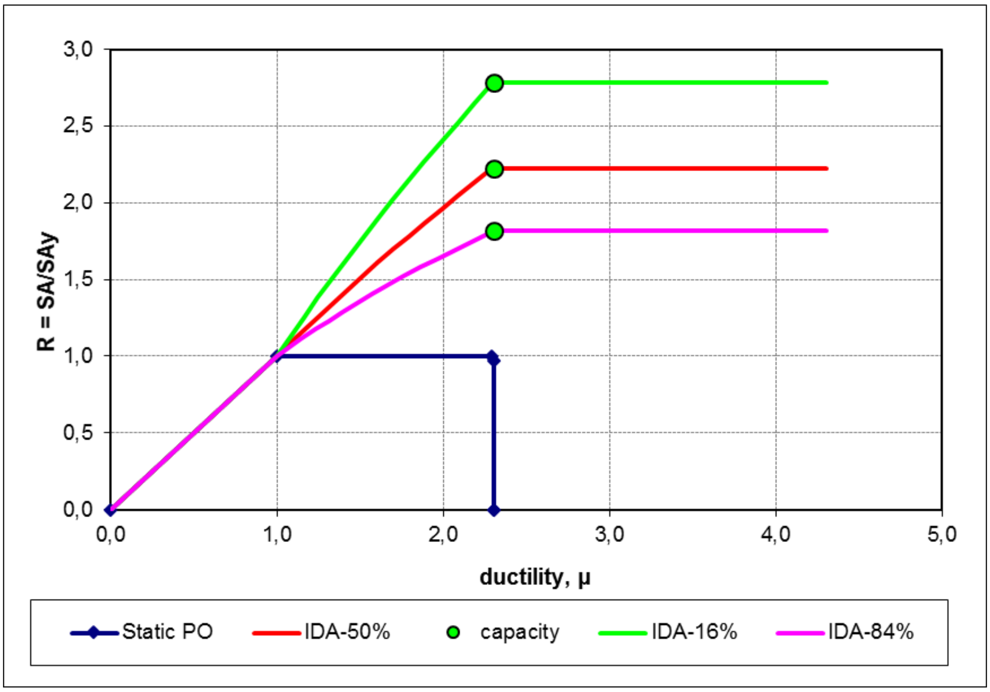
3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

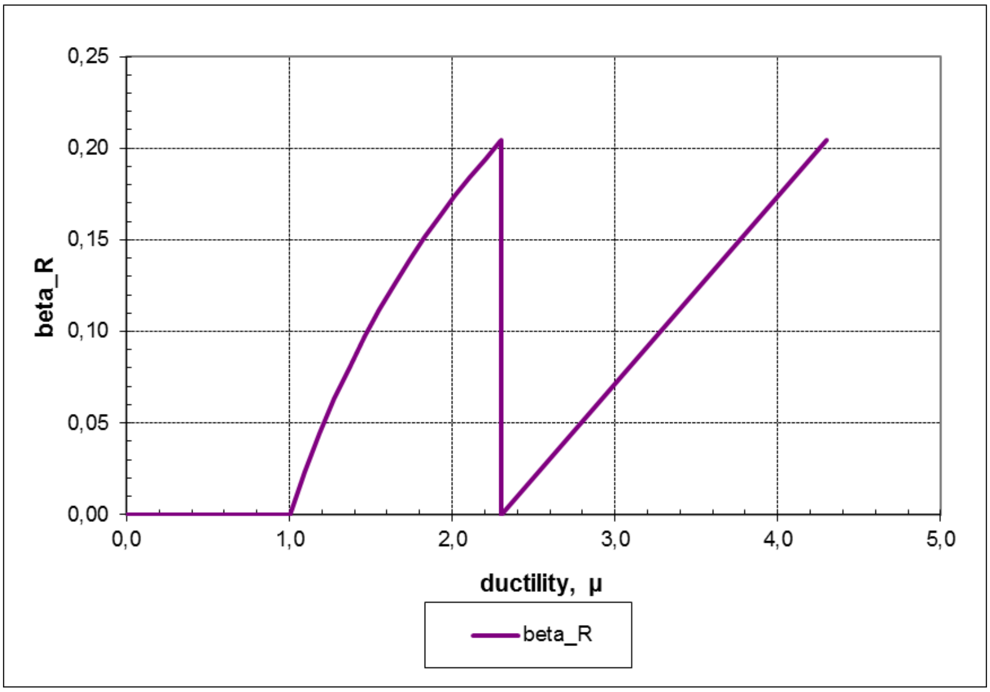


0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

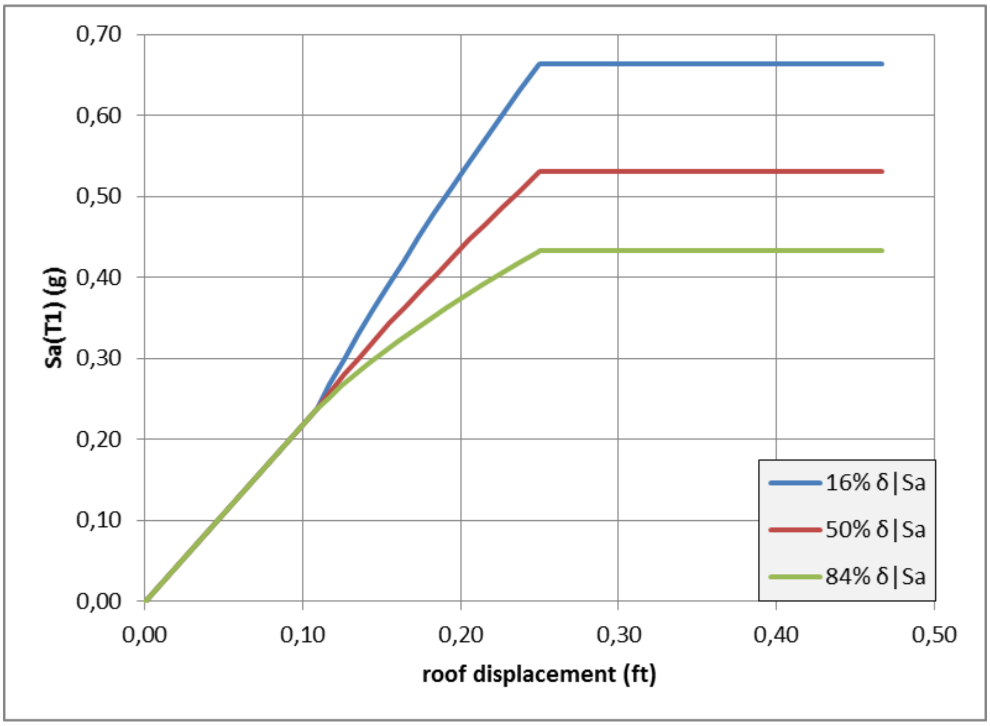
6.5 Model 4

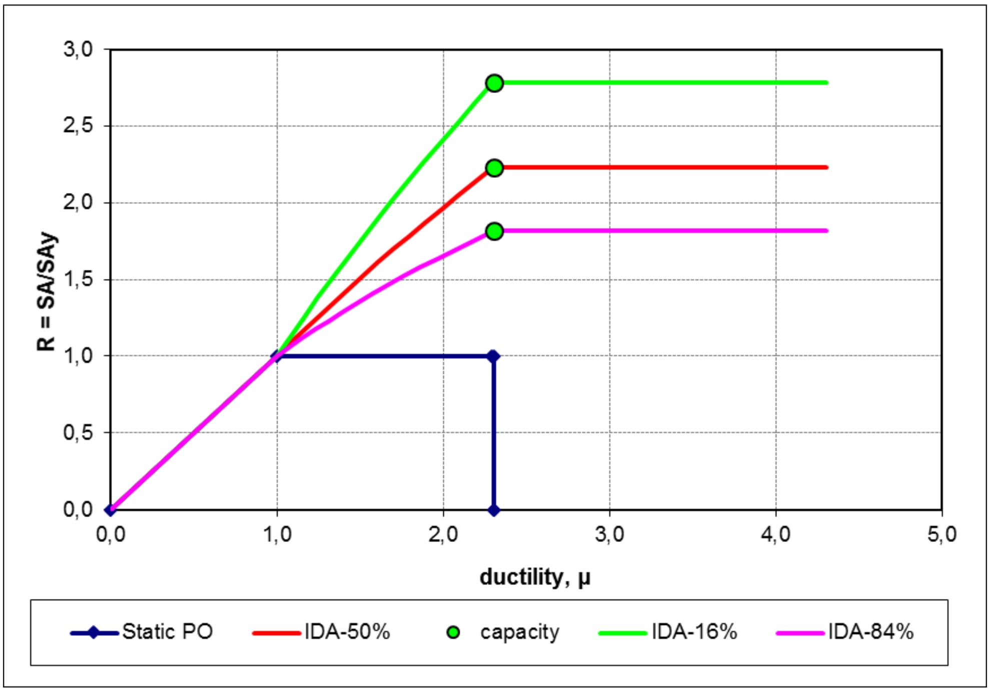

0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa


3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

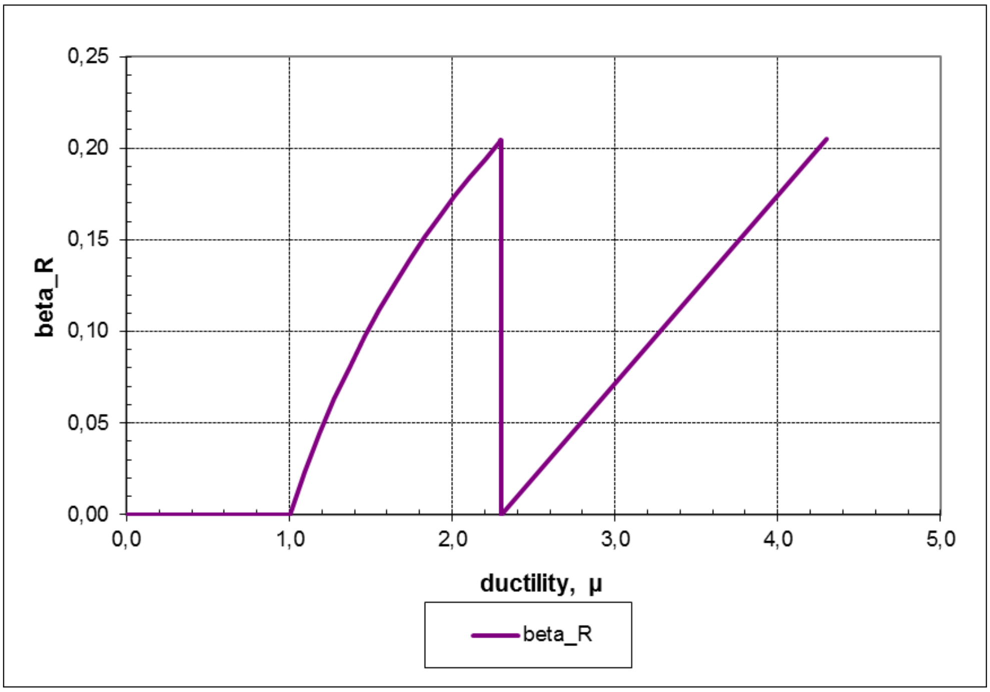

0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

6.6 Model 5


0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

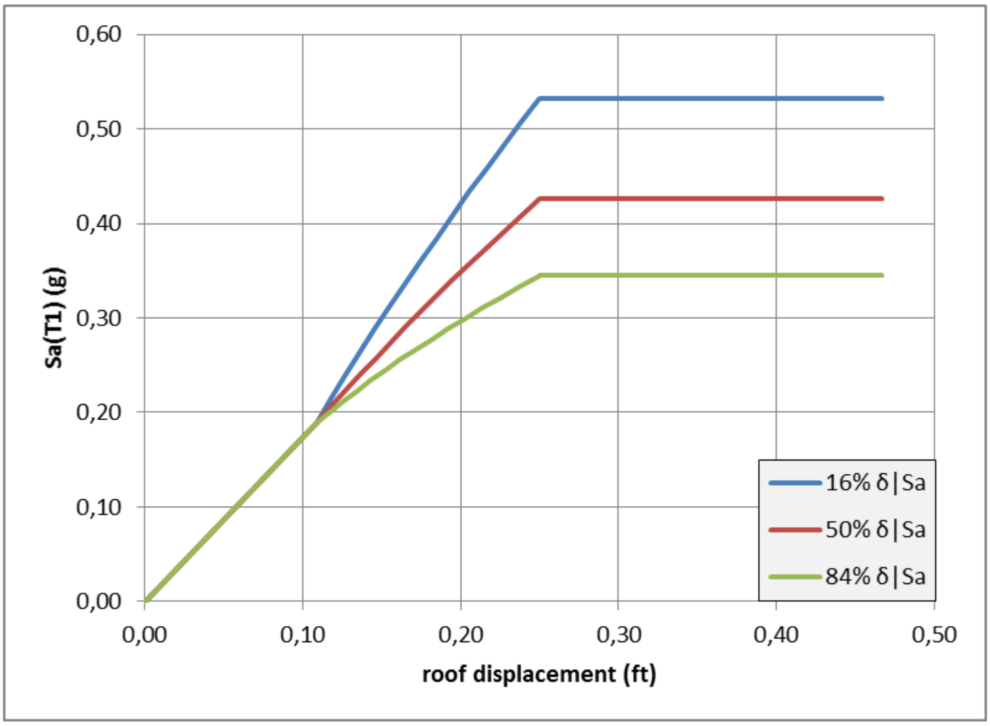


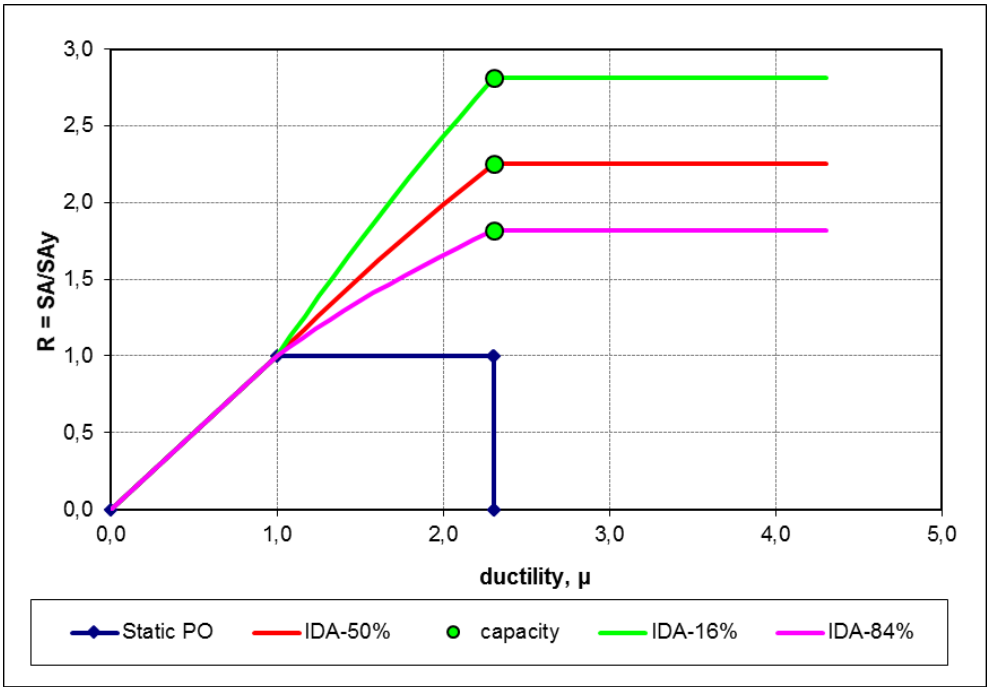
3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%



0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

6.7 Model 6


0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa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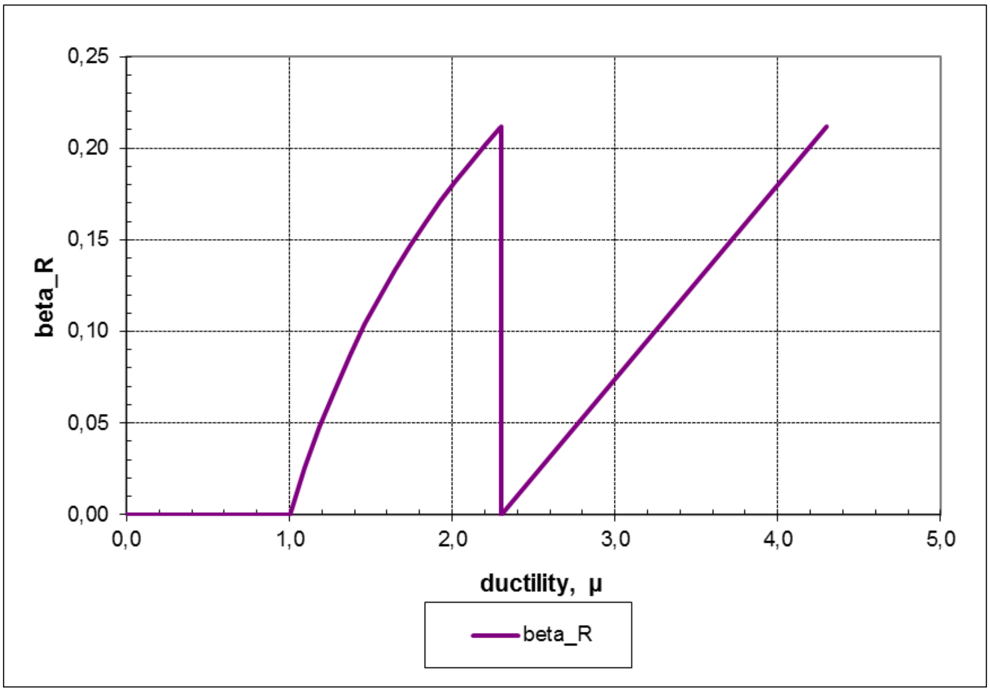
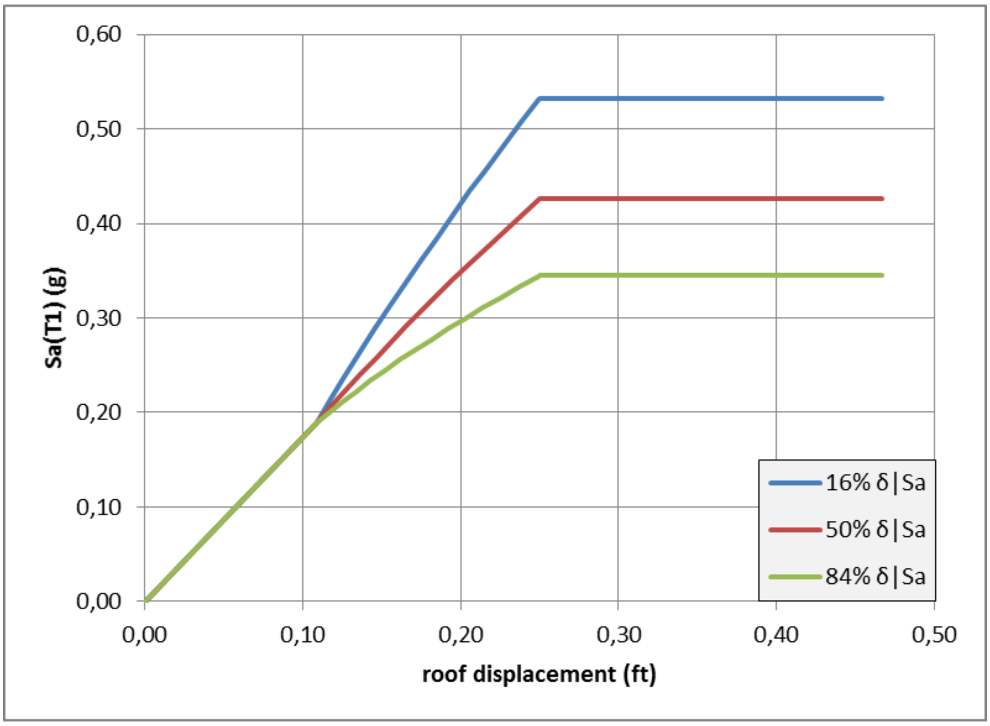
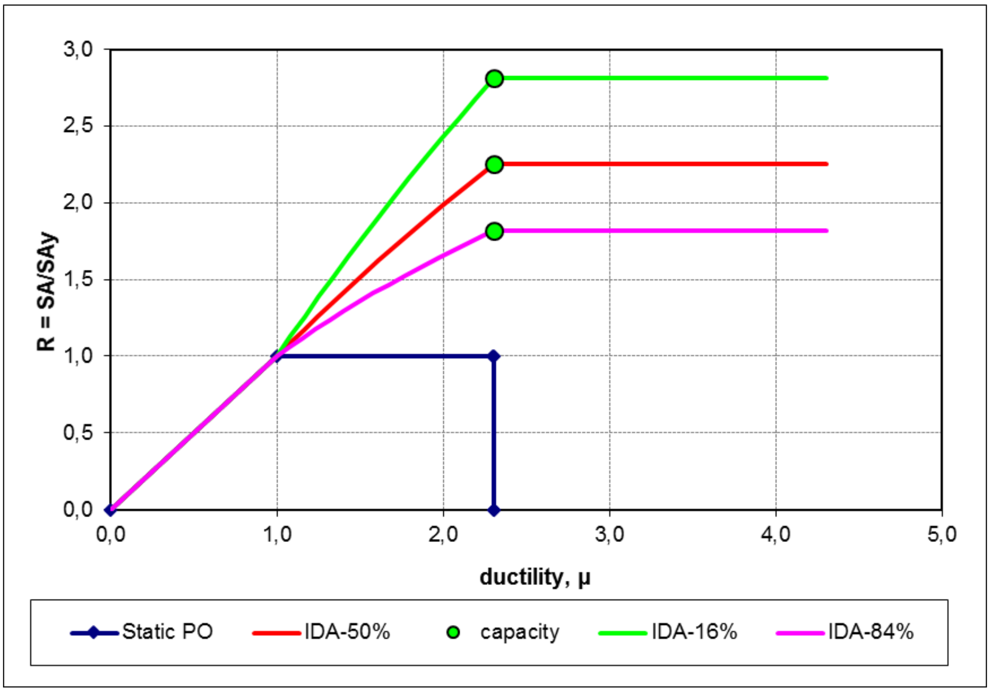
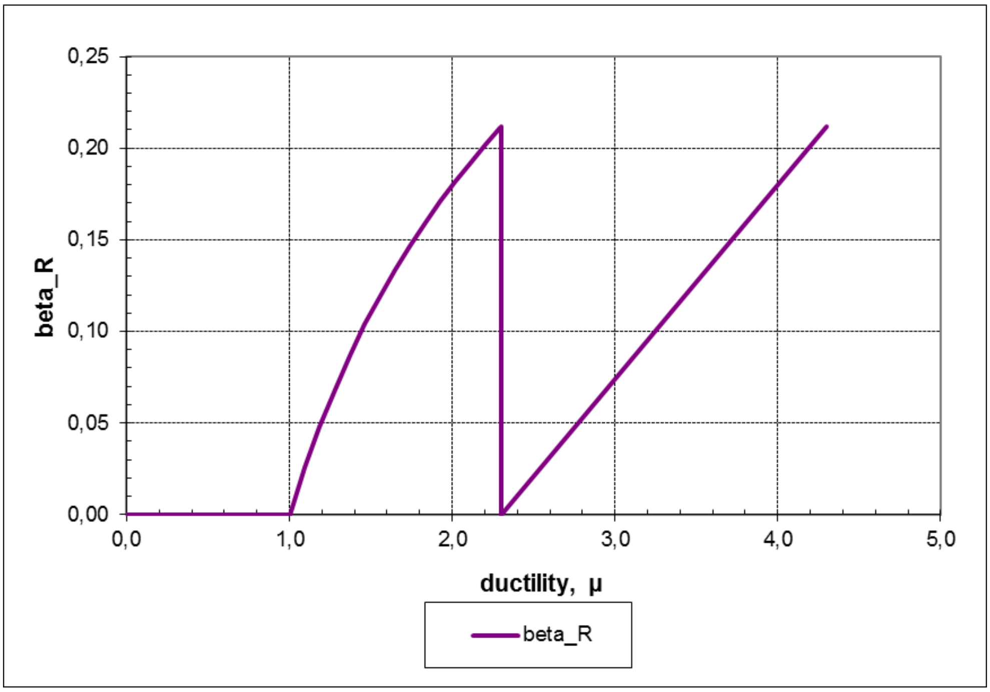
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%


0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

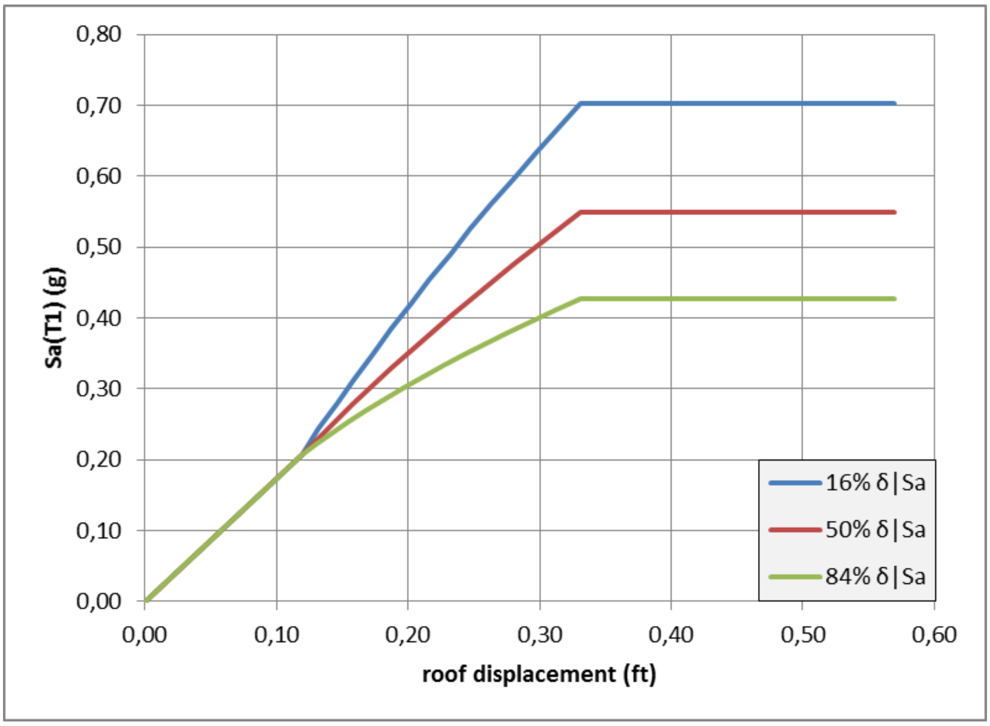
6.8 Model 7


0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

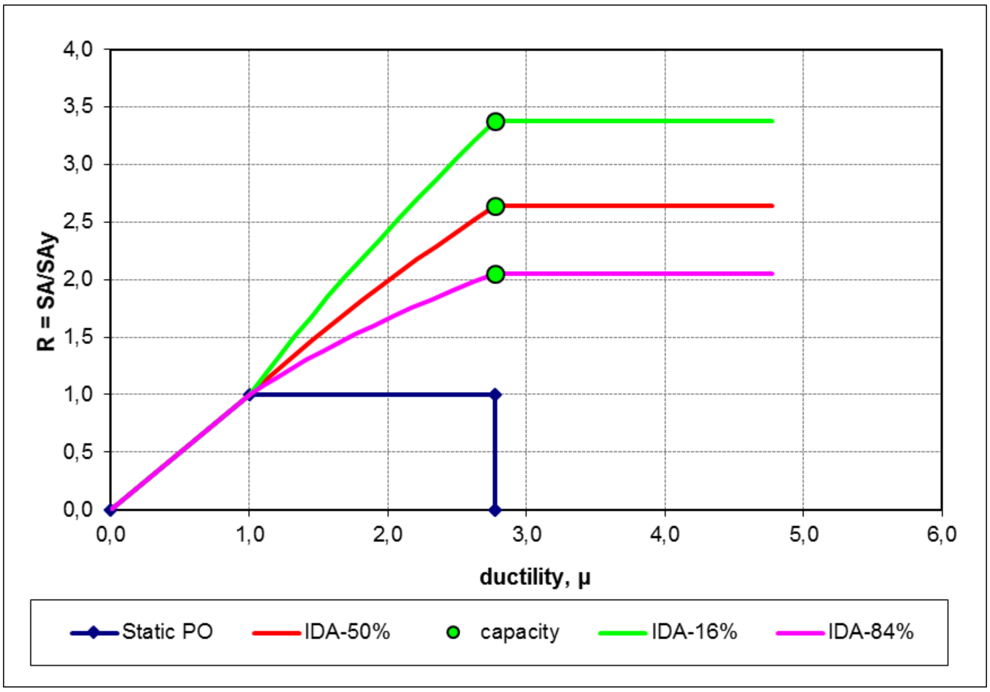

3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%


0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
ductility, μ
beta_R

6.9 Model 8



0,80
0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
0,60
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa



4,0
3,5
3,0
2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
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2,0
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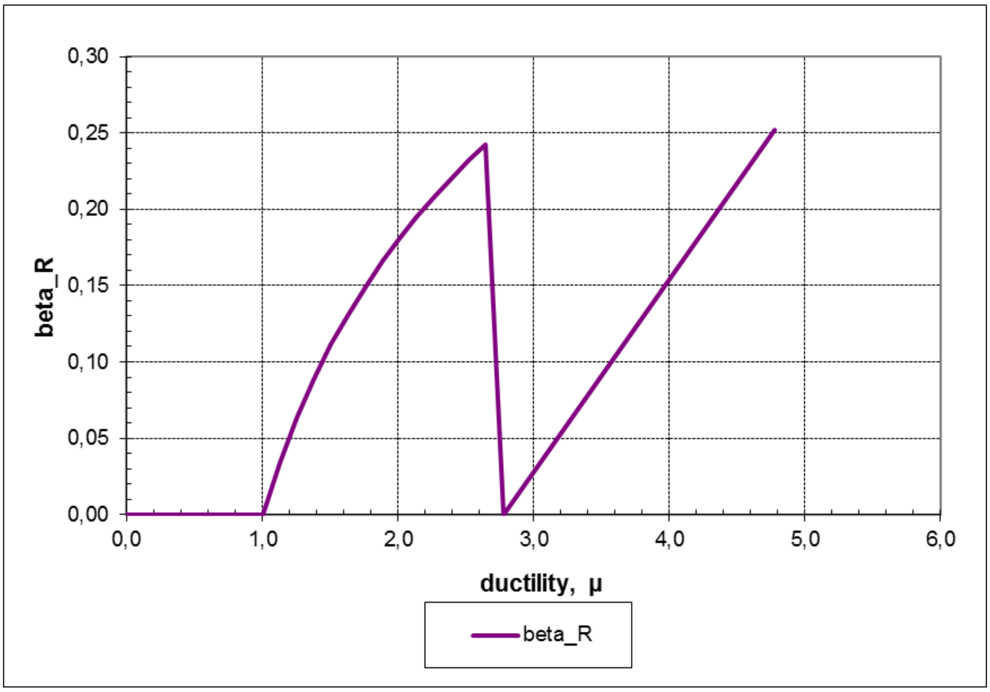
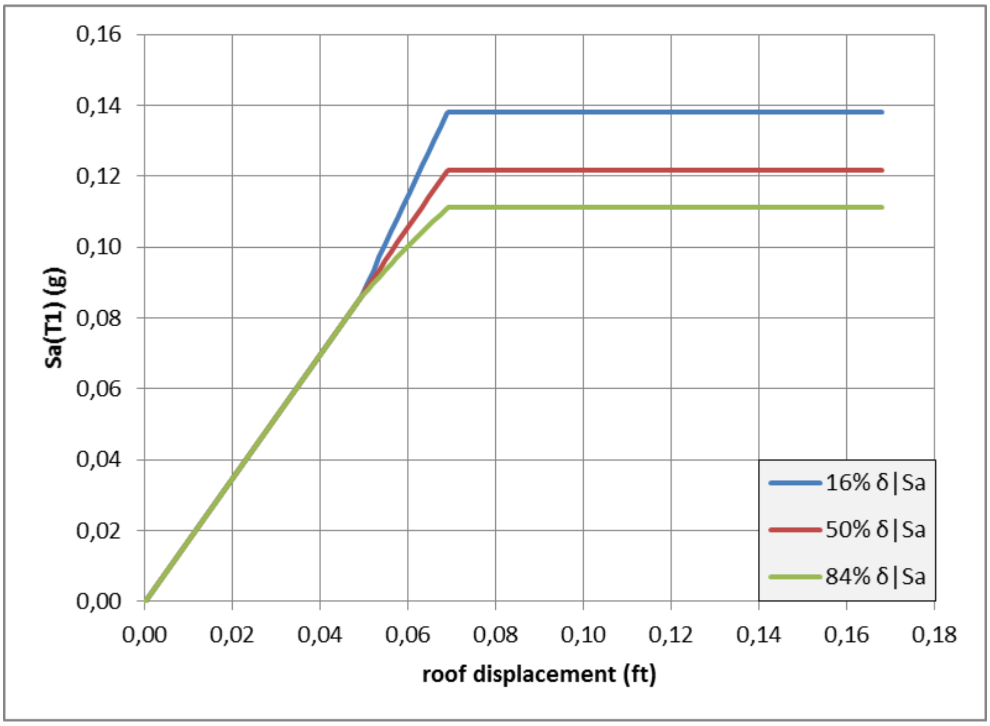
4,0
5,0
6,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

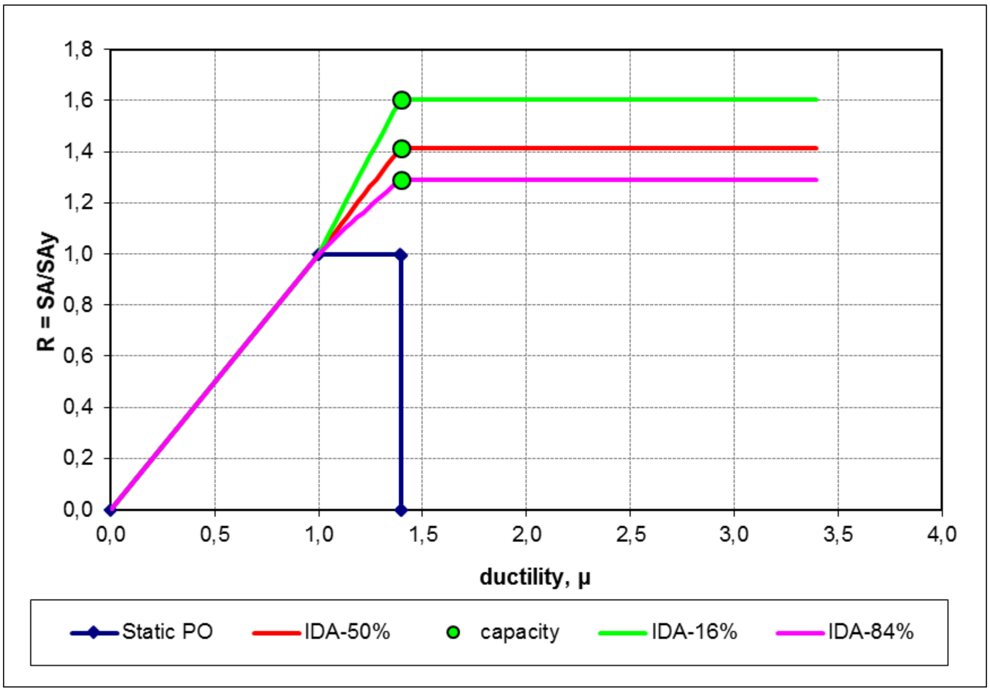


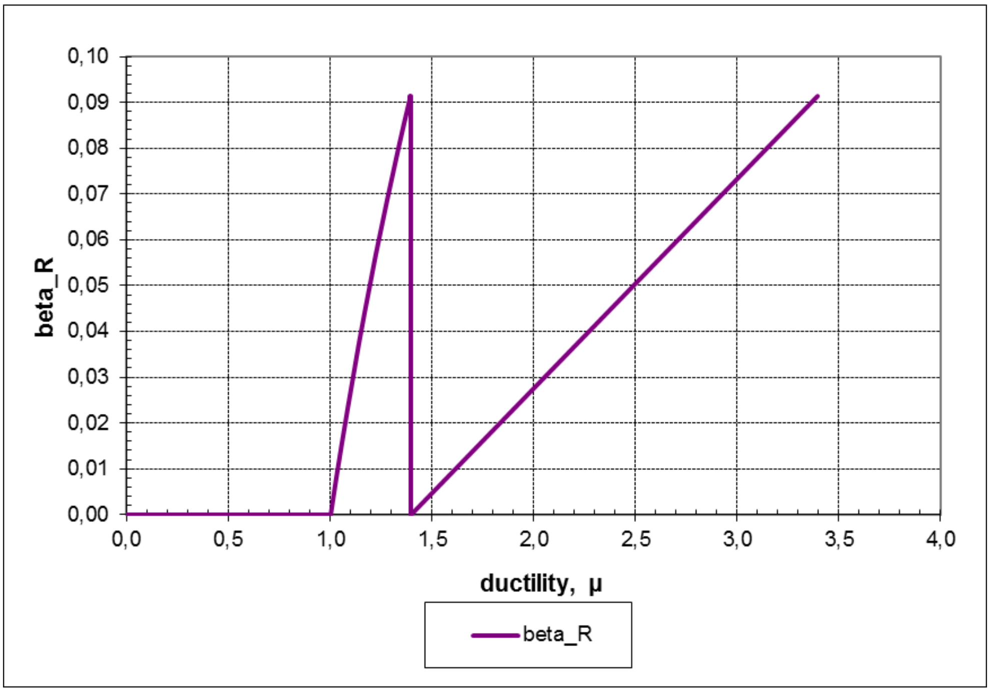
0,30
0,25
0,20
0,15
0,10
0,05
0,00
beta_R
0,0
1,0
2,0
3,0
4,0
5,0
6,0
ductility, μ
beta_R

6.10 Model 9

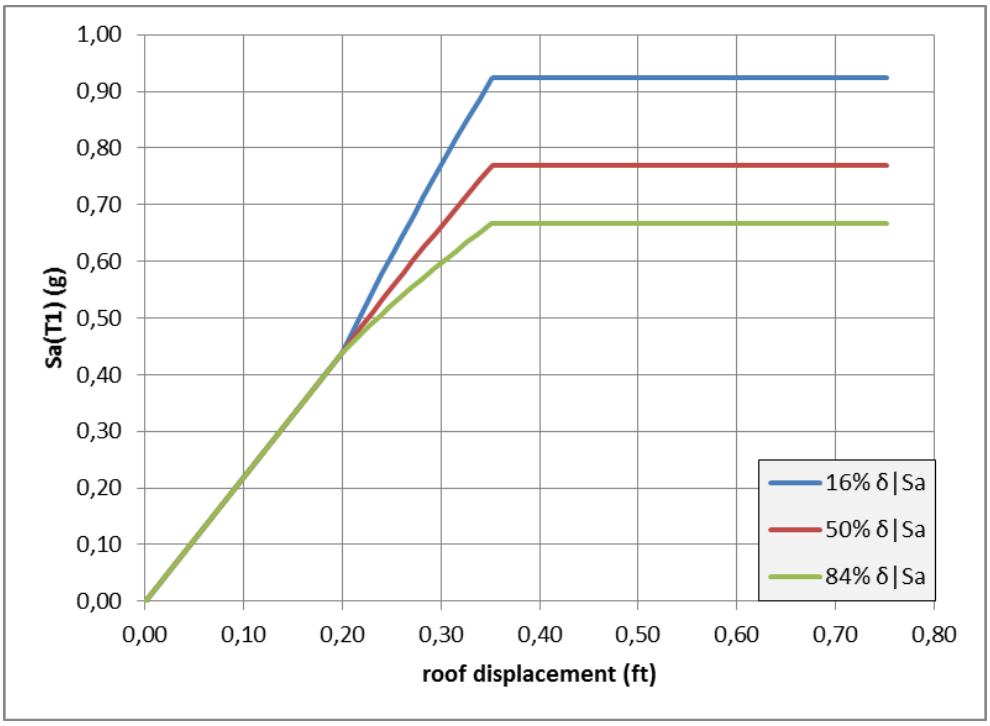


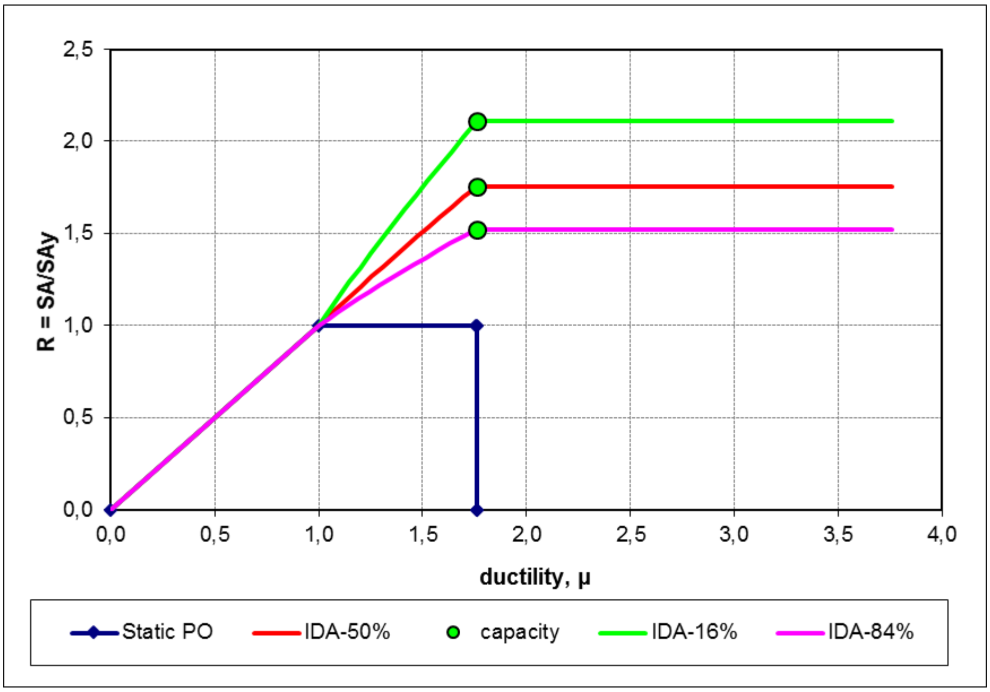
0,16
0,14
0,12
0,10
0,08
0,06
0,04
0,02
0,00
Sa(T1) (g)
0,00
0,02
0,04
0,06
0,08
0,10
0,12
0,14
0,16
0,18
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

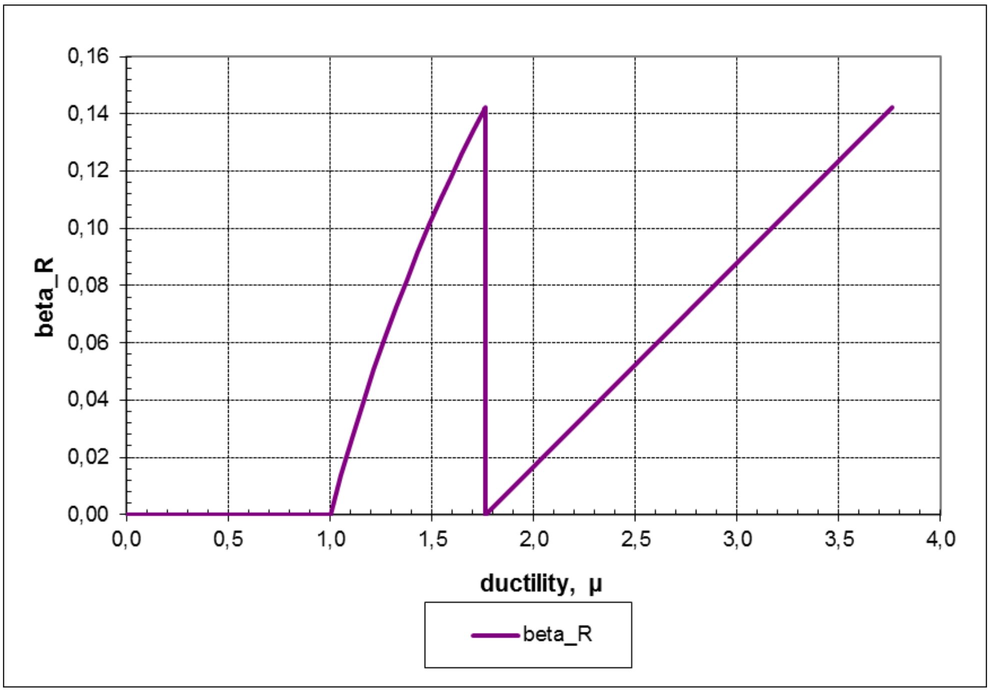

R = SA/SAy
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%


beta_R
ductility, μ
beta_R

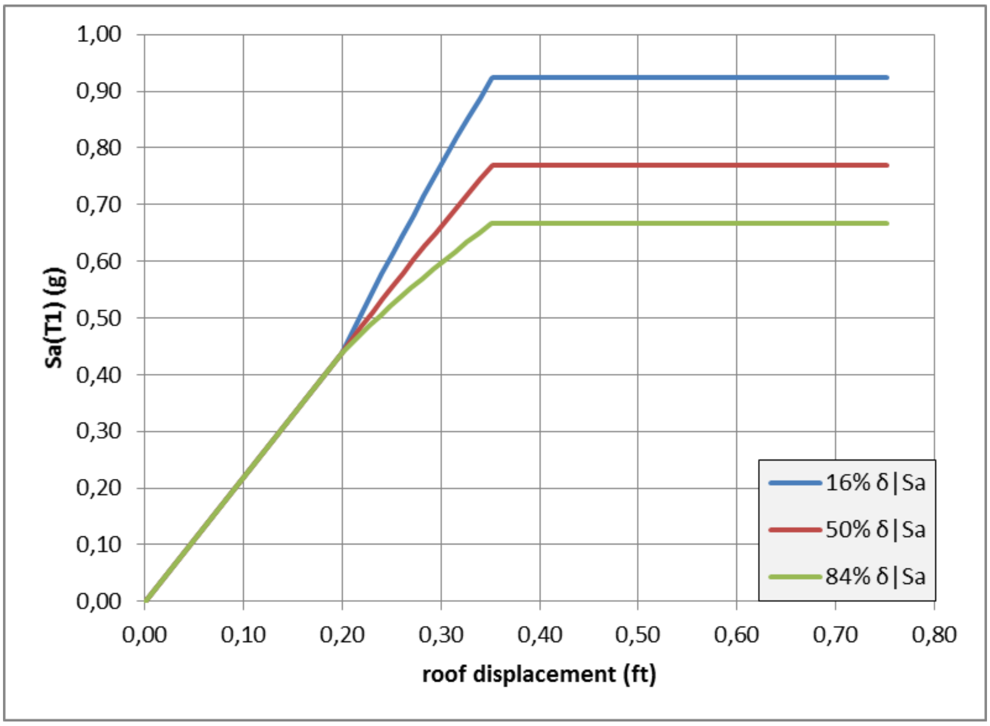
6.11 Model 10

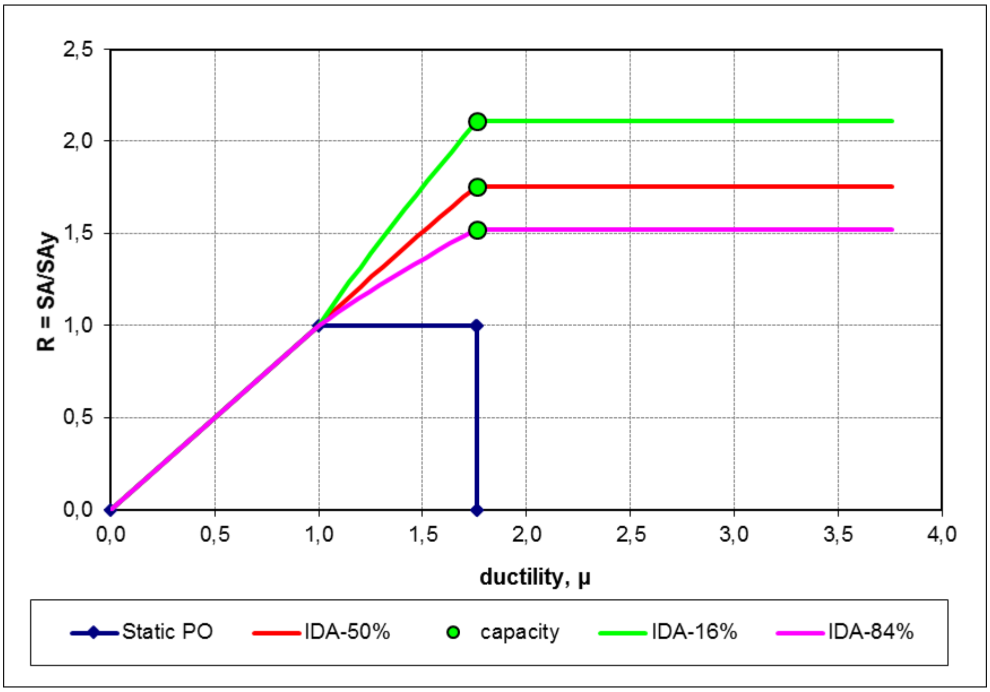

1,00
0,90
0,80
0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
0,60
0,70
0,80
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa


2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

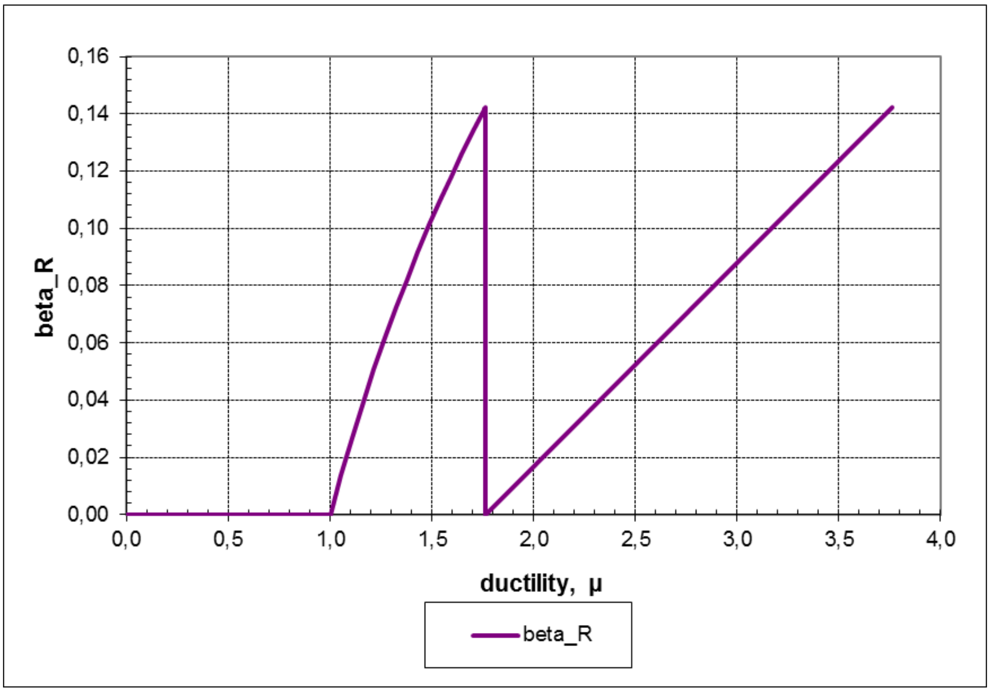

0,16
0,14
0,12
0,10
0,08
0,06
0,04
0,02
0,00
beta_R
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
beta_R

6.12 Model 11


1,00
0,90
0,80
0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
0,60
0,70
0,80
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa

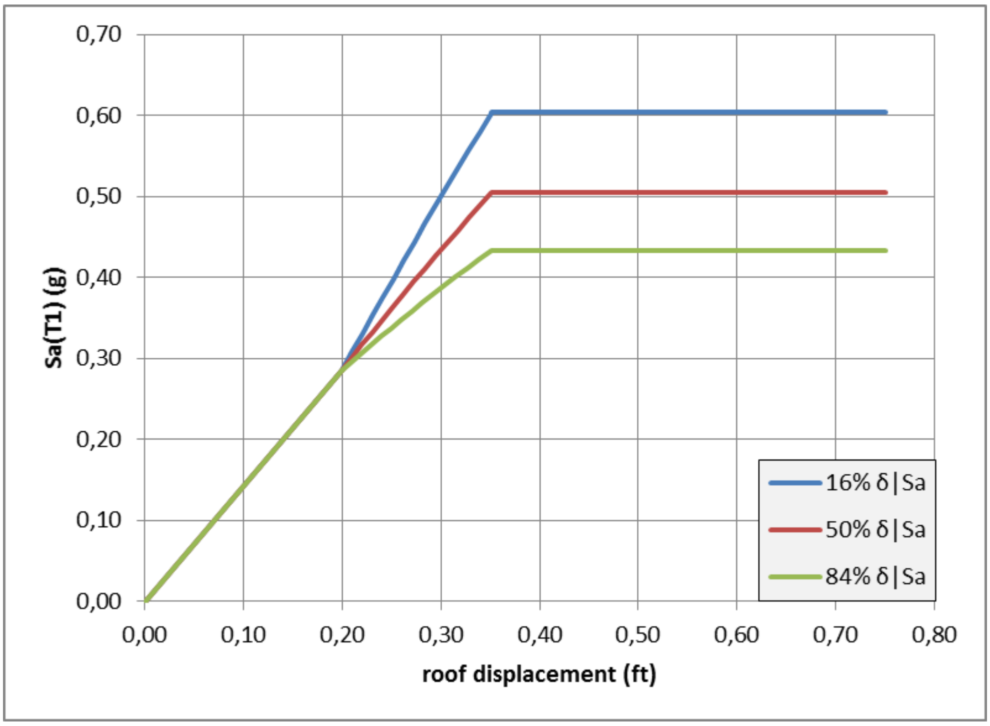


2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%

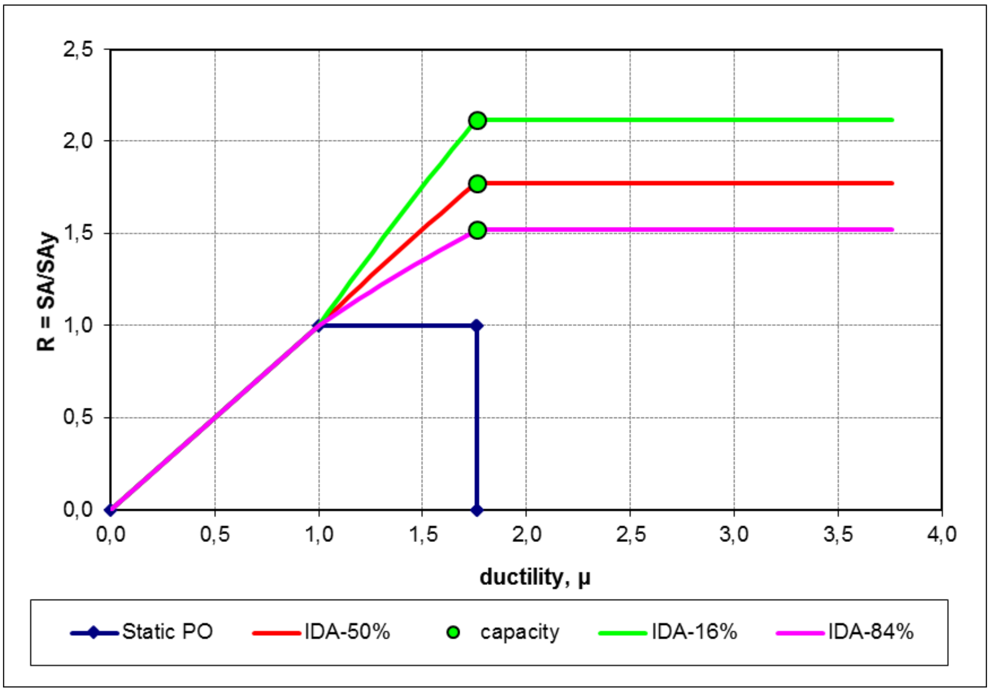


0,16
0,14
0,12
0,10
0,08
0,06
0,04
0,02
0,00
beta_R
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
beta_R

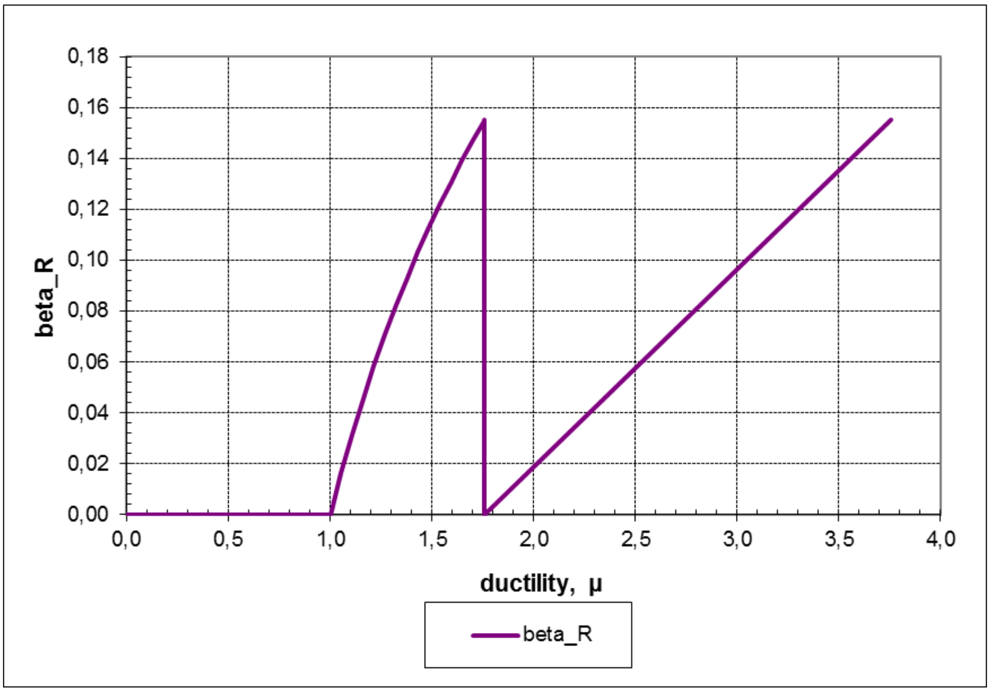
6.13 Model 12



0,70
0,60
0,50
0,40
0,30
0,20
0,10
0,00
Sa(T1) (g)
0,00
0,10
0,20
0,30
0,40
0,50
0,60
0,70
0,80
roof displacement (ft)
16% δ|Sa
50% δ|Sa
84% δ|Sa



2,5
2,0
1,5
1,0
0,5
0,0
R = SA/SAy
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
Static PO
IDA-50%
capacity
IDA-16%
IDA-84%


0,18
0,16
0,14
0,12
0,10
0,08
0,06
0,04
0,02
0,00
beta_R
0,0
0,5
1,0
1,5
2,0
2,5
3,0
3,5
4,0
ductility, μ
beta_R

Conclusions

In this work, a procedure for seismic vulnerability assessment of classes of R.C. buildings, which were designed using the seismic code in force at time of their construction, has been implemented in probabilistic terms.

To achieve an accurate and realistic prediction of the seismic response of a structure is necessary to have analytical tools that allow to figure out the nonlinear behavior and its evolution over time.

The used approach is "multi-level", for classes of buildings that represent the building types that are in the examined area.

The starting point was the observation of an area inside the City Hall of Zografou, the district within which the NTUA (National Technic University of Athens) is located, by detecting some significant features of 305 surveyed buildings (such as number of floors, irregularities in height and in plant, year of construction).

Each of these characteristics has been considered as discriminatory for the belonging of the particular building to a specific group. Homogeneous groups were then treated with techniques of statistical type, including the Clustering method, by which the number of the models (12 models) is resulted much lower than the number of the buildings analyzed, representative of the structures present in the whole area examined

Through the SPO and SPO2IDA, the final result are in terms of nonlinear dynamic analysis.

The approach based on "damage factor" compared to other models for which are known seismic losses, led to further evaluation in terms of statistical dispersion of results.

The steps are repeatable, with the necessary precautions, in other areas, and they give the opportunity to describe the seismic fragility of the heritage of entire cities. The results are useful to provide valuable information to organizations such as the Civil Protection and / or insurance agencies.

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