

***Vulnerability Curves for the Seismic Assessment of
Reinforced Concrete Building Populations***

by

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Abstract

Existing vulnerability relationships are reviewed with a view to their application to a European risk assessment scenario. New empirical fragility curves for reinforced concrete building populations are derived from an extensive data bank of post-earthquake building damage distributions. The heterogeneous observational data is reinterpreted in terms of the newly proposed Homogenised Reinforced Concrete damage scale (HRC-Scale). The latter is calibrated experimentally and allows distinctions to be made between the seismic resistances of different structural systems. The feasibility of using observation-based data for the generation of vulnerability curves for different strong ground motion parameters is investigated. The notion of developing a set of “homogeneous” vulnerability relationships, applicable to different lateral-load resisting systems is explored. A series of relationships for different building height and age-classes are proposed. Although large uncertainties are associated with the empirical relationships, they are seen to be a significant improvement over existing observation-based curves. A new procedure for analytical vulnerability curve generation is proposed. The methodology represents an optimum between reliability and computational efficiency. Adaptive pushover analyses are adopted within a capacity spectrum framework of assessment to determine the performance of a population of building models under increasing ground motions. Material uncertainty is considered in the generation of the population for analysis, with design of experiment techniques used to optimise its size. Suites of accelerograms are selected for the analyses, in accordance with spectra derived for scenario events, which are consistent with the hazard levels of the limit states. Structure performance is assessed using the HRC-Scale. The assessment results are used to construct response surfaces, from which the damage statistics forming the basis of the vulnerability curves are generated via re-sampling. The proposed methodology is illustrated for the case of low-rise, infilled RC frames with inadequate seismic design and the curves are compared to observational data.

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Chapter 1

Introduction

1.1 Preamble

Seismic risk assessments are carried out on populations of buildings to identify the urban areas most likely to undergo large life and economic losses during an earthquake. The results of such studies are important in the mitigation of losses under future seismic events, as they allow strengthening intervention and disaster management plans to be drawn. The determination of seismic risk involves the integration of inventory data, seismic hazard assessments and vulnerability studies. Each element of the risk model is associated with large uncertainties, and requires further development and refinement for the creation of reliable risk assessment tools. The proliferation of accurate inventory data is expected in the near future, due to the current development of GIS based survey procedures and programs for the interpretation of aerial photography. Concerted efforts have been made to improve the prediction of seismic hazard. Attenuation models for the prediction of spectral ordinates, which account for the effects of source, site and path on ground motion characteristics, have been developed. Following observation of the poor performance of existing buildings in recent earthquakes, a multitude of tools and procedures have been developed for the vulnerability assessment of single structures and building populations.

The development of simple tools for vulnerability assessment has been slower than for the other constitutive elements of seismic risk. This is mainly due to the expense associated with the dynamic testing of structures, and the large computational effort and specialist programs required for the analytical simulation of building population seismic response. Existing vulnerability relationships only cover a limited number of regions and structural types. They are mostly developed from the effort of individuals rather than a united research community, and little

agreement exists regarding the derivation methodology, performance criteria and ground motion characterisation adopted for their development.

1.2 The Problem

In Europe the majority of reinforced concrete buildings are not designed to seismic codes. These buildings tend to fail in a brittle manner when subjected to strong lateral loads, due to the absence of capacity design and inadequate reinforcement detailing. Nevertheless, most have an inherent lateral resistance, born of over-strength factors in design codes, which may be sufficient to resist the moderate earthquakes that typify European seismicity with an acceptable degree of damage. The criteria that define an *acceptable* degree of damage vary according to the importance of the building, its use and the owner's requirements. Typically, multiple performance criteria are required to be satisfied such as that the buildings show no damage under a frequent earthquake event, suffer repairable damage in a rare event and ensure life-safety in cases of extreme ground motion. The satisfaction of multiple behaviour objectives lies at the basis of Performance Based Design. The development of methodologies for these designs is a subject of active research in the field of earthquake engineering and it is believed that the same concept should also be applied to building assessment. Vulnerability assessment tools are therefore required, which will allow the prediction of the probability of occurrence of several damage states, (rather than failure only), under different severities of ground motion.

Vulnerability (or fragility) curves provide a possible solution for such an assessment tool. They consist of a set of relationships between ground motion and the probability of exceedence of certain thresholds of damage. In mathematical terms, the probability of reaching or exceeding damage state d_i given that the ground motion level is gm_k , is given by:

$$P_k = P[D \geq d_i | GM = gm_k] = \sum_{j=1}^n P[D = d_j | GM = gm_k] \quad [1. 1]$$

If P_{ik} is evaluated by varying k , (i.e. the ground motion severity), whilst keeping i constant, a vulnerability curve is obtained for the damage state i .

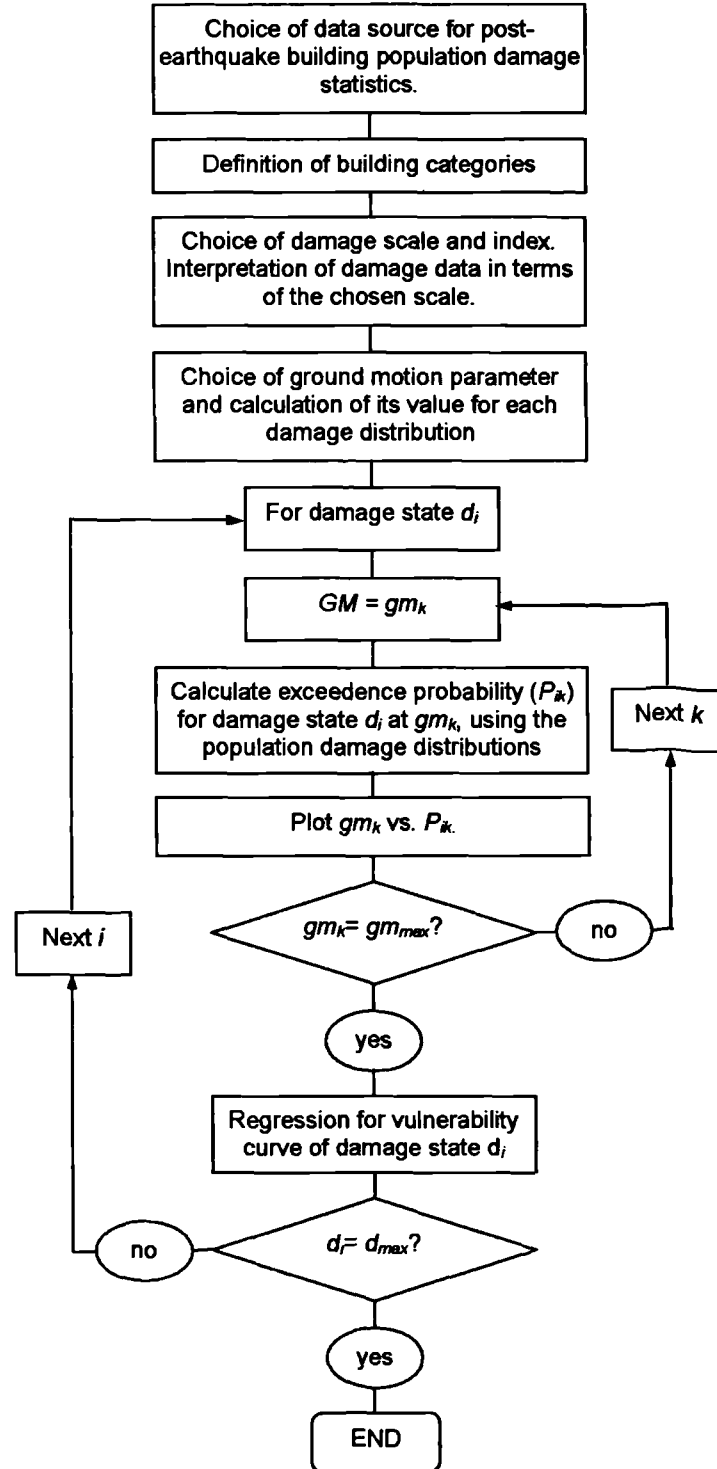


Figure 1. 1: Flowchart of the general methodology for vulnerability curve generation

Vulnerability curve shapes change for different structural types, due to variations in their rate of accumulation of damage with increasing ground motion. However, the same general procedure is followed for the derivation of vulnerability curves in the case of all structural classes. This is presented in a flowchart in Figure 1. 1. The main choices to be made at each stage of the derivation process are:

- Choice of a source/s, (empirical, analytical, judgment), for the building population damage distributions and associated ground motion values, used as the statistical basis for the vulnerability curve generation.
- Choice of a ground motion parameter representative of the damage potential of earthquake time histories.
- Determination of a building classification system, for the grouping together of damage statistics concerning buildings of similar dynamic response characteristics.
- Selection of a damage scale and the definition of limit states for the assessment of building performance.
- Choice of a structural response parameter for the estimation of global building damage, and determination of its value at the thresholds of the chosen limit states.
- Determination of a procedure for the interpretation of the building damage statistics in terms of the chosen damage scale.
- Choice of a methodology for the damage data combination and confidence bound determination.
- Selection of shape functions for the vulnerability curves and of a regression procedure.

A variety of damage data sources, damage scales, ground motion parameters and curve derivation methodologies exist. All choices made have a large influence on the final result and determine the range of application of the curves. Each must be addressed in the assessment of existing curves, and for the proposal of new vulnerability curve generation methodologies. Furthermore, the effect on the curves of the uncertainty associated with each chosen parameter should be considered.

1.3 Aim and Scope

The research here presented, is carried out within the framework of Task 4.2 of the Safety Assessment For Earthquake Risk Reduction (SAFERR) European Research Training Network (Contract: HPRN-CT-1999-00035). The Network aims to refine the available tools and propose new techniques for the prediction of each element contributing to seismic risk, and Task 4.2 deals with the definition of vulnerability relationships based on deformations.

The aim of the research is to investigate the state-of-the-art of vulnerability representation for populations of reinforced concrete structures, highlight the fallacies and limitations of current vulnerability assessment methods and propose new techniques for the derivation of displacement-based vulnerability curves from observational and analytical sources of damage statistics, which produce more reliable predictions of seismic risk. Within the scope of this research only populations of reinforced concrete structures of typical European construction are considered in the illustration of the proposed curve derivation methodologies. However, the proposed techniques can be potentially applied to any structural type through consideration of observational data that is consistent with the assessed building type in the empirical methodology or through appropriate building and uncertainty modelling in the analytical technique.

Within this study, a critique of existing vulnerability curves for reinforced concrete structures is firstly carried out and the fundamental ingredients of a vulnerability assessment tool are discussed in detail. Subsequently, the use,

application and reliability of observational post-earthquake damage data alone, as a source for the development of new vulnerability relationships for populations of RC buildings is explored. A concerted effort is dedicated to the collection of as wide an observational data bank as internationally possible. A methodology for the observational data processing and vulnerability curve generation is explained. For the purpose of damage data harmonisation, a new damage scale (Homogenised Reinforced Concrete Scale, HRC-Scale) is defined. This scale is correlated to other existing damage scales and calibrated to a deformation measure of structural response, using experimental observations. Various parameters for strong ground motion characterisation are considered and conclusions are made as to their suitability. Using the newly defined damage scale, the concept of a single set of “homogeneous” vulnerability relationships applicable to different lateral-load resisting systems, is set forth and series of regression and sensitivity analyses are carried out to assess its feasibility.

In light of the observations made during the empirical curve derivation, structural analyses are investigated as a source of damage data for vulnerability curve generation. A new procedure is proposed for the derivation of analytical vulnerability curves for the seismic assessment of populations of reinforced concrete structures. The proposed methodology is regarded to represent an optimum solution between reliability and computational efficiency. It adopts an adaptive pushover technique to analyse a series of structural models. Within the analysis, both structural and ground motion variability are considered, with design of experiment techniques being used to limit the number of analyses required for full representation of these sources of uncertainty. The performance of the buildings under increasing ground motion intensities is assessed using a modified capacity spectrum method. Response surfaces are constructed from the results of the population assessment. Building damage statistics are generated from the response surfaces through re-sampling, and are used to derive the final vulnerability curves. The methodology is illustrated through application to the category of low-rise infilled moment resisting frames of typical European construction. The validity of the analysis technique and derived curves is assessed with respect to experimental and field-survey data.

Chapter 2

Existing Vulnerability Functions

2.1 Introduction

Vulnerability curves relate the probability of exceedence of multiple damage limit states to a measure of ground motion severity. Few vulnerability relationships have been specifically derived for European countries; however several relationships for reinforced concrete (RC) structures have been proposed by government agencies and research institutes worldwide. These are derived for different structural systems, using a variety of methodologies and parameters for the representation of damage and seismic demand. Although not specifically derived for Europe, it is possible that existing relationships derived for reinforced concrete buildings of similar structural characteristics and construction to those composing the European stock, may be used for their seismic assessment. The salient features of the most important existing relationships of this kind are summarised in Table 2. 1. These relationships are also reviewed individually in Appendix A.

In the following section the main factors found to influence the reliability of existing vulnerability curves are presented, both generally and with a view to their application in a European earthquake risk assessment scenario. Examples from the literature are used throughout to illustrate the conclusions made. Finally, the existing vulnerability relationships regarded to be most relevant to European constructions are compared to existing observational post-earthquake damage statistics.

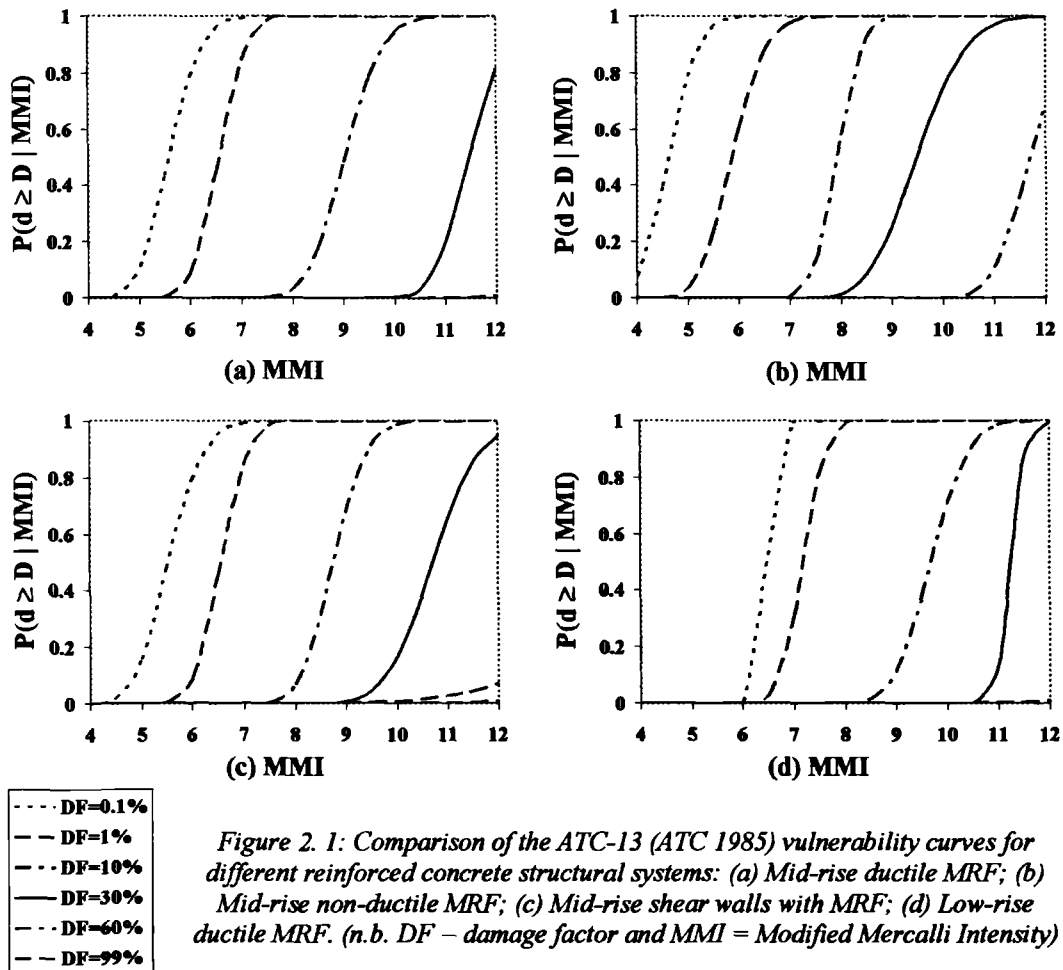
Source	Type ⁽¹⁾	Country	Structural Unit	Curve Function ⁽²⁾	GMP ⁽³⁾	Damage Scale	No. of LS ⁽⁴⁾	E/Q or A-grms ⁽⁵⁾	Datasets ⁽⁶⁾ (Units)	Mean R ² (7)
Spence et al. (1992)	E	S. Europe	Pre-Code MRF	Normal	PSI ⁽⁸⁾	MSK	5	13	20-100	- (COV 36%)
Orsini (1999)	E	Italy	Apartments	Normal	PSI	MSK	5	1	41 (12,522)	0.45
Yamazaki & Murao (2000)	E	Japan	Pre-, Old- & New-Code MRF	Lognormal	pgv	AIJ (1995)	2	1	16 (3,814)	0.95
Miyakoshi et al. (1997)	E	Japan	Pre-, Old- & New-Code MRF	Lognormal	pgv	Property Loss	2	1	Not known	0.90
ATC-13 (ATC 1985)	J	USA	Bare & Infilled MRF of varied heights & codes	Lognormal	MMI	ATC-13 (ATC 1985)	5	N/A	79 experts	N/A
Onose (1982)	A	Japan	Low/Mid-rise MRF	Lognormal	pga	Ductility	6	206	N/A (405)	N/A
Singhal & Kiremidjian (1997)	A	USA	Low, Mid & High-rise MRF	Lognormal	Sas _w (T)	Park & Ang (1985a)	4	3800 ⁽⁹⁾	38 (300)	N/A
Mosalam et al. (1997)	A	USA	Low-rise MRF	1- α exp ⁸	pga	ISD (%)	4	600	600	0.72
Mosalam et al. (1997)	A	USA	Low-rise Infilled MRF	1- α exp ⁸	pga	ISD (%)	4	600	600	0.68

⁽¹⁾Type of vulnerability curve: Empirical (E), Analytical (A), Judgement (J); ⁽²⁾ Cumulative distribution function or equation defining the vulnerability curve shape; ⁽³⁾ Ground motion parameter used by the curves; ⁽⁴⁾ Number of Limit States; ⁽⁵⁾ Number of earthquakes or accelerograms used; ⁽⁶⁾ Number of observational damage datasets used (Number of structures or structural models considered); ⁽⁷⁾ Average correlation coefficient reported by the curve authors; ⁽⁸⁾ Parameterless Scale of Intensity Pomonis et al. (1992); ⁽⁹⁾ 100 accelerograms scaled to 38 intensity levels.

Table 2. 1: Summary of existing vulnerability curve characteristics.

2.2 Factors Influencing Existing Vulnerability Function Reliability

Existing vulnerability relationships vary according to the lateral-load resisting system, seismic design and detailing of the building they consider (e.g. Figure 2. 1). These factors influence the building dynamic characteristics and rate of accumulation of structural damage with increasing ground motion severity. For any one structural system however, several choices exist for the source of the building damage data, structural unit, damage scale, damage index and ground motion parameter used in the vulnerability curve derivation. This is illustrated in Figure 2. 2, where a comparison is made between four vulnerability curve sets for reinforced concrete bare moment resisting frames. The general influence of these factors on loss prediction reliability is discussed in the following sections.



2.2.1 Damage Data Source

Existing vulnerability curves can be classified into the four generic groups of empirical, judgmental, analytical and hybrid, according to whether the damage data used in their generation stems mainly from observed post-earthquake surveys, expert opinion, analytical simulations, or combinations of these, respectively. Each data source has associated advantages and disadvantages.

The observational source is the most realistic, as all practical details of the exposed stock are taken into consideration alongside soil-structure interaction effects, topography, site, path and source characteristics. However, the same aspects that render observational data the most realistic are also responsible for the severe limitations in their application potential. The derivation of empirical vulnerability relationships requires large quantities of reliable observational post-earthquake building damage statistics, covering a wide range of ground motions. Ideally, the observational damage statistics should consist of post-earthquake surveys carried out for large populations of buildings of similar construction, over areas of uniform soil conditions, in close proximity to ground motion recording stations. This is a necessary condition for a uniform seismic demand to be assumed across the population, and for the reported damage distributions to be representative of the variation in building resistance. In practice, most existing post-earthquake surveys report observed building damage over very large areas, encompassing multiple soil conditions. Even in cases such as that of Yamazaki and Murao (2000), where the aerial distribution of building damage is known in detail, large uncertainties are associated with the determination of an average seismic demand value for the site. This uncertainty is accentuated by the typical lack of local ground motion recordings. As post-earthquake surveys are not carried out with the purpose of providing data for vulnerability curve generation, the level of refinement, in terms of both structure and damage classification is typically low. Existing post-earthquake surveys often do not distinguish between damage observed in structures of different materials, lateral-load resisting system, heights and seismic design provisions. Consequently, existing empirical ground motion - damage relationships developed from very few damage surveys carried

out for single locations or earthquake events are highly specific to a particular seismo-tectonic, geotechnical and built-environment. Observations of damage made after single events often do not encompass a range of ground motions sufficient for the reliable determination of the entire vulnerability curve shape (e.g. Orsini 1999, Yamazaki and Murao 2000). In practice, the latter requires the combination of observations from numerous different earthquakes and countries. However in the literature, only Spence et al. (1992) are seen to successfully combine multiple earthquakes worldwide. The aforementioned low level of refinement in structure and damage classification that typifies post-earthquake survey statistics, poses a real hindrance to the combination of damage data for building populations of different composition. Moreover, due to the infrequency of large magnitude earthquake events near densely populated areas, even when observations from multiple events are combined, the data used in the curve generation tends to be scarce and highly clustered in the low-damage, low ground-motion severity range. Therefore, a reliable determination of the vulnerability curve shape is not guaranteed by the combination of earthquake observations, with considerable uncertainties being associated with loss predictions for large magnitude events.

Empirical data is also associated with inherent uncertainties that cannot be removed via manipulation of the damage statistics. Errors in building damage classification are introduced in the statistics at source, due to the typically rapid execution of post-earthquake surveys by engineers of varied experience and the use of poorly defined damage scales. This is reflected in the consistently larger scatter observable within a set of empirical curves for the lighter damage states compared to the higher ones, as in the former case errors in building classification are expected to be more significant due to the difficulties associated with the damage identification. Moreover, damage due to multiple earthquakes may be aggregated and attributed to a single event, or buildings damaged as a consequence of phenomena other than ground shaking (e.g. ground subsidence, landslides, flooding and fire) included in the data. These errors can lead to a large data scatter even in cases where a single event and limited survey area are considered (e.g. Orsini 1999).

Judgement-based curves are not associated with the same problems regarding the quantity and quality of building damage statistics that typify empirical relationships. Expert panels of civil engineers with experience in the field of earthquake engineering, are asked to make estimates of the likely damage distribution within building populations when subjected to earthquakes of different intensities. Probability distribution functions are fitted to the expert predictions to represent the range of damage estimates at each intensity level. The probability of a specified damage state is derived from the resulting distributions and plotted against the corresponding ground-motion level to obtain a set of vulnerability curves and associated uncertainty bounds. Since the experts can be asked to provide damage estimates for any number of structural types, the curves can easily be made to include all the factors affecting the seismic response of different structures. Hence, expert opinion is the predominant source used by most current rehabilitation codes in the United States of America for the generation of damage probability matrices and vulnerability curves (e.g. ATC 1985 and ATC 1996). The reader is referred to Appendix D of ATC-13 (ATC 1985) for an extensive literature survey of motion-damage relationships based on expert opinion.

The reliability of judgement-based curves is questionable however, due to their dependence on the individual experience of the experts consulted. Furthermore, where intensity is used to characterise the ground motion severity (as in ATC-13), concerns arise as to how experts interpret a scale that does not contain information on building performance for all levels of ground motion (Reitherman 1985). It is practically impossible to evaluate the degree of conservatism associated with the judgement-based source. Furthermore, inherent in the expert vulnerability predictions is a consideration of local structural types, typical configurations, detailing and materials. Thus, if the country of the vulnerability curve derivation is characterised by construction practices that differ significantly from those used in Europe, their application to the latter may be precluded.

Analytical vulnerability curves adopt damage distributions simulated from the analyses of structural models under increasing earthquake loads as their statistical basis. Compared to expert opinion, analyses can result in a reduced bias and

increased reliability of the vulnerability estimate for different structures. Despite this fact, few analytical vulnerability curves for reinforced concrete buildings have been generated in the past. This is mainly due to the substantial computational effort involved and limitations in modelling capabilities. Architectural finishes cannot be accounted for, detailed modelling of infill walls remains a challenge, elaborate soil models cannot be accommodated alongside elaborate structural models and rocking and uplifting of structures poses serious numerical modelling difficulties. Many existing analysis environments also have difficulties converging when structures are subjected to large demands, with numerical collapse frequently preceding structural failure. However, recent developments have led to the inclusion of a number of response features into analytical methods, such as shear-flexure-axial interaction, soil-structure interaction, interactive confinement on concrete members and reinforcing bar buckling. Furthermore, new analysis techniques and solution procedures have been established, which enable the generation of damage data for large and complex structures at very high speeds. Hence, analytical approaches to vulnerability curve generation are becoming ever more attractive in terms of the ease and efficiency by which data can be generated, but have not yet been fully exploited to the limits of their potential.

Existing analytical vulnerability relationships are derived using a variety of analysis procedures. Reliability based methods are used by Onose (1982), which adopt assumed building capacity and seismic demand parameter probability distributions to estimate the population damage state exceedence probabilities. Though the methodology is applicable to any location, the curves resulting from this particular study are highly dependent on the local building stock composition and seismicity. Several analytical vulnerability curves are developed using damage distributions simulated from the analyses of structural models under increasing earthquake loads. The analytical procedure followed ranges in complexity from the elastic analysis of equivalent single degree of freedom systems (Mosalam et al. 1997), to non-linear time history analyses of 3D models of RC structures (Singhal and Kiremidjian 1997). The choices made for the analysis method, structural idealisation, seismic hazard and damage models, strongly influence the derived curves and have been seen to cause significant

discrepancies in seismic risk assessments made by different authorities for the same location, structure type and seismicity, (Priestley 1998). Therefore, their application to European sites may not be justified unless an appropriate degree of uncertainty in the structural models and ground motions are considered.

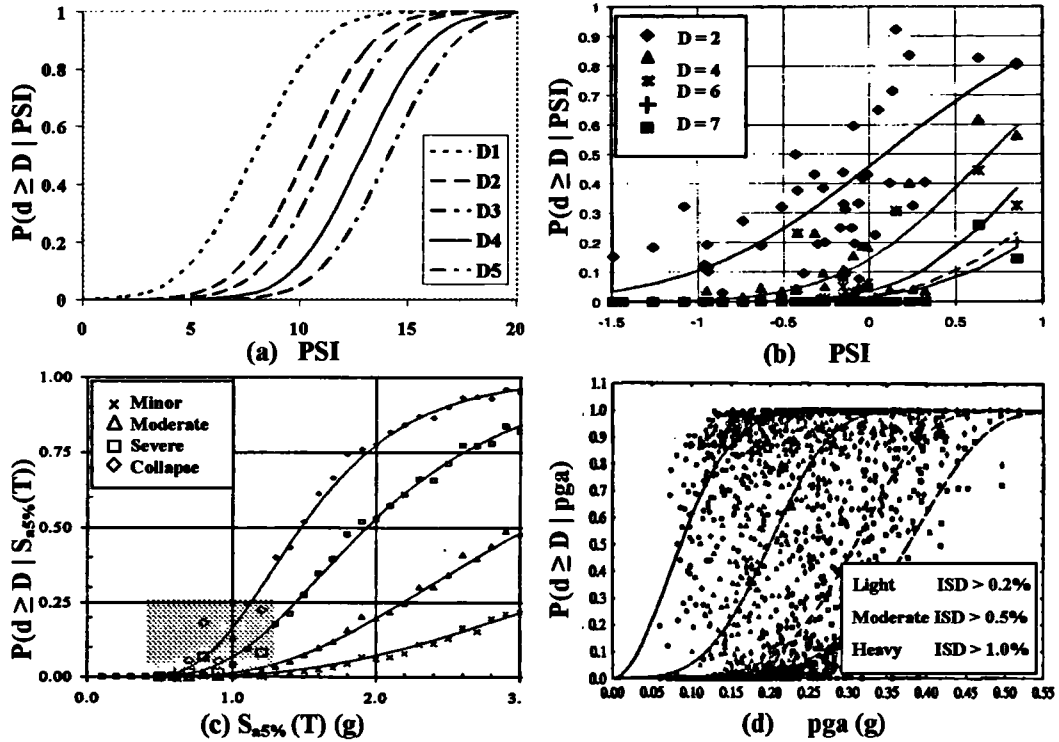


Figure 2: Comparison of existing vulnerability curves for reinforced concrete frames; (a) Spence et al. (1992) curves for RC frames using the MSK (Medvev and Sponheuer 1969) damage scale; (b) Orsini (1999) curves for RC frames using the MSK (Medvev and Sponheuer 1969) damage scale; (c) Singhal and Kiremidjian (1997) curves for low-rise RC MRF (points updated with observational data are shown as the larger symbols in the shaded area) using the Park and Ang (1985) damage index; (d) Mosalam et al. (1997) curves for low-rise RC MRF using inter-storey drift values to define damage.

Hybrid vulnerability curves attempt to compensate for the scarcity of observational data, subjectivity of judgemental data and modelling deficiencies of analytical procedures, by combining data from the different sources. The most complete discussion for this type of curve is given by Kappos et al (1998). Existing examples of hybrid curves typically involve the modification of analytical or judgement based relationships with observational data. In most cases, the data deriving from the additional sources is, however, very limited in quantity and scope. This is highlighted in a review of thirty earthquake damage estimation methods proposed for use in America from 1972 to 1984 by Reitherman (1985), where it is seen that the reviewed methodologies rely almost entirely on

observations of damage made by Algermissen and Steinbrugge (1978) during the San Fernando earthquake (USA, 1971). The curves proposed in ATC-13 (ATC 1985) and ATC-40 (1996), though based heavily on expert opinion, also incorporate limited observational data from the San Fernando (USA, 1971) and Northridge (USA, 1994) earthquakes, respectively. In the latter case, the observational and judgemental data is further supplemented with experimental test results. The experimental source has gained importance in recent years, as the testing of large and reasonably realistic structures has become more frequent. However, due to the time and costs associated with full scale testing, and the non-conclusiveness of small scale testing on similitude grounds, a very limited number of parameters can be investigated, with parametric variations not being possible. Therefore, experimental data is currently only used for verification purposes rather than as an additional source of building damage statistics. Singhal and Kiremidjian (1997) adopt a Bayesian technique to update analytical curves for low-rise frames with observational damage data from a tagging survey of only 84 buildings affected by the Northridge earthquake (USA, 1994). As previously stated, observations taken from a single earthquake event will cover only a small range of ground motions. Nevertheless, their inclusion may have a significant influence on the vulnerability plot and may result in a greater uncertainty, as highlighted in Figure 2. 2c. It is therefore concluded that the consideration of multiple data sources is necessary for the correct determination of vulnerability curve reliability.

2.2.2 Structural Unit and Damage Measure

The choice of damage scale and measure is fundamental to the vulnerability curve generation. In the case of empirical curves, it is essential that the damage scale used is clearly defined in terms of the damage expected in the structural and non-structural elements of buildings with different lateral load resisting systems. The use of performance states defined by economic criteria, (e.g. Algermissen and Steinbrugge 1978, Miyakoshi et al. 1997), can preclude the vulnerability curve application to locations other than those considered. Furthermore, a time dependency may be introduced if the limit-state values are linked to the financial

situation in place at the time of the earthquake. No simple relationship exists between structural damage and monetary risk, and the direct use of performance limit states based property loss are seen by Miyakoshi et al (1997) to under-predict the damage observed in RC buildings after the Kobe earthquake (Japan, 1995). The use of apartments as the structural unit by Orsini (1999), is not justified physically if soft-storey failure mechanisms are considered and, given the approximate nature of existing inventory data, results in limited practical application. RC buildings and global structural damage must therefore be used as the structural unit and loss evaluation parameter, respectively.

It is practically impossible to obtain an observational damage distribution that provides a compositional match to the building population within an assessed urban area. Therefore, vulnerability curves derived from observational data that aggregates the damage incurred by different structural systems, (i.e. considers populations with varied building composition) do not provide a reliable estimate of the seismic risk for all locations. A series of vulnerability curves for different structural systems, building heights and design codes are necessary for application in conjunction with an inventory of the local building composition, in the determination of the seismic risk in an urban area. The lack of large quantities of refined observational data promotes the use of analytical techniques for the derivation of such curves. Indeed, none of the existing vulnerability curve sources provides a comprehensive set of vulnerability curves for the diverse range of buildings found in Europe. Hence, at present, if a vulnerability assessment were to be made, multiple vulnerability curve sources might have to be considered.

In order for vulnerability curves to be used in a performance-based framework, it is desirable that the selected damage scale is defined in terms of at least three damage limit states, corresponding to serviceability, damage control and collapse prevention. Miyakoshi et al. (1997) and Yamazaki and Murao (2000), both fall short of this requirement. The scale's limit states must be clearly defined in terms of the damage expected in the structural and non-structural elements of buildings with different lateral load resisting systems. For analytical curve derivation, the damage scale should further be calibrated to a measurable structural response parameter. The choice of response parameter for the calibration and its values, are

important in determining the reliability of the vulnerability relationships. The use of structural ductility for this parameter, as in Onose (1982), is deemed unsuitable due to its inability to detect failures arising from damage concentrated at the storey level. Maximum inter-storey drift is able to detect such failure modes and is used by Mosalam et al. (1997). However, the latter use engineering judgement to assign parameter values to the limit states that are observed to be very low and result in curves, which despite being labelled “Heavy” and “Major”, are unsuitable for high damage state assessments. An in depth discussion of damage scales and indices for vulnerability curve derivation is given in Chapter 3.

2.2.3 Ground Motion Characterisation

In a vulnerability relationship, the horizontal axis represents the seismic demand. The parameter chosen to represent the seismic demand should be capable of representing the influence of source, path and site on the strong ground motion and should be evaluated independently of the seismic vulnerability of the building stock to which it is imposed. Nevertheless, many existing vulnerability curves adopt intensities to characterise the ground motion, which are evaluated from observations of earthquake effects and thus incorporate elements of structural vulnerability, (e.g. ATC 1985). This is also true of the Parameterless Seismic Intensity (PSI) scale, which is specifically proposed by Pomonis et al. (1992) for use in vulnerability curve derivation and is adopted by both Spence et al. (1992) and Orsini (1999). The use of a ground motion parameter that is related to structural damage, results in a reduced data scatter and misleading sense of confidence in the risk assessment results.

A detailed discussion of ground motion characterisation in vulnerability studies is given in Chapter 4, where it is concluded that only parameters that can be evaluated from ground motion records, and for which reliable attenuation relationships exist, are suitable for the representation of seismic demand in both empirical and analytical vulnerability curves. In practice this means using either peak or spectral ground motion values. However, in choosing a ground motion parameter, the vulnerability curve derivation methodology must also be

considered. For example, if inelastic capacity and demand is included in the generation of the analytical damage statistics, or if observational damage statistics are used, characterisation of the horizontal axis of the curves via inelastic indices of ground motion is superfluous. Care must also be taken to ensure that the methodology adopted for the curve derivation does not alter the ground motion parameter values and their distributions. For example, Yamazaki and Murao (2000) adopt an iterative derivation methodology wherein the peak ground velocity (*pgv*) distribution derived by Yamaguchi and Yamazaki (1999) for the Nada Ward after the Kobe (Japan, 1995) earthquake, is modified using preliminary vulnerability functions. The resulting *pgv* distribution is then re-applied in the derivation of new fragility curves. The process of *pgv* re-estimation and vulnerability curve updating is repeated several times and the final *pgv* distribution obtained differs significantly from the original. The axes of these vulnerability curves are effectively inter-dependent and the horizontal axis can no longer be regarded as representative of the observed strong ground motion.

2.2.4 Verification with Observational Building Damage Data

The validity of vulnerability relationships should be determined through comparison with actual observations of post-earthquake damage. Surprisingly, existing vulnerability curves are rarely verified for field observations other than those used in their derivation. The limited nature of the empirical data incorporated in hybrid curves has been discussed above. Of the reviewed curves, only Singhal and Kiremidjian (1997) and Onose (1982) make specific attempts at verification with limited quantities of observational post-earthquake damage data. Onose (1982) compares his damage estimates with those observed in three districts of Sendai city after the Miyagi-ken Oki earthquake (Japan, 1978). Singhal and Kiremidjian (1997) verify their vulnerability curves for low-rise frames with damage data from the Northridge earthquake (USA, 1994). In both cases, a very limited range of ground motion values is covered, almost no data is available for high damage states. Furthermore, in the latter case, the data refers to observations made in only 84 buildings. Notwithstanding the small amount of data, both report a limited success in the damage prediction.

2.3 Comparison of Existing Vulnerability Curves with Observational Damage Data

It has been previously stated that vulnerability relationships developed for non-European countries may be site-specific due to inherent assumptions as to the building population composition. However, it is possible that if the vulnerability relationships distinguish between the seismic resistance of structures with distinct lateral-load resisting system, heights and seismic design codes, they may be used in the assessment of European buildings with similar structural characteristics. This statement assumes that a sufficient degree of structural uncertainty is included in the derivation of the curves to overcome any differences in construction practice and detailing that may distinguish the different locations. To assess the validity of this assumption, a comparison is made between existing vulnerability curve damage predictions and observational damage data collected from the literature for European RC structures.

The vulnerability relationships used for the comparison are chosen from the literature as those most pertinent to European buildings. The observational damage data used for the comparison derives from the empirical database described in Chapter 5. The building damage distributions are converted to the damage scale associated with the analysed curves, by assigning Homogenised Reinforced Concrete Damage index values (DI_{HRC} , see Section 5.3.1) to each of their damage states using Tables 3.2 to 3.6. The corresponding maximum inter-storey drift ratio ($ISD_{max\%}$) limits, are calculated using Equations 3.20 to 3.23 in Section 3.4.3. In the case of Singhal and Kiremidjian (1997), the conversion between Park and Ang (1985a) damage index and $ISD_{max\%}$ is carried out using the conversion equations derived by the authors. The DI_{HRC} and $ISD_{max\%}$ limits assigned to the damage states of each existing curve set are presented in Table 2. 2 These are used to obtain the corresponding damage state exceedence probabilities, from the cumulative distribution functions fit to each observational damage dataset, as described in Section 5.3.1.

Existing Curve Source	Structure Type ⁽¹⁾	Damage Limit State ⁽²⁾	Vulnerability Curve Equation	Assigned Lower Limit DI_{HRC}				Corresponding ISD_{max} (%)			
				N-D MRF ⁽³⁾	IF ⁽⁴⁾	S-W ⁽⁵⁾	General ⁽⁶⁾	N-D MRF	IF	S-W	General
Spence et al. (1992)	Non-seismic Frames	D1	y=inorm (PSI, 7.9, 2.5)	0	0	0	0	0.33	0.05	0.26	0.13
		D2	y=inorm (PSI, 10.3, 2.5)	20	30	10	20	0.58	0.19	0.33	0.28
		D3	y=inorm (PSI, 11.3, 2.5)	40	50	40	40	1.03	0.47	0.72	0.56
		D4	y=inorm (PSI, 12.9, 2.5)	70	70	80	70	2.43	1.14	1.99	1.65
		D5	y=inorm (PSI, 14.1, 2.5)	100	100	100	100	5.75	4.34	3.31	4.84
Orsini (1999)	RC Apartments	D2	y=inorm (PSI, 0.092, 0.87)	20	30	10	20	0.58	0.19	0.33	0.28
		D4	y=inorm (PSI, 0.696, 0.657)	70	70	80	70	2.43	1.14	1.99	1.65
		D6	y=inorm (PSI, 1.022, 0.567)	80	80	90	80	3.24	1.78	2.56	2.36
		D7	y=inorm (PSI, 1.318, 0.638)	90	90	95	90	4.32	2.78	2.91	3.38
		D8	y=inorm (PSI, 1.476, 0.689)	100	100	100	100	5.75	4.34	3.31	4.84
ATC-13 (1985)	Low-rise non-ductile bare MRF	Light	y = ilognorm (MMI, 1.79, 0.075)	10	N/A	N/A	N/A	0.44	N/A	N/A	N/A
		Moderate	y = ilognorm (MMI, 2.08, 0.05)	35	N/A	N/A	N/A	0.89	N/A	N/A	N/A
		Heavy	y = ilognorm (MMI, 2.28, 0.08)	60	N/A	N/A	N/A	1.83	N/A	N/A	N/A
		Major	y = ilognorm (MMI, 2.71, 0.188)	80	N/A	N/A	N/A	3.24	N/A	N/A	N/A
		Destroyed	y = ilognorm (MMI, 2.75, 0.09)	100	N/A	N/A	N/A	5.75	N/A	N/A	N/A
ATC-13 (1985)	Low-rise non-ductile bare MRF	Light	y = ilognorm (MMI, 1.77, 0.09)	10	N/A	N/A	N/A	0.44	N/A	N/A	N/A
		Moderate	y = ilognorm (MMI, 2.07, 0.05)	35	N/A	N/A	N/A	0.89	N/A	N/A	N/A
		Heavy	y = ilognorm (MMI, 2.25, 0.08)	60	N/A	N/A	N/A	1.83	N/A	N/A	N/A
		Major	y = ilognorm (MMI, 2.46, 0.05)	80	N/A	N/A	N/A	3.24	N/A	N/A	N/A
		Destroyed	y = ilognorm (MMI, 2.75, 0.09)	100	N/A	N/A	N/A	5.75	N/A	N/A	N/A

⁽¹⁾Type of RC structures for which the existing curves are derived; ⁽²⁾Damage limit state definition for each curve, (see also Table 2.1); ⁽³⁾Non-ductile bare moment resisting frame; ⁽⁴⁾Infilled moment resisting frame; ⁽⁵⁾Shear-wall structure; ⁽⁶⁾General reinforced concrete structure.

Table 2. 2: Summary of the HRC damage index values (DI_{HRC}) assigned to each damage state of existing vulnerability curves, for their comparison with the observational post-earthquake damage statistics presented in Chapter 5. The DI_{HRC} values are assigned using the damage state conversions in Tables 3.2 to 3.6, and the corresponding maximum inter-storey drift ratios, for different RC structures, are calculated using Equations 3.20 to 3.23.

Existing Curve Source	Structure Type ⁽¹⁾	Damage Limit State ⁽²⁾	Vulnerability Curve Equation	Assigned Lower Limit D_{LRC}				Corresponding ISD_{max} (%)			
				N-D MRF ⁽³⁾	IF ⁽⁴⁾	S-W ⁽⁵⁾	General ⁽⁶⁾	N-D MRF	IF	S-W	General
Singhal and Kiremidjian (1997)	Low-rise bare MRF	Minor Moderate Severe Collapse	$y = \text{lognorm}(Sa, 0.31, 0.40)$ $y = \text{lognorm}(Sa, 0.56, 0.44)$ $y = \text{lognorm}(Sa, 1.00, 0.50)$ $y = \text{lognorm}(Sa, 1.39, 0.60)$	38.6	N/A	N/A	N/A	0.99	N/A	N/A	N/A
				52.7	N/A	N/A	N/A	1.48	N/A	N/A	N/A
				76.7	N/A	N/A	N/A	2.95	N/A	N/A	N/A
				97.8	N/A	N/A	N/A	5.40	N/A	N/A	N/A
Singhal and Kiremidjian (1997)	Mid-rise bare MRF	Minor Moderate Severe Collapse	$y = \text{lognorm}(Sa, -0.29, 0.30)$ $y = \text{lognorm}(Sa, 0.053, 0.32)$ $y = \text{lognorm}(Sa, 0.67, 0.35)$ $y = \text{lognorm}(Sa, 1.17, 0.37)$	40.4	N/A	N/A	N/A	0.99	N/A	N/A	N/A
				52.7	N/A	N/A	N/A	1.48	N/A	N/A	N/A
				74.9	N/A	N/A	N/A	2.95	N/A	N/A	N/A
				95.1	N/A	N/A	N/A	5.40	N/A	N/A	N/A
Mosalam et al. (1997)	Low-rise bare MRF	Insignificant Moderate Heavy Major	$y = 1 - \exp(-87pga^2)$ $y = 1 - \exp(-244pga^{3.6})$ $y = 1 - \exp(-220pga^{4.9})$ $y = 1 - \exp(-143pga^{5.5})$	-17.1	N/A	N/A	N/A	0.20	N/A	N/A	N/A
				14.8	N/A	N/A	N/A	0.50	N/A	N/A	N/A
				39.0	N/A	N/A	N/A	1.00	N/A	N/A	N/A
				53.1	N/A	N/A	N/A	1.50	N/A	N/A	N/A
Mosalam et al. (1997)	Low-rise infilled MRF s	Insignificant Moderate Heavy Major	$y = 1 - \exp(-24pga^{1.2})$ $y = 1 - \exp(-12pga^{1.8})$ $y = 1 - \exp(-3pga^{2.8})$ $y = 1 - \exp(-2pga^{3.6})$	-100.8	N/A	N/A	N/A	0.02	N/A	N/A	N/A
				-64.8	N/A	N/A	N/A	0.05	N/A	N/A	N/A
				-37.5	N/A	N/A	N/A	0.10	N/A	N/A	N/A
				-21.6	N/A	N/A	N/A	0.15	N/A	N/A	N/A

⁽¹⁾Type of RC structures for which the existing curves are derived; ⁽²⁾Damage limit state definition for each curve, (see also Table 2.1); ⁽³⁾Non-ductile bare moment resisting frame; ⁽⁴⁾Infilled moment resisting frame; ⁽⁵⁾Shear-wall structure; ⁽⁶⁾General reinforced concrete structure.

Table 2.2: Continued.

Various ground motion parameters are adopted in the existing curve definition. Peak and spectral ground motion parameter values are assigned to each of the observational damage data locations, either using measurements of ground motion made at nearby recording stations, or using adequate attenuation relationships. The adopted attenuation relationships are chosen according to the location of the survey and the relevant fault mechanism, and are further discussed in Section 4.3. In the case of Spence et al (1992) and Orsini (1999), the horizontal axis of the curves is converted from PSI to peak ground acceleration (pga) using empirical relationships derived by Pomonis et al. (1992). Conversion from the Modified Mercalli Intensity (MMI) scale to elastic spectral acceleration at 5% critical damping ($S_{a5\%}(T)$) is also carried out for the ATC-13 (ATC 1985) curves using the empirical relationships derived for California by Singhal and Kiremidjian (1997).

Source	Structure type	Comparison with observations for European structures of same lateral-load resisting system.				Comparison with observations for structures of same type, country and earthquake.			
		Sample Size		$\mu(\epsilon)^{(3)}$	$R^2^{(4)}$	Sample Size		$\mu(\epsilon)^{(3)}$	$R^2^{(4)}$
		DS ⁽¹⁾	Bldgs ⁽²⁾			DS ⁽¹⁾	Bldgs		
Spence et al. (1992)	Non-seismic Bare MRF	28	3306	0.39	0.55	-	-	-	-
Orsini (1999)	Bare MRF	26	293636	0.64	0.50	16	2208	0.77	0.50
ATC-13 (ATC 1985)	Non-ductile low-rise MRF	30	3682	0.44	0.52	4	1476	1.82	0.49
ATC-13 (ATC 1985)	Non-ductile mid-rise MRF	8	290832	0.31	0.42	-	-	-	-
Singhal & Kiremidjian (1997)	Low-rise bare MRF	12	1842	0.10	0.38	4	1476	0.15	0.44
Singhal & Kiremidjian (1997)	Mid-rise bare MRF	8	290832	0.26	0.52	-	-	-	-
Mosalam et al. (1997)	Low-rise bare MRF	14	291847	0.41	0.44	4	1476	1.54	0.49
Mosalam et al. (1997)	Infilled Frame	8	1544	0.78	0.39	-	-	-	-

⁽¹⁾Number of datasets; ⁽²⁾Total number of buildings in the datasets; ⁽³⁾ $\mu(\epsilon)$ = mean error in prediction; ⁽⁴⁾ R^2 = coefficient of correlation of the observational data with the fitted curves.

Table 2. 3: Comparison of existing vulnerability curves with observational post-earthquake building damage statistics.

Two sets of observational damage data are used for the comparison and the results are summarised in Table 2. 3. In the first instance (*Case I*), data relating to populations of European structures with the same lateral load resisting system as assumed by the existing curves, is used. In the second case (*Case II*), the comparison is carried out using observations made only for the structures, countries and earthquakes specified by the existing vulnerability curves as being within their limit of application. A fairly poor correlation is generally seen between the observed statistics and the existing curves (Table 2. 3). Due to the limited amount of observational data, the value of the coefficient of correlation is regarded to have little significance. The validity of the curves must therefore be assessed from observation of the mean prediction error. It is evident from Table 2. 3, that the empirical vulnerability curves proposed by Orsini (1999) show no correlation to the observed data in either *Cases I* or *II*. The empirical vulnerability curves of Spence et al. (1992), show significantly better correlation, but still exhibit large errors (Figure 2. 3a). The non-empirical curves show a closer fit to the observations than the empirical curves studied, with the analytical vulnerability curves of Singhal and Kiremidjian (1997) giving the best results, (Figure 2. 3b).

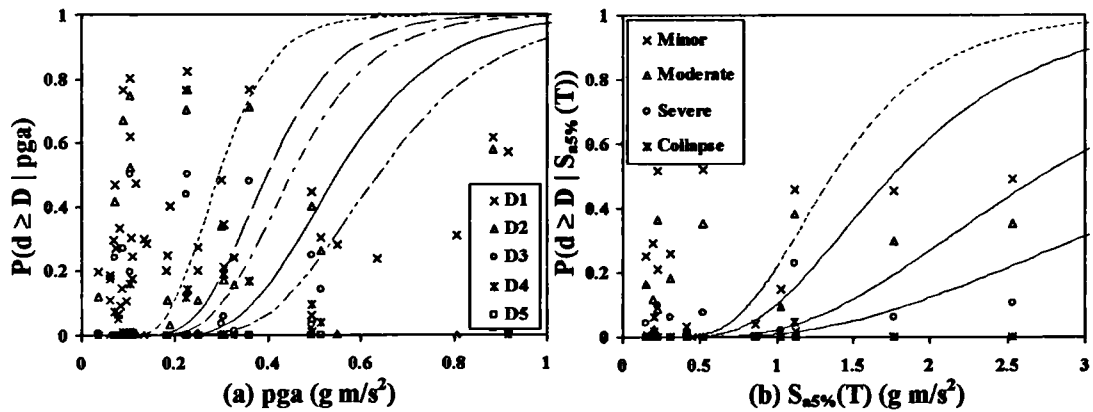


Figure 2. 3: Comparison of existing curves with unweighted observational damage data for European structures with the same lateral load resisting system, as assumed by the curves (a) Spence et al (1992); (b) Singhal and Kiremidjian (1997) Low-rise RC MRF.

A significant degree of scatter may be attributed to the natural variation in the structural characteristics present within each building lateral-load resisting system category. All existing curves yield consistently larger mean errors in *Case II* than

in *Case I*. Although based on very few observations, this suggests that the uncertainty in prediction associated with the application of the curves to building types specific to the country of their origin, is comparable to that obtained from their application to European buildings of similar structural system. This implies that the use of non-European vulnerability curves in a European risk assessment is indeed possible, given appropriate consideration of structural similitude. Furthermore, it would seem to justify the combination of observational damage data for similar constructions deriving from different countries and earthquakes in the derivation of new empirical relationships.

Despite these positive conclusions, the reviewed curves are not recommended for use in a European seismic risk assessment. The small quantity and limited ground motion range of the data used herein for the verification, results in misleadingly high correlation coefficient values, with large errors in prediction seen in all cases (Table 2. 3). The poor representation of observed building damage by the curves in the case of single earthquake events is clearly evident in Figures B.7 and B.10 (Appendix B). These observed prediction errors are believed to arise from a combination of errors inherent in the empirical data used for the comparison, and from the assumptions made regarding damage data source, structural unit, damage and ground motion characterisation, by the existing curves. From the above discussion, it is concluded that there are no vulnerability curves, at present, that meet all the criteria necessary for a reliable European seismic risk assessment tool. The need for new methodologies for the development of empirical and analytical vulnerability curves for Europe is therefore established.

Chapter 3

Damage Characterisation

3.1 Introduction

The choice of a damage scale for the assessment of buildings is fundamental to the definition of vulnerability curves. To allow a multi-level performance assessment, the adopted scale must consist of at least 3 limit states. Each damage state is required to be defined in terms of the structural and non-structural damage expected in different building classes. Use of acceptability criteria based on loss of function or value, instead of structural damage, introduces a time and location dependency that can limit the scale application and lead to erroneous physical damage predictions (Miyakoshi et al. 1997). The use of a single damage description for all buildings is also incorrect, as the progression of damage and rate of attainment of the damage limit states varies for structures with different dynamic characteristics. Furthermore, the inclusion of non-structural damage in the global limit state definition is important, as it can pose serious risk to life safety at large deformations, and at low deformations is the main reason for loss of function. In the generation of empirical vulnerability curves, consideration of the performance of different building types in the damage scale definition, aids the combination of observational damage statistics from different countries and events. For this purpose, a correlation between the chosen scale and other existing damage scales is also required. Furthermore, if the scale is to be used in the derivation of analytical curves, the descriptive damage states must be calibrated with a measurable structural response parameter.

A variety of descriptive damage scales for the evaluation of reinforced concrete buildings exist in the published literature. Those most commonly used for the derivation of empirical and judgement-based vulnerability curves are proposed by the Applied Technology Council (ATC-13 and ATC-40), the Federal Emergency Management Agency (FEMA273 and HAZUS99), Structural Engineers Association of California (Vision-2000), and the Architectural Institute of Japan (AIJ 1995).

Furthermore, in Europe the EMS-98 (CdE 1998) and MSK (Medvev and Sponheuer 1969) damage scale descriptions are frequently adopted. None of these existing damage scales is however found to be adequate for use in the derivation of both empirical and analytical vulnerability curves. Unclear or ambiguous wording is used by some scales in the definition of their damage limit states. This leads to misinterpretation when employed in the evaluation of post-earthquake or experimental structural damage. Many scales do not distinguish between structures with different lateral load resisting systems (e.g. MSK scale, see Figure 3. 1). Even when disparate damage state definitions are used for shear wall and frame structures, a common scale is often adopted in the evaluation of bare and infilled frames. Furthermore, damage scales derived for use in non-European countries often do not include damage definitions for all the structural classes composing the European building stock. For example, ATC-13 (ATC 1985) and FEMA 273 (ATC 1997) do not include damage descriptions for moment resisting frames with infills, which is the most common construction type in Europe. Moreover, the contribution of non-structural damage to the global building performance state is not recognised or explicitly quantified in many existing scales.

	Grade 1: Negligible to slight damage (no structural damage) fine cracks in plaster over frame members and in partitions.
	Grade 2: Moderate damage (slight structural damage, moderate non-structural damage) hair-line cracks in columns and beams; mortar falls from the joints of suspended wall panels; cracks in partition walls; fall of pieces of brittle cladding and plaster.
	Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage) cracks in columns with detachment of pieces of concrete, cracks in beams.
	Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage). severe damage to the joints of the building skeleton with destruction of concrete and protrusion of reinforcing rods; partial collapse; tilting of columns.
	Grade 5: Destruction (very heavy structural damage) total or near total collapse.

Figure 3. 1: The MSK scale of damage for buildings.

In codified scales, the assignment of structural response parameter values to damage limit states is infrequent. When done, the values are either assigned via engineering

judgement, or through the interpretation of a small number of experimental test results that do not concern the full range of structures covered by the scales. For example, the experimental data used in the calibration of FEMA 273 (ATC 1997) is limited to tests on low-rise concrete shear walls, new RC structures and non-structural elements published in Kircher et al. (1997), Ferritto (1983), and Division of the State Architect Report (DSA 1996), respectively. The lack of an extensive experimental calibration leads to the assignment of over-conservative values of response parameters to damage limit states (see Section 3.4.1). Regardless of the amount of experimental data used, the implicit inclusion of local construction practice and materials in the calibration means that damage scales proposed for use in USA and Japan are not appropriate for the assessment of European buildings. Furthermore, in the case of rehabilitation codes that adopt force-based methods of structural assessment, the parameter values proposed may be unreliable even when empirically calibrated. This is due to the fact that they may represent specified reductions in building strength, rather than observed states of damage. In view of these facts, it is concluded that a new descriptive damage scale is required for the characterisation of damage in European reinforced concrete building populations.

3.2 Definition of a New Damage Scale for RC Buildings

A new damage scale named the Homogenised Reinforced Concrete damage scale (HRC-Scale) is proposed for use in the derivation of vulnerability curves. The scale is subdivided into seven damage states and is presented in Table 3.1. Existing scales and experimental observations reported in the literature are considered in order to define each damage state in terms of the typical structural and non-structural damage expected in the four main types of reinforced concrete structure found in Europe. It is noted that the bare MRF structure class is considered to include frames with weak infills (e.g. made of hollow ceramic/clay blocks) that are not expected to significantly contribute to the lateral resistance of the structure. The infilled MRF structure class only contains frames with infills made of brick masonry (reinforced and unreinforced) or concrete blocks that are well connected to the surrounding frames and that are expected to provide a significant lateral-load resistance.

D_{HRC}	DAMAGE STATE	DUCTILE MRF	NON-DUCTILE MRF	INFILLED MRF	SHEAR-WALL
0	None	No damage	No damage	No damage	No damage
10	Slight	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills	Fine cracks in plaster partitions/infills
20	Light	Start of structural damage	Start of structural damage	Cracking at wall-frame interfaces	Start of structural damage
30		Hairline cracking in beams and columns near joints (<1mm)	Hairline cracking in beams and columns near joints (<1mm)	Cracking initiates from corners of openings	Hairline cracking on shear-wall surfaces & coupling beams
40				Diagonal cracking of walls. Limited crushing of bricks at b/c connections	Onset of concrete spalling at a few locations
50	Moderate	Cracking in most beams & columns	Flexural & shear cracking in most beams & columns	Increased brick crushing at b/c connections	Most shear walls exhibit cracks
60		Yielding in a limited number of beams & columns	Yielding in a limited number of beams & columns	Start of structural damage	Some walls reach yield capacity
70		Larger flexural cracks & start of concrete spalling	Shear cracking & spalling is limited	Some diagonal shear cracks in members especially for exterior frames	Increased diagonal cracking & spalling at wall corners
80	Extensive	Ultimate capacity reached in some elements – large flexural cracking, concrete spalling & re-bar buckling	Loss of bond at lap-splices, bar pull-out, broken ties	Extensive cracking of infills, falling bricks, out-of-plane bulging	Most shear walls have exceeded yield, some reach ultimate capacity, boundary element distress seen.
90		Short column failure	Main re-bar may buckle or elements fail in shear	Partial failure of many infills, heavier damage in frame members, some fail in shear	Re-bar buckling, extensive cracking & through-wall cracks. Shear failure of some frame members
100	Partial Collapse	Collapse of a few columns, a building wing or single upper floor	Shear failure of many columns or impending soft-storey failure	Beams &/or columns fail in shear causing partial collapse. Near total infill failure	Coupling beams shattered and some shear walls fail
	Collapse	Complete or impending building collapse	Complete or soft-storey failure at ground floor	Complete or impending building collapse	Complete or impending building collapse

Table 3. 1: The HRC-Scale: Typical damage expected in ductile, non-ductile and infilled RC moment resisting frames and in RC shear-wall structures

<i>DI_{HRC}</i>	HRC	HAZUS 1999	VISION 2000	FEMA 273	EMS98	MSK	AJJ	ATC-13	ATC- 21	EPPO
0	None	No damage								
10	Slight	Slight	Fully operational	Immediate occupancy	Grade 1	D1	Light	Slight	Green Tag	Green Tag
20	Light		Operational	Damage control	Grade 2	D2	Minor	Light		
30								Light		
40								Moderate		
50	Moderate	Moderate	Life safe	Life safe	Grade 3	D3	Moderate	Moderate	Yellow Tag	Yellow Tag
60										
70										
80	Extensive	Extensive	Near collapse	Limited safety	Grade 4	D4	Major	Heavy	Red Tag	Red Tag
90				Collapse prevention				Major		
100	Partial Collapse		Collapse					Partial collapse		
	Collapse		Collapse limit state							

Table 3. 2: The approximate equivalence between existing damage scales and the HRC-Scale for general RC structures.

The generation of empirical vulnerability curves requires the re-interpretation of post-earthquake damage survey data in terms of the HRC damage scale. This requires the correlation of the HRC-scale with other existing damage scales. Poland (1999) attempted to correlate three damage scales in America, however there is no such published relationship of equivalence on an international level. Four approximate correlation tables, (corresponding to the different structural systems of Table 3.1), are therefore drawn between the HRC and other existing damage scales from the authors' personal interpretation of the relative limit state damage descriptions. The correlation tables are presented in Tables 3.3 to 3.6. A further correlation table for general RC structures is shown in Table 3.2. The latter is proposed for use, when the predominant lateral-load resisting system of a building population is ill defined by the post-earthquake damage surveys.

The response parameter most commonly used to describe damage at the storey level is the maximum inter-storey drift ratio ($ISD_{max\%}$). Many researchers have found that $ISD_{max\%}$ is closely correlated to observed damage, (Scawthorn et al 1981). Unlike the case of member deformation, inter-storey drift response is capable of detecting global, storey-level and non-structural damage. No information regarding local damage is given by $ISD_{max\%}$ but its overall effect on the structure is implicitly included in the parameter. $ISD_{max\%}$ is therefore suitable for the definition of a wide range of structural damage limit states. Maximum inter-storey drift ratio values are used by Mosalam et al. (1997) to characterise damage in their analytical vulnerability curves. FEMA273 (ATC 1997) also associates $ISD_{max\%}$ values to different levels of seismic performance. Maximum inter-storey drift ratio values, given in the literature as being characteristic of different degrees of damage in structural and non-structural elements, are summarised in Tables D.1 to D.3 of Appendix D. These tables clearly show the large variation in values within each damage category which arises from the different building configurations, seismic detailing and construction materials.

At the structural level, the overall drift ratio and structural displacement ductility are typically used to define the damage state. The two measures are linked, as the structural ductility is the ratio of the overall to yield drift. Uncertainty in the definition of structural yield makes the drift ratio the preferred parameter for use in building damage scale calibration. Many sources assign drift ratio values to damage limit states, for structures with different lateral-load resisting systems and design codes, (e.g. FEMA 273, Kircher et al.1997, HAZUS99). However, in Table D.3, (Appendix D), a larger variation in the values proposed for similar structures by different authors, is observed for total drift, than for $ISD_{max\%}$. The overall drift ratio is only representative of global building damage when the structure deforms linearly with height. In cases where structural dynamic response is strongly affected by higher modes of vibration, the overall drift ratio may decrease, despite a continued increase in inter-storey drift and observed damage. Moreover, parameters defined on a structural level are unsuitable for the assessment of non-capacity designed buildings, as they are incapable of accounting for concentrations of damage at a single storey. Also, most parameter values proposed in the literature, are evaluated from opinion and may not yield reliable estimates of damage. These may, therefore, not be used in the damage scale calibration.

From the above discussion, it is concluded that the maximum inter-storey drift ratio, ($ISD_{max\%}$), provides the optimum parameter for the representation of both global and storey-level building damage. The response parameter may be easily evaluated from the results of structural analyses. Furthermore, a relatively large amount of experimental data is available for use in its correlation to the proposed damage index. Maximum inter-storey drift ratio will therefore be used in the derivation of both the analytical and empirical vulnerability curves.

3.4 Experimental Calibration of the HRC Damage Scale

Different mechanisms contribute to the occurrence and progression of damage, (e.g. flexural cracking, shear cracking, loss of bond and the interaction between structural and non-structural elements). The quantification of damage via one or more response parameters is therefore prone to large uncertainties. Numerous measurements of the damage parameter and associated observed damage state are required for a reliable damage index calibration. Ideally, damage observations and response measurements from instrumented buildings subjected to earthquakes should be used for the calibration. However, as such data is not currently available, the results of dynamic or pseudo-dynamic experiments on large-scale structural models (representative of the system for which the damage index calibration is desired) should be used. It is noted that soil-structure interaction and strain rate effects that may affect the accumulation of damage within a structure, may not be represented within the experiments. However, the sensitivity of a storey-level peak response parameter such as $ISD_{max\%}$ to the omission of these effects is expected to be less than that of local damage indices evaluated from the moment-rotation response of structural members.

With the exception of the Park and Ang (1985a) damage index, all existing damage indices are calibrated using few experimental observations. This is mainly due to the prohibitive costs associated with the dynamic testing of large structures. Existing reports on shake-table and pseudo-dynamic testing of structures are few in number and cover only a limited number of lateral load resisting systems, configurations, and seismic codes. Even in the case of local damage indices, available experimental data for calibration is scarce. Extensive cyclic testing of reinforced concrete members has

only recently been carried out. Hence, a very limited number of section, member configurations, and axial loading conditions are covered by existing experiments. Moreover, most cyclic member tests concern well-detailed structural elements, failing in a flexurally dominated manner (Rossetto 2002). This can lead to a poor representation of shear-dominated or mixed flexure-shear failure modes by existing local damage indices. In both structure and member tests, damage observations are not commonly made at times other than yield or collapse. Consequently, large uncertainties are associated with response parameter values assigned to intermediate damage states by existing damage scales.

The practice of calibrating new damage indices using existing damage indices is not correct. Within this study, an independent calibration of the HRC-scale is carried out, using experimental observations of maximum inter-storey drift ratio ($ISD_{max\%}$) and corresponding structural damage. Preliminary relationships between the HRC damage index (DI_{HRC}) and $ISD_{max\%}$ are found using data from published experimental reports concerning the dynamic testing of structural models. These relationships are validated and updated using the results of pseudo-dynamic tests on full-scale reinforced concrete structures, carried out within the Innovative Seismic Design Concepts for New and Existing Structures (ICONS) European Union funded network. Although the latter experiments yield a limited number of observations, they provide a valuable source of reliable damage index calibration data due to the highly controlled conditions of the experiment, the large scale of the models and the use of buildings with design and detailing typical of the existing European construction. The final $DI_{HRC} - ISD_{max\%}$ equations are used to define the boundaries of the HRC damage states in terms of maximum inter-storey drift ratio. The $ISD_{max\%}$ values assigned by the derived relationships to existing damage scale limit-states, are compared to those prescribed in the latter scale definitions.

3.4.1 Preliminary Calibration of the HRC Damage Scale

A new calibration of the proposed damage index (DI_{HRC}) with $ISD_{max\%}$ is carried out. The calibration data is assembled from published reports and papers and consists of descriptions of structural damage and corresponding inter-storey drift ratio ($ISD_{max\%}$)

observed at different times during the dynamic testing of reinforced concrete structure models. Through interpretation of the reported global damage, HRC-Damage Index (DI_{HRC}) values are assigned to the test specimens at different times during the experiments. These are reproduced in Table D.1 of Appendix D. $ISD_{max\%}$ values obtained from tests on non-structural infill-walls are also reported in Table D.2 of Appendix D. The deformation capacity of infill-walls determines the permissible inter-storey drift limits of the lower damage limit states in reinforced concrete infilled-frames. In cases where the inter-storey-drifts are not reported, it is often possible to obtain values of the maximum observed building drift ratios. These can be assumed equal to $ISD_{max\%}$ in cases where the building response is dominated by its first vibration mode. Hence, the drift ratios obtained for low-rise structures are used as an additional source of calibration data.

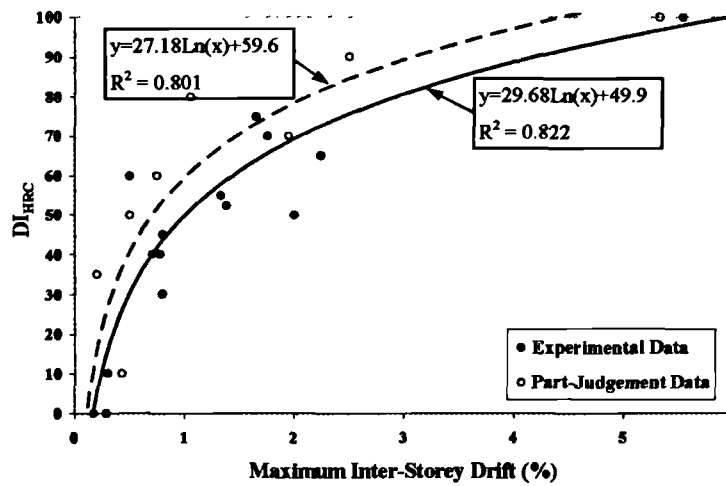


Figure 3. 2: The DI_{HRC} - ISD_{max} relationships for general RC structures, derived using experimental data only, and experimental with part-judgement based data, (the latter being obtained by combining experimental data with that obtained from published codes, see Table D.3, Appendix D).

Structural Type	Number of $ISD_{max\%}$ - DI_{HRC} Experimental Observations				
	Non-ductile MRF	Ductile MRF	Infilled MRF	Shear-wall Structures	All Structures
Preliminary Eq.ns	20	4	14	6	44
ICONS Eq.ns	8	-	24	8	-
Final Eq.ns	28	4	24	6	62

Table 3. 7: Summary of the number of experimental observations used in the regression for the $ISD_{max\%}$ - DI_{HRC} relationships for different RC structure types.

Drift and inter-storey drift limits in worldwide codes of practice are summarised in Bertero et al. (1991). These and other damage state inter-storey drift limits in the published literature are reproduced in Table D.3 of Appendix D. Most are defined solely on the basis of experience and engineering judgement, whilst others are partly based on experimental research. For example, in FEMA273, the inter-storey drift ranges are based heavily on the work of Kircher et al (1997) who, in his research, also refers to the experimental work of others, (e.g. Ferrito 1983). These inter-storey drift values are over-conservative, and when combined with the experimental data, result in a relationship with a more rapid rate of damage development with drift than when the experimental data alone is used (see Figure 3. 2). Only data derived directly from experimental testing is therefore used for the derivation of the DI_{HRC} - $ISD_{max\%}$ relationships.

A total of 44 pairs of $ISD_{max\%}$ - DI_{HRC} values are collected for the damage scale calibration from various published experimental reports and papers (see Table 3.7 and Appendix D). Non-linear regressions are carried out on the $ISD_{max\%}$ - DI_{HRC} points and relationships derived for each of the building categories of Table 3.7. Little experimental data, covering few damage states, is available for new (ductile) bare-moment resisting frames. A separate relationship for this structural class is therefore not defined. Until further experimental data is available, the the $ISD_{max\%}$ - DI_{HRC} relationship for non-seismically designed (non-ductile) moment resisting frames, may be used to obtain conservative damage estimates for this structural class.

$$DI_{HRC} = 34.20Ln(ISD_{max\%}) + 38.66; R^2=0.953 \text{ for non-ductile MRF} \quad [3.13]$$

$$DI_{HRC} = 33.70Ln(ISD_{max\%}) + 47.52; R^2=0.956 \text{ for infilled frames} \quad [3.14]$$

$$DI_{HRC} = 39.31Ln(ISD_{max\%}) + 52.98; R^2=0.985 \text{ for shear-wall structures} \quad [3.15]$$

A further relationship is derived from non-linear regression on all $ISD_{max\%}$ - DI_{HRC} values. This allows the damage scale to be used in the derivation of vulnerability curves from observational damage statistics for structures of unspecified lateral load-resisting system.

$$DI_{HRC} = 28.06 \ln(ISD_{max\%}) + 47.95; R^2 = 0.776 \text{ for general RC structures} \quad [3.16]$$

In all the above equations the inter-storey drift is expressed as a percentage. If the value of DI_{HRC} is predicted to be less than 0 or greater than 100, the structure is assumed to be in the “no damage” or “collapse” limit state, respectively.

3.4.2 Validation of the Preliminary DI_{HRC} - $ISD_{max\%}$ Relationships

The interpretation of experiments carried out by others is subjective. A validation of the preliminary DI_{HRC} - $ISD_{max\%}$ relationships using independent sources of reliable experimental data is therefore carried out. This is done using the results of recent pseudo-dynamic tests on full-scale reinforced concrete structures. The testing was carried out at the European Laboratory for Structural Assessment (ELSA), of the Joint Research Centre (JRC) at Ispra (Italy), as part of the ICONS research initiative. The test specimens consist of two seismically designed shear-wall structures and two identical, non-seismically designed, moment resisting frames, one with masonry infill-walls and one without. A full description of the test frames and testing procedure is given in Pinto et al. (1999) and Reynouard and Fardis (2001). Though few damage observations were made, the test results are very well documented and fully available. Closely controlled conditions were enforced during the testing procedure. These tests therefore provide an invaluable, reliable source of data for use in the calibration of DI_{HRC} .

3.4.2.1 Validation of the Shear-wall Structure DI_{HRC} - $ISD_{max\%}$ Relationship

The performance of seismically designed shear-wall structures is investigated by the ICONS Network (Reynouard and Fardis 2001). Pseudo-dynamic tests are carried out on two geometrically identical frames, each consisting of two coupled shear-walls and two sets of columns connected by beams (see Figure 3.3). The first structure is designed according to the seismic design provisions of Eurocode 8 (EC8, CEN 1996), whereas a displacement-based design (DBD) procedure is adopted in the second case. C25/30 concrete and S500 steel are used in the construction. The reinforcement

detailing within the frames differs according to the requirements of the adopted design method. The EC8 shear-wall was observed to contain a significantly greater quantity of reinforcement than the DBD structure.

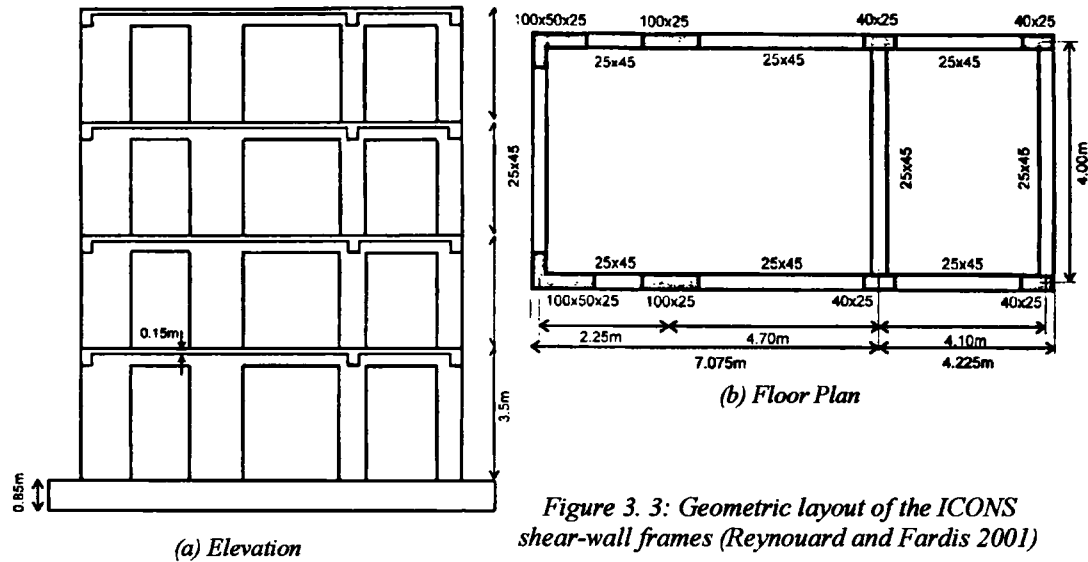


Figure 3. 3: Geometric layout of the ICONS shear-wall frames (Reynouard and Fardis 2001)

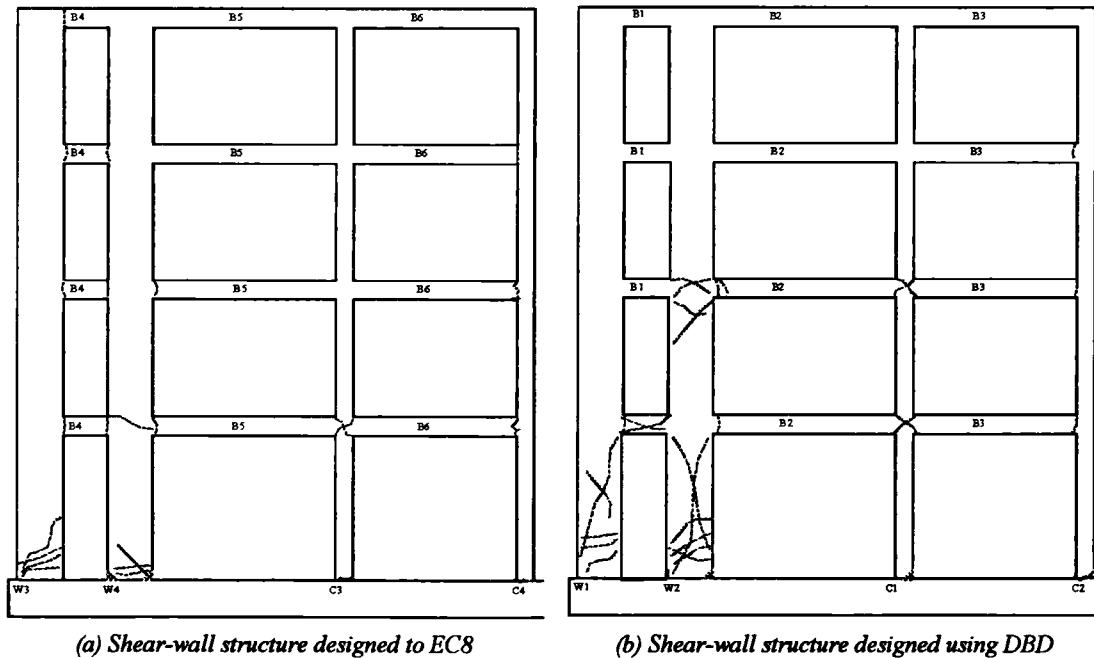


Figure 3. 4: Schematic drawing of the damage observed at the end of pseudo-dynamic testing of the ICONS shear-wall structures, (Reynouard and Fardis 2001). The green/shaded areas indicate concrete crushing, whilst the lines indicate cracking.

The frames, joined via a rigid slab, are tested in parallel using artificial accelerograms derived for 1.0 and 1.5 times the design earthquake. The testing is stopped during the latter earthquake due to the formation of substantial damage in the structure (Figure 3.4). Details of the observed damage and recorded structural displacements are reported in Reynouard and Fardis (2001). A summary of the DI_{HRC} values assigned to the structures as a whole, and to each individual storey, are given in Table 3.8, together with the recorded $ISD_{max\%}$. These values are compared to those obtained from the recorded $ISD_{max\%}$ using the preliminary DI_{HRC} - $ISD_{max\%}$ relationship for shear-wall structures, (Equation 3.15).

Floor	EC8-Shear-wall Structure				DBD-Shear-wall Structure			
	ISD_{max} (%)	Damage Description	Observed DI_{HRC}	Predicted DI_{HRC}	ISD_{max} (%)	Damage Description	Observed DI_{HRC}	Predicted DI_{HRC}
3 rd	1.58	light	30	71.0	1.58	slight	10	71.0
2 nd	2.00	light	30	80.2	2.00	light	40	80.2
1 st	2.43	light	40	87.9	2.43	moderate	70	87.9
Ground	2.86	moderate	70	94.3	2.86	extensive	80	94.3
Overall	2.86	moderate	70	94.3	2.86	extensive	80	94.3

Table 3. 8: Comparison of the observed damage and maximum inter-storey drift ratios for the ICONS shear-wall pseudo-dynamic tests, with the HRC damage index values predicted by the preliminary DI_{HRC} - $ISD_{max\%}$ equation for shear-walls (Eq. 3.15).

The large discrepancy between the predicted and observed damage indices indicates that the preliminary equations are not adequate for use in assessing the tested structures. Non-linear regression on the DI_{HRC} - $ISD_{max\%}$ values observed during the ICONS shear-wall tests, result in the following relationship.

$$DI_{HRC} = 107.37 \ln(ISD_{max\%}) + 43.17; R^2 = 0.918 \text{ for ICONS shear-walls} \quad [3.17]$$

From Figure 3. 5, it is evident that this equation provides a less conservative damage estimate than the preliminary equation for all $ISD_{max\%}$ values. The higher vulnerability predicted by the preliminary equation is explained by the inadequate seismic design of the shear-wall structures used in its derivation.

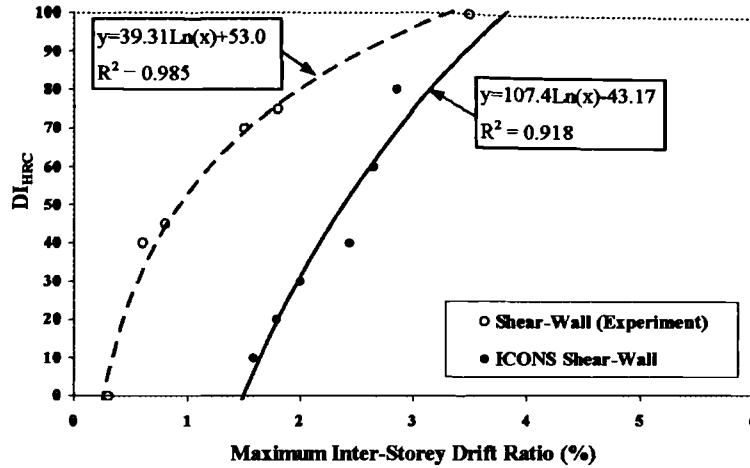


Figure 3. 5: Comparison of the preliminary DI_{HRC} - $ISC_{max\%}$ relationship for shear wall structures with that obtained from the ICONS pseudo-dynamic testing of shear-walls.

Most existing shear-wall structures are not designed to comply with modern seismic codes. Therefore, the data obtained from the ICONS tests is not used to update the preliminary equation shear-wall equation. It is recommended that the preliminary equations be used for the vulnerability assessment of urban areas with predominant shear-wall type constructions. The ICONS shear-wall relationship should only be applied in the case that modern seismic codes are predominantly used in the assessed area.

3.4.2.2 Validation of the Bare and Infilled Frame DI_{HRC} - $ISC_{max\%}$ Relationships

The reinforced concrete bare and infilled frames tested at Ispra are representative of the existing building stock in Europe. They are designed by Carvahlo et al. (1999), according to typical 1950s codes of practice. The configuration of the frames is shown in Figure 3. 6. The frames do not satisfy capacity design principles, and contain irregularities in plan and elevation. Weak concrete and un-deformed reinforcing steel bars are used in the construction of the frames. The latter are joined via short lap-splices, positioned at plastic hinge locations and confinement reinforcement is minimal. Further information on the vertical loads applied during testing and the member reinforcement details, can be found in Carvalho et al. (1999) or Pinto et al. (1999).

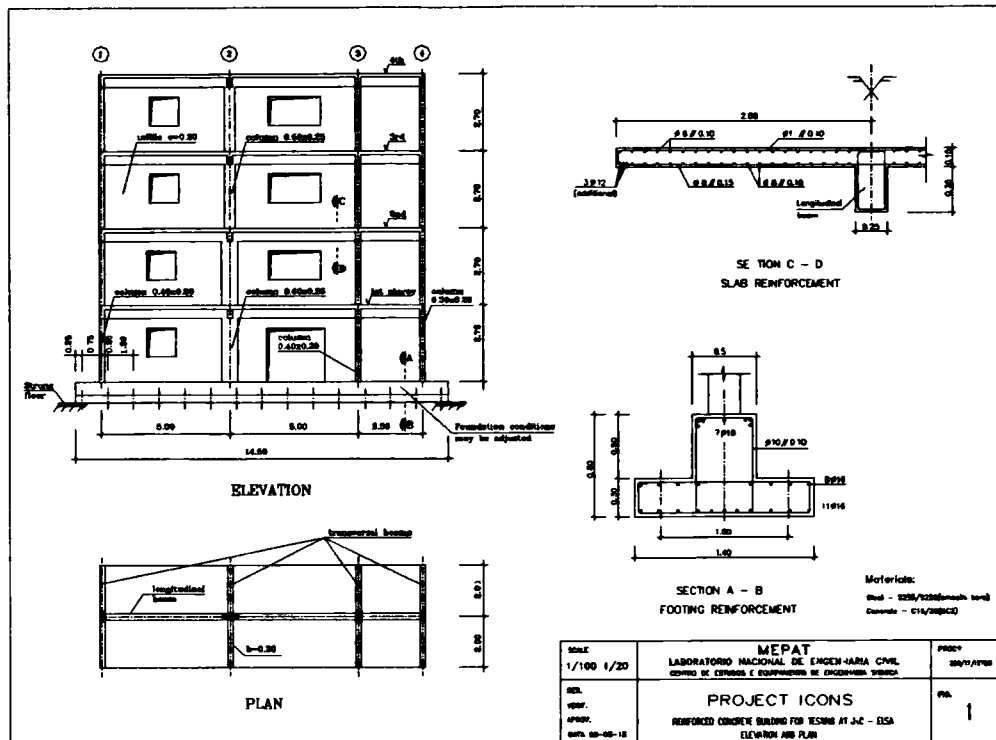


Figure 3. 6: Configuration of the ICONS bare and infilled moment resisting frames designed by Carvalho et al. (1999) and tested pseudo-dynamically at the ELSA laboratories in Ispra, Italy.

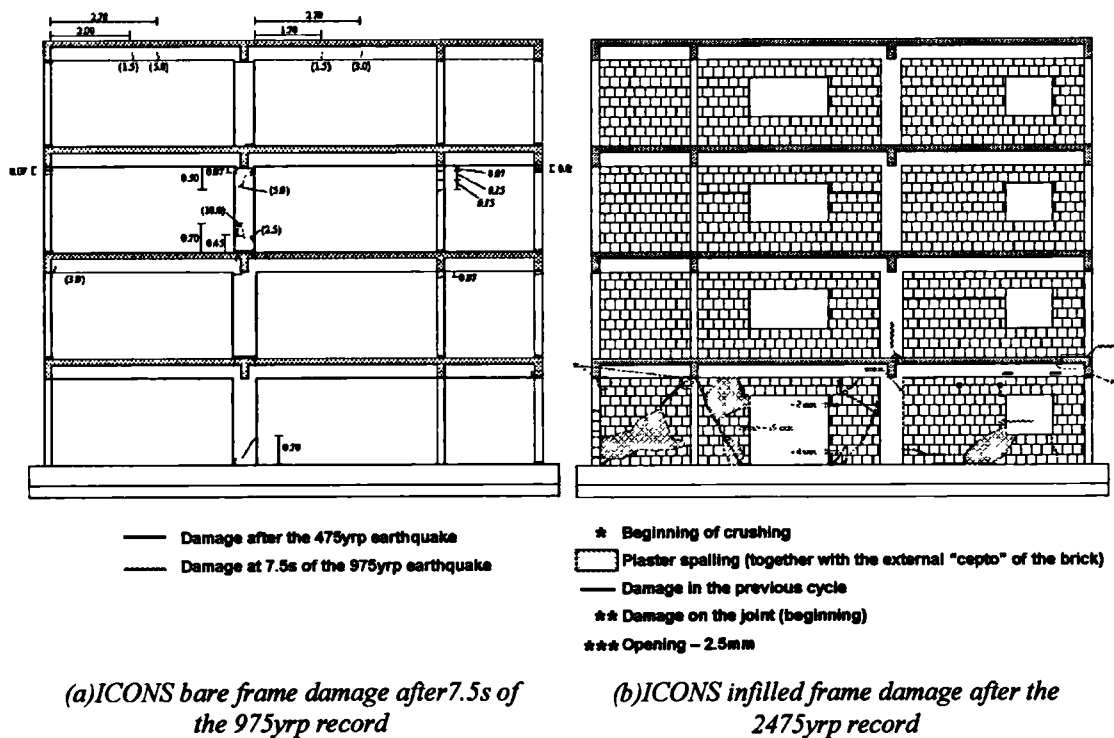


Figure 3. 7: Damage observed in the bare and infilled ICONS frames at the end of the pseudo-dynamic testing.

The frames were tested to near failure, repaired and then tested again to failure. Only tests carried out on the bare- and infilled-frames before repair are considered in the damage scale calibration. Three earthquake records, developed by Campos-Costa and Pinto (1999), are used in the testing of the frames. The records are derived from uniform risk spectra, for return periods of 475, 975 and 2000 years, developed from catalogues of European earthquakes, (NGDC and NEIC (1996)). The infilled frame was subjected to all three records. However, testing of the bare structure was stopped during the application of the 975-year return period record, due the formation of a soft-storey mechanism at the third floor. The damage in the frames after testing is illustrated in Figure 3. 7.

Floor	475yrp at 15s				975yrp at 15s			
	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}
3 rd	0.46	light	15	12.8	0.91	light/moderate	40	36.8
2 nd	0.80	light	30	32.3	2.41	moderate/extensive	70	71.0
1 st	0.74	light	30	29.5	1.03	moderate/extensive	40	41.2
Ground	0.44	light	10	11.3	0.63	light/moderate	20	23.9
Overall	0.80	light	30	32.3	2.41	moderate/extensive	70	71.0

Table 3. 9: Comparison of the observed damage and maximum inter-storey drift ratios for the ICONS bare frame at different times during the pseudo-dynamic tests, with the HRC damage index values predicted by the preliminary DI_{HRC} -ISD_{max%} equation for non-ductile MRFs (Eq.3.13).

The frames were fully instrumented, with recordings of the element displacements and rotations made throughout the loading process. 6 and 2 observations of structural damage are made, at different times during the loading sequence, for the infilled and bare frames, respectively. The reported damage is used to assign damage index values to the frames at both the storey and global level. The DI_{HRC} values are compared, in see Table 3.9 and Table 3.10, to those obtained using the preliminary bare and infilled DI_{HRC} -ISD_{max%} relationships (Equations 3.13 and 3.14).

Time	475yrp at 5s					475yrp at 6.57s					475yrp at 12.17s				
	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Predicted DI _{HRC}
Floor															
3 rd	0.04	no damage	0	0.0	0.06	slight	5	0.0	0.06	light	5	0.0	0.06	light	0.0
2 nd	0.05	no damage	0	0.0	0.07	slight	5	0.0	0.08	light	10	0.0	0.08	light	0.0
1 st	0.06	no damage	0	0.0	0.10	slight	5	0.0	0.12	light	10	0.0	0.12	light	0.0
Ground	0.07	slight	5	0.0	0.11	light	10	0.0	0.12	light	10	0.0	0.12	light	0.0
Overall	0.07	no/slight	5	0.0	0.11	slight	10	0.0	0.1	light	10	0.0	0.1	light	0.0

Time	975yrp at 6.53s					975yrp at 12.43s					2000yrp at 15s				
	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Observed DI _{HRC}	Predicted DI _{HRC}	ISD _{max} (%)	Damage Description	Predicted DI _{HRC}
Floor															
3 rd	0.07	slight	5	0.0	0.11	slight	5	0.0	0.11	Slight	5	0.0	0.11	Slight	0.0
2 nd	0.09	light	10	0.0	0.15	light/mod	40	0.0	0.15	slight/moderate	40	0.0	0.15	slight/moderate	0.0
1 st	0.14	light	10	0.0	0.27	moderate	50	4.1	0.27	Moderate	50	4.1	0.27	Moderate	4.1
Ground	0.14	slight	15	0.0	0.43	moderate	55	19.4	1.23	Moderate	60	53.8	1.23	Moderate	53.8
Overall	0.14	light/slight	15	0.0	0.43	moderate	55	19.4	1.23	Extensive	60	53.8	1.23	Extensive	53.8

Table 3. 10: Comparison of the observed damage and maximum inter-storey drift ratios for the ICONS infilled frame at different times during the pseudo-dynamic tests, with the HRC damage index values predicted by the preliminary $DI_{HRC}-ISD_{max}$ equation for infilled MRFs (Eq 3.14).

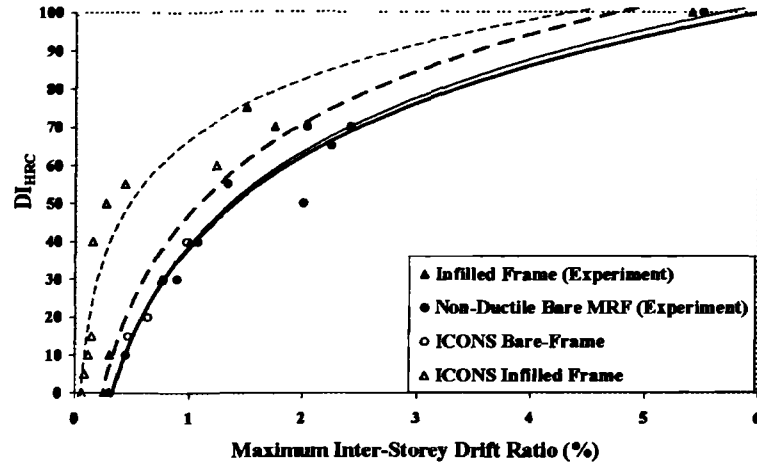


Figure 3. 8: Comparison of the preliminary DI_{HRC} - $ISD_{max\%}$ relationships (darker lines) with those obtained from the ICONS pseudo-dynamic tests on bare and infilled frames (full and dashed lines, respectively).

In Figure 3.8, the preliminary equations are also compared to those derived solely from the ICONS frame pseudo-dynamic test results. The regression is carried out on the damage indices evaluated at the storey level for the frames, and their related $ISD_{max\%}$ values. The preliminary equation for non-ductile frames gives a close prediction of the damage state of the ICONS bare frame. Equation 3.13 is therefore updated using the data obtained from the ICONS bare-frame tests. The resulting relationship is given by Equation 3.18.

$$DI_{HRC} = 34.89 \ln(ISD_{max,\%}) + 39.39; R^2=0.991 \text{ for non-ductile MRF} \quad [3.18]$$

$$DI_{HRC} = 22.49 \ln(ISD_{max,\%}) + 66.88; R^2=0.822 \text{ for ICONS infilled frame} \quad [3.19]$$

The preliminary infilled-frame relationship is instead seen to under-predict the observed damage in the ICONS infilled frame, for all values of inter-storey drift. Due to the scarce data used to derive the preliminary equation, it is discarded. Equation 3.19, derived solely from the ICONS infilled frame response, is recommended for use in the assessment of this building class.

3.4.3 Final Calibration of the HRC Damage Scale

Following a process of validation and updating, the final damage index calibration relationships for shear-walls, infilled and bare non-ductile MRFs are given by:

$$DI_{HRC} = 34.89Ln(ISD_{max,\%}) + 39.39; R^2=0.991 \text{ for bare non-ductile MRF} \quad [3. 20]$$

$$DI_{HRC} = 22.49Ln(ISD_{max,\%}) + 66.88; R^2=0.822 \text{ for infilled non-ductile MRF} \quad [3. 21]$$

$$DI_{HRC} = 39.31Ln(ISD_{max,\%}) + 52.98; R^2=0.985 \text{ for shear-wall structures} \quad [3. 22]$$

The general RC structure equation is also updated using the ICONS bare and infilled frame data:

$$DI_{HRC} = 27.89Ln(ISD_{max,\%}) + 56.36; R^2=0.760 \text{ for general RC structures} \quad [3. 23]$$

In the above equations, the inter-storey drift is expressed as a percentage. If the predicted value of DI_{HRC} is less than 0 or greater than 100, the “no damage” and “collapse” limit states are assumed, respectively.

The final damage index calibration relationships are illustrated in Figure 3. 9, together with their weighted derivation data. A large difference in the progression of damage within different structural types is seen. Table 3.11 summarises the $ISD_{max\%}$ values predicted by Equations 3.20 and 3.22 for the FEMA273 (ATC 1997) damage limit states, using the damage scale conversions of Table 3.6 and Table 3.5, respectively. Significant discrepancies are seen. The FEMA273 response parameter values are more conservative for “Life Safety”, and less conservative for the “Collapse” limit state. These differences are attributed to the limited use of experimental data in the FEMA273 damage scale calibration. Test data regarding low-rise concrete shear walls, new RC structures and non-structural elements only, is used. The inclusion of some experimental data for shear walls, results in a better correlation of the $ISD_{max\%}$ values for this structural class than for the non-ductile frames.

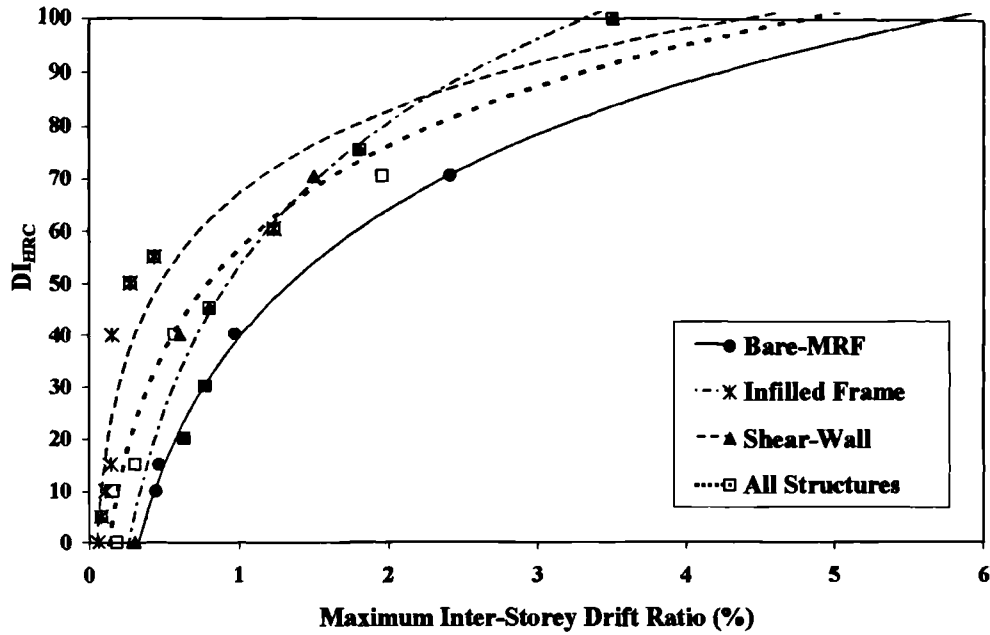


Figure 3. 9: Illustration of the final DI_{HRC} - $ISD_{max\%}$ HRC Damage Scale calibration relationships.

$ISD_{max\%}(\%)$	RC Structure Class			
FEMA273 Damage State	Non-ductile MRF		Shear walls	
	HRC	FEMA273	HRC	FEMA273
Immediate Occupancy	0.00	0.00	0.00	0.00
Life Safety	1.36	1.00	0.93	0.50
Collapse Prevention	3.20	4.00	1.99	2.00

Table 3. 11: Comparison of the $ISD_{max\%}$ limit state threshold values in FEMA273 (ATC 1997), with those predicted by the HRC-Scale correlation relationships, using Tables 3.5 and 3.6.

In Table 3.12, the HRC damage state definition, obtained for non-ductile moment resisting bare frames, is compared to the Park and Ang (1985) global damage index (PA). The relationships proposed by Singhal and Kiremidjian (1997) for mid-rise moment resisting frames, are used to convert the maximum inter-storey drift ratio ($ISD_{max\%}$) boundary values to equivalent PA values. The damage state definition of Hwang and Huo (1994) is then used to assign damage state descriptions to the calculated PA values. A reasonable correlation between the two scales results, with the HRC damage scale observed to be the more conservative. The difference between the damage state definitions is attributed to the consideration, by Singhal and Kiremidjian (1997), of structures of better

seismic design than those used for the calibration of the HRC-Scale. This further outlines the importance of seismic code and construction material consideration in vulnerability analyses, and enforces the statement that damage scales developed for non-european countries may not be adequate for use in European risk assessments.

HRC Damage State	Non-ductile MRF Threshold $ISD_{max\%}(\%)$	Mid-rise Frame PA	PA Damage State
None	0.00	-0.136	Non-structural damage
Slight	0.32	-0.064	Non-structural damage
Light	0.43	-0.039	Non-structural damage
Moderate	1.02	0.096	Slight structural damage
Extensive	2.41	0.411	Moderate damage
Partial Collapse	4.27	0.834	Severe damage
Collapse	> 5.68	> 1.155	Collapse

Table 3. 12: Correlation between the HRC and Park and Ang (1985) damage limit-states for non-ductile moment resisting reinforced concrete frames, using Equation 3.20 and the relationships between $ISD_{max\%}$ and PA of Singhal and Kiremidjian (1997) for mid-rise frames, ($ISD_{max\%} = 100*(0.044PA + 0.006)$).

$ISD_{max\%}(\%)$	RC Structure Class			
HRC Damage State	All Structures	Non-ductile MRF	Infilled MRF	Shear-wall Structures
None	0.00	0.00	0.00	0.00
Slight	0.13	0.32	0.05	0.26
Light	0.19	0.43	0.08	0.34
Moderate	0.56	1.02	0.30	0.72
Extensive	1.63	2.41	1.15	1.54
Partial Collapse	3.34	4.27	2.80	2.56
Collapse	> 4.78	> 5.68	> 4.36	> 3.31
Mean (p/o)	0.94	1.00	1.19	1.07
COV (p/o)	0.64	0.07	0.29	0.23

Table 3. 13: HRC damage limit-state threshold $ISD_{max\%}$ values. The mean and coefficient of variation of the ratio of predicted-to-observed HRC damage index, associated with the empirical calibration equations (Equations 3.20 to 3.23).

The empirical calibration equations 3.20 to 3.23, are used to define the maximum inter-storey drift ratio values characterising the HRC damage limit state boundaries (Table 3.13). These values are used to determine the damage state exceedence probabilities in both the empirical and analytical vulnerability curve generation.

It is acknowledged that the proposed relationships are only based on limited quantities of experimental data. A large part of this data does however derive from well-controlled, heavily instrumented, full-scale pseudo-dynamic tests on buildings representative of existing designs and construction. The detailed damage evaluations made throughout the course of the tests are therefore regarded as very reliable and the structural response as realistic and representative of that of structures in the field. The coefficient of variation associated with the ratio of predicted-to-observed DI_{HRC} is also presented in Table 3.13. For a conservative estimate of vulnerability a lower-bound $ISD_{max\%}$ value should be used in the structural assessment. Consideration of uncertainty in the damage state bounds is deemed especially important in the case that the predominant structure type of the assessed building population is not known.

Chapter 4

Ground Motion Characterisation

4.1 Introduction

The parameter chosen to represent ground motion in the construction of vulnerability curves must be representative of the damage potential of earthquakes. Furthermore, in order for the developed vulnerability curves to be of practical use, the chosen ground motion parameter must be easily quantifiable from knowledge of the earthquake characteristics. Within this chapter, some of the most commonly used ground motion parameters are considered for use in the construction of vulnerability curves from post-earthquake damage surveys. The same parameter value may be obtained from innumerable earthquake time-histories. Thus, within the second part of the chapter, a procedure for the selection of “performance consistent” acceleration record suites, appropriate for use in the generation of damage statistics via analysis, is proposed. The record selection technique is illustrated for the case of low-rise infilled frames of typical European construction, and the uncertainty in this structure’s response to strong-motion is evaluated.

4.2 Choice of a Ground Motion Parameter

An accelerogram is the most complete representation of earthquake ground motion. The damage potential of earthquake time-histories varies according to many factors. These factors include the number of load cycles, amplitude and duration of the ground motion, and the relationship between record frequency content and the dynamic characteristics of the assessed buildings. Many ground motion damage potential indices have been proposed in the literature in an attempt to quantify the multiple aspects of the recorded motion that affect earthquake damage potential (e.g. Clough and Penzien 1975, Nau and Hall 1984, Araya and Saragoni 1984, Benedetti et al. 2001). Different degrees of success are achieved,

but most require the earthquake record for the site to be known prior to their evaluation. Thus, problems arise when dealing with post-earthquake damage statistics collected at sites for which locally measured earthquake records are unavailable. Furthermore, the large uncertainties associated with the prediction of earthquake record traces, do not render such parameters amenable for use in a pre-earthquake assessment context. Only ground motion parameters that can be evaluated from existing attenuation relationships are, therefore, considered for use in this study.

The main ground motion parameters used to characterise seismic demand in existing vulnerability studies may be summarised into the three categories of peak ground motion parameters, felt intensities and parameters derived from the response of SDOF systems. These are briefly reviewed and their adequacy as ground motion damage potential predictors and suitability for use in vulnerability curves, is discussed.

4.2.1 Peak Ground Motion Parameters

Peak ground motion parameters are measured directly from earthquake records and represent the maximum absolute amplitude of the recorded ground motion. Numerous attenuation relationships exist for the evaluation of these parameters from knowledge of the earthquake magnitude, fault distance and site soil conditions. This is a necessary requirement for the vulnerability curves to be used in a predictive capacity. A further advantage of peak ground motion values is that within an analytical procedure for vulnerability curve generation, direct scaling of the records may be carried out using these parameters, and the distribution of vulnerability observations along the horizontal axis of the plots controlled.

In published papers, there is little agreement as to which peak ground motion parameter is the most suitable for earthquake damage potential representation. Liu and Zhang (1984), consider peak ground acceleration (*pga*) to be the most appropriate index for this purpose. Due to its traditional use in the definition of design loads for structures, *pga* is often used to represent the severity of an

earthquake in vulnerability studies (e.g. Mosalam et al. 1997). However, it is well known that *pga* does not correlate well with observed damage. This is clearly seen in Figure 4.1a, where no trend can be identified between *pga* and the weighted observational RC frame damage statistics presented in Chapter 5. *pga* is physically associated with a sudden rupture over a small portion of the causative fault, and its recorded value at small distances from the fault is virtually independent of the earthquake magnitude. In engineering terms, *pga* is related to the dynamic loads imposed on very stiff structures and, therefore, may not be adequate for the characterisation of seismic demand in medium or high rise building populations.

Peak ground velocity (*pgv*), is associated with the energy imposed by earthquakes, and is more representative of the seismic demand on structures of intermediate natural period. *pgv* is generally calculated through direct integration of accelerograms and may be sensitive to both the record noise content and frequency filtering process. The parameter is often adopted in vulnerability relationships derived for Japan, (e.g. Yamazaki and Murao 2000). Following a comparison of strong motion and response based parameters, Naeim and Anderson (1996), recommend the use of *pgv* for earthquake damage potential characterisation. However, in a study made of the Kobe earthquake (Japan, 1995), Miyakoshi et al. (1997) find a poor correlation between *pgv* and damage.

No vulnerability studies currently exist that adopt peak ground displacement (*pgd*) in the ground motion characterisation. This parameter is associated with the maximum imposed structural deformation, and is more adequate for the characterisation of seismic demand in long period structures. Few attenuation relationships exist for the prediction of *pgd*, due to its high sensitivity to noise in accelerograms and to filtering techniques adopted in the elimination of unwanted frequencies from raw records. The *pgd* attenuation relationships that do exist, (e.g. Kawashima et al. 1986, Theodulidis and Papazachos 1992, Sadigh and Egan 1998), are based on few digital records and cover limited areas and fault mechanisms. These do not provide reliable estimates of *pgd* for all sites of recorded damage present in the observational database of Chapter 5.

In Figure 4.1, *pgd* correlates better with the observational damage statistics than *pga*. However, the overall correlation remains poor. This is attributed to the fact that peak ground motion parameters only represent the record amplitude. They are unable to capture features of the earthquake record, such as frequency content, duration or number of cycles, which also affect the response and consequent damage of a structure. This observation is confirmed by Penazzo (2001), who demonstrates that all three peak ground motion parameters correlate badly with measures of earthquake energy, spectral intensity and duration. The latter comparison considers 473 natural records measured at different soil sites, and selected from a world wide databank of earthquakes with surface magnitude and source-site distance ranges of 4.5–7.6 and 0–150km, respectively.

Composite indices, which combine peak ground motion parameters, have been proposed in the literature in an attempt to improve representation of earthquake record energy and frequency content. The composite indices proposed by Zhu et al (1988), and Newmark and Rosenbleuth (1971), are defined in Equations 4.1 and 4.2, respectively.

$$GM = \frac{pga}{pgv} \quad [4.1]$$

$$GM = \frac{pga.pgd}{(pgv)^2} \quad [4.2]$$

Both equations are closely related to the energy content of the record in the frequency domain, (e.g. a high (*pga/pgv*) ratio, indicates that the energy is concentrated in the short period range). Only a very limited number of relationships for combined peak ground motion parameter attenuation exist, (e.g. Sadigh and Egan 1998). These are not available for all areas and types of fault mechanisms. Although attenuation relationships exist for predicting the individual parameters composing the indices, their use may not yield reliable index values, as they do not account for attenuation of the parameter combination.

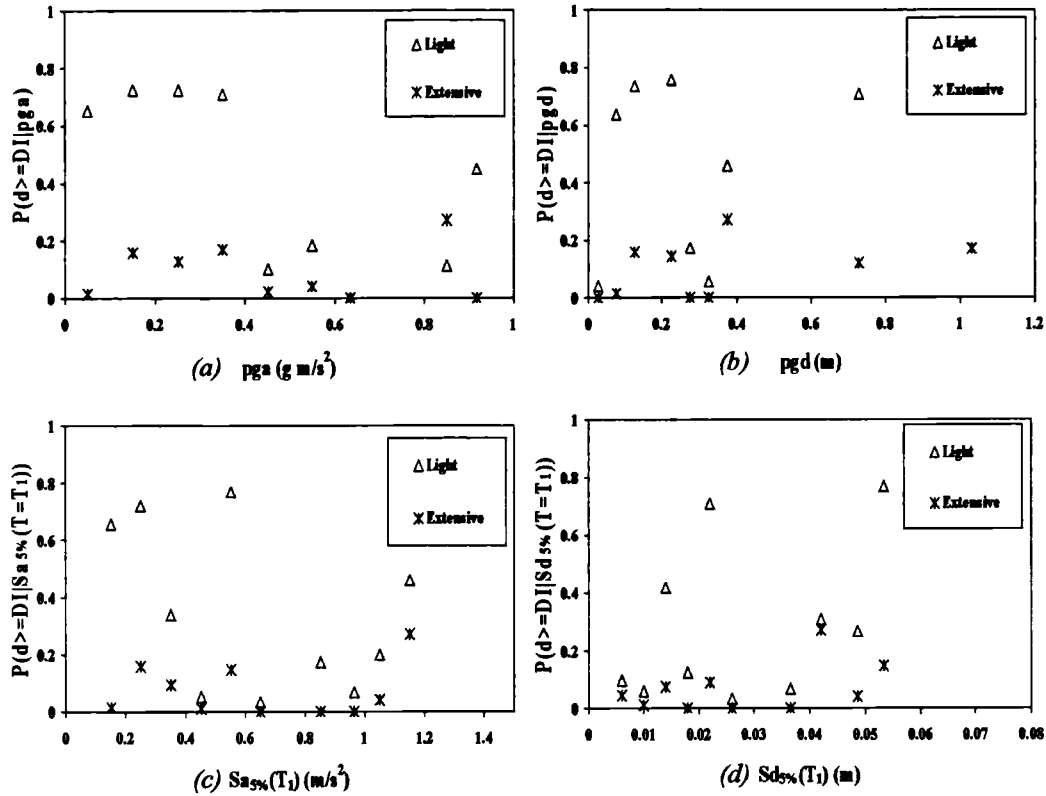


Figure 4. 1: Comparison of the correlation of the weighted bare RC frame observational post-earthquake damage statistics for the “Light” and “Extensive” HRC damage states of Chapter3 with different ground motion parameters: (a) pga ; (b) pgd ; (c) $Sa_{5\%}(T_1)$; (d) $Sd_{5\%}(T_1)$.

It is concluded that indices consisting of peak ground motion parameters, or their combination, are not suitable for use in vulnerability curve generation. It is further recommended that only ground motion parameters, which can be evaluated directly from single attenuation relationships, (developed from extensive and reliable earthquake record databases), should be considered for the characterisation of the seismic demand.

4.2.2 Felt intensities

Most codified vulnerability relationships adopt intensity measures to represent seismic demand, (e.g. ATC-13 (ATC 1985)). Many different intensity scales are used around the world. Those most frequently adopted are the Modified Mercalli Intensity (MMI), Japan Meteorological Agency (JMA), Mercalli-Cancani-Sieberg (MCS), Medvedev-Sponheuer-Karnik (MSK) and European Magnitude Scale (EMS98). Different criteria are adopted in the definition of the intensity levels

within each scale. Consequently, vulnerability curves derived from their use are not directly comparable. Although Intensity is seen to correlate well with observed damage, many disadvantages are associated with its use in vulnerability curve derivation.

Intensity is a subjective measure for which few unreliable attenuation relationships exist. Intensity should not strictly be used as a numerical scale, as it is a discrete measure associated with intervals between unit values that are not necessarily equal. Furthermore, due to the relatively large width of these intervals, numerous statistical damage data sets, deriving from different earthquake scenarios will be assigned the same intensity value. This results in a large scatter in the vulnerability data associated with each intensity value. Hence, the definition of a reliable vulnerability curve shape is more difficult. Pomonis et al (1992), propose a continuous intensity scale called the Parameterless Seismic Intensity scale (ψ , or PSI scale), for use in the derivation of vulnerability curves. Relationships between PSI and *pga* are proposed for use together with existing attenuation equations. This allows the PSI vulnerability curves to be used in predictive earthquake risk assessments. However, the PSI-*pga* relationships are derived from a limited number of field observations and corresponding ground motion recordings. They are associated with large scatter and require greater verification before they can be deemed reliable.

Regardless of the scale continuity, the use of an intensity value to characterise ground motion for use in vulnerability curve derivation is fundamentally wrong. Intensity is evaluated directly from observed earthquake effects and, hence, its value is partly dictated by the building stock vulnerability. This causes a degree of inter-dependence between the two axes of the fragility curves. In order for the derived vulnerability curves to be widely applicable, it is important to decouple the seismic demand from the structural vulnerability. Therefore, only ground motion parameters that can be evaluated directly from the earthquake characteristics should be considered for vulnerability curve derivation.

4.2.3 Ground Motion Parameters from SDOF System Response

One of the reasons why felt Intensity is so popular in risk assessment, is that it inherently accounts for the influence of founding soil and structural dynamic properties on the seismic demand imposed on buildings. Spectral values derived from the dynamic time-history analysis of single degree of freedom (SDOF) systems, are regarded to give a good representation of these combined effects. As for peak ground motion values, the choice of which spectral value to use poses a problem to which there is no universally accepted solution. Acceleration spectra are representative of the imposed seismic forces on the structure over a wide range of frequencies, and are used by most modern design codes in the determination of structure loads. In view of this, attenuation relationships have been proposed by various authors (e.g. Joyner and Boore 1988, Ambraseys et al. 1996), for the evaluation of $S_a(T)$ from knowledge of the earthquake characteristics, site conditions and structural period. From the engineering point of view, the adoption of $S_a(T)$ in risk assessments is advantageous, as the design forces for strengthening measures required for the fulfilment of performance objectives, can be determined directly from the vulnerability curves. $S_a(T)$ is adopted by Singhal and Kiremidjian (1997), in the derivation of vulnerability curves for California and is observed in Figure 4.1, to give a better correlation with observed damage than pga . However, damage is more closely related to the seismic energy imparted to the structure and imposed relative displacements than to imposed forces.

Spectral velocity ($S_v(T)$), is indicative of the earthquake energy, and probably gives the best representation of earthquake damage potential. Many parameters based on energy dissipation concepts have been proposed in the literature for structural damage quantification (see Section 3.3.1). Although many attenuation relationships exist for the parameter evaluation, (e.g. Spudich et al. 1997, Shabestari et al. 2000), $S_v(T)$ has not yet been used in the development of vulnerability curves, due to the limited practical usefulness of energy measures in the design of structures. For similar reasons, no vulnerability curves have as yet been derived in terms of spectral displacement ($S_d(T)$). However, $S_d(T)$ is gaining importance in seismic risk assessment due to the recent development of

displacement based methods of seismic design (e.g. Moehle 1992, Priestley et al. 1996), and ongoing research for the proposal of design equations for reinforced concrete members based on measures of deformation (Panagiotakos and Fardis 1999). Furthermore, the installation of digital strong-motion measuring instruments in countries worldwide, has eliminated the uncertainty in the spectral displacement determination from accelerograms, associated with noise contamination and record frequency filtering procedures. Consequently, reliable attenuation relationships for $S_d(T)$ have been derived (Bommer et al. 1998 and Borzi et al. 1998). The importance in structural damage determination of spectral displacement is recognised by the Federal Emergency Management Agency (FEMA), in the development of their risk assessment tool HAZUS99 (FEMA 1999). This uses capacity curve methods, (see Section 7.2), to assess building risk and hence, expresses the seismic demand in terms of both $S_d(T)$ and $S_a(T)$. In Figure 4.1, the superiority of $S_d(T)$ compared to $S_a(T)$ in representing earthquake damage potential, is illustrated by the identifiable tendency of the exceedence probabilities to increase with the parameter value. This parameter is, therefore, chosen to represent seismic demand in the developed vulnerability relationships.

The main difficulties associated with the use of elastic spectral values are the determination of equivalent vibration periods and damping coefficients for the characterisation of buildings as SDOF systems. Consideration must be given to the likely structural failure mode and its strength and stiffness degradation during ground excitation. It may be possible to identify the likely failure mode from the structural configuration and seismic code used in construction. However, estimation of the probable degradation is extremely difficult as this depends not only on the structure, but also on the ground motion characteristics. Parameters found from the integration of the structural response over a given period range have been used in the past to account for structural softening with increasing sustained damage. Beniof (1934), was the first to suggest that an integral value of the acceleration spectrum be used to represent ground motion damage potential, but left the limits of integration to be defined from future research. Housner (1952) and Kappos (1990) propose Spectral Intensity values derived from integration of elastic acceleration response spectra between the periods of 0.1 - 2.5 seconds, and $0.8 T_f - 1.2 T_f$, (T_f being the fundamental period of the structure), respectively.

The use of a range of integration that is dependent on the structural period, such as that proposed by Kappos (1990), provides a better representation of the seismic demand on a particular structure. However, the proposed limits may not be appropriate for all ground motions, lateral load resisting systems, types of member detailing and material properties. For example, period shifts of 100% and 150% were observed in the fundamental periods of 4 and 11 storey non-seismically designed bare frames tested pseudo-dynamically at ISPRA (Pinto et al. 1999) and by Galambos et al. (1982), respectively. In the latter case, a 200% period shift was observed for the same 11 storey frame when infills were present. Given the difficulty associated with the definition of a range of period values for the integration and the extra computational effort involved, integrated spectral displacement values are not considered for the vulnerability curve definition.

In displacement based design and assessment, the effective period, evaluated from the structure secant stiffness at the point of maximum applied ductility, is used to evaluate $S_d(T)$. In the present study, maximum inter-storey drift ratio is used as a damage index, which in a population of regular structures with similar yield displacements, is effectively a measure of the structural ductility level. Each damage state is therefore associated with a ductility level, and the damage state exceedence probability implicitly represents the structures' ability to provide this ductility. This implies that different effective periods should be used for the determination of the elastic spectral displacement demand for each damage state vulnerability curve. However, observational data implicitly includes the effects of inelasticity in both the seismic demand and structural response. Consequently, inelasticity and period elongation during the earthquake, is taken into account in the determination of the exceedence probabilities, (i.e. in the y-axis of the curves). The inclusion of inelasticity in the x-axis is, therefore, unnecessary and the elastic period can be used to characterise the ground motion demand for all damage state curves. The same conclusion holds for the derived analytical curves, as long as inelasticity is considered in the determination of the structure response during the analysis and performance evaluation.

4.3 Spectral Displacement Calculation for Empirical Curve Derivation

In the generation of vulnerability curves from observational post-earthquake damage statistics, spectral displacement values are assigned to each survey site, using the elastic uniform hazard spectra attenuation relationships of Borzi et al. (1998). The latter relationship is based on a large, homogeneous and consistent database of recorded motions. Furthermore, surface magnitude (M_s) is adopted by the relationship, which is not prone to rapid saturation for large events. The attenuation relationship takes the form of Equation 4.3, where the attenuation coefficients C_1 , C_2 , C_4 , C_a , C_s , and σ vary in value according to the elastic period (T).

$$\log[S_d(T)] = C_1' + C_2 M_s + C_4 \log(r) + C_a S_a + C_s S_s + \sigma P \quad [4.3]$$

In Equation 4.3, $r = \left(\sqrt{d^2 + h_0^2}\right)$, represents the attenuation path, where d is the shortest distance to the surface projection of the fault rupture (in km). h_0 is a factor found from regression, which accounts for the difference between the region of largest energy release on the causative fault surface and the closest part of the fault to the site. The coefficients S_a and S_s , are included in the equation to account for amplification of ground motion due to soil conditions at the site. S_a takes a value of 1 for stiff soils (with shear velocities $360 < V_s \leq 750$ m/s), and 0 otherwise. S_s instead takes a value of 1 for soft soils (with shear velocities $180 < V_s \leq 360$ m/s), and 0 otherwise. P is included to account for uncertainty in the relationship. Values for P of +1, -1 and 0 will result in upper bound, lower bound and mean estimates of $S_d(T)$ respectively. Mean values of the spectral displacement demand are assigned to the observational damage survey sites to provide an assessment tool that is realistic and not over-conservative. The $S_d(T)$ values resulting from Equation 4.3, are associated with a fraction of critical damping (ξ) of 1%. A correction factor of $(3/7)^{1/2}$ is applied to these values as per Eurocode 8 (CEN 1996), to convert the response to 5% of critical damping ($S_{d5\%}(T)$), which is more characteristic of a reinforced concrete structure responding elastically.

The assignment of spectral displacement demand values to observational damage survey data sites, implies the estimation of an elastic period of vibration that is representative of the structure population. The use of a realistic period for stiffer structures such as shear walls and infilled frames is important as the value of predicted $S_d(T)$ may vary significantly (see Section 7.5.1). However, the accurate Eigen-value analysis of a representative finite element building model requires detailed information on the material properties, seismic detailing and structural configuration of the surveyed population, which are often unavailable. Therefore, unless a specific estimate of the typical structural period is given by the survey sources, it is only possible to make crude estimates of the elastic period. The latter can be done using equations such as that proposed in Eurocode 8 (CEN 1996) and presented in Equation 4.4, which are based on the predominant building height (H in meters) within the surveyed population.

$$T_1 = C_t H^{\frac{3}{4}} \quad [4.4]$$

If no data is available regarding the distribution of the building heights within the surveyed population, then reference is made to published literature, for information on the predominant number of stories in the population. Average storey heights of 3m and 3.5m are assumed typical for European buildings in rural communities and urban areas, respectively, (Calvi 1999). An average storey height of 3m (10ft) as is given by FEMA 273 (ATC 1997), is instead used for buildings in the United States. The coefficient C_t in Equation 4.4, takes a value of 0.075 for moment resisting reinforced concrete frames, and 0.05 for all other lateral load resisting systems except for shear wall structures. In the latter case, C_t is calculated as $(0.075/A_c^{0.5})$, where A_c is the combined effective area of the shear walls in the first storey of the building, expressed in m^2 . The latter parameter is

defined by $A_c = \sum \left[A_i \left(0.2 + \frac{l_{wi}}{H} \right)^2 \right]$, where A_i and l_{wi} are the effective cross-

sectional area and length parallel to the applied forces of shear wall i , with the restriction that $(l_{wi}/H) \leq 0.9$.

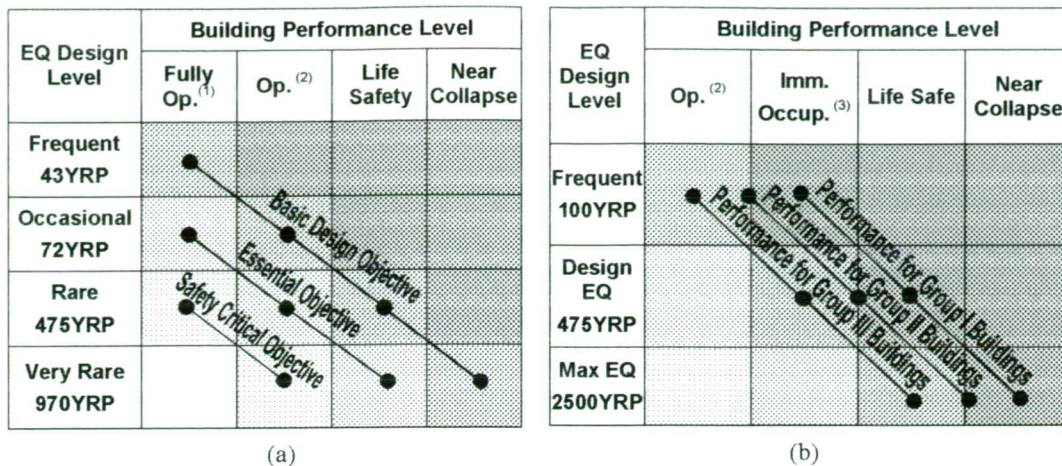
4.4 Earthquake Record Selection Procedure for Vulnerability Analysis

In the generation of vulnerability curves, random variability in the ground motion parameter, (i.e. $S_{d5\%}(T)$), is not considered, as this should be included in the determination of the hazard within the total risk assessment. Within an analytical curve derivation procedure, the ground motion parameter is deterministic, and is evaluated from the earthquake records used as input to the analyses. However, the characteristics of records producing any single value of the ground motion parameter can vary significantly and the effect of this variation on the structural response and consequent damage prediction uncertainty must be considered. This may be achieved through the use of suites of accelerograms with different peak amplitude, frequency content, cycle number and duration characteristics. However, no formal guidance for the selection of record suites for vulnerability curve generation exists.

The structural response must be evaluated over a very wide range of ground motion severities. In practice, it is difficult to obtain natural records for the specific fault mechanisms and soil conditions of the assessed area, which will cover the necessary range of ground motions. Therefore, in most existing vulnerability studies (e.g. Singhal and Kiremidjian 1997), a set of natural records is selected, (or records are artificially generated), to be consistent with a single response spectrum that represents the seismic hazard in the assessed region. These records are then scaled by various means, to represent the entire range of ground motion severities used for the vulnerability curve generation. Use of a single earthquake scenario for the selection of all records will, however, not account for the change in the constitutive characteristics of records deriving from earthquake events of very different magnitudes and return periods. Mosalam et al. (1997) adopt two sets of synthetic ground motions for fragility analysis generated for the Mississippi embayment by Hwang and Huo (1994) and Horton et al. (1996). The synthetic records consider wave propagation and duration effects and are generated using multiple probability based scenario earthquakes with moment magnitude and fault distance ranges of 5.9-7.7 and 82-100km, respectively. Use of such record suites gives a more rigorous representation of the ground motion

variability over a wider range of ground motion severities, than is achieved by the scaling of records deriving from a single earthquake scenario. However, use of these suites in a vulnerability curve generation framework, may not necessarily eliminate the need for record scaling. For example, notwithstanding consideration of earthquakes with return periods of up to 18182 years by Hwang and Huo (1994), the magnitude-fault distance pairs selected for the accelerogram synthesis, result in *pga* values that are insufficiently large for a reliable and complete determination of the higher damage state curves in three out of four vulnerability curve sets derived by Mosalam et al. (1997). If it is further considered, that the damage states defined by Mosalam et al. (1997) correspond only to the “Slight” to “Moderate” damage range of the HRC Scale, it can be concluded that large scaling factors are required if the same suite is used in the development of analytical vulnerability curves herein. An extensive hazard analysis and large computational effort is involved in the synthesis of hazard consistent record suites for Europe. A simpler methodology for the selection of natural records that considers multiple earthquake scenarios is therefore desired.

The analytical methodology for vulnerability curve generation proposed in Chapters 6 and 7, involves the use of an adaptive pushover technique of analysis. Within the latter method, the applied force distribution is updated considering the relative ratio between the spectral ordinates of the response modes of importance. This means that the same push-over curve results from any single record, irrespective of the scaling. However, the structural performance under several ground motion intensities can be assessed from a single pushover curve, through spectrum scaling within the capacity spectrum framework of assessment proposed in Chapter 7. The number of records used, therefore, directly determines the number of analyses required for the vulnerability study. A satisfactory representation of ground motion uncertainty effects on structural response may be achieved using a relatively small number of earthquake records. Consequently, a procedure for the selection of earthquake records, which combines the use of different earthquake scenarios with the use of record scaling, should be considered to optimise the computational effort.



⁽¹⁾Fully Operational; ⁽²⁾Operational; ⁽³⁾Immediate Occupancy

Figure 4. 2: Diagram of the performance objectives defined by (a) Vision 2000 (SEOAC 1995) and (b) FEMA 273 (ATC 1997).

Seismic performance is assessed conditionally to the probability of occurrence of ground motion (i.e. earthquake return period, RP). Typically, multiple performance objectives must be checked, such as “Serviceability” under frequent earthquakes and “Collapse prevention” under very rare events. The attainment of each damage limit state should, therefore, be assessed using records that are representative of seismic hazard scenarios consistent with the performance objective. However, no official guidelines exist for assigning RPs to performance states. In the literature, the requirements of acceptable risk are observed to change according to the agency carrying out the assessment and the consequences of structural failure. For example, the US Department of Energy (DOE 1994) assigns earthquake return periods between 500 and 10,000 years to the serviceability limit state, depending on the performance category of the facility assessed/designed. According to Bentler (2001), the US Army Corps of Engineers adopt values of 144 and 950 years to assess non-critical structures for serviceability and collapse prevention, respectively. In Figure 4.2, the performance objectives of FEMA 273 (ATC 1997) and Vision 2000 (SEOAC 1995) are illustrated. Both sources define different design objectives for buildings of diverse importance. However, FEMA 273 assigns a much more severe earthquake hazard to the collapse prevention objective than Vision 2000.

Within this research, three scenarios corresponding to the performance states of “serviceability”, “damage control” and “collapse prevention” are considered in the

definition of earthquake characteristics for the selection of ground motion records. Following a review of existing literature, the earthquake RPs in Table 4.1, are chosen as representative of those assigned to buildings of ordinary importance. These hazard scenarios are adopted in Section 4.4.1, to define earthquake spectra for use in the selection of “performance consistent” accelerograms for use in building damage statistics simulation analyses.

Performance Limit State	Earthquake Return Period (years)
Serviceability	95
Damage Control	475
Collapse Prevention	2475

Table 4. 1: Performance states and associated earthquake return periods used to define the earthquake “target” spectra.

The number of accelerograms selected for each scenario is determined from consideration of ground motion uncertainty. The selected acceleration records are scaled to increasing values of pga , in order to cover the full range of ground motions required for the vulnerability curve definition. Such a scaling procedure is adequate for structures with natural periods below 0.5s (Chandler and Duan 1991, Tso et al. 1992). The structural population assessment results corresponding to the different “performance consistent” record sets, are considered separately in the definition of each limit-state vulnerability curve. The “Slight” and “Light”, “Moderate” and “Extensive”, and “Partial” and “Complete Collapse” damage state curves, are derived considering structural response observed for the “Serviceability”, “Damage control” and “Collapse prevention” record suites, respectively. The resulting set of “performance-consistent” vulnerability curves is appropriate for use in the multiple assessment of performance objectives, at any ground motion level. In order to fully define the vulnerability curve shape, the methodology requires the scaling of records associated with smaller, more frequent earthquakes, by large factors. However, in Section 7.4.2, the use of multiple scenario earthquakes for the definition of single damage state curves is not shown to alter significantly the vulnerability curve shape obtained for the performance consistent curves, in the region of ground motions that interest the performance objective satisfaction.

4.4.1 “Target” Earthquake Spectra Determination

The comprehensive effects of magnitude, distance and soil condition on structural demand are best represented through a response spectrum. Thus, three “target” spectra are developed that are consistent with the return period earthquakes and performance states of Table 4.1. The definition of a spectrum involves the choice of an attenuation relationship for the spectral ordinates. This in turn, requires as input, values of peak ground acceleration, earthquake magnitude, site soil conditions and source distance. In order to determine values for these parameters that are representative of European seismicity, the hazard maps for Italy produced by Albarello et al. (1999), are analysed. These hazard maps are derived using the average of the mean plus one standard deviation predictions of acceleration response, obtained from the attenuation relationships of Sabetta and Pugliese (1996) and Ambraseys et al. (1996). The hazard for each of the 8100 municipalities of Italy, is summarised for each of the three considered return periods, in terms of the peak ground acceleration and spectral acceleration ordinates at 10 discrete values of natural frequency. In order to determine values of peak ground acceleration (*pga*) that are appropriate for the characterisation of the different hazard scenarios, the frequency of occurrence in each return period hazard map, of the reported *pga* values for the 7724 seismically rated municipalities, are plotted. Lognormal cumulative probability functions, of the form of Equation 5.2 (Chapter 5), are then fit to the data using the non-linear regression module of STATISTICA (StatSoft Inc. 1995).

The fitted curves are shown in Figure 4.3. The parameters defining the curves and mean *pga* values observed across all the municipalities are shown in Table 4.2. Also shown in the table, are the *pga* values predicted to be exceeded and the mean *pga* values in the three seismic zones of Italy, (Zone 1 having the highest seismicity and Zone 3 the lowest). A moderate-high hazard scenario is represented by the mean peak ground acceleration over Zone 2. These correspond well with the values of 0.106g, 0.222g and 0.41g, for 100, 475 and 2475 year return period earthquakes typical of Portugal, proposed by Campos-Costa and Pinto (1999), to represent the same hazard scenario.

Return Period (years)	Fit Curve Parameters		Mean pga values across zone (g m/s ²)				Threshold pga values (g m/s ²)		
	μ	σ	All Italy	Zone1	Zone2	Zone3	Zone1	Zone2	Zone3
95	-2.808	0.535	0.069	0.138	0.107	0.068	0.133	0.081	0.055
475	-2.175	0.575	0.128	0.288	0.211	0.130	0.265	0.156	0.103
2475	-1.640	0.619	0.221	0.542	0.379	0.224	0.484	0.273	0.174

Table 4. 2: Cumulative peak ground acceleration fitted curve parameters (see Eq.5.2), mean and lower bound peak ground acceleration values for the seismic zones of Italy for return periods of 95, 475 and 2475 years and the hazard maps of Albarello et al. (1999).

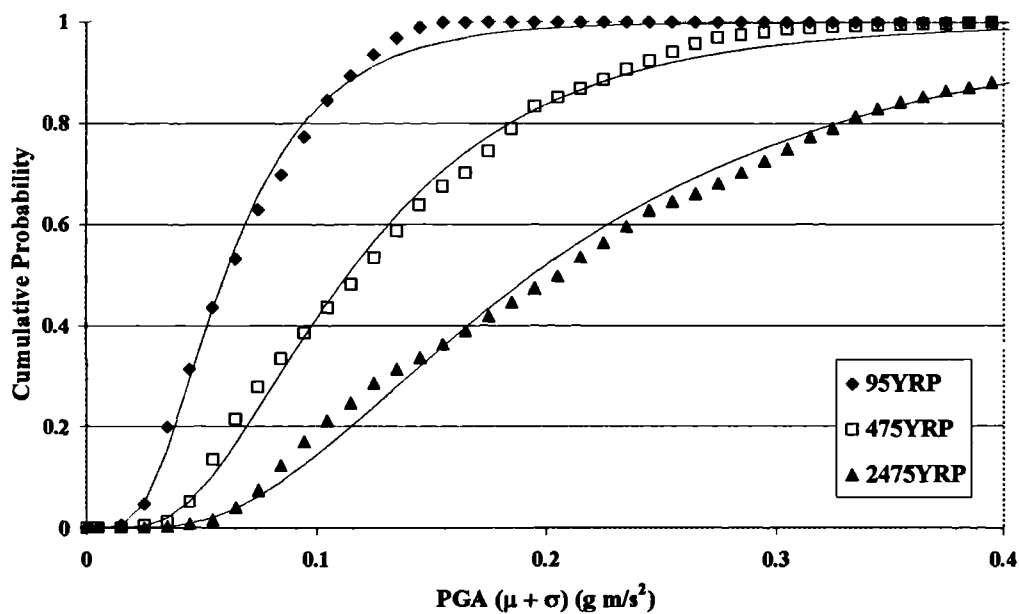


Figure 4. 3: The cumulative frequency of occurrence of (mean plus one standard deviation) pga values for 7724 seismically rated municipalities in Italy. Plots are shown for earthquakes with 475, 975 and 2475 year return periods, as predicted by Albarello et al. (1999). The smooth lines represent lognormal curves fitted to the data.

In order to determine a target spectral shape, values for the magnitude, distance and soil-type, associated with the selected *pga*-value and return period, must be determined. The attenuation relationships of Sabetta and Pugliese (1996) and Ambraseys et al (1996) are used to carry out a back analysis for the combination of magnitude, distance and soil-type resulting in a best fit to the spectral ordinates for each municipality. The process is iterative and is carried out using a program specifically written for this task. For each return period earthquake, the program selects the municipalities associated with *pga* values that lie between the upper and lower bounds of Zone 2. A soil category is chosen and the error in prediction

of the recorded spectral ordinates is calculated for each municipality, for all possible combinations of magnitude and distance variables. The parameter combination resulting in the lowest mean squared error throughout the period range, is then assigned to the municipality for the specified soil category. The next soil category is then selected and the process of magnitude-distance pair assignment is repeated. The magnitude-distance pairs are tallied and the most frequent parameter pair occurrence for each soil category and return period is determined and reported in Table 4.3.

Return Period (years)	Target pga (g m/s ²)	pga range for record selection (g m/s ²)	Target Surface Magnitude (Ms)	Target fault distance (km)	No. of records/ earthquakes/ stations ⁽¹⁾
ROCK Soil Site Conditions					
95	0.067	0.041-0.093	5.75	22	12/5/6
475	0.122	0.076-0.185	6.05	14	6/3/3
2475	0.217	0.138-0.349	6.25	8	10/4/5
FIRM Soil Site Conditions					
95	0.061	0.035-0.087	5.55	30	10/4/5
475	0.119	0.073-0.182	5.85	18	12/5/6
2475	0.207	0.128-0.339	6.15	12	22/10/11
SOFT Soil Site Conditions					
95	0.061	0.035-0.087	4.95	16	2/1/1
475	0.118	0.072-0.181	5.35	10	8/4/4
2475	0.223	0.144-0.355	5.45	4	1/1/0

⁽¹⁾The number of records available from the European Strong Motion Database (Ambraseys et al. 2002), for representation of the target spectra and the number of corresponding earthquakes and recording stations.

Table 4. 3: Peak ground acceleration, surface magnitude and fault distance values defining the target spectra for 95, 475 and 2475 year return period earthquakes under different soil conditions. The values relate to the hazard maps for Italy of Albarello et al. (1999).

Using the determined parameter combination, the attenuation relationships of Ambraseys et al. (1996) and Sabetta and Pugliese (1996), are used with equal weighting, (as per the derivation of the hazard maps), to establish “target” acceleration spectra for the three return periods considered. These are illustrated in Figure 4.4a. Equivalent displacement “target” spectra are also derived using the attenuation relationship of Borzi et al. (1998), and are shown in Figure 4.4b.

These target spectra are adopted in the selection of time-histories for the representation of ground motion uncertainty in vulnerability analyses.

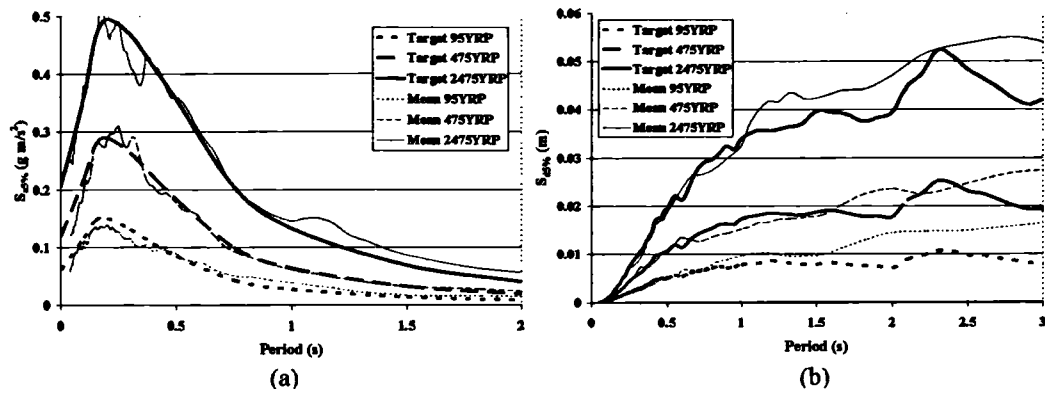


Figure 4. 4: Target and ensemble record suite mean spectra for 95, 475 and 2475year return period earthquake scenarios; (a) Acceleration spectra for 5% damping ($g \cdot m/s^2$), (b) Displacement spectra for 5% damping (m)

4.4.2 Choice of “Performance Consistent” Earthquake Records

It can be inferred from the work of Wen and Wu (2001), that 10 accelerograms are sufficient for the representation of the ground motion variability associated with a given spectral shape. This assumption is conditional to their being selected so that their ensemble median spectral shape closely resembles that of the chosen spectrum. Three suites of 10 records corresponding to the three damage-state spectra are, therefore, selected from the European Strong-Motion (ESM) Database (Ambraseys et al. 2002), for use as input to the analytical vulnerability curve generation. The ESM Database consists of a total of 1068 uncorrected and uniformly corrected records from Europe and adjacent regions, recorded at 376 stations following 432 shallow and intermediate depth earthquakes. Suites of possible natural records for each of the return periods are selected for rock, firm and soft soil sites, considering earthquakes of surface magnitude and distances within $\pm 0.5Ms$ and $\pm 10km$ of the values defining the target curves. Ranges of pga are also used in the record selection, which are defined by considering the deviation in pga values characterising the hazard in the Italian seismic Zone 2 (see Table 4.3). Only time-histories that do not show any special features, peculiarities or near source effects are considered in the selection. From these possible record

groups, the ten giving the best fit (in a least square error sense) to the target spectra are determined and used in the analytical vulnerability curve derivation. The characteristics of the selected records are summarised in Tables 4.4 to 4.6, where their fit to the target spectra is also quantified. The variation of the spectral shapes within each suite is shown in Figure 4.5, whilst the fit of their ensemble mean to the target spectral shapes is illustrated in Figure 4.4.

Although the adopted record selection technique is time consuming, the resulting record suites may be used to carry out vulnerability analyses in all areas with similar seismic activity and fault mechanisms to those considered (i.e. medium European seismicity, shallow crustal faulting). For areas with very different seismicity or characterised by subduction earthquakes, ranges of peak ground accelerations, earthquake surface magnitudes and fault distances can be found from historical earthquake catalogues or from local hazard maps using a similar method to that shown above. Target spectra can then be developed using appropriate attenuation relationships and suites of records for analysis chosen from catalogues of earthquakes with similar fault mechanisms.

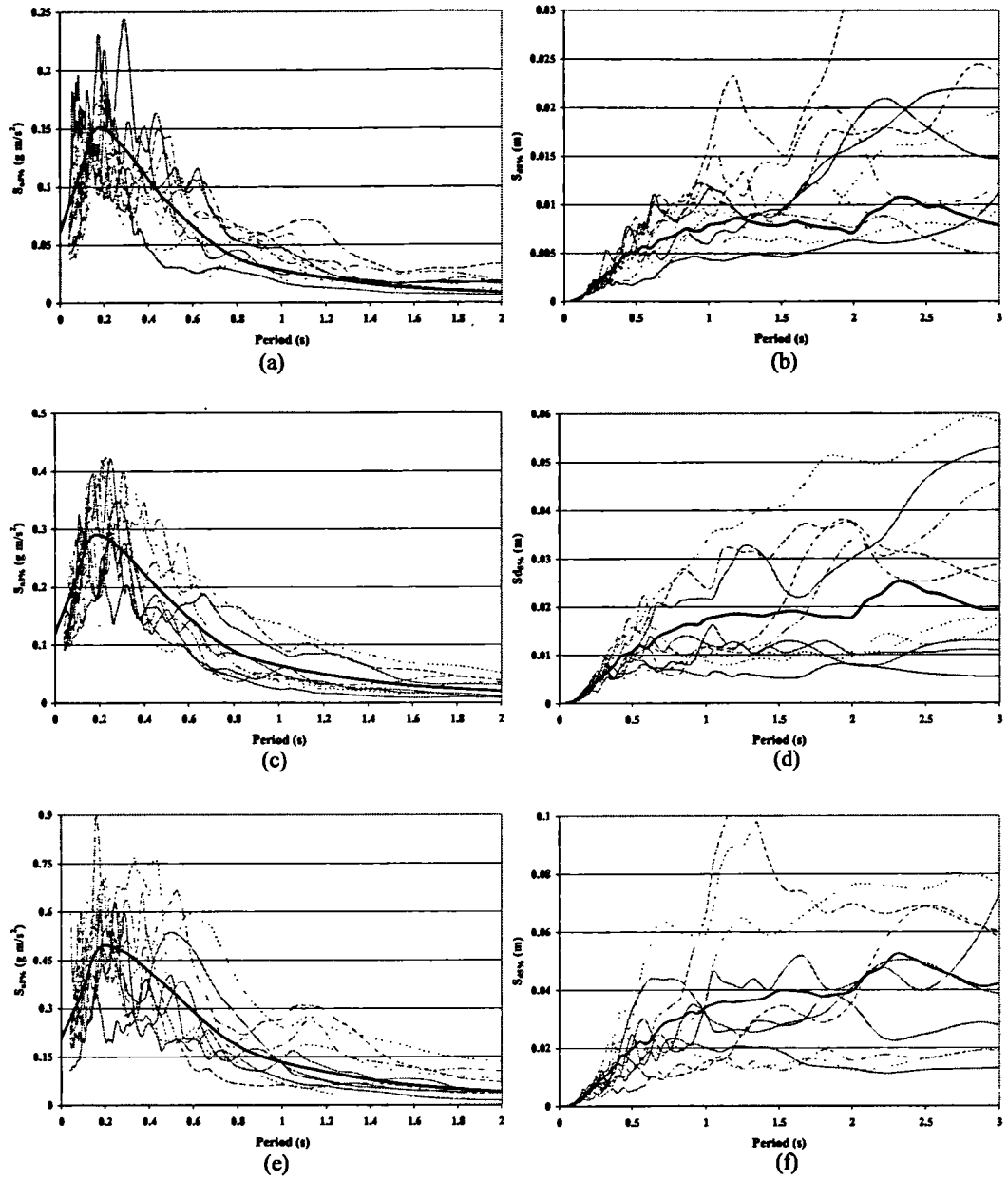


Figure 4. 5: Target and chosen record spectra at 5% damping; (a), (c), (e) Acceleration spectra, (b), (d), (f) Displacement spectra, for the 95, 475 and 2475year return period hazard scenarios respectively.

Earthquake Name	Country	Date	Record reference	Station Name and direction	Latitude	Longitude	Fault Mechanism	Depth (km)	Ms ⁽¹⁾	r ⁽²⁾ (km)	Soil	pga (g m/s ²)	pgv (cm/s)	Duration (s) ⁽³⁾	R ² SA ⁽⁴⁾	R ² SD ⁽⁵⁾
Aftershock of Friuli	Italy	15/09/1976 03:15:19	134y	Forgaria-Cornio, EW	46.29°	13.2°	oblique	15	6.06	12	stiff	0.217	10.24	21.99	0.757	0.768
Aftershock of Friuli	Italy	15/09/1976 09:21:19	147y	San Rocco, EW	46.32°	13.16°	thrust	12	5.98	9	stiff	0.236	18.70	16.75	0.752	0.826
Basso Tirreno	Italy	15/04/1978 23:33:48	171x	Naso, NS	38.27°	14.86°	thrust	15	5.83	16	stiff	0.152	8.31	28.97	0.696	0.442
Aftershock of Montenegro	Yugoslavia	24/05/1979 17:23:18	228x	Bar-Skupstina Opstina, NS	42.23°	18.76°	thrust	5	6.34	12	stiff	0.202	15.47	34.34	0.749	0.866
Valnerina	Italy	19/09/1979 21:35:37	242x	Cascia, NS	42.76°	13.02°	normal	4	5.84	6	rock	0.154	7.58	24.98	0.669	0.410
Kalamata	Greece	13/09/1986 17:24:31	414x	Kalamata-OTE Building, N10W	37.1°	22.18°	normal	8	5.75	5	stiff	0.272	23.55	29.73	0.651	0.660
Kilini	Greece	18/10/1988 12:34:05	436y	Zakynthos-OTE Building, S78E	37.9°	21.06°	thrust	12	5.65	11	stiff	0.151	8.01	27.82	0.738	0.825
Umbria-Marche	Italy	26/09/1997 09:40:30	592y	Colfiorito, EW	43.031°	12.862°	normal	6	5.91	3	stiff	0.330	14.67	48.32	0.419	0.739
Umbria-Marche	Italy	26/09/1997 09:40:30	600y	Castelnuovo-Assisi, EW	43.031°	12.862°	normal	6	5.9	23	soft	0.110	10.86	55.08	0.494	0.930
Ionian	Greece	23/03/1983 23:51:05	2015x	Argostoli-OTE Building, NS	38.23°	20.29°	strike slip	3	6.16	17	stiff	0.182	4.97	32.31	0.581	0.352

⁽¹⁾Ms is the surface magnitude; ⁽²⁾r is the fault distance in km; ⁽³⁾Duration of recorded accelerogram; ⁽⁴⁾Correlation coefficient with respect to the target acceleration spectrum; ⁽⁵⁾Correlation coefficient with respect to the target displacement spectrum.

Table 4. 4: Summary of the chosen acceleration records for the 2475 year return period hazard scenario and their fit to the target spectra.

Earthquake Name	Country	Date	Record reference	Station Name and direction	Latitude	Longitude	Fault Mechanism	Depth (km)	Ms ⁽¹⁾	r ⁽²⁾ (km)	Soil	pga (g m/s ²)	pgv (cm/s)	Duration (s) ⁽³⁾	R ² SA ⁽⁴⁾	R ² SD ⁽⁶⁾
Aftershock of Friuli e/q	Italy	11/09/1976 16:35:03	120y	San Rocco, EW	46.29°	13.18°	thrust	16	5.52	17	stiff	0.089	4.45	13.00	14.97	29.54
Aftershock of Friuli e/q	Italy	15/09/1976 09:21:19	138x	Kobarid-Osn. Skola, NS	46.32°	13.16°	thrust	12	5.98	19	alluvial	0.142	8.05	24.56	23.58	15.99
Aftershock of Montenegro e/q	Yugoslavia	24/05/1979 17:23:18	233x	Kotor-Zovod za Biologju Mora, NS	42.23°	18.76°	thrust	5	6.34	19	alluvial	0.118	7.26	30.67	27.27	21.42
Aftershock of Montenegro e/q	Yugoslavia	24/05/1979 17:23:18	234y	Herceg Novi-O.S. D. Pavicic, EW	42.23°	18.76°	thrust	5	6.34	18	rock	0.077	6.49	28.80	0.804	0.812
Preveza	Greece	10/03/1981 15:16:20	337y	Lefkada-OTE Building, EW	39.2°	20.8°	unknown	10	5.26	21	soft	0.099	5.87	0.813	0.673	0.630
Lefkas	Greece	27/05/1981 15:04:02	340x	Lefkada-OTE Building, NS	38.79°	21.01°	unknown	15	4.82	11	soft	0.119	4.55	0.808	0.741	0.755
Lazio Abruzzo	Italy	07/05/1984 17:49:43	365x	Atina, NS	41.73°	13.9°	normal	8	5.79	12	rock	0.100	3.68	0.687	0.582	0.572
Lazio Abruzzo	Italy	07/05/1984 17:49:43	378y	Cassino-Sant' Elia, EW	41.73°	13.9°	normal	8	5.79	17	alluvium	0.114	7.90	0.635	0.791	0.800
Aftershock of Spitak e/q	Armenia	07/12/1988 07:45:45	440x	Gukasian, NS	40.96°	44.27°	thrust	11	5.8	10	soft	0.147	10.77	0.634	0.342	0.689
Umbria-Marche	Italy	26/09/1997 09:40:30	602x	Matelica, NS	43.031°	12.862°	normal	6	5.9	21	stiff	0.101	9.12	0.814	0.717	0.809

⁽¹⁾Ms is the surface magnitude; ⁽²⁾r is the fault distance in km; ⁽³⁾Duration of recorded accelerogram; ⁽⁴⁾Correlation coefficient with respect to the target acceleration spectrum;

⁽⁶⁾Correlation coefficient with respect to the target displacement spectrum

Table 4. 5: Summary of the chosen acceleration records for the 475 year return period hazard scenario and their fit to the target spectra.

Earthquake Name	Country	Date	Record reference	Station Name and direction	Latitude	Longitude	Fault Mechanism	Depth (km)	M _s ⁽¹⁾	r ⁽²⁾ (km)	Soil	pga (g m/s ²)	pgv (cm/s)	Duration (s) ⁽³⁾	R ² SA ⁽⁴⁾	R ² SD ⁽⁶⁾
Aftershock of Friuli e/q	Italy	15/09/1976 09:21:19	140y	Robic, EW	46.32°	13.16°	thrust	12	5.98	12	rock	0.049	1.785	29.35	26.68	28.22
Aftershock of Friuli e/q	Italy	15/09/1976 09:21:19	142x	Codroipo, NS	46.32°	13.16°	thrust	12	5.98	38	stiff	0.045	2.657	22.96	24.84	28.48
Aftershock of Montenegro e/q	Yugoslavia	24/05/1979 17:23:18	232x	Kotor Nas Rakit, NS	42.23°	18.76°	thrust	5	6.34	19	rock	0.057	3.634	32.40	23.63	46.11
Valnerina	Italy	19/09/1979 21:35:37	246x	Arquata del Tronto, NS	42.76°	13.02°	normal	4	5.84	21	rock	0.061	4.031	13.76	0.606	0.620
Valnerina	Italy	19/09/1979 21:35:37	247y	Spoletto, EW	42.76°	13.02°	normal	4	5.84	21	stiff	0.039	2.052	0.807	0.686	0.680
Umbria	Italy	29/04/1984 05:02:59	360y	Citta di Castello-Regnano, EW	43.25°	12.47°	normal	7	5.38	33	soft	0.048	2.579	0.713	0.796	0.618
Lazio Abruzzo	Italy	07/05/1984 17:49:43	368y	Ponte Corvo, EW	41.73°	13.9°	normal	8	5.79	31	rock	0.068	3.854	0.477	0.600	0.590
Aftershock of Lazio Abruzzo e/q	Italy	11/05/1984 10:41:50	386y	Cassino-Sant'Elia, NS	41.77°	13.91°	normal	8	5.27	28	alluvium	0.037	2.602	0.853	0.815	0.776
Umbria-Marche	Italy	28/09/1997 00:33:16	595x	Bevagna, NS	43.023°	12.892°	normal	7	5.5	25	stiff	0.034	4.401	0.786	0.784	0.788
Umbria-Marche	Italy	28/09/1997 09:40:30	758y	Norcia-Altavilla, EW	43.031°	12.862°	normal	6	5.9	30	alluvium	0.046	3.042	0.937	0.654	0.900

⁽¹⁾M_s is the surface magnitude; ⁽²⁾r is the fault distance in km; ⁽³⁾Duration of recorded accelerogram; ⁽⁴⁾Correlation coefficient with respect to the target acceleration spectrum;

⁽⁶⁾Correlation coefficient with respect to the target displacement spectrum

Table 4. 6: Summary of the chosen acceleration records for the 95 year return period hazard scenario and their fit to the target spectra.

4.5 Effect of Ground Motion Variability on Structural Response

Notwithstanding the close fit of the mean spectrum of the records with the “target” shape (Figure 4.4), a large variation in the time-histories and spectra used to represent each earthquake return period scenario, is seen (Figure 4.5). This variation introduces uncertainty in the seismic response of structures. Figure 4.6 illustrates the results of adaptive pushover response of a three storey infilled moment resisting frame designed according to the codes in use in Italy in 1982, when subjected to the 2475 year return period suite of records.

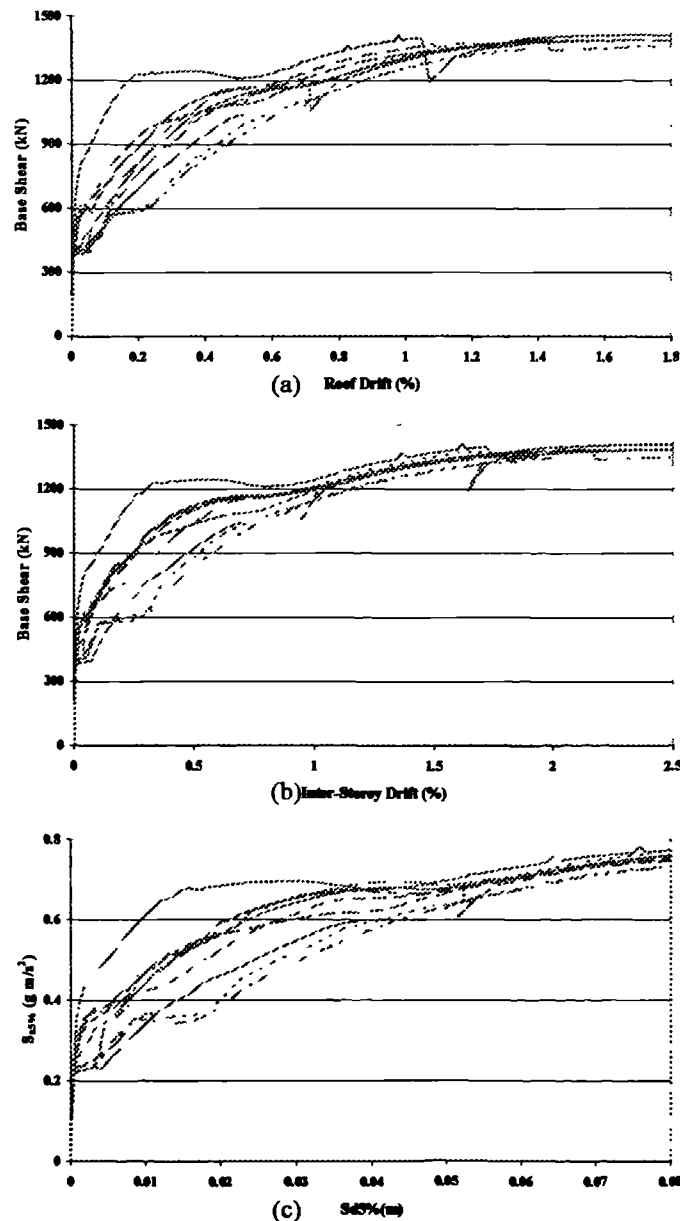


Figure 4. 6: Variation in the (a) base shear vs roof drift, (b) base shear vs inter-storey drift and (c) spectral acceleration vs displacement, adaptive pushover of a 3 storey infilled RC frame designed to the Italian codes in place in 1980, subjected to the 2475 year return period suite of records.

Similar induced uncertainties are also observed in the infilled frame response for the 95 and 475year return period earthquake sets. The average variation in maximum inter-storey drift ratio ($ISD_{max\%}$), and elastic spectral displacement ($S_{d5\%}(T)$) due to ground motion uncertainty, are summarised in Table 4.7.

Return Period	Scaled pga (gm/s^2)	$ISD_{max\%}$ (%)		$S_{d5\%}(T)(m)$	
		Mean	COV (%)	Mean	COV (%)
95YRP	0.10	0.0275	23.25	0.00094	28.18
	0.25	0.1090	28.35	0.00227	25.88
	0.49	0.3730	48.64	0.00471	25.74
	0.74	0.6829	43.20	0.00719	26.62
475YRP	0.10	0.0226	22.60	0.00077	29.00
	0.26	0.1378	22.16	0.00192	17.17
	0.51	0.4753	32.13	0.00393	16.91
	0.77	0.8450	28.14	0.00621	17.02
2475YRP	0.07	0.0168	41.42	0.00066	36.21
	0.29	0.1365	47.50	0.00222	34.50
	0.51	0.4171	57.59	0.00422	32.20
	0.73	0.7291	51.79	0.00592	34.30

Table 4. 7: Summary of the average variation in the maximum inter-storey drift ratio ($ISD_{max\%}$) and 5% critically damped spectral displacement response ($S_{d5\%}(T)$) observed in each of the 25 infilled structures composing the population for the analytical curve generation, when subjected to all earthquake records within each scenario suite, scaled to 4 pga values.

The reported values are calculated from the mean variation in the response of each of the 25 frames of different material combination, when subjected to the ten earthquakes composing each suite, scaled to a series of pga values. The ground motion variation results in an average coefficient of variation of 27.0% in the $S_{d5\%}(T)$, over all scenarios. This leads to average variations in the $ISD_{max\%}$ predicted for each structure, at a given level of pga , of up to 37.2% (COV). These induced response uncertainties are greater than the average COV of 21% resulting from material property variation within the structural population (See Section 6.3.3). This observation emphasises the importance of including ground motion uncertainty in analytical vulnerability evaluations.

Chapter 5

Empirical Vulnerability Curves

5.1 Introduction

Empirical curves are derived from observations of damage distributions in buildings, made during past earthquakes. This assumes that the earthquake damage observed in buildings in the past is representative of the future performance of similar constructions, subjected to comparable events. The advantage of the observational source of damage statistics is that all aspects of the ground motion and structural characteristics are represented. However, severe practical limitations on the quantity and quality of this data exist. These are discussed in Section 2.2.1, and are mainly attributed to the infrequent occurrence of damaging earthquakes near densely populated areas and the rapid execution of post-earthquake surveys.

The viability of deriving reliable vulnerability curves from observational statistics is herein assessed. In Chapter 2 and Appendix A, existing empirical relationships are shown to be inadequate for use in the seismic assessment of European buildings. An attempt is therefore made to derive new empirical vulnerability curves for this purpose, using a large databank of observed post-earthquake damage distributions, (see Section 5.2). The HRC scale and spectral displacement (for 5% critical damping), are used to characterise structural damage and seismic demand, respectively. In view of the limitations imposed by existing observational data, a new methodology for the derivation of empirical curves is proposed, which allows the combination of damage distributions for all structural systems, countries and earthquakes. The problem of data scatter is addressed within the empirical data processing, and is accounted for in the definition of confidence bounds for the curves. Sensitivity analyses are carried out to assess the influence of assumptions made during the curve derivation, on the resulting vulnerability predictions.

5.2 Observational Damage Databank

A database of observed post-earthquake damage distributions is assembled with the purpose of deriving new empirical vulnerability relationships for the assessment of European reinforced concrete building populations. A total of 99 datasets is collected from survey reports published by research groups and governmental authorities. Each dataset consists of the damage distribution observed in a population of reinforced concrete buildings surveyed at a specific location after the occurrence of an earthquake. The database comprises a total of 340,000 reinforced concrete buildings and includes observations made in 19 earthquakes (Table 5.1). Although numerous sources of possible damage statistics were consulted, only 29 were found to present damage statistics in an appropriate format for inclusion in the empirical curve derivation. Incomplete statistics, where all buildings in an affected area are not surveyed or where damage statistics are presented only as proportions of the buildings damaged, are excluded. Furthermore, surveys concerning special building types only, (e.g. schools or hospitals), which are not representative of the construction practice used in the general building stock, (e.g. have special seismic detailing), are also not considered in the database assembly.

To allow ground motion parameter values to be determined for each dataset, site recordings of the ground motion and information regarding the earthquake, fault location and site geology are collected from the literature. In order to obtain sufficient data for the vulnerability curve regression and to cover a wide range of ground motions, a large number of damage statistics for non-European earthquakes are included in the database. However, it is important to note that it is not the number of datasets in absolute terms, but the dataset distribution with ground motion severity, that determines the vulnerability curve shape. Figure 5.1, shows that across most of the ground motion range, the proportion of European data dominates both in terms of dataset and building numbers. However, the figure also shows the clustering of damage observations in the low ground motion range. This leads to uncertainty in the determination of the vulnerability curve shape for high ground motions and for the severe damage states.

Country	E/q ref ⁽¹⁾	Earthquake	M _s	No. of Dsets ⁽²⁾	No. of Bldgs ⁽³⁾	Source
Algeria	1	Chenoua, 1989	5.7	1	1,073	Farsi & Belazougui (1992)
Chile	2	Valparaiso, 1985	7.8	1	322	Spence (1998)
Greece	3	Korinthos, 1981	6.7	1	97	Pomonis et al. (1992)
	4	Alkyonides Gulf, 1981	6.4	2	286	Pitilakis et al. (1982)
	5	Kalamata, 1986	5.9	2	625	Pomonis et al. (1992)
	6	Aegion, 1995	6.5	1	1,157	Karantoni et al. (1997)
	7	Konitsa, 1996	5.4	3	264	Dandoulaki et al. (1997)
Italy	8	Campania, 1980	6.9	28	3,306	Martin Center (1998)
	9	Umbria-Marche, 1997	5.9	2	753	Sabetta & Capotorti (1998) Di Pasquale et al. (1998)
Japan	10	Miyagi-ken-oki, 1978	7.7	3	349	Onose (1982)
	11	Kobe, 1995	6.8	37	33,857	AIJ (1995), Inoue et al. (1999), Akamatsu et al. (1995), Murao & Yamazaki (1999), Otani (1999), Nakano (1996), Whittaker et al. (2000), Yamaguchi et al. (1998)
Mexico	12	Mexico City, 1985	8.1	3	4,896	Jara et al. (1992), Otani (1999)
Philippines	13	Luzon, 1990	7.8	1	181	Otani (1999)
Turkey	14	Erzincan, 1992	6.8	2	447	Pomonis et al (1992) Otani (1999)
	15	Adana-Ceyhan, 1998	6.2	1	65	Wenk et al. (1998)
	16	Kocaeli, 1999	7.8	7	290,718	Ascheim (2000), Sengezer et al. (1999)
USA	17	Santa Barbara, 1925	6.3	1	24	Thiel & Zsutty (1987)
	18	Long Beach, 1933	6.2	1	22	Thiel & Zsutty (1987)
	19	Northridge, 1994	6.8	2	1,430	Singhal & Kiremidjian (1997)
Total				99	339,872	

⁽¹⁾Earthquake reference number; ⁽²⁾Number of datasets; ⁽³⁾Total number of buildings in the datasets.

Table 5. 1: The composition of the observational damage database used for the empirical vulnerability curve derivation.

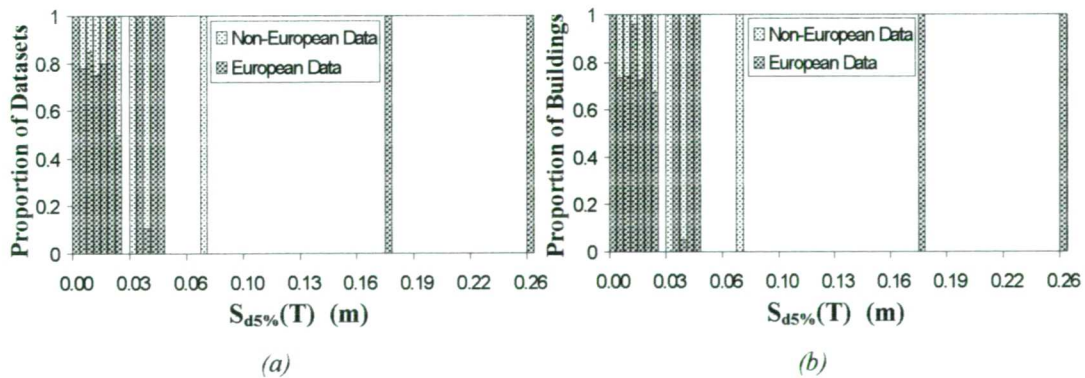


Figure 5. 1: Distribution with spectral displacement ($S_{d5\%}(T)$) of the relative proportion of European and Non-European collected observational post-earthquake survey data in terms of (a) damage distribution datasets and (b) surveyed buildings.

Only observational statistics concerning structures of similar lateral-load resisting system to those constituting the existing European stock are considered. When the exact structural composition of the surveyed population is not specified by the damage data source, alternative literature sources are consulted to assign a predominant building type, height and seismic code to the datasets. Damage statistics concerning populations of mixed structure composition are assigned to a “general” building class. Three height classes are considered, with height ranges chosen in accordance with HAZUS99 (FEMA 1999). These classes represent the changes in construction materials, member dimensioning and detailing, and wind resistance provisions specified in most existing design codes, for buildings of increasing height. New, old and pre-seismic code building design categories are considered to distinguish between the level of capacity design and seismic detailing adopted in the buildings. The structural composition of the observational damage database is summarised in Table 5.2. As mentioned in Section 3.2, the bare MRF structure class is considered to include frames with weak infills not expected to significantly contribute to the lateral resistance of the structure, whilst the infilled MRF structure class only contains frames with infills made of brick masonry or concrete blocks that are well connected to the surrounding frames and that are expected to provide a significant lateral-load resistance.

Structural Type		Bare MRF	Infilled MRF	Shear-wall Structures	Undefined System	Total
Total No. of Datasets		43	8	1	47	99
Total No. of Buildings		296513	2720	322	40317	339872
Datasets Discretised by Height	Low-Rise (≤ 3 floors)	36	8	0	10	54
	Med-Rise (4-7 floors)	13	2	1	43	59
	High-Rise (≥ 8 floors)	6	2	0	8	16
Datasets Discretised by Seismic Code	Pre-Code	33	4	0	28	65
	Old-Code	10	3	1	21	35
	New-Code	0	1	0	2	3

Table 5. 2: Structural composition of the observational damage database used for the empirical vulnerability curve derivation.

The non-uniform distribution of the number of surveyed buildings between different countries, earthquake events and structural systems is considered in the empirical curve derivation in Section 5.3. Prediction errors, introduced by differences in construction practice of the structures within each building class, are shown to be of comparable size on a national/single event and international/multiple event scale in Section 2.3. Thus, given that the chosen ground motion parameter realistically represents the imposed seismic demand, the combination of observational data for similar structures, deriving from different locations and earthquakes is justified.

5.3 Empirical Curve Derivation

In order to carry out risk assessments for building populations of varied composition, it is either necessary to develop a series of vulnerability curve sets for different building types, or use observational damage statistics that provide a compositional match to the assessed building stock. In practice, it is virtually impossible to find the observational data required for the latter method, especially if similitude in founding soil characteristics and seismic activity are also considered. The former method requires the grouping of buildings into structural classes with similar dynamic behaviour, and the definition of performance criteria that account for the structural features characterising each class. In practice, this means the separate treatment of buildings of different construction material, horizontal load resisting system, configuration, height, age and seismic detailing. From Table 5.2, it is clear that sufficient observational data, for the derivation of vulnerability curves specific to a single lateral-load resisting system, is only available for the category of reinforced concrete non-ductile bare frames. However, even in the latter case, the ground motion range covered by the data is insufficient for a reliable determination of the curve shape. Furthermore, the low level of refinement of the survey damage statistics impedes the separate treatment of data regarding structures of different heights and seismic codes. Consideration of other features influencing building seismic resistance, such as configuration, regularity in plan and elevation, previous damage, repair and strengthening is virtually impossible.

A new approach to the derivation of empirical vulnerability curves is therefore proposed, wherein data for different structural systems is combined to produce a single set of “homogenised” or “general” curves applicable to all. The difference in the damage rate of disparate systems is accounted for through the use of the HRC damage scale, (presented in Chapter 3). This scale is calibrated to the parameter of maximum inter-storey drift ratio ($ISD_{max\%}$), for different lateral-load resisting systems. The reinterpretation of the heterogeneously defined observational damage statistics to the HRC damage scale, according to the predominant structural system in the surveyed population, is discussed in the following section. The procedure followed for the development of vulnerability curves from this data, is then explained. Finally, the success in homogenising damage data for different structures, associated with different construction practices, countries and earthquakes, is discussed.

5.3.1 Homogenisation of Observational Data to the HRC Scale

Each dataset can be visualised as a bar chart (Figure 5.2a), wherein the proportion of the surveyed buildings lying within each damage state are plotted side-by-side, in order of increasing damage severity. The collected post-earthquake damage statistics are associated with different damage scales. Hence, these bar charts cannot be directly compared, or combined in the derivation of vulnerability curves. Furthermore, the assumption of a uniform distribution of the damage states along the horizontal axis of the plots is physically unrealistic due to the non-linear nature of damage progression in structures. In order to obtain a uniform criterion for the damage evaluation, the 99 datasets are re-interpreted in terms of the HRC-Scale. Four approximate correlation tables, (corresponding to different structural systems), between the HRC and other existing damage scales are presented in Tables 3.3 to 3.6. These tables are used to assign DI_{HRC} values to the damage states of every dataset, according to the predominant lateral-load resisting system of the surveyed buildings. Where the surveyed population is of mixed composition, the correlation table for general structures is used (Table 3.2). Equations 3.20 to 3.23, are then used to convert the thus assigned damage indices

into equivalent $ISD_{max\%}$ values. The 99 datasets are re-plotted using maximum inter-storey drift ratio (normalised to 6%) as the horizontal axis (Figure 5.2b). In this way a continuous scale is used to define the spread of the observed damage frequencies along the damage-axis of the histograms. In order to calculate damage state exceedence probabilities for the HRC limit states, continuous cumulative distribution functions are fit to the observed damage frequencies of each dataset (Figure 5.2c).

The histograms of the observed reinforced concrete building damage (Figure 5.3), reveal that in locations of low ground motion severity, the frequency distributions

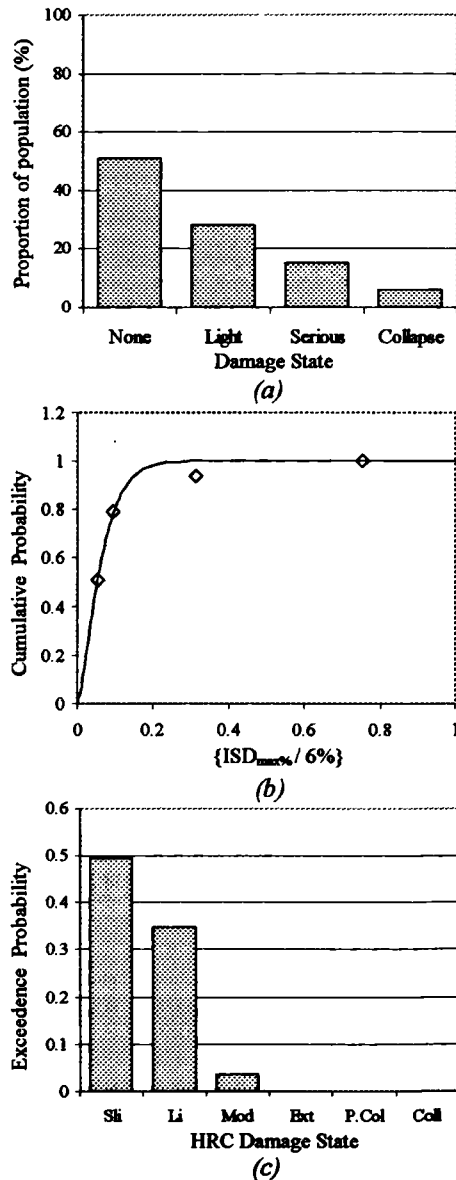


Figure 5. 2: Conversion of the RC building damage statistics observed in Plateas after the Korinthos Earthquake (Greece, 1981) to the HRC-Scale

skew heavily towards the lower damage states. Vice-versa, in the case of severe ground motions there is a shift in the distributions towards the higher damage states. These observations promote the use of either the Beta or Lognormal probability distribution functions (pdf). The latter is used by Singhal and Kiremidjian (1997) to represent the damage observed in low-rise RC frames after the Northridge Earthquake (USA, 1994). The ability of the Lognormal pdf to skew to the left is seen to be adequate for representing the predominantly low levels of damage that are observed in the latter case. The Beta pdf, which is able to skew both left and right, provides a greater flexibility for the modelling of distributions dominated by high levels of damage. It is used in the characterisation of the expert damage predictions, which form the statistical basis of the ATC-13 (ATC 1985) vulnerability curves. The

cumulative Beta probability distribution function $F_\beta(x)$, is defined by:

$$F_\beta(x) = \int_{-\infty}^x \frac{\Gamma(\alpha + \beta)}{\Gamma(\alpha)\Gamma(\beta)} x^{\alpha-1} (1-x)^{\beta-1} \quad \text{for } 0 < x < 1, \alpha > 0, \beta > 0 \quad [5. 1]$$

$$\text{where } \Gamma(c) = \int_0^\infty x^{c-1} e^{-x} dx; \quad \mu = \frac{\alpha}{\alpha + \beta}; \quad \sigma^2 = \frac{\alpha \cdot \beta}{(\alpha + \beta)^2 (\alpha + \beta + 1)}$$

The cumulative Lognormal distribution function $F_L(x)$ is defined by the equations:

$$F_L(x) = \int_{-\infty}^x \frac{1}{\sqrt{2\pi\beta}} x^{-1} e^{-(\ln x - \mu)^2 / 2\beta^2} \quad \text{for } x > 0, \beta > 0 \quad [5. 2]$$

$$\text{where } \mu = e^{\alpha + \beta^2 / 2}; \quad \sigma^2 = e^{2\alpha + \beta^2} (e^{\beta^2} - 1)$$

In the present case, x is the maximum inter-storey drift ratio value, (corresponding to the reported damage state and scale of each dataset), normalised to a value of 6%. μ and σ^2 are the mean and variance of x , whilst α and β are parameters describing the shape of the distribution functions.

Beta and Lognormal cumulative probability functions are fit to each of the 99 re-interpreted damage datasets, via non-linear regression. The program STATISTICA (Version 5.0, StatSoft Inc. 1995), is used to evaluate the model function shape parameters through minimisation of the square of the residuals. Kolmogorov-Smirnov one-sample tests at significance levels of 1%, 5% and 10% are carried out to assess the goodness of fit of the functions with the data. The Kolmogorov-Smirnov method is chosen as, unlike other goodness of fit tests such as the chi-square test, it can be applied to very small samples. This is important, as the fit can only be checked at three or four locations for most datasets, (i.e. corresponding to the number of defined damage states). If the maximum absolute difference between the fitted curve and the sample is less than the value pre-

defined by the method for a specified sample size and level of significance, then the distribution is said to satisfy the test at that significance. The higher the significance level satisfied, the better the fit of the continuous distribution to the

observational data. The results of the Kolmogorov-Smirnov tests are presented in

Appendix E. The Beta distribution is seen to satisfy the 10% significance test in the case of all 99 datasets. The Lognormal distribution is instead seen to fail the 5% and 10% significance tests in 2 and 7 cases, respectively. Furthermore, the maximum discrepancy values between the observational data and the latter fitted distributions, is on average greater than in the case of the Beta distribution.

Beta cumulative probability distribution functions are therefore used to model the observational damage distributions. All datasets are now expressed as continuous distributions of normalised $ISD_{max\%}$, and datasets pertaining to a structural class may be directly compared. Furthermore, damage state exceedence probabilities for all systems may be compared, if due consideration is taken of the different $ISD_{max\%}$ limit state values, defined for disparate lateral load resisting systems, in

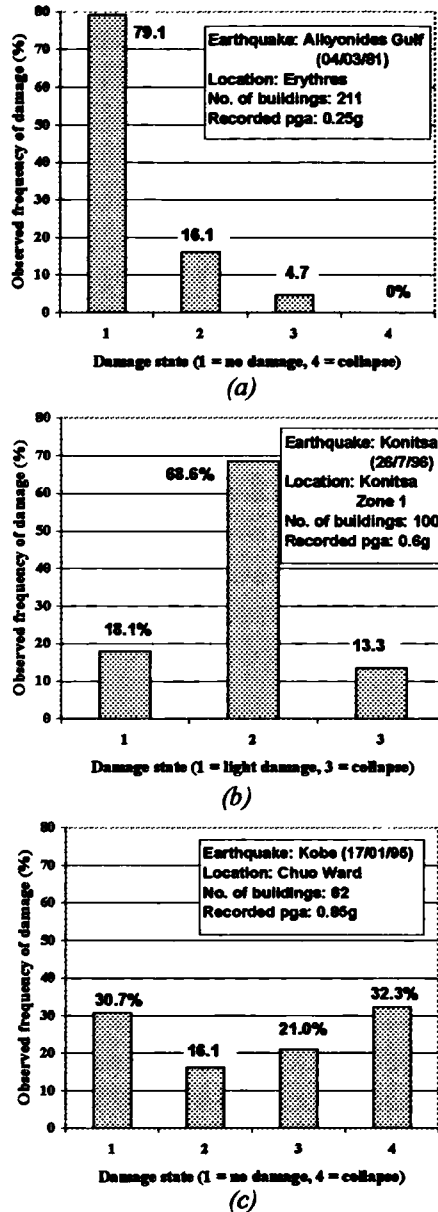


Figure 5.3: Example of the change in the distribution of building damage frequencies observed for (a) Low, (b) Medium and (c) High ground motion severities.

Table 3.13. The Beta cumulative function shape parameters, fit to each dataset, are presented in Appendix E and form the statistical basis for the empirical vulnerability curve derivation.

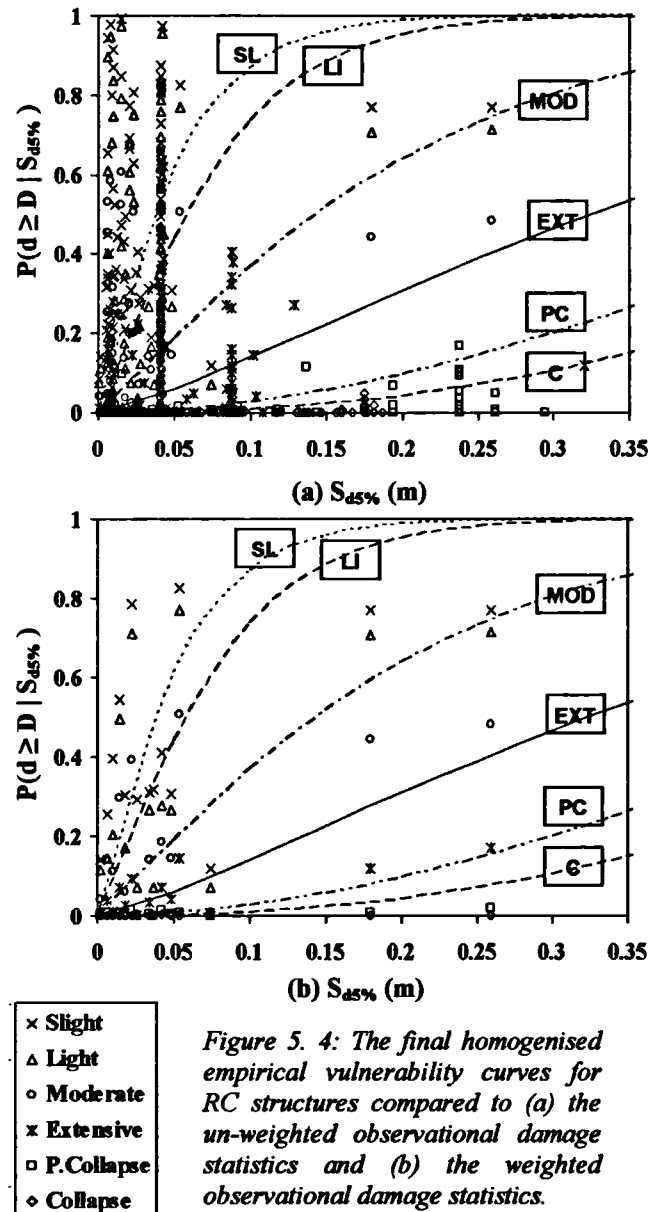
5.3.2 Weighting of the Observational Data

Exceedence probabilities for the HRC damage limit states are determined from the Beta distributions fit to each dataset. The limit state values of $ISD_{max\%}$ defined in Table 3.13, chosen in accordance with the predominant structure type of the dataset, are used for this purpose. The attenuation of Borzi et al. (1998) is used to assign spectral displacement values to each dataset, as described in Section 4.3. 99 values of exceedence probability result for each HRC damage limit state. These are plotted against the corresponding ground motion parameter values to form the vulnerability plot for the homogeneous curve derivation.

Equal weighting of the exceedence probability points is commonly assumed in the generation of existing empirical curves due to the small quantities of observational data considered. Therefore, in the present case, a weighting procedure is required to deal with the substantial amount of raw data available and large associated scatter (Figure 5.4a). In the presence of similar ground motion severities, the damage state exceedence probabilities derived from each dataset are combined according to relative numbers of surveyed buildings. The weighted statistics are plotted against the median value of the corresponding ground motion interval, and non-linear regressions carried out to derive the vulnerability relationships (Figure 5.4b).

Apart from facilitating the identification of trends in the data, the weighting procedure reduces the influence on the curve shape, of single earthquake events and of non-European earthquakes (in view of the relative proportions of European and non-European data in Figure 5.1). The high frequency of small magnitude earthquake observations at low ground motions lessens the influence of any single such event, (and of any observation associated with large magnitude events), in this region. Unlike the case of the vulnerability curves derived by Borzi (1999), probabilities of exceedence of damage states, rather than damaged building numbers, are here combined for the curve determination. Hence, the influence of datasets with large sample sizes is attenuated and is not seen to unduly influence the curve shape. This is observed to be true even in the specific case of the

Kocaeli earthquake (Turkey, 1999), which has datasets of large sample size. The latter earthquake provides a vast source of damage data for a variety of structural classes and a unique source for European damage observations in the large ground motion range. Its influence on the vulnerability curve shape in this range is seen to be significant but is justified by the high reliability of the data and its relevance to a European risk assessment scenario.



5.3.3 Vulnerability Curve Fitting

S-shaped functions are used to model building population vulnerability in existing relationships. They are able to account for the fact that the onset of damage occurs

at ground motion parameter values greater than zero, and that once a certain level of ground motion is reached, the occurrence of a given damage state is certain. The functions most commonly used to represent the shape of vulnerability curves in existing relationships, are cumulative normal and lognormal distributions. These and other cumulative distribution functions are assessed to model the vulnerability curve shapes. Non-linear regressions are carried out to fit the considered function to the observational statistics, using the program STATISTICA (Version 5.0, StatSoft Inc. 1995). A comparison of the models is carried out considering both acceleration and displacement-based peak and spectral ground motion parameters.

$$P(d \geq DI_{HRC} | GM) = 1 - \exp(-\alpha \cdot GM^\beta) \quad [5.3]$$

The determination of the curve shape for severe damage states and at intermediate and high levels of ground motion, are prone to large uncertainties due to the scarcity of observational data. Indeed, the vulnerability curves could be extrapolated in a number of different shapes and still maintain the same fit to the observational data. Choice of a distribution function to model the curve shape is therefore based on observation of the coefficient of correlation (R^2) and the maximum difference between the data and the fitted curves, for the “Light” to “Extensive” HRC damage states. Following many trials, cumulative lognormal functions (Equation 5.2) and relationships of the form of Equation 5.3, are found to give the best fit to the data when $S_{a5\%}(T)$ is used to characterise the seismic demand. However, the latter relationship is seen to yield the optimum fit for all considered ground motion parameters (GM). This functional form is therefore used to derive the empirical vulnerability curves in this study, with the parameters α and β being found from non-linear regression on the plotted observational data.

The thus derived homogeneous empirical vulnerability curve shape parameters are summarised in Table 5.3 and illustrated in Figure 5.4. Notwithstanding the weighting process, the latter Table clearly shows the existence of a large data scatter around the derived curves. Confidence bounds are required for use with the empirical curves to account for the scatter and compensate for the scarcity of observations for the high damage states and ground motions.

HRC Damage State	Curve Parameters						Bldgs	Non-weighted data				Weighted data			
	Mean		U90%		L90%			Data sets (3)	$\mu(\epsilon)$ (4)	cov (ϵ) ⁽⁵⁾	R ² (6)	Data sets	$\mu(\epsilon)$	cov (ϵ)	R ²
	$\alpha^{(1)}$	$\beta^{(2)}$	α	β	α	β									
Slight	25.82	1.10	76.45	1.10	13.72	1.10	339872	99	0.23	0.83	0.37	15	0.23	0.68	0.53
Light	21.08	1.20	73.88	1.20	8.350	1.20	339187	90	0.23	0.93	0.30	15	0.22	0.73	0.54
Moderate	6.500	1.15	29.57	1.15	2.342	1.15	331702	69	0.17	1.02	0.30	14	0.14	0.83	0.59
Extensive	3.000	1.30	17.52	1.30	1.323	1.30	329152	49	0.07	1.32	0.28	13	0.06	1.25	0.65
P.Coll.	2.500	2.00	13.45	2.00	1.200	2.00	292839	35	0.03	1.37	0.28	12	0.02	1.86	0.53
Collapse	2.000	2.40	9.37	2.40	1.119	2.40	77876	24	0.01	1.62	0.36	6	0.02	2.14	0.50

⁽¹⁾ α and ⁽²⁾ β are the shape parameters (Eq.5.3) for the mean, upper and lower 90% confidence bounds; ⁽³⁾Number of datasets used for the derivation and comparison (weighted and non-weighted respectively); ⁽⁴⁾Mean error in prediction; ⁽⁵⁾Coefficient of variation of the error in prediction; ⁽⁶⁾Coefficient of correlation of the data to the fitted curves.

Table 5. 3: Summary of the homogeneous empirical vulnerability relationships, using the HRC damage scale and $S_{avg}(T)$ to characterise the damage and ground motion, respectively.

5.3.4 Confidence Bounds

A large scatter is associated with the observational data used to derive the empirical curves. This uncertainty may partly arise from the procedure followed in the combination of heterogeneous damage data for the homogeneous curve derivation, (this is further discussed in Section 5.4). Representation of each dataset via a single ground motion value, soil type and predominant structure system does not account for the natural variation in these parameters at the survey site. Hence damage distributions from different sites, with the same predominant structure and ground motion properties, may show significant differences. A substantial part of the scatter may also be attributed to errors in damage or building classification, born of the rapid survey procedure and the use of coarse damage scales. Both these sources of uncertainty are inherent in the data and may not be removed via data manipulation.

Confidence bounds are therefore defined to allow conservative risk assessments to be made. It is not sufficient to assume a constant standard deviation from the mean prediction, for all damage state curves and ground motions (e.g. Spence et al. 1992). This is due to the fact that the relationships are non-linear, the distribution of scatter around the curves is not constant and there is a bias in the quantity of

data for low damage states and ground motions. The confidence associated with each damage state curve is therefore assessed across the vulnerability plot, by dividing the ground motion axis into intervals. Each interval is taken in turn and the standard deviation of the plotted data about each damage state curve, is calculated. A probability distribution function is then defined to model the spread of the observations about the mean curve, for each ground motion interval and damage limit state. If the number of observations within an interval is large, (greater than 30 in this case), the error is assumed to follow a normal distribution about the mean curve. If the sample is small, there is an uncertainty associated with the determination of the standard deviation of the predicted error. Student T-distributions are adopted, which are able to account for this added uncertainty. The fitted error distributions are used to locate the position of the 95% and 5% confidence levels, associated with each damage state curve, at each ground motion interval. Non-linear regressions are carried out on the latter points, to define relationships of the form of Equation 5.3 for the upper and lower 90% confidence bounds of the damage state curves.

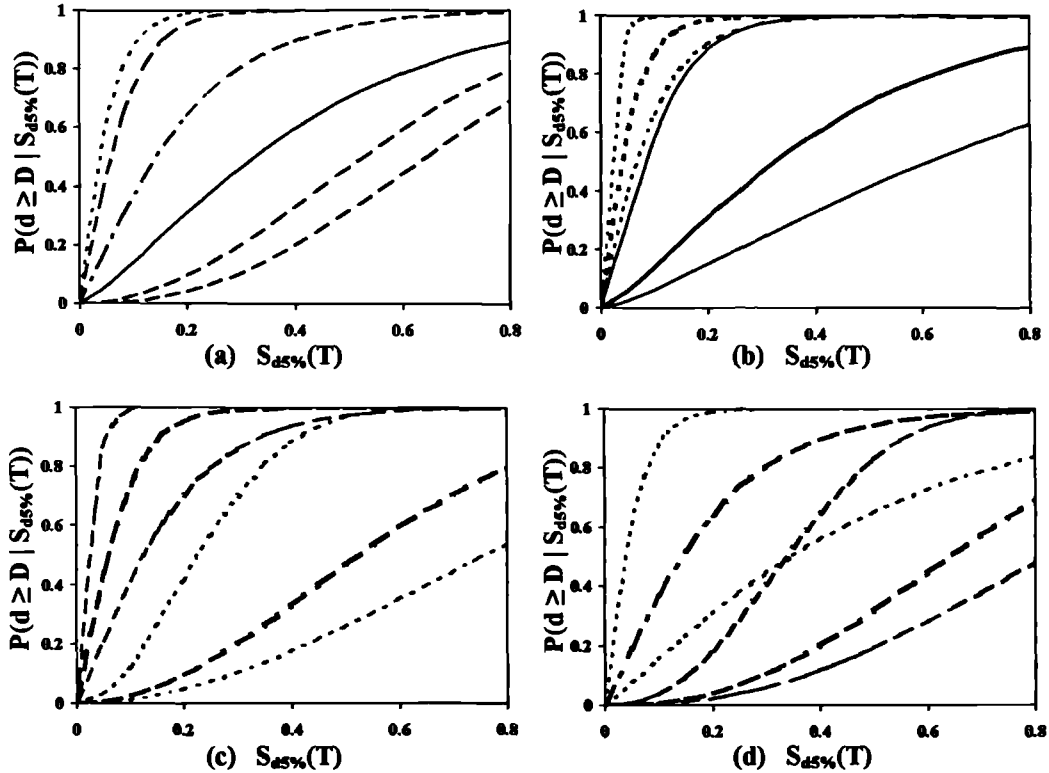


Figure 5. 5: The homogeneous empirical vulnerability curves (dark lines) and 90% confidence bounds (light lines): (a) Mean curves; (b) Slight and Extensive damage states; (c) Light and Partial Collapse damage states; (d) Moderate and Collapse damage states.

Wide confidence bounds are observed for all limit states, (Figure 5.5), which are slightly larger in size to those derived by Singhal and Kiremidjian (1997) for their analytical vulnerability curves (see Appendix A). The latter, account for the variation in damage index, which arises from uncertainty in the building material properties. However, they do not account for other sources of uncertainty associated with observational data, such as damage classification errors (which introduce scatter at low damage states), or scarcity of data (which causes large bounds to be defined by the Student T-distributions for the high damage state empirical curves). The similarity in the confidence intervals of the proposed homogeneous curves and those of Singhal and Kiremidjian (1997), which are derived for a single structural system, is regarded as proof of the success of the proposed methodology in reducing the effect of structure type on vulnerability curve shape. However, the degree of this success may only be ascertained via sensitivity analyses.

5.4 Sensitivity Analysis and Discussion of the Empirical Curves

In Table 5.3, the proposed relationships are seen to give a better fit to the damage data used for their derivation, than is reported by the authors of other existing empirical curves, despite the substantially larger number of observations and structural types considered. This indicates that the procedure used to define the damage states, exceedence probabilities and choice of ground motion parameter, have succeeded to significantly homogenise the damage database. The success in achieving this goal is assessed through observation of the influence on the curve shape and data scatter, of the three main assumptions made for the vulnerability curve derivation:

1. Seismic demand characterisation via elastic spectral displacement at 5% critical damping, $S_{d5\%}(T)$.
2. Combination of damage distributions deriving from multiple earthquakes and survey sites.
3. Combination of damage distributions for all building types

In the following sections each of these assumptions are discussed and the limitations of the homogeneous curves, are highlighted.

5.4.1 Effect of Ground Motion Parameter Choice

The requirements of a ground motion parameter for use in vulnerability curve derivation are discussed in detail in Chapter 4. Here, the influence of ground motion parameter choice on vulnerability curve shape and data scatter is assessed. Vulnerability curves are derived using peak ground acceleration (pga), elastic spectral acceleration and displacement for 5% of critical damping ($S_{a5\%}(T)$ and $S_{d5\%}(T)$) and inelastic spectral displacement for a ductility based damping value ($S_{d1\mu\%}(T_{inelastic})$), (see Figure 5.6 and Figures B.1 to B.4 in Appendix B)

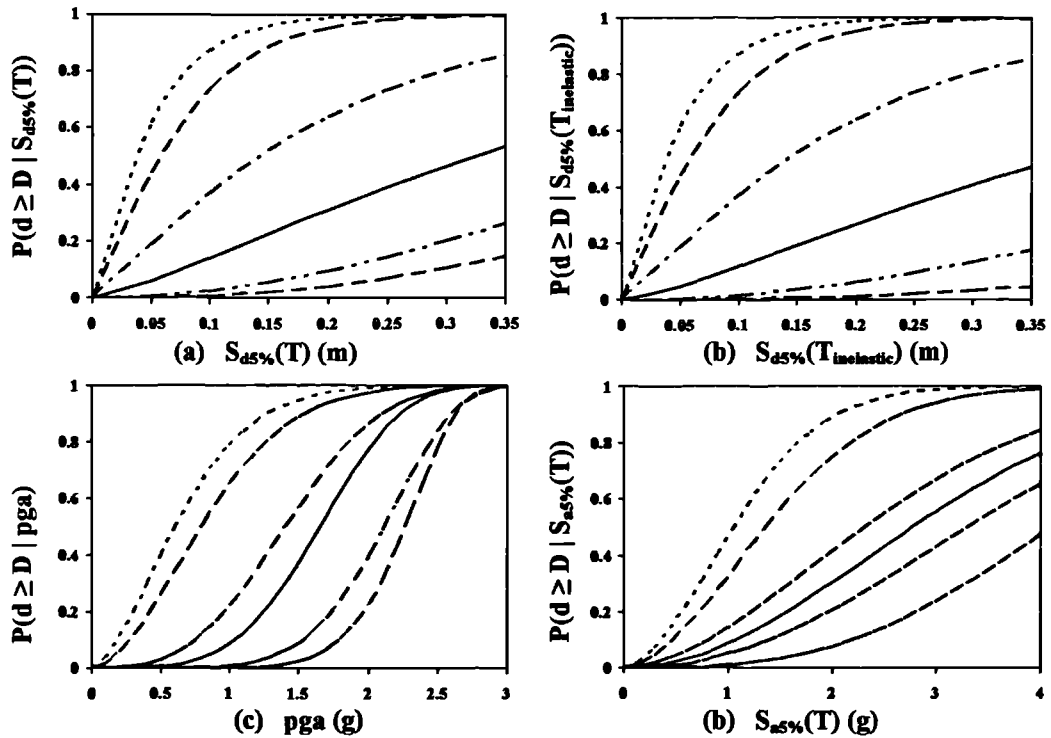


Figure 5. 6: Homogeneous empirical vulnerability curves generated for different ground motion parameters.

Where ground motion records are available for the survey location, these are used to evaluate the considered parameters, otherwise attenuation relationships are selected based on the survey site location and relevant fault mechanism. In the

case of pga and $S_{a5\%}(T_{elastic})$, the relationships of Ambraseys et al. (1996) are used for datasets regarding European and Middle Eastern countries, Joyner and Boore (1988) for the United States of America and Youngs et al. (1997) for Japan, Chile, Mexico and other subduction zones. For the spectral displacements, the attenuation relationships of Borzi et al. (1998) are used for all site locations, and the ground motion parameters calculated as per Section 4.3.

HRC Damage State	Curve Parameters						Bldgs	Non-weighted data				Weighted data			
	Mean		U90%		L90%			Data sets (3)	$\mu(\epsilon)$ (4)	cov (ϵ)(5)	R^2 (6)	Data sets	$\mu(\epsilon)$	cov (ϵ)	R^2
	$\alpha^{(1)}$	$\beta^{(2)}$	α	β	α	β									
pga															
Slight	1.556	1.60	3.950	1.60	0.830	1.60	339872	99	0.27	0.80	0.37	9	0.37	0.77	0.33
Light	1.055	1.80	2.732	1.80	0.620	1.80	339187	90	0.24	1.04	0.29	8	0.37	0.76	0.35
Moderate	0.250	3.00	0.903	3.00	0.102	3.00	331702	69	0.21	0.96	0.34	8	0.26	0.74	0.36
Extensive	0.093	4.00	0.538	4.00	0.010	4.00	329152	49	0.08	1.32	0.27	7	0.08	0.92	0.36
P.Coll.	0.009	5.80	0.162	5.80	0.001	5.80	292839	35	0.03	1.41	0.26	7	0.01	0.78	0.39
Collapse	0.001	8.00	0.005	8.00	0.001	8.00	77876	24	0.01	1.42	0.26	4	0.00	1.51	0.28
S _{a5%(T)}															
Slight	0.633	1.80	1.865	1.80	0.192	1.80	339872	99	0.25	0.84	0.39	15	0.30	1.04	0.39
Light	0.396	1.80	1.356	1.80	0.116	1.80	339187	90	0.22	1.05	0.30	15	0.29	1.05	0.37
Moderate	0.153	1.80	0.524	1.80	0.041	1.80	331702	69	0.17	1.07	0.28	14	0.19	0.99	0.39
Extensive	0.090	2.00	0.447	2.00	0.036	2.00	329152	49	0.08	1.13	0.26	14	0.08	1.00	0.52
P.Coll.	0.050	2.20	0.265	2.20	0.031	2.20	292839	35	0.04	1.40	0.40	13	0.05	1.68	0.49
Collapse	0.010	3.00	0.056	3.00	0.006	3.00	77876	24	0.01	2.29	0.44	8	0.03	2.27	0.49
S _{d5%(T)}															
Slight	25.82	1.10	76.45	1.10	13.72	1.10	339872	99	0.23	0.83	0.37	15	0.23	0.68	0.53
Light	21.08	1.20	73.88	1.20	8.350	1.20	339187	90	0.23	0.93	0.30	15	0.22	0.73	0.54
Moderate	6.500	1.15	29.57	1.15	2.342	1.15	331702	69	0.17	1.02	0.30	14	0.14	0.83	0.59
Extensive	3.000	1.30	17.52	1.30	1.323	1.30	329152	49	0.07	1.32	0.28	13	0.06	1.25	0.65
P.Coll.	2.500	2.00	13.45	2.00	1.200	2.00	292839	35	0.03	1.37	0.28	12	0.02	1.86	0.53
Collapse	2.000	2.40	9.37	2.40	1.119	2.40	77876	24	0.01	1.62	0.36	6	0.02	2.14	0.50
S _{dμ%(T_{inelastic})}															
Extensive	2.500	1.30	10.18	1.30	0.926	1.30	329152	49	0.08	1.27	0.28	19	0.07	1.56	0.57
P.Coll.	1.600	2.00	7.497	2.00	0.740	2.00	292839	35	0.03	1.40	0.27	16	0.06	1.60	0.48
Collapse	0.600	2.40	1.076	2.40	0.125	2.40	77876	24	0.01	1.42	0.27	5	3e-3	0.76	0.45

(1) α and (2) β are the shape parameters (Eq.5.3) for the mean curves, upper and lower 90% confidence bounds (U90% and L90% respectively); (3) Datasets = no. of datasets used for the derivation and comparison (weighted and non-weighted respectively); (4) $\mu(\epsilon)$ = mean error in prediction; (5) cov(ϵ) = the coefficient of variation of the error in prediction; (6) R^2 = coefficient of correlation of the data to the fitted curves.

Table 5. 4: Summary of the homogeneous empirical vulnerability relationships, developed for different ground motion parameters, and their fit to the observational data

$S_{d\mu\phi}(T_{inelastic})$ is introduced to investigate the effect of including the influence on the seismic demand characterisation, of period elongation and increased energy dissipation due to building damage. The ductility values associated with the different damage state curves are calculated assuming the “Moderate” damage state corresponds to structural yield. The top drifts characterising each damage limit state are assumed equal in value to the inter-storey drifts defined in Table 3.13, (i.e. linear variation in structural displacement with height is assumed). Different ductility values are defined for each structural class. For example, in the case of the general structure class, ductility values (μ) of 2, 4.15 and 6, are obtained for the “Extensive”, “Partial Collapse” and “Collapse” damage states, respectively. The resulting ductilities are used to calculate the damping, inelastic period and corresponding inelastic spectral displacement ($S_{d\mu\phi}(T_{inelastic})$), for all limit states exceeding “Moderate”, using the ductility-damping relationships in Borzi et al. (1998).

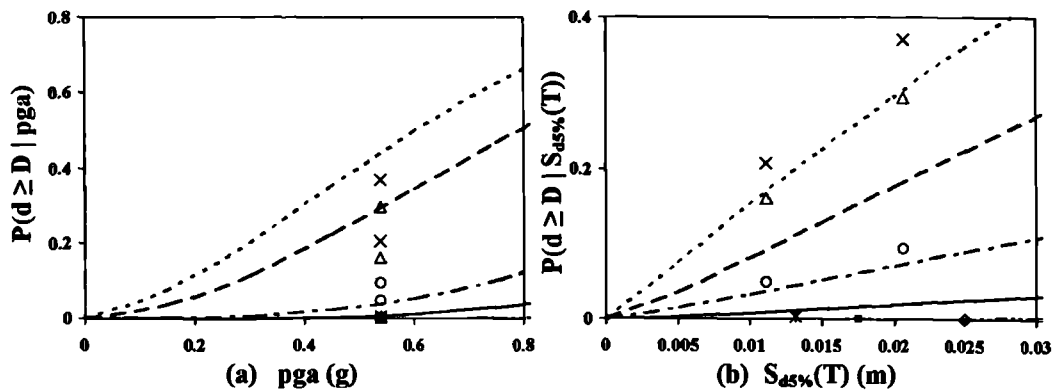


Figure 5. 7: Example of the difference in the vulnerability point distribution for different ground motion parameters, using observations of low and mid-rise building damage made in Aegion, after the Aegion (Greece, 1995) earthquake.

The empirical vulnerability relationships for the different ground motion parameters are summarised in Table 5.4. A direct comparison of the variation in the damage state predictions, with increasing site-distance, may be made in Appendix B, where the curves are plotted for two different magnitude events, (using the attenuation relationships of Ambrasseys et al 1996). In Appendix B, the curves are also compared to the existing curves reviewed in Chapter 2. Comparison of Table 5.4 and Table 2.3, shows that the homogeneous empirical

curves provide a better correlation with the observational data, than existing curves derived using the same parameter for ground motion characterisation.

An increase in the damage probability of buildings is expected to coincide with an increase in seismic demand. Therefore, as the same sources are used for the attenuation relationships in each case, the correlation of the curves with the observational data gives a direct indication of the relative worth of the parameters as measures of earthquake damage potential. From Table 5.4, (and Figures B.6 to B.12 in Appendix B), the correlation can be seen to be fairly poor, and the mean prediction error substantial, in all cases. *pga* yields a consistently worse fit than the spectral parameters. This is expected, as it is unable to account for the contribution of earthquake duration, record cycle, peak and frequency content on the ground motion damage potential, and the effects of structural dynamic properties and site geology on the seismic demand (Figure 5.7). These factors are seen to contribute to the large under-prediction of damage observed by Rossetto (2001), when the derived *pga* curves are applied to a case study of the Bhuj Earthquake (India, 2001).

The vulnerability curves derived using spectral displacement show a better correlation to the empirical data than those using spectral acceleration. This demonstrates the superiority of displacements in the representation of earthquake damage potential. In the case of $S_{d,\mu\%}(T_{inelastic})$, the prediction error and width of confidence bounds for higher damage states is considerably reduced compared to $S_{d5\%}(T)$. The limited data available for higher damage states, means that further investigation is required. The effects on the seismic demand imposed on structures, due to their inelastic behaviour and degradation during earthquakes, are, however, implicitly included in the observational damage statistics used to derive the empirical curves. This means that they are incorporated into the y-axis of the curves. Hence, it is unnecessary to characterise the x-axis using an inelastic spectral value. It is therefore concluded that elastic spectral displacement (at 5% critical damping, $S_{d5\%}(T)$), is the optimum parameter for ground motion characterisation in the vulnerability curve derivation.

5.4.2 Effect of Earthquake Combination

When a damage survey is carried out over a restricted area after the occurrence of an earthquake, the results will reflect the local building stock, attenuation characteristics of the earthquake motion and soil amplification effects. In the derivation of homogenous curves, it is assumed that damage data deriving from different countries and earthquakes, for similar lateral load resisting systems, can be combined, (see Section 2.3). The combination of data for different structure types, defined in terms of varied damage scales, is carried out using a damage index that accounts for differences in typical load resisting systems. Despite the homogenisation procedure, significant scatter is still observable in the plotted empirical data. In order to assess the success achieved in combining data from different locations and events, the homogeneous curves are re-derived using data from the Kobe (Japan, 1995) and Campania (Italy, 1980) earthquakes only (Figure 5.8 and Table 5.6).

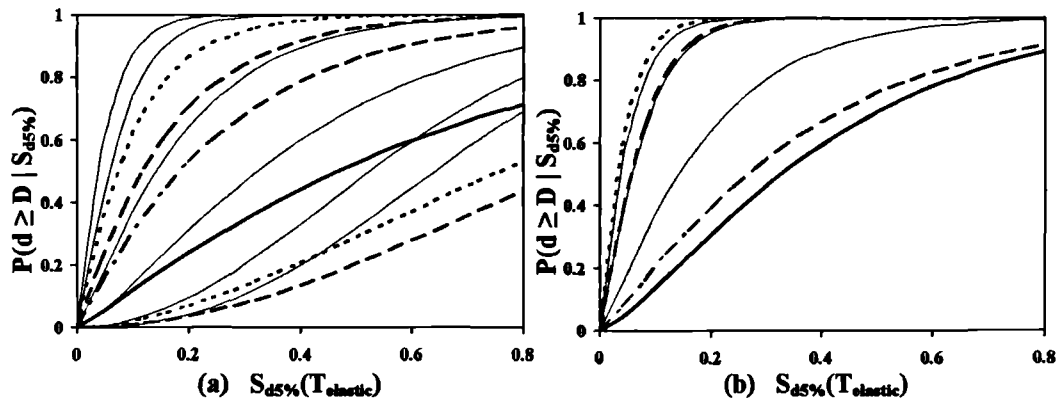


Figure 5. 8: Comparison of homogeneous curves (full grey lines) with vulnerability curves derived for specific earthquake events (dashed black lines): (a) Kobe earthquake (Japan, 1995); (b) Campania earthquake (Italy, 1980).

The use of single event data is seen to have an effect on the shape of the derived vulnerability curves. However, comparison of the correlation coefficients and mean error values for the single event data relative to its own regressed curves, does not show a great improvement over those of the homogeneous curves. No reduction in scatter is observed in the case of the Kobe earthquake. Only a slight improvement is observed in the case of Campania earthquake, with similar values being obtained for the mean prediction errors. Indeed, the difference in shape

between the fragility curve sets can almost solely be attributed to the influence of the predominant structure classes, (mid-rise, old-seismic code and low-rise, pre-seismic code frames for Kobe and Campania respectively). The single-event curves are based on a smaller number of observations and more homogeneous building stock, than the homogeneous curves. This indicates that the homogenisation process has been successful in allowing data from many sources to be combined, without introducing a significant additional scatter in the data. The similar scatter for single and multiple events, also tends to confirm the stipulation that much of the uncertainty arises from damage classification errors inherent in the observational data.

5.4.3 Effect of Structure Combination

The vulnerability curves described in Table 5.3 are “homogeneous” as they combine all the available observational data regardless of structural type. This assumes that the use of the HRC damage scale, which distinguishes between buildings with different dynamic properties, is sufficient to eliminate the curve shape dependency on structure type. In order to test this assumption, the vulnerability curves are re-derived considering sub-sets of the observational data appertaining to the different lateral-load resisting system, seismic design code and height classes of Table 5.2. The resulting “class-specific” curves are summarised in Table 5.5. Where insufficient empirical data is available for the class-specific curve derivation, the observations are compared directly to the general curves to assess the building class influence.

In the case of infilled frames, the vulnerability is slightly under-predicted by the general curves in the low damage range, whilst no difference is observed at high levels of damage. Only slight differences between the general and bare MRF vulnerability curve shapes are seen for the “Moderate” damage state. Furthermore, similar correlation values and prediction errors obtained when the structure-specific and homogenised curves are applied in the damage prediction of infilled and bare frames. This indicates that the influence of non-structural walls on the building response is mostly removed from the curve shape. The homogeneous

curves can therefore be used with a good degree of confidence in the assessment of these systems. Insufficient data is available for any conclusion to be made regarding shear-wall structures, and it is recommended that the homogenised vulnerability curves not be used, or that the lower bound 90 % confidence curve adopted, in this case.

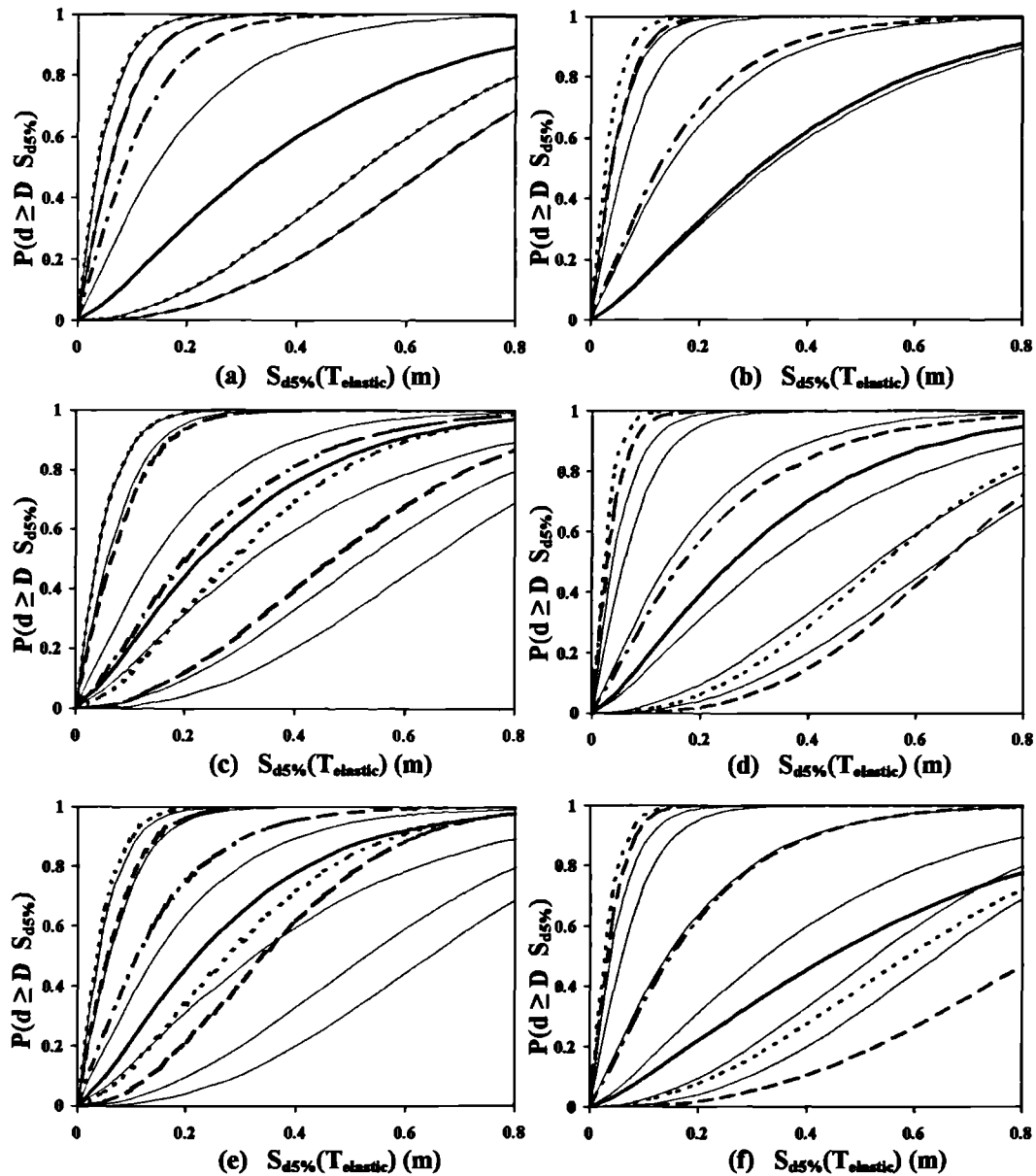


Figure 5. 9: Comparison of homogeneous curves (full grey lines) with vulnerability curves derived for specific structural categories (dashed black lines): (a) Bare frames; (b) Infilled frames; (c) Low-rise bare frames; (d) Mid-rise bare frames; (e) Pre-seismic code bare frames; (f) Old seismic code bare frames.

HRC Damage State	Class-specific Curve Parameters						Correlation with weighted class-specific data						Correlation with non-weighted class-specific data									
	Mean		U90%		L90%		Bldgs ⁽³⁾	DS ⁽⁴⁾	Class-specific curves		Homogeneous curves		Bldgs	DS	Class-specific curves		Homogeneous curves					
									$\mu(\epsilon)^{(5)}$	$\text{cov}(\epsilon)^{(6)}$	$R^{2(7)}$	$\mu(\epsilon)$			$\text{cov}(\epsilon)$	R^2	$\mu(\epsilon)$	$\text{cov}(\epsilon)$	R^2	$\mu(\epsilon)$	$\text{cov}(\epsilon)$	R^2
	$\alpha^{(1)}$	$\beta^{(2)}$	α	β	α	β																
Bare MRF																						
Slight	23.830	1.05	76.451	1.10	13.716	1.10	296513	12	0.174	0.724	0.635	0.175	0.752	0.541	296513	43	0.159	0.717	0.603	0.168	0.700	0.595
Light	21.270	1.20	73.881	1.20	8.350	1.20	296300	12	0.201	0.718	0.598	0.200	0.725	0.557	296300	37	0.150	0.924	0.541	0.150	0.926	0.540
Moderate	13.296	1.20	29.570	1.15	2.342	1.15	293579	10	0.195	0.679	0.639	0.149	0.776	0.444	293579	18	0.156	0.851	0.610	0.140	0.899	0.553
Extensive	3.000	1.30	17.518	1.30	1.323	1.30	292828	9	0.099	0.865	0.524	0.099	0.865	0.283	292828	13	0.080	1.053	0.522	0.080	1.053	0.522
P.Collapse	2.500	2.00	13.450	2.00	1.200	2.00	258976	9	0.040	1.312	0.395	0.040	1.312	0.055	258976	10	0.036	1.404	0.401	0.036	1.404	0.401
Collapse	2.000	2.40	9.365	2.40	1.119	2.40	67602	4	0.024	1.494	0.453	0.024	1.494	0.006	67602	4	0.024	1.494	0.453	0.024	1.494	0.453
Infilled MRF																						
Slight	40.000	1.10	76.451	1.10	13.716	1.10	2720	4	0.329	0.720	0.466	0.409	0.650	0.448	2720	8	0.436	0.661	0.438	0.523	0.557	0.447
Light	35.000	1.20	73.881	1.20	8.350	1.20	2720	4	0.374	0.755	0.430	0.434	0.705	0.427	2720	8	0.470	0.629	0.434	0.531	0.561	0.442
Moderate	7.500	1.15	29.570	1.15	2.342	1.15	2720	4	0.220	1.027	0.363	0.226	1.009	0.365	2720	8	0.238	0.868	0.379	0.244	0.847	0.382
Extensive	3.200	1.30	17.518	1.30	1.323	1.30	2460	4	0.004	0.819	0.757	0.003	0.829	0.789	2460	7	0.006	0.864	0.444	0.006	0.889	0.427
Low-Rise Bare MRF (<3 Floors)																						
Slight	25.817	1.10	125.27	1.10	2.390	1.10	5372	9	0.104	0.791	0.637	0.104	0.791	0.637	5372	35	0.140	0.704	0.552	0.140	0.704	0.552
Light	18.560	1.20	80.480	1.20	4.100	1.20	5159	8	0.090	0.901	0.588	0.112	0.794	0.579	5159	29	0.099	0.942	0.395	0.103	0.932	0.430
Moderate	5.500	1.30	30.560	1.30	2.000	1.30	2438	5	0.050	0.991	0.255	0.076	0.919	0.369	2438	10	0.087	1.419	0.179	0.091	1.200	0.197
Extensive	4.671	1.30	16.380	1.30	0.600	1.30	1667	3	0.080	1.320	0.346	0.091	1.304	0.329	1667	5	0.061	1.418	0.284	0.066	1.475	0.266
P.Collapse	5.100	1.60	17.115	1.60	0.500	1.60	1455	3	0.033	1.321	0.346	0.043	1.356	0.310	1455	3	0.033	1.338	0.349	0.043	1.355	0.311
Collapse	3.160	2.00	10.320	2.00	0.300	2.00	1455	3	0.005	1.633	0.301	0.007	1.659	0.262	1455	3	0.005	1.648	0.304	0.007	1.658	0.262

⁽¹⁾ α and ⁽²⁾ β shape parameters (Eq.5.3) for the mean curves, upper and lower 90% confidence bounds (U90% and L90% respectively); ⁽³⁾ Bldgs = total no. of buildings within the datasets used; ⁽⁴⁾ Datasets = no. of datasets used for the derivation and comparison (weighted and non-weighted); ⁽⁵⁾ $\mu(\epsilon)$ = mean error in prediction; ⁽⁶⁾ $\text{cov}(\epsilon)$ = the coefficient of variation of the prediction error; ⁽⁷⁾ R^2 = coefficient of correlation of the data to the fitted curves.

Table 5. 5: The proposed structure-type, height and seismic-code specific empirical curves and their correlation to the weighted and non-weighted observational data.

HRC Damage State	Class-specific Curve Parameters						Correlation with weighted class-specific data						Correlation with non-weighted class-specific data									
	Mean		U90%		L90%		Bldgs ⁽³⁾	DS ⁽⁴⁾	Class-specific curves			Bldgs	Datasets	Class-specific curves			Homogeneous curves					
	$\alpha^{(1)}$	$\beta^{(2)}$	α	β	$\mu(\epsilon)^{(5)}$	$\text{cov}(\epsilon)^{(6)}$			$R^{2(7)}$	Homogeneous curves				$\mu(\epsilon)$	$\text{cov}(\epsilon)$	R^2						
										$\mu(\epsilon)$	$\text{cov}(\epsilon)$											
Mid-Rise Bare MRF (4-7 Floors)																						
Slight	48.410	1.05	150.00	1.05	3.000	1.05	290912	7	0.167	0.770	0.658	290912	10	0.151	0.778	0.651	0.261	0.469	0.585			
Light	39.000	1.10	140.00	1.10	5.400	1.10	290912	7	0.199	0.682	0.650	290912	10	0.180	0.724	0.643	0.276	0.575	0.572			
Moderate	5.300	1.15	105.00	1.15	0.800	1.15	290912	7	0.178	0.699	0.589	290912	10	0.172	0.793	0.561	0.175	0.752	0.584			
Extensive	4.000	1.30	21.600	1.30	0.800	1.30	290832	6	0.127	0.959	0.560	0.101	0.794	0.581	290832	8	0.108	1.065	0.567	0.087	0.961	0.579
P.Collapse	3.000	2.40	14.700	2.40	0.500	2.40	257172	6	0.028	1.216	0.526	0.038	1.410	0.516	257172	6	0.028	1.217	0.526	0.038	1.411	0.516
Collapse	2.500	3.00	9.600	3.00	0.300	3.00	66261	2	0.022	1.350	0.500	0.039	1.270	0.500	66261	2	0.022	1.350	0.500	0.040	1.270	0.500
Pre-Seismic Code Bare MRF																						
Slight	29.430	1.10	222.00	1.10	3.470	1.10	3655	8	0.122	0.676	0.563	0.110	0.821	0.557	3655	31	0.127	0.758	0.582	0.134	0.093	0.564
Light	23.000	1.20	163.00	1.20	2.500	1.20	3442	7	0.123	0.809	0.551	0.109	0.851	0.553	3442	25	0.096	0.938	0.493	0.093	0.084	0.473
Moderate	8.860	1.15	75.000	1.15	2.000	1.15	721	2	0.069	1.322	0.584	0.047	1.085	0.635	721	6	0.085	1.183	0.319	0.085	0.113	0.235
Extensive	5.000	1.30	80.000	1.30	1.700	1.30	256	2	0.099	1.361	0.354	0.113	1.400	0.339	256	3	0.069	1.578	0.287	0.079	0.126	0.267
P.Collapse	5.500	1.60	60.000	1.60	1.200	1.60	64	2	0.041	1.376	0.362	0.055	1.405	0.335	64	2	0.041	1.350	0.358	0.055	0.077	0.335
Collapse	6.000	2.00	36.800	2.00	1.000	2.00	25	1	0.010	0.000	0.5	0.019	0.000	0.500	25	1	0.009	0.000	0.500	0.019	0.000	0.500
Old-Seismic Code Bare MRF																						
Slight	42.000	1.10	320.00	1.10	10.400	1.10	292572	8	0.228	0.611	0.605	0.256	0.462	0.601	292572	10	0.229	0.573	0.557	0.281	0.446	0.543
Light	45.000	1.20	308.00	1.20	2.400	1.20	292572	8	0.240	0.562	0.616	0.266	0.545	0.596	292572	10	0.242	0.537	0.574	0.298	0.511	0.545
Moderate	6.700	1.20	71.200	1.20	1.800	1.20	292572	8	0.187	0.666	0.603	0.188	0.624	0.609	292572	10	0.195	0.670	0.567	0.193	0.645	0.573
Extensive	2.000	1.30	23.500	1.30	1.100	1.30	292572	8	0.093	0.630	0.631	0.089	0.829	0.608	292572	10	0.084	0.986	0.587	0.080	0.959	0.589
P.Collapse	2.000	2.00	10.000	2.00	0.600	2.00	258912	8	0.026	1.437	0.526	0.031	1.533	0.520	258912	8	0.026	1.438	0.526	0.031	1.534	0.520
Collapse	1.100	2.50	4.770	2.50	0.300	2.50	67577	3	0.012	1.691	0.499	0.025	1.713	0.499	67577	3	0.012	1.691	0.499	0.025	1.713	0.499

Table 5.5 Continued

HRC Damage State	Event- specific Curve Parameters		Correlation with weighted event-specific empirical data								Correlation with non-weighted event-specific empirical data							
	Mean		Bldgs ⁽³⁾	DS ⁽⁴⁾	Event-specific curves			Homogeneous curves			Bldgs	Data sets	Event-specific curves			Homogeneous curves		
	$\alpha^{(1)}$	$\beta^{(2)}$			$\mu(\epsilon)^{(5)}$	$cov(\epsilon)^{(6)}$	$R^2^{(7)}$	$\mu(\epsilon)$	$cov(\epsilon)$	R^2			$\mu(\epsilon)$	$cov(\epsilon)$	R^2	$\mu(\epsilon)$	$cov(\epsilon)$	R^2
Kobe (1995)																		
Slight	10.697	1.05	33857	3	0.077	0.357	0.555	0.121	0.047	0.549	33857	37	0.901	0.807	0.248	0.298	0.523	0.097
Light	6.532	1.05	33857	3	0.058	0.715	0.490	0.075	0.033	0.587	33857	37	0.901	0.797	0.262	0.254	0.598	0.025
Moderate	4.038	1.05	29694	3	0.056	0.465	0.109	0.047	0.040	0.208	29694	33	0.622	0.819	0.167	0.160	1.055	0.126
Extensive	1.591	1.10	29188	2	0.012	1.061	0.315	0.012	0.014	0.361	29188	26	0.366	0.826	0.167	0.096	1.125	0.168
P.Collapse	1.100	1.70	28848	2	0.005	1.340	0.326	0.006	0.007	0.352	28848	21	0.135	0.832	0.258	0.038	1.122	0.263
Collapse	0.898	2.00	5518	2	0.001	0.022	0.086	0.001	0.001	0.208	5518	17	0.037	0.853	0.237	0.010	1.251	0.253
Campania (1980)																		
Slight	30.81	1.10	3306	8	0.126	0.628	0.567	0.11	0.091	0.561	3306	28	0.127	0.797	0.583	0.13	0.73	0.56
Light	22.00	1.20	2621	7	0.118	0.791	0.562	0.11	0.091	0.563	2621	19	0.093	0.992	0.514	0.09	0.98	0.50
Moderate	3.16	1.15	372	2	0.005	0.271	0.981	0.04	0.055	0.661	372	3	0.140	1.483	0.230	0.14	1.06	0.19
Extensive	2.96	1.30	25	1	0.226	0.000	0.500	0.23	0.000	0.500	25	1	0.226	0.000	0.500	0.22	0.00	0.50

⁽¹⁾ α and ⁽²⁾ β shape parameters (Eq. 5.3) for the mean curves; ⁽³⁾ Bldgs = total no. of buildings within the datasets used; ⁽⁴⁾ Datasets = no. of datasets used for the derivation and comparison (weighted and non-weighted); ⁽⁵⁾ $\mu(\epsilon)$ = mean error in prediction; ⁽⁶⁾ $cov(\epsilon)$ = the coefficient of variation of the prediction error; ⁽⁷⁾ R^2 = coefficient of correlation of the data to the fitted curves.

Table 5. 6: The proposed single event empirical curves and their correlation to the weighted and non-weighted observational data.

The general curves tend to over-predict the damage observed in low-rise bare moment resisting frames for the low damage states and under-predict that for higher limit states (Figure 5.9c). The reverse is true in the case of mid-rise frames (Figure 5.9d). The damage observations available for high-rise bare MRF are over-predicted by the homogeneous curves at high ground motions. The influence of building height on seismic demand is taken into account through the use of spectral displacement (for 5% critical damping) as the ground motion parameter. Use of this parameter is shown in Section 5.4.1 to have a positive effect on the correlation of the fitted curves with the empirical data. The discrepancies between the homogeneous and “height-specific” curves are attributed to the typically different building regulations used in the design of buildings of disparate height-classes. More stringent requirements for construction materials, section dimensions and detailing are generally imposed for tall structures. Furthermore, wind loads are often considered in their design. These factors increase the redundancy and lateral-load resistance of these structures. Furthermore, the characteristic rate of change of stiffness and energy dissipation capacity with increasing damage, differ from those of low-rise buildings. Therefore the derivation of “height-specific” vulnerability curves is recommended.

When the re-derived code-specific curves are compared to the general curves, the latter are seen to significantly under- and over-predict the vulnerability of pre- and old-seismic code structures, respectively. Due to the lack of observational data, no observations can be made regarding structures designed to modern seismic codes. The difference in the curve shapes is mainly due to the absence of capacity design in pre-seismic code structures, which causes their failure in earthquakes via predominantly soft storey modes. This mode of failure is associated with a rapid transition between low levels of damage and the collapse limit state. It is reflected in the pre-code vulnerability relationships (Figure 5.9e), by a closer proximity of the curves and their more rapid ascension with increasing ground motion. The contribution of poor construction quality and irregularity in precipitating localized structure failure modes, is observed to be the main cause of the large under prediction of damage to RC structures in Ahmedabad after the Ms 7.9 Bhuj Earthquake (India, 2001) by the homogenised curves (Rossetto 2001).

Use of the HRC damage scale is seen to significantly reduce the influence of lateral-load resisting system on the derived vulnerability curve shapes. However, from the above observations, it is concluded that the influence of different levels of seismic design and construction quality are not successfully removed from the homogeneous curves. This is attributed to the lack of experimental data for the inclusion of these factors in the calibration of the HRC-Scale. The structure type, height and code-specific vulnerability relationships are associated with higher correlation coefficients and lower prediction errors than the homogenised curves. They show a significant improvement on existing empirical relationships, however the relationships are based on limited data and the size of the scatter remains large. These problems may only be removed via the analytical generation of damage statistics.

Chapter 6

Analytical Vulnerability Curves

Part 1:- Derivation Methodology, System Design and Modelling

6.1 General Analytical Curve Generation Methodology

The main problem associated with the derivation of empirical vulnerability curves is the lack of an extensive database of consistently evaluated damage statistics for homogeneous building populations, covering a large range of ground motion severities. Within this chapter, a procedure for vulnerability curve derivation is presented, wherein building damage statistics are simulated through analysis. The procedure is summarised in Figure 6.1 and can be subdivided into four main steps:

1. System definition
2. Input parameter generation
3. Model evaluation
4. Statistical processing of analysis results

In the case of this study, the “*system*” is the building class to be assessed. It is defined by a single structure with material, configuration and seismic resistance characteristics that are representative of the entire class. Deviation in seismic resistance of buildings within the class is considered through the analysis of a population of building models, generated from the general *system* design by varying its structural properties. The population of models is analysed under a series of ground motion records to account for the effect of ground motion uncertainty on the structure seismic response. The proposed methodology considers design of experiment procedures for the selection of the population and earthquake record suite sizes used in analysis. This is done in order to optimise computational effort and to guarantee convergence of results. A method for the selection of performance-consistent earthquake records is presented in Chapter 4. Hence, within this chapter, discussion of Step 2, “*input parameter generation*”, focuses on the determination of structural property value combinations for the

building population definition. Step 3, the “*model evaluation*”, is carried out using an innovative adaptive pushover analysis technique. The maximum inter-storey drift response of each structure within the population is assessed, under increasing intensities of ground motion, using a modified capacity spectrum method. Step 4, the “*statistical processing of analysis results*”, involves the definition of response surfaces, relating the observed inter-storey drift response to the structural property and ground motion parameter values. Response surface equations are defined for each hazard scenario, through separate consideration of the analyses statistics resulting from each suite of performance-consistent records. A re-sampling technique is adopted to generate the system vulnerability curves from the surfaces, with the relationships between maximum inter-storey drift and the HRC scale presented in Chapter 3, used to determine the damage state exceedence probabilities. The response surface equation used to develop each damage state curve, is selected according to the hazard level associated with the satisfaction of a desired performance objective. The generated vulnerability curves may, therefore, also be defined as “performance consistent”. The overall uncertainty in the seismic resistance of the structure population under different ground motions is represented by the fit of the analysis data to the mean response surface equations. The latter is considered for the definition of confidence bounds.

Any structural typology may be adopted within the framework of the proposed analytical methodology. The results are, however, strongly dependent on the choices made for the structural configuration, the design of the system representing the building class, the structural properties to be varied in the population generation and their corresponding probability distributions. The *system* definition, design and structure population simulation methodology, are discussed in the following sections. In the next Chapter, the techniques proposed for use in the analysis and assessment of the structure population, and the statistical processing for the vulnerability curve generation, are presented. Throughout, the application of the method is illustrated for the case of typical low-rise European reinforced concrete infilled frames. Finally, the adequacy of the derived curves is assessed from a comparison with observational post-earthquake damage statistics and equivalent, empirically derived, curves.

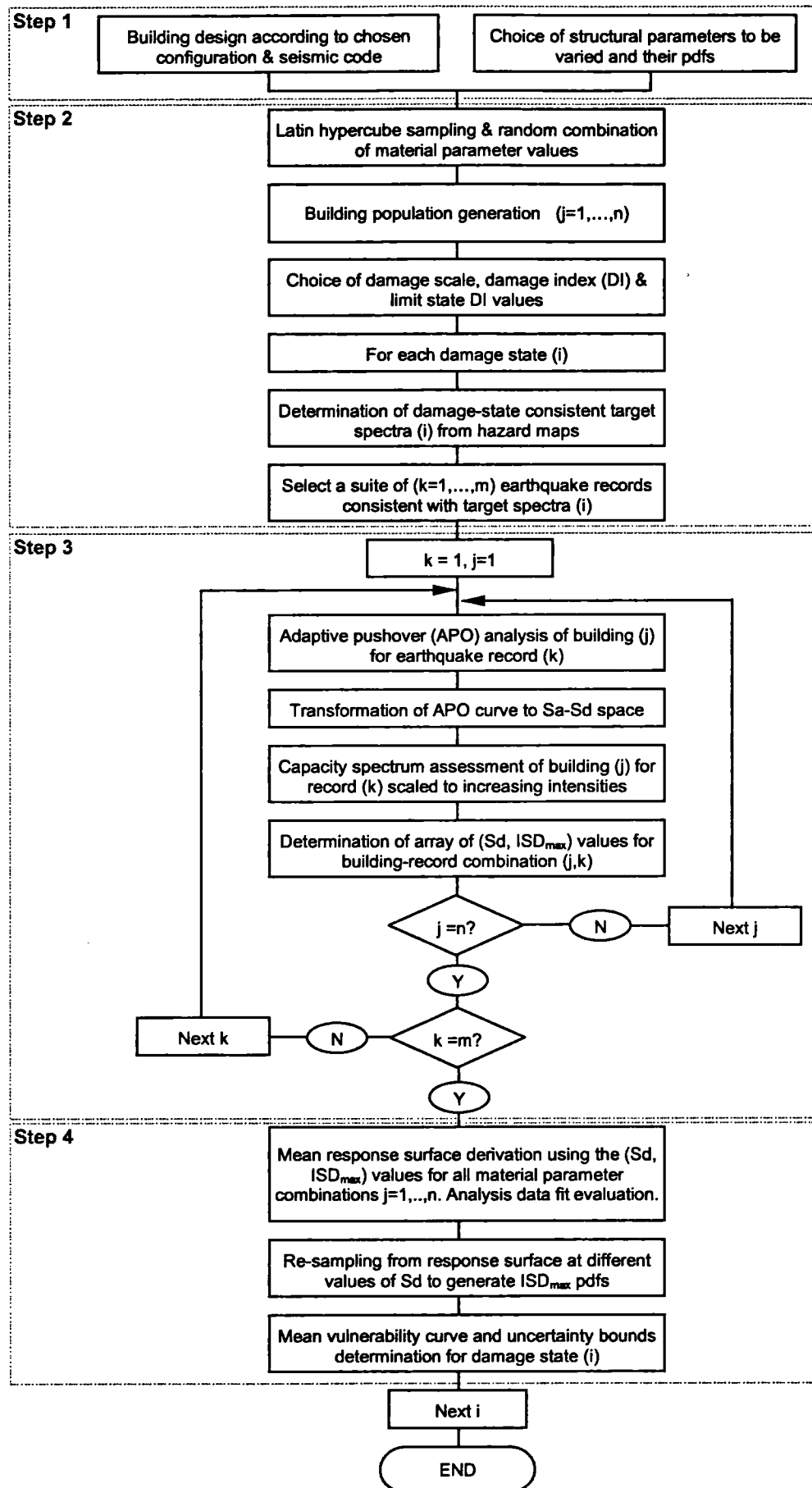


Figure 6. 1: Flowchart of the proposed analytical method for vulnerability curve generation

6.2 Definition of the System

The choice of building design and material properties assumed for the model are dependent on the composition of the building stock in the assessed region. The buildings must be grouped into categories with similar lateral load resistance and different vulnerability curves must be derived for each building class. Detailed information regarding configuration, material properties and design of each building is not available in most current structure inventories. However, a reasonable building classification system that is indicative of the load resistance may be obtained considering the main construction material (steel, reinforced concrete etc), lateral load resisting system (e.g. bare-MRF, infilled-MRF, shear wall), height range (low-, mid- or high-rise) and seismic code used (e.g. none, old seismic code, new/EC8 equivalent). Such building classification systems can be found in Fema 273 (ATC (1997)) or ATC-13 (ATC (1985)), however building classes tailored to the construction practice in the assessed area should be considered for a reliable risk evaluation. Furthermore, local knowledge of building plan configurations and materials should be considered in designing a model that is representative of each class, with the consequent degree of capacity design of the structure, member sizes and detailing being determined in accordance with the design code defining the class.

In order to illustrate the analytical vulnerability curve derivation methodology, only one building class is considered here. The analytical model is however chosen to be representative of the existing reinforced concrete building stock in Europe. In order to do this, data regarding the spatial and temporal distribution of building types in European countries was collected. As few pre-earthquake vulnerability studies have been carried out in the past, only a limited quantity of such data was found. In Table 6.1, the results of recent pre-earthquake vulnerability studies in Italy are presented. Table 6.2 instead, summarises the predominant structural types in areas hit by past Mediterranean earthquakes, reported by the literature sources consulted in the Empirical damage database compilation (in Chapter 5). Differences exist between the predominant building categories defined in Table 6.2, and those of the survey statistics in the empirical

database, for the same earthquakes. These arise as the latter may only concern a sub-set of structures within the population. From the tables, it is observed that the building stock predominantly consists of low-rise infilled frames built according to old seismic design codes. In practice, frames assigned to the “bare” frame category often contain weak infill-walls that are poorly connected to the surrounding frame. A three storey infilled frame designed to an old seismic code is, therefore, chosen to define the structural system.

Location	No. of Bldgs ⁽¹⁾	Structure Type	Height ⁽²⁾	No. of Storeys	Seismic Code
Catania	-	Weakly infilled MRF	Medium	5	None
Abruzzo, Basilicata, Molise, Campania, Puglia, Calabria, W.Sicily	14762	Infilled frames	Low	3	None
	9311	Weakly infilled MRF	Low	3	None
	833	Shear-wall structures	Low	3	None
Castelnuovo (Tuscany)	124	Infilled frames	Low	3	None

⁽¹⁾. Number of buildings surveyed; ⁽²⁾. Predominant building height category.

Table 6. 1: Reinforced concrete census data for Italy (GNDT).

Country	Earthquake	Structure Type	Height ⁽¹⁾	No. of Storeys	Seismic Code
Italy	Campania 1980	Bare frame	Low	3	None
	Umbria-Marche, 1997	Infilled frame	Low	3	None
Greece	Konitsa, 1996	Unspecified	Low	2	Old
	Korinthos, 1981	Infilled frames	Low	3	Old
	Kalamata, 1986	Infilled frames	Low	3	Old
	Alkyonides-Gulf, 1981	Bare frames	Low	2	Old
	Aegion, 1995	Infilled frames	Low	2	Old
Turkey	Erzincan, 1992	Infilled frames	Low	3	Old
	Adana-Ceyhan, 1998	Infilled frames	Medium	5	Old
	Kocaeli, 1999	Infilled frames	Medium	5	Old

⁽¹⁾Predominant building height category

Table 6. 2: Predominant reinforced concrete building categories for the regions hit by various Mediterranean earthquakes, as reported by the post-earthquake survey literature sources used in the compilation of the empirical database of Chapter5.

The structural configuration in plan and elevation, the storey heights and bay dimensions contribute to the global seismic behaviour of the system. These are also chosen in accordance with the literature sources mentioned above. A regular

configuration is selected to maintain generality, consisting of a 3x3 bay plan configuration with constant inter-storey heights and bay-widths of 3m and 4.5m, respectively. The system design and member dimensioning are carried out according to the Italian seismic codes of 1982 and corresponding gravity design codes (see Table 6.3). This code is chosen as being representative of the seismic design of the existing European building stock and furthermore, is consistent with the hazard maps used for the earthquake record selection in Chapter 4.

6.2.1 System Design

The three storey infilled frame defining the system is designed according to the prescriptions for loading, material, member dimensioning and detailing of the seismic and gravity load design codes in place in Italy in 1982. These design codes consist of major laws or royal decrees, which have been amended over time through the publication of a series of governmental decrees. Those relevant in the year assumed for the design are listed in Table 6.3. Of special notice is Law no.64 of 1974 and subsequent Governmental Decree in March 1975, which introduce the concept of response spectra to the seismic design of buildings in Italy. According to the codes, two conditions of loading must be considered in the building design and verification: the “Normal” load case, which considers all possible gravity and live load combinations, and the “Exceptional” load case, which considers all possible seismic, gravity and live load combinations, excluding wind actions.

In the gravity load calculation, the infill is assumed to be made of perforated bricks of density 11kN/m^3 and to consist of two leaves of 10cm and 6cm thickness. Rectangular openings of 2m x 1.5m for windows are assumed within each infill panel, resulting in a distributed beam span load, due to the infill, of 4.11kN/m on the external frame. The contribution to structural resistance of the infill panels is not accounted for in the design but is considered within the analyses. A density of 25kN/m^3 is taken for the reinforced concrete, and distributed floor and roof loads of 1.5 kN/m^2 and 0.7kN/m^2 , respectively, are used to account for internal

partitions, floor and roof coverings. The dead loads resulting from these considerations are summarised in Figure 6.1.

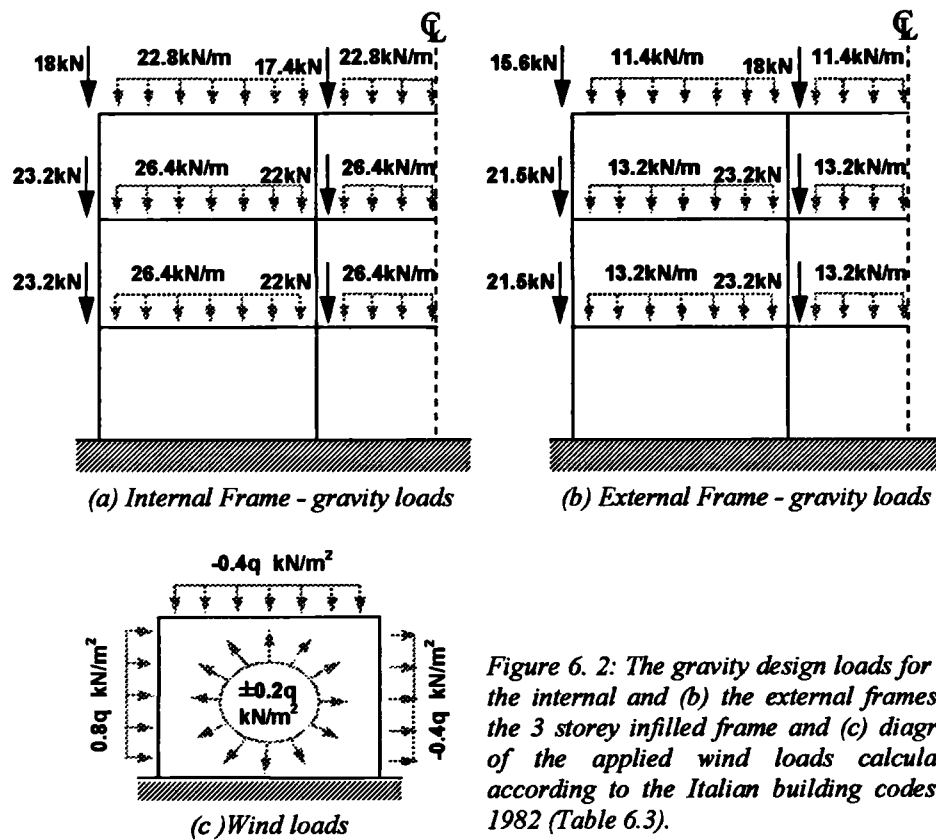


Figure 6. 2: The gravity design loads for (a) the internal and (b) the external frames of the 3 storey infilled frame and (c) diagram of the applied wind loads calculated according to the Italian building codes of 1982 (Table 6.3).

Italian standard specifications for reinforced concrete construction		
Royal Decree no.2229	16/11/1939	Standards for the execution of structures in unreinforced or reinforced concrete.
Law no.1086	5/11/1971	Standards for normal and pre-compressed concrete and steel structures.
Government Decree	30/5/1974	Technical standards for normal and pre-compressed concrete and steel structures.
Government Decree	3/10/1978	Synthesis of design loads for structures
Government Decree	26/3/1980	Synthesis of design loads for structures
Circular LLPP no. 22631	24/5/1982	Instructions regarding the loading and overloading of structures and general criteria for their safety verification.
Italian standard specifications for constructions in seismic areas		
Law no. 64	2/2/1974	Standards for constructions in seismic areas
Government Decree	3/3/1975	Approval of the technical standards for construction in seismic areas and dispositions for the code application.

Table 6. 3: Summary of the design codes for reinforced concrete buildings in Italy in 1982.

A live load of 2kN/m^2 on the floors and roofs is prescribed by the codes for habitations or public office buildings. An additional roof live load of 0.9kN/m^2 is assumed to account for snow, as prescribed by the codes for northern and central Italian regions. For the three storey building, a uniform wind pressure of 0.6kN/m^2 is obtained assuming an intermediate wind zone classification (Zone 2). The wind loads are considered to act as static forces in the horizontal direction, with modification coefficients applied to the basic wind pressure to account for element exposure and inclination. Two combinations of internal and external wind actions are checked (Figure 6.1), and the worst case adopted in the design of each building element.

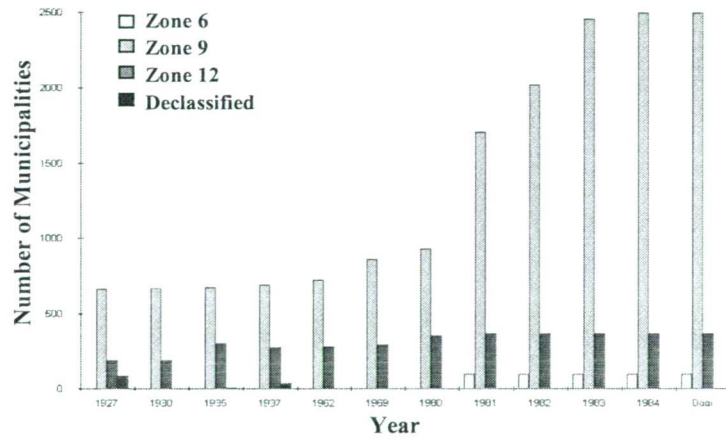


Figure 6. 2: The evolution of the seismic classification of Italian municipalities according to De Marco et al. (2003). Municipalities with no seismic zone rating are not illustrated.

The seismic actions are determined from a static analysis of the structure, subjected to a system of lateral forces applied non-simultaneously in the two main perpendicular directions of the building plan. The total horizontal seismic load F is calculated from Equation 6.1.

$$F = C.R.I.E.\beta.W \quad [6. 1]$$

$$C = \frac{S-2}{100} \quad [6. 2]$$

$$T_0 = 0.1 \frac{H}{\sqrt{B}} \quad [6. 3]$$

Where, C is the numerical coefficient of seismic intensity, calculated from the seismicity coefficient S , according to Equation 6.2. The intermediate seismic zone defined by the code, Zone 2 ($S=9$), is assumed in the seismic force determination. This is the most frequent classification for Italian municipalities (Figure 6.2), and corresponds to a pga value of approximately 0.07g, with an associated exceedence probability of 10% in 50years. It is noteworthy that this value is lower than that of 0.11g predicted for the equivalent seismic zone and earthquake return period in the current maps by Albarello et al. (1999), (see Table 4.2). In Equation 6.1, I and ε are the importance and foundation coefficients, respectively, which are assumed to equal 1 for structures of normal importance founded on firm soil. β is the structural coefficient, which takes a value of 1.2 for frame structures. W is the weight of the structure, calculated from the combination of the permanent dead loads and 1/3 of the live loads. R is the response coefficient, which takes a unit value for structures with natural periods below 0.8s, or $(0.862/T_0^{2/3})$ otherwise. The natural period of the structure T_0 is calculated as 0.245s using Equation 6.3, where H and B are the building height and width dimensions in meters. These considerations result in a design base shear of 8.4% of the structure weight. In the static analysis, the total seismic force, F , is applied as a series of horizontal storey forces, F_i , determined according to Equation 6.4.

$$F_i = \gamma_i \cdot F \cdot W_i \quad , \quad \gamma_i = h_i \frac{\sum_{j=1}^N w_j}{\sum_{j=1}^N w_j h_j} \quad [6.4]$$

In Equation 6.4, i designates the storey number, N is the maximum number of storeys, γ_i is the distribution coefficient, h_i is the storey height above ground level and w_i is the weight of the storey. The static analyses for the design moments, axial and shear forces are carried out using the analysis program QSE Space Version 6.0 (STAAD/Pro-QSE 1999). The program assumes elastic behaviour of materials and adopts the Stiffness Matrix Method with Choleski Decomposition for the analysis of the 2D frame models. A continuous beam analogy is adopted in the slab design, with all possible support and loading conditions being checked to determine the maximum design moment and shear forces. A working stress

method is prescribed by the Italian codes for member dimensioning and detailing. Within the method, the material stresses at critical sections resulting from application of the “Normal” and “Exceptional” load cases, are checked for compliance with maximum stress values defined in the code. Concrete with a characteristic compressive strength (f_{ck}) of 300kg/cm² (30MPa), and FeB38 grade ribbed steel bars with characteristic yield stresses (f_{yk}) of 3800kg/cm² (380MPa), are used in the frame design. The admissible stresses for these materials, under the “Normal” load condition, are summarised in Table 6.5, with those for the “Exceptional” load case design being 1.125 times these values.

Material	Action	Admissible stress	Units
Concrete	Flexure	$\sigma'_b = 60 + \frac{(f_{ck} - 150)}{4}$ for columns	Kg/cm ²
		$\sigma'_b = 0.9 \left(60 + \frac{(f_{ck} - 150)}{4} \right)$ for beams	Kg/cm ²
	Axial compression	$\sigma'_b = 0.7 \left(60 + \frac{(f_{ck} - 150)}{4} \right)$	Kg/cm ²
	Combined axial compression and flexure	Mean σ over section $\leq \sigma'_b$ compression Maximum $\sigma \leq \sigma'_b$ flexure	
Concrete	Shear	$\tau_0 = 4 + \frac{(f_{ck} - 150)}{75}$	Kg/cm ²
		$\tau_1 = 14 + \frac{(f_{ck} - 150)}{35}$ τ must be $< \tau_1$ and only minimum shear reinforcement is required if $\tau < \tau_0$	kg/cm ²
Steel	All	$\sigma'_s = 2200$ for FeB38 ribbed bars	Kg/cm ²

Table 6. 4: Summary of the admissible stresses for concrete with compressive strength of 30MPa and FeB38 steel defined by the Italian design codes of 1982 (Table 6.3), for use in conjunction with the working stress design and verification method.

The design of the 3 storey infilled reinforced concrete moment resisting frame is presented in Figures 6.4 to 6.6. The frames are symmetrical in plan and elevation, with the external and internal frame designs differing due to changes in the design loads (Figure 6.3). A summary of the more important limitations on element sizes and detailing are summarised in Table 6.6. The resulting roof and floor slab designs for the 3 storey frame are illustrated in Figure 6.6. A grid of reinforcement is provided in both the top and bottom of each slab. The grid bar spacing within

1.125m of each beam-axis is double that within the central 2.25m of each span. The beam and column member designs for the internal and external frames are illustrated in Figures 6.5 and 6.4, respectively. Despite the inclusion of seismic actions in the design, the shear reinforcement in the latter members is insufficient for significant levels of section confinement.

Element	Object	Detailing Limitations
Beams and Columns	Dimensions	Concrete cover: $c \geq 2\text{cm}$ Effective flange width: $b_{eff} = B_{web} + 10t_{slab}$ for internal beams $b_{eff} = B_{web} + 5t_{slab}$ for external beams
	Longitudinal reinforcement	Longitudinal bar diameter: $12\text{mm} \leq \phi_l \leq 30\text{mm}$ for columns $5\text{mm} \leq \phi_l \leq 30\text{mm}$ for beams Minimum bar spacing: $s_l = \max(\phi_l, 2\text{cm})$ Minimum reinforcement ratio: $\gamma_{min} = 0.8\%$ for $A < 2000\text{cm}^2$ $\gamma_{min} = 0.5\%$ for $A > 8000\text{cm}^2$ $\gamma_{min} = [0.8 - 0.3((A - 2000)/6000)]\%$ for $2000 \leq A \leq 8000\text{cm}^2$ Maximum reinforcement ratio: $\gamma_{max} = 5.0\%$ Minimum straight anchorage length: $l_a = \max(12\phi_l, 15\text{cm})$ Minimum lap splice length: $l_{spl} = 35\phi_l$ Minimum inter-lap splice distance = $60\phi_l$ Lap splices must not involve more than 1/3 of the steel in the cross-section.
	Shear reinforcement	Stirrup spacing: $s \leq \min(0.5B, 15\phi_l, 25\text{cm})$ for columns $s \leq \min(0.5B, 0.8d, 15\phi_l, 25\text{cm})$ for beams Beams: If B_{web} or $D > 400\text{mm}$ at least 4 stirrup legs must be used Minimum cross sectional area of stirrups/m = $3\text{ cm}^2/\text{m}$ At least 3 hoops/m must be provided Columns: If B or $D > 400\text{mm}$ at least 4 stirrup legs must be used Transverse reinforcement bar diameter: $\phi_w \geq 6\text{mm}$
Slab	Dimensions	Concrete cover: $c \geq 0.8\text{cm}$ Minimum slab thickness: $t = \max(\text{span}/30, 8\text{cm})$
	Reinforcement	Minimum reinforcement ratio: $\gamma_{min} = 0.15\%$ Maximum reinforcement ratio: $\gamma_{max} = 4.0\%$ Reinforcement in any one direction must not be less than 20% of that in the orthogonal direction. Minimum anchorage length: $l_a = \max(10\phi_l, 15\text{cm}, (\phi_l \cdot f_{cd}/36\tau_0))$ Minimum lap splice length: $l_{spl} = \max(20\phi_l, (\phi_l \cdot f_{cd}/12\tau_0))$

Table 6. 5: Summary of the detailing prescriptions for reinforced concrete structural members given by the Italian codes of Table 6.3. The values reported assume a 30MPa concrete grade and 380MPa grade steel ribbed bar reinforcement

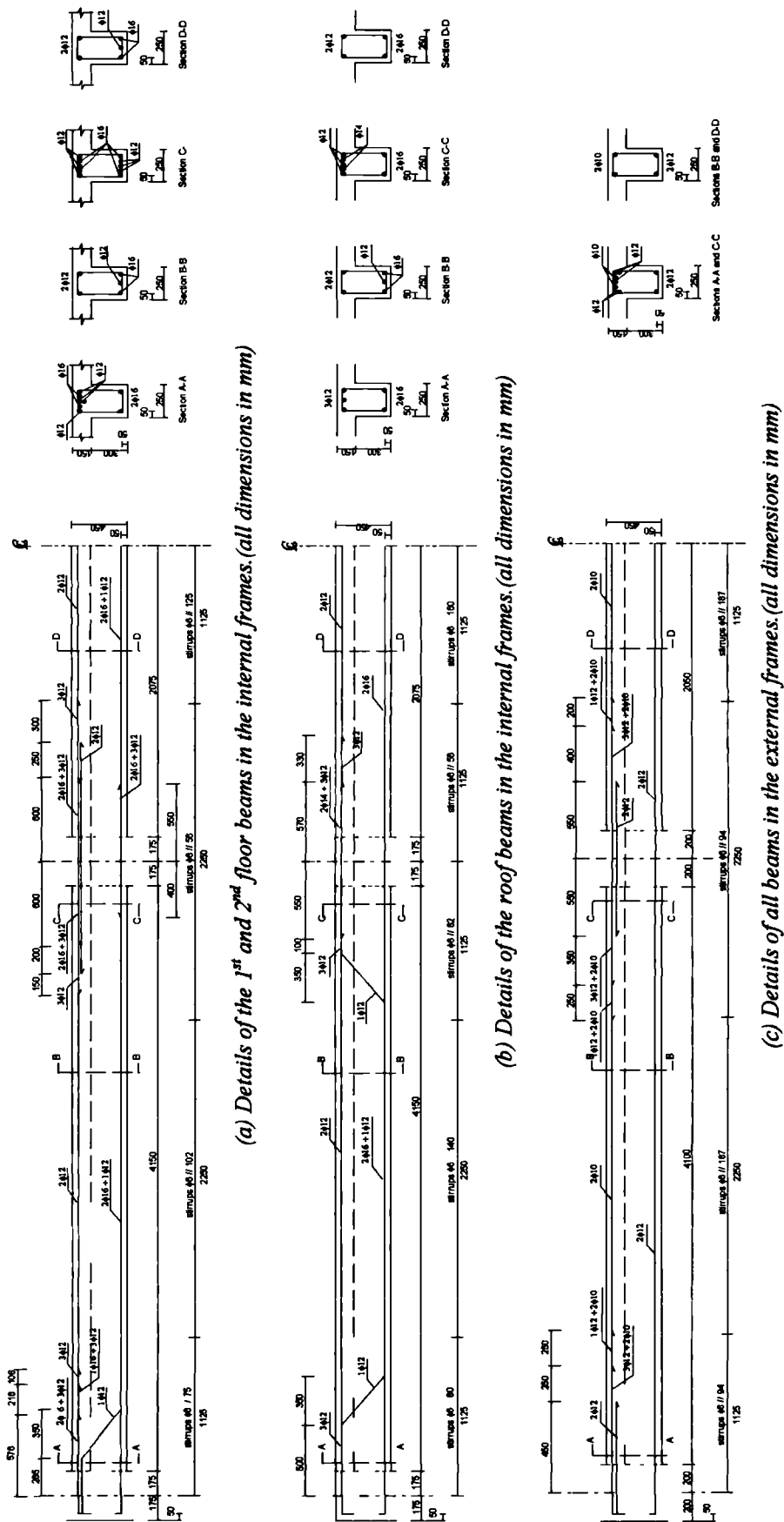


Figure 6. 5: Details of the beams in the 3 storey infilled frame designed to the 1982 Italian seismic codes.

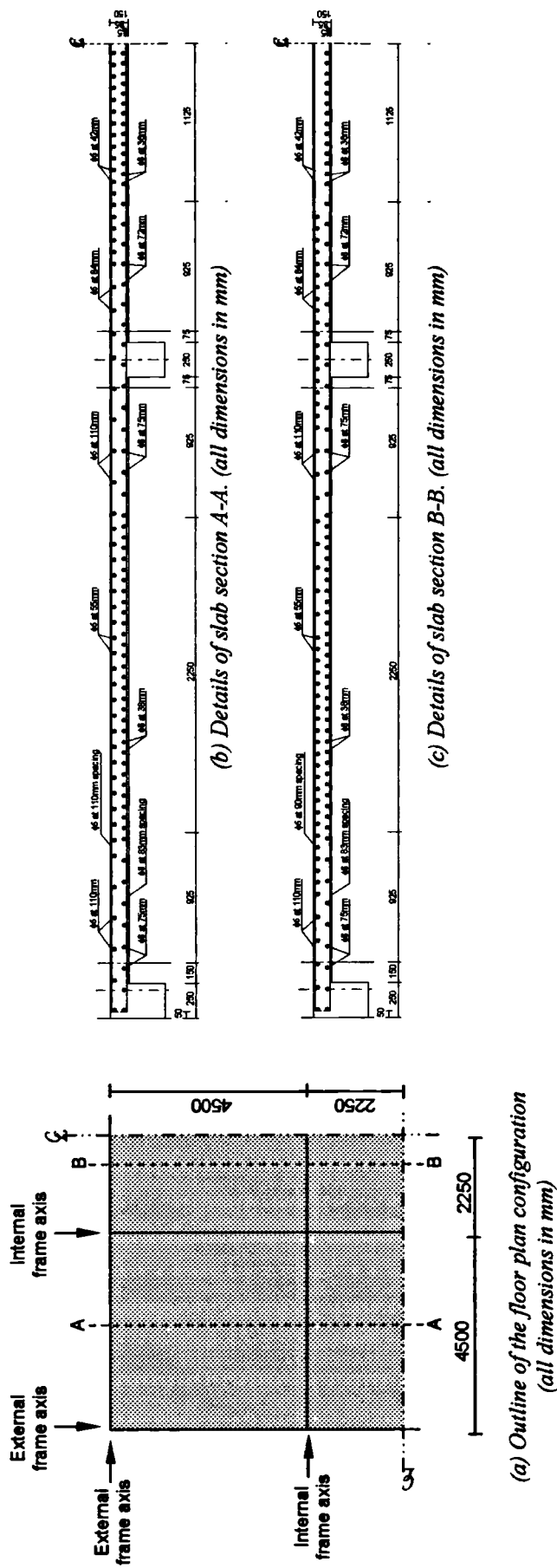


Figure 6. 6: Details of the floor and roof slabs of the 3 storey infilled structure, designed to the 1982 Italian seismic codes.

6.2.2 System Modelling

The degree of idealisation in structural modelling and the choice of modelling tool, can significantly affect the results obtained from analyses. Through consideration of different lumped plasticity model idealisation techniques, Park and Ang (1985b) obtain coefficients of variation of 0.12, 0.29 and 0.52 for the yield strength, elastic stiffness and first mode damping response values in reinforced concrete frames, respectively. In the present case, the 3 storey infilled frame is modelled using the Inelastic Dynamic Analysis of Structure finite element package, INDYAS (Elnashai et al. 2000). The program is developed from ADAPTIC (Izzudin and Elnashai 1989) and has been verified elsewhere, (e.g. Elnashai and Elghazouli 1993, Broderick and Elnashai 1994, Elnashai and Pinho 1998). The program allows a three-dimensional finite-element model of the building to be constructed, wherein the distribution of reinforcement within sections is modelled bar-by-bar and the non-linear behaviour of materials is taken into account. This high level of refinement in material, section and structural modelling is regarded as sufficient for prediction errors due to structure idealisation to be assumed minimal.

Symmetry in the building allows only half the structure to be modelled for the analysis. This results in the same periods of vibration and displacement response as for the whole building, but at half the value of base shear. The element and node pattern used to model the frame is illustrated in Figure 6.6. Inertia masses and vertical loads are placed at each beam-column joint to represent the self weight and gravity loads. The slab is not modelled as an individual element, but is represented by joints, which link the interior and exterior frames at the beam-column connections and transfer all horizontal and vertical movements, but no rotations, between the frames. The contribution of the slab to the beam resistance is accounted for by modelling the beams as T-sections, with effective widths equal to the sum of the web width and 0.14 times the span length. Cubic elasto-plastic 3D elements are used to model the beams and columns. These account for the spread of inelasticity along the member length and across the section depth, by dividing the cross-section, at the two element Gauss points, into a number of monitoring areas to which the material models are applied. To obtain more

accurate inelastic modelling, each beam and column is subdivided into 6 and 5 elements, respectively, as shown in Figure 6.6. The location of the first node is chosen such that the second element Gauss point approximately coincides with the location of the first member Gauss point. The second node location is determined so as to enclose the expected spread of plasticity, estimated using the plastic hinge length equations of Paulay and Priestley (1992) and Rossetto (2002).

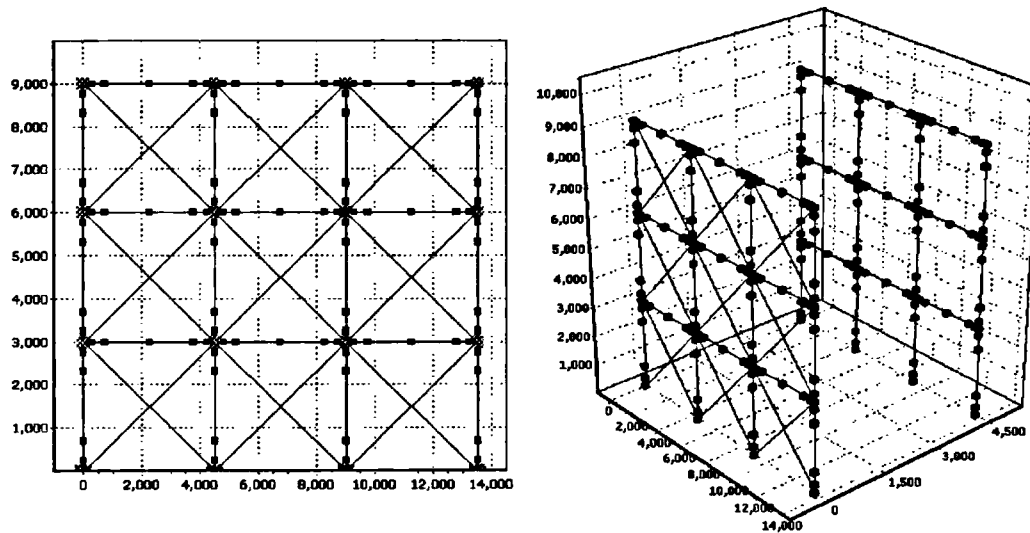


Figure 6. 7: Element and node pattern of the infilled frame finite element model used within INDYAS (Elnashai et al. 2000), for the analyses.

The CON2 model shown in Figure 6.7, is used to represent the concrete behaviour. This is an uni-axial constant confinement model and is defined by the compressive strength (f_c), tensile strength (f_t), crushing strain (ϵ_{co}) and confinement factor (k) of the concrete. The Mander et al. (1988) model is used to represent the effects of concrete confinement. The predicted concrete behaviour at large strains is sensitive to the choice of confinement model. This is confirmed by Kappos, Chryssanthopoulos and Dymiotis. (1998), who evaluated the ultimate concrete strain predicted by different confinement models, for a series of concentrically loaded reinforced concrete members of different configuration, tested in 5 experimental programs. The choice of confinement model is seen to introduce an average variation of approximately 0.1 in the COV of the ratio of experimental to predicted ultimate concrete strain. This implies that the confinement model choice affects the predicted damage and, therefore, that its contribution to the structural response uncertainty should be evaluated. However, in the case of existing buildings, due to inadequate detailing provisions, most

structural members are either only lightly confined, or totally unconfined. For example, in the present case all the beams are effectively unconfined whilst the columns have confinement factors of 1.004. Confinement model choice can therefore be regarded as a negligible source of response uncertainty. Nevertheless, in a previous study by the author, (Rossetto 2002), it is shown that of the five most commonly used confinement models, Mander et al. (1988) yields the best prediction of the experimentally observed deformation capacity of 726 reinforced concrete members. This model is therefore assumed adequate for the representation of concrete material behaviour herein. The STL1 material model is used to represent the steel behaviour. This is a bilinear elasto-plastic model with kinematic strain hardening. It is defined by the Young's modulus (E_s), yield strength (f_y) and strain hardening coefficient (μ_s) of the steel, as shown in Figure 6.7a.

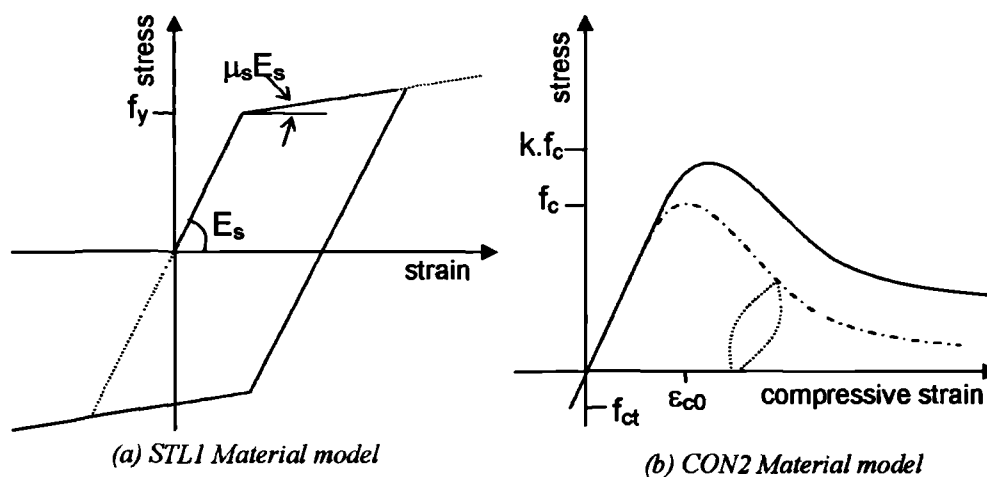


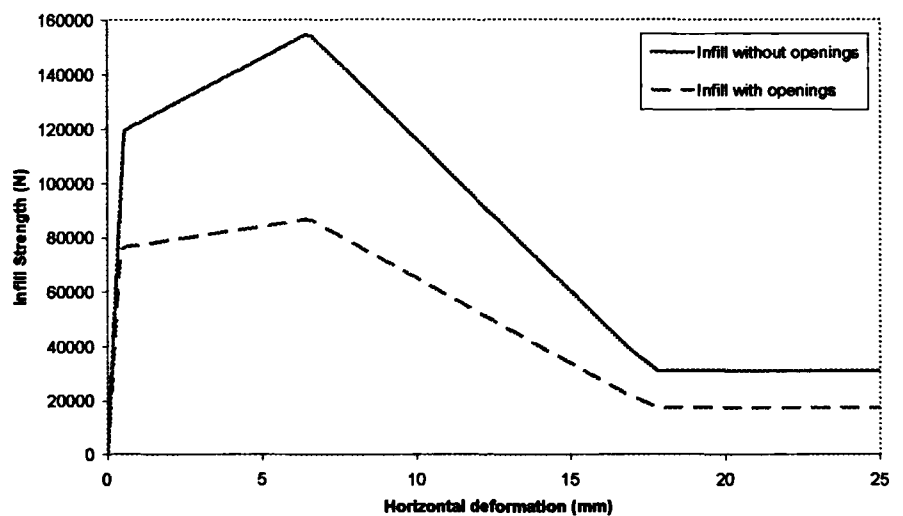
Figure 6. 8: The steel and concrete material stress-strain curves used within INDYAS (Elnashai et al. 2000)

The infill panel response is modelled via inclined rectangular struts that act in compression only. The lateral force-displacement curve adopted to model the response of the infill panels, is that of Panagiotakos and Fardis (1994). This is a tri-linear curve of the form shown in Figure 6.8a, and is formulated to account for the interaction between the infill and surrounding frame. Values of 0.26MPa and 1240MPa are used for the mean diagonal infill cracking strength and infill shear modulus, respectively. The mean horizontal infill compressive stress and Young's modulus are 2.4MPa and 2515MPa, respectively. These material property values

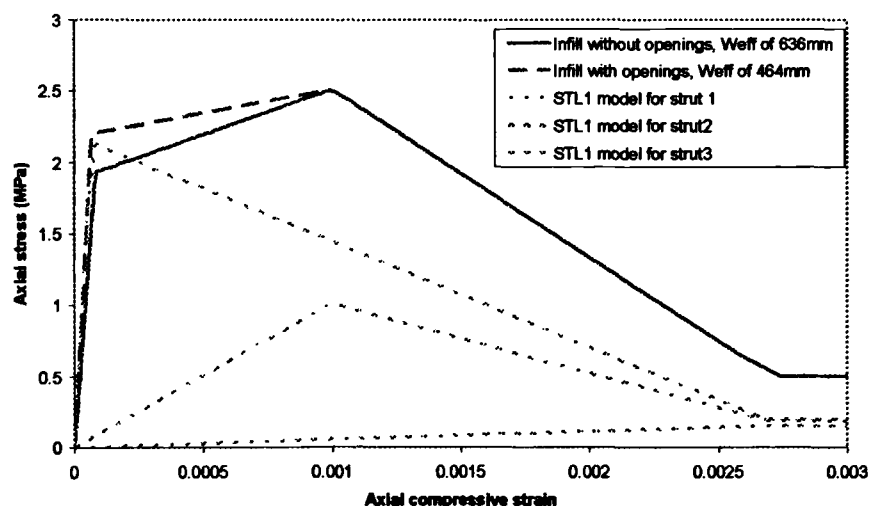
are taken from tests carried out on wallettes at the University of Pavia in Italy, (reported in Fardis ed. 1996), and are representative of typical masonry infill materials in Europe. The maximum strength value is calculated as 1.3 times the cracking strength. The stiffness of the softening branch is taken as 5% of the initial stiffness, and the residual post-ultimate strength as 0.2 of the ultimate strength. To define the equivalent strut behaviour, the lateral load-displacement curve is transformed into an axial stress-strain curve with the Mainstone (1971) formula being used to evaluate the diagonal strut effective width. Use of this strut width equation and material model combination, was observed to suitably represent the infill panel response of a full-scale infilled frame tested pseudo-dynamically at the ELSA laboratories in Ispra, Italy (reported in Fardis ed. 1996).

The effect of openings (e.g. windows and doors) on infill strength and stiffness is recognised by many authorities (e.g. CEB (1996)) and has been observed in experimental tests (e.g. Moghaddam and Dowling (1987)). However, few studies exist that quantify the effect of openings on infill panel behaviour, and none exist that comprehensively cover the large variety of masonry materials, degree of infill to frame connection and construction methods observed in practice. In this study, the relationships derived by Mosalam (1996) for unreinforced masonry infill walls with openings, are applied as modification factors to the infill panel load-deformation model of Panagiotakos and Fardis (1994). The relationships distinguish between pre- and post-cracking stiffness reduction due to openings, and are shown in Figure 6.9. No infill material model exists within INDYAS, nor are there any material models comparable to that of Figure 6.8 in the analysis program. The CON2 model may be modified to represent the infill strut axial load-deformation curve, via the selection of appropriate material parameter values. However, the fitting of these parameters is not simple and a bad representation of the degradation slope is observed. Furthermore, the CON2 model does not allow the distinct representation of both infill cracking and ultimate strength. Therefore, within the analysis, the infill is represented by three diagonal struts of equal rectangular cross-sections, acting in parallel and undergoing the same deformation. The parameters of the STL1 models defining the stress-strain curves of the struts, are selected such that the combined behaviour of the three struts results in the desired infill material response (see Figure 6.8b). The characteristics of the

individual stress strain curves are found deterministically through geometrical considerations (e.g. the sum of the initial slopes of the three material curves must equal the initial slope of the infill response model, etc). The distribution in stiffness and strength between the three parallel struts are chosen in such a way as to minimise numerical instabilities, which can arise due to sudden changes in stress-strain curve slopes. This method for representing infill behaviour is seen to reproduce the Panagiotakos and Fardis (1994) model well when applied in the pushover analyses of a single-storey, single-bay, frame and of the complete 3 storey building.



(a) Horizontal load – deformation curve of the infill panel with and without opening effect consideration (dashed and full lines respectively)



(b) Axial stress-strain curve of the infill panel (heavy dashed line) and of the three equivalent struts (lightly dashed lines)

Figure 6. 9: Horizontal load-deformation and axial stress-strain curve for the struts representing the infill panel behaviour. The curves are derived according to the Fardis ed. (1996) infill behaviour model, the Mainstone formula (Mainstone 1971) of effective strut width, Mosalam (1996) model for opening effects and the mean material properties.

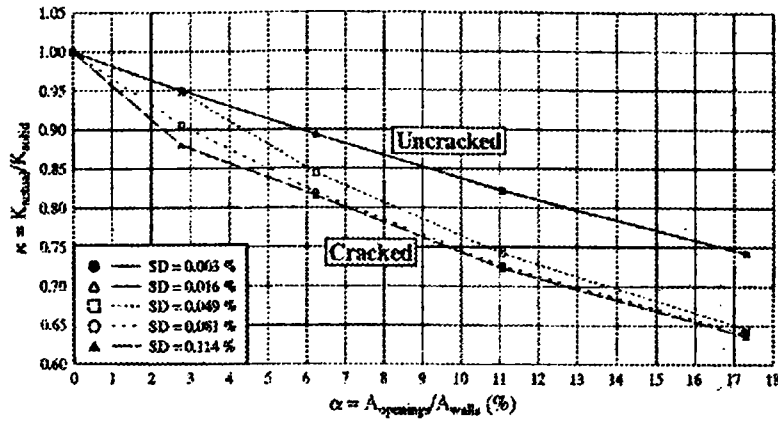


Figure 6. 10: Illustration of the effect of opening size on the lateral stiffness of single infilled frames for different applied lateral displacements (SD), according to Mosalam (1996).

Varying degrees of soil-structure interaction and response spectral amplification can arise for different soil conditions, depending on the relative stiffness of soil and structure (Jennings and Bielak (1973)). The complex modelling of soil and foundations substantially increases both the computation time number of analyses required, as the soil properties introduce a further large source of uncertainties. Soil-structure interaction is therefore omitted from the structural assessments herein. This omission is regarded to be conservative for most structural configurations and soil conditions. This is because existing studies (Merrit and Housner (1954), Castellani (1970), Veletsos and Prasad (1989), Luco and Mita (1987)) show that for buildings with relatively low natural periods, the combined soil-structure system results in a reduced fundamental frequency of the structure and an increased damping compared to a rigid base assumption. These result in a lower seismic demand (lower spectral amplification) and reduced building vibration response. This is not necessarily the case when displacement demand is considered, as the increase in demand due to the global stiffness reduction may not be fully counteracted by the increase in system damping due to energy dissipation in the soil (Lycom and Newmark (1961)). Within this study soil-structure interaction is not modelled explicitly in the analysis, however the effect of soil type on the spectral amplification, frequency content and duration of ground motion is accounted for by the selection of accelerograms for the analysis over a variety of soil sites.

6.3 Generation of Input Parameters

The proposed methodology for vulnerability curve derivation, prescribes the analysis of a population of frames with different structural properties, subjected to a number of earthquake records with distinct characteristics. In this way, the method is able to account for the effect of variability in seismic input and structural characteristics on the damage statistics simulated for the building class (*system*), and evaluate the associated uncertainty in the vulnerability prediction. The selection of “performance-consistent” earthquake records is discussed in Chapter 4, where the influence of seismic input variability on the vulnerability curves derived herein, is evaluated. A population of frames with different dynamic response characteristics can be generated through the treatment of selected structural parameters as random variables. Once the variables are selected, they are assigned probability distribution functions based on experimental observations. Values of the parameters are then extracted from the distributions using appropriate sampling techniques. Finally, the sampled values are combined to define a series of frames with different characteristics, all nominally representing the same structure.

Clearly, consideration of all possible uncertainties in the global and local structural characteristics and ground motion yields an extremely large number of permutations for the analysis. This is neither practical nor justified, considering the general nature of the application of the final vulnerability curves. In the following sections, the determination of the variable structural properties, their combination for the building population definition and the effect of structural variability on vulnerability uncertainty, are discussed.

6.3.1 Selection of Variable Structural Parameters

Uncertainty in the response of a structure can be represented via the direct variation of its global dynamic response characteristics. This is done by Mosalam et al (1997), who generate a population of SDOFs for use in analysis, by idealising their structure as a single degree of freedom system (SDOF) and

varying the slopes of its tri-linear force displacement response curve. The curve slopes are varied within an uncertainty envelope, defined around the main structure response curve. However, the width of the uncertainty envelope is determined arbitrarily and the method may yield results that are over conservative, or unrepresentative of the assessed system. The results of the analyses do not allow the explicit evaluation of the influence of parameter variation at the member, section or material level, on the global vulnerability. The use of such a method for the generation of a structure population is, therefore, only deemed suitable if the structure has to be simplified significantly for analysis. This is not the case here, where detailed modelling of the structure is possible, and an accurate, computationally efficient technique is proposed for the population analysis and assessment (see Sections 7.1 and 7.2).

In the proposed vulnerability curve generation approach, the general plan and elevation configuration of the system is treated deterministically. Changes in these are expected to alter the design forces, code prescriptions, member design and detailing, and hence warrant a separate vulnerability curve derivation. Within a class of buildings, differences exist at the member level in the terms of member length, section area, quantity and spacing of transverse reinforcement. In the present case, the configuration of the system is chosen to represent that typical of the structural class, with the section sizes and reinforcement detailing assigned according to the code prescriptions. The latter are not expected to vary significantly for small changes in member length. Consequently, the section size and transverse steel variation are not expected to introduce significant uncertainties in the global response. In the case of the 3 storey building presented in Section 6.2.1, the design code used provides insufficient transverse reinforcement for significant member confinement. Hence, the possible influence on confinement, of changes in reinforcement due to small variations in member length, is regarded to be particularly insignificant here.

At the section level, variations in section geometry, concrete cover, placement and area of steel reinforcement may occur. These variations are determined by the design and construction practice in the assessed area and are difficult to quantify. However, if the typical practice of providing minimum section sizes and

reinforcement for the code defined loads is considered, the variation in moment and shear resistance of sections within buildings of similar configuration, may be assumed small. This is supported by the work of Grant et al. (1978) and Frangopol et al. (1996), who show that the variability in section dimensions, (as built versus design), do not significantly contribute to the member response.

Variation in concrete, steel and infill material properties is typically large within a population, and derives from differences in manufacturing processes and local construction practices. Material variability can be regarded as the most important source of uncertainty in the determination of structural response, with all other sources either deriving directly from its effects, or being insignificant in size compared to it. For example, many authors have shown material variability to be more influential on section response than geometric variability involving gross steel areas and concrete cross-sectional areas (eg Grant et al 1978). It is noteworthy that this type of uncertainty is also considered in the design process, where characteristic values for material strengths are used and are further reduced using partial safety factors, to ensure structural safety. In most commonly used concrete, steel and infill material models, the following parameters determine the material behaviour:

- Concrete compressive strength (f_c) and ultimate strain (ϵ_{cu}).
- Yield strength (f_y) and strain (ϵ_y) of reinforcing steels.
- Ultimate strength (f_t) and strain (ϵ_t) of reinforcing steels.
- Infill panel compressive strength, (f_{wc}).

Use of all 7 parameters for the representation of material variability in the system, leads to the generation of approximately 143 building models for the analysis (considering the 2ⁿ Factorial Composite Method of experiment design (Hicks 1982)). This number increases significantly if the parameters are assumed to vary spatially in the structures. A reduced number of models for analysis, and consequently variable parameters, are desired. Therefore, the relative influence on the global response uncertainty of each parameter's variability is assessed in the following sections. Their spatial variation within single structures is discussed and

probability distribution functions are assigned to the chosen parameters from experimental observations.

6.3.1.1 Variation in Concrete Properties

The value of concrete compressive strength completely defines the material behaviour for an assumed confinement model. The influence of confinement model choice on structure response uncertainty is discussed in Section 6.2.2. There it is concluded that variation in the confinement model is of relatively little importance compared to that of the concrete unconfined strength, due to the structural members being effectively unconfined. For similar reasons, the member response is also assumed insensitive to variations in the ultimate strain, as this will equal the unconfined concrete strain and can be regarded as constant. Moreover, variability in the ultimate concrete strain does not affect the attainment of low damage states. Its influence on the global response uncertainty is therefore neglected.

Uncertainty in the concrete compressive strength is characterised by a probability distribution function, which describes the variation in the parameter mean value. The mean material parameter value used in the analysis is derived directly from the distribution using a sampling procedure. However, the method implicitly assumes a fixed value for the coefficient of variation of f_c , (COV_{f_c}). This model is adequate for the representation of concrete compressive strength, as experiments show that for a given level of control, significant variations in the mean value of f_c are observed, whilst approximately constant values of COV_{f_c} are maintained (Stewart 1991). In existing building assessments, the nominal value of concrete strength is typically assumed equal to the material characteristic value, and a mean strength value, which is larger than the nominal, is adopted for the building analysis (e.g. Singhal and Kiremidjian 1997). Experience has shown that the nominal value of compressive concrete strength is not always guaranteed in existing buildings due to lack of control, poor maintenance or material degradation with time (Petersons 1968). It is therefore proposed that the nominal concrete compressive strength be used to define the mean for the parameter

variation herein. This assumption also avoids the possibility of sampling concrete unconfined strength values that are comparable to confined strengths.

Information on the variability of f_c is assembled from various sources of experimental data (Conrad 1966, Petersons 1968, Mirza Hatzinikolas and MacGregor 1979, Hwang and Huo 1998). Normal distributions are generally used to describe the variation in mean concrete compressive strength, with $COVf_c$ values in the range of 10-15% adopted according to the conditions of control during mixing and curing (Mirza, Hatzinikolas and MacGregor 1979). The dependence of concrete strength variation on curing conditions is also recognised by Conrad (1966), whom suggests equations for $COVf_c$ for different degrees of workmanship. Within this study, medium control conditions are assumed, and from consideration of the collected experimental results, a value of 12% for the $COVf_c$ is adopted. In the case of the 3 storey infilled frame designed in Section 6.2.1, an insignificant effect on the confinement factor of the structural elements is observed when extreme material property values are assumed. Constant values for the member confinement factor are therefore used in the analyses. Spatial variation in the concrete strength is neglected within the current building class assessment. As the frame is of low height, changes in the concrete properties through the frame are not expected to be significant.

6.3.1.2 Variation in Steel Properties

Structural members are designed to be under-reinforced to avoid brittle failure. Consequently, uncertainty in the yield strength of steel is expected to have an important influence on the global behaviour of the structure, and to influence the attainment of low damage states. The influence of steel yield strength uncertainty on the axial load – bending moment interaction diagram of an under reinforced concrete column is shown in Figure 6.10, (Dymiotis 2000).

The Young's modulus of steel, (E_s), can be considered constant for a given class of steel, (i.e. $E_s = 200,000\text{Mpa}$). Steel yield strain, (ϵ_{sy}), can therefore be directly related to yield stress, ($\epsilon_{sy}=f_y/E_s$) and perfect correlation of the parameters is

assumed. Experiments show that ratio of maximum tensile to yield stress, (f_t/f_y) , does not exhibit a large variation for a given type of steel, (e.g. for European S500 Tempcore steels $f_t/f_y \approx 1.2$, whilst for high ductility mild steels of grade S400 or lower $f_t/f_y \approx 1.5$, Panagiotakos and Fardis 1999). (f_t/f_y) is therefore considered constant herein. Similarly to ε_{cu} , variations in the steel ultimate stress and strain, $(f_t$ and ε_t , respectively), are only important if member failure is dominated by rupture of the steel reinforcement. The occurrence of this failure mode is rare, especially in the presence of axial loads, and the parameters are not expected to significantly influence the rate of attainment of lower damage states. Uncertainty in these parameters is therefore disregarded.

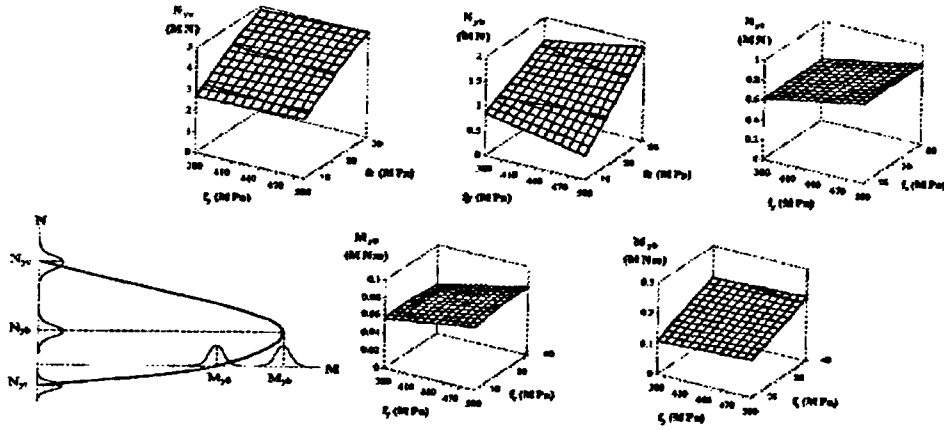


Figure 6. 11: Response surfaces for a 300 x 300 column section expressed in terms of concrete compressive strength and steel yield strength, (Dymiotis, 2000)

The probability distribution function used to represent the steel yield strength is described in terms of the uncertainty in its coefficient of variation (COV_{f_y}). A subtle difference exists between this approach and that used to model the concrete compressive strength, as here a fixed characteristic parameter value is implicitly assumed. This method is preferable in the case of the reinforcement yield strength, as the characteristic strength is a requirement of the design and is usually guaranteed, whilst the coefficient of variation of f_y varies according to the manufacturer. Experimental data regarding steel yield strength variation was collected from literature sources, (Baker 1972, Mirza, Hatzinikolas and MacGregor 1979, MacGregor et al. 1983, Pipa and Carvalho 1995, Hwang and Huo 1998). From this data, COV_{f_y} is seen to vary between 2 and 10%, depending on the type of steel and bar diameter. The frequency of occurrence of COV_{f_y} for

the 68 independent experimental observations collected from the literature (totalling 17,484 specimen tests), is plotted in Figure 6.11. The distribution of the number of specimens with $COVf_y$ is approximately uniform, and hence the latter values can be assumed independent of the sample size. A probability distribution function is fit to the plot via least-squares regression. Both normal and log-normal functions are considered to describe the distribution (see Figure 6.11). The latter function with a mean of 6% and standard deviation of 1.52%, is found to give the best data fit. The mean yield strength value ($f_{y\mu}$) used in the analysis, is determined indirectly from the $COVf_y$ value obtained from the sampling procedure, as $f_{y\mu} = f_{yk} / (1 - 1.645 COVf_y)$.

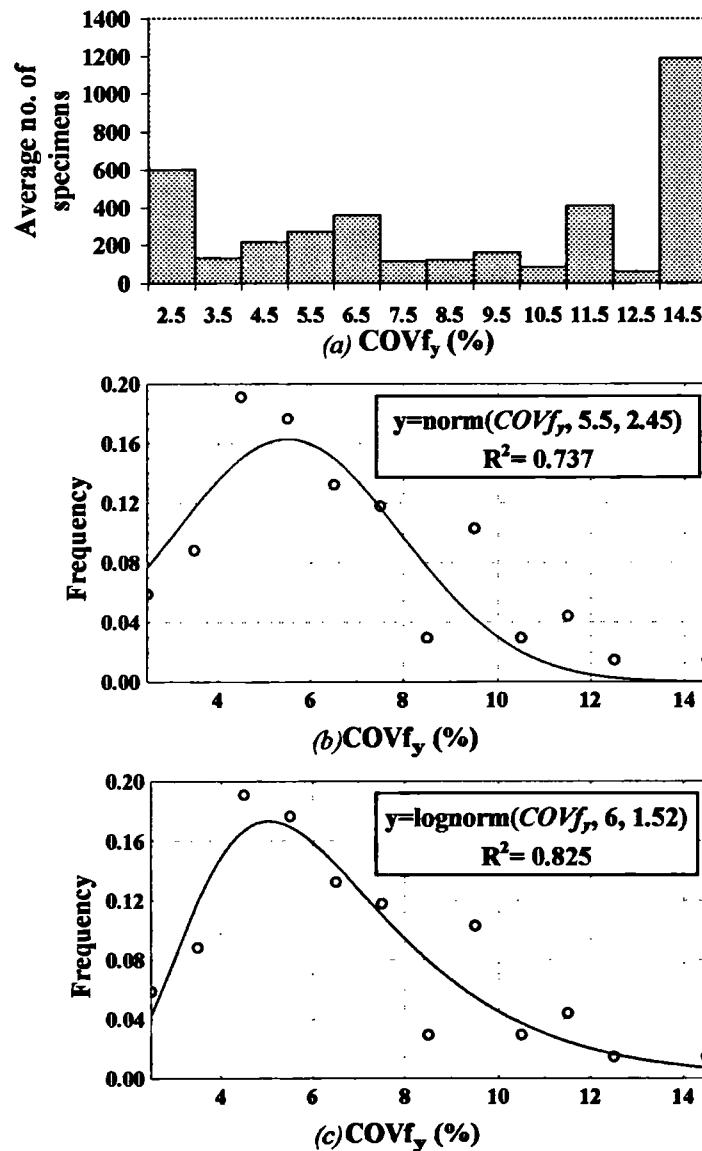


Figure 6.12: Plots of (a) the variation in sample size across the range of $COVf_y$ (%), (b) the normal and (c) lognormal functions fit to the $COVf_y$ frequency plot through regression (STATISTICA, StatSoft Inc. 1995).

Though typically associated with smaller coefficients of variation than concrete compressive strength, the relative values of steel strength in the beams and columns play an important role in the sequence of member yielding and the accumulation of damage. In practice, significantly different bar sizes tend to be used in the horizontal and vertical frame members. It is observed from the collected experimental results that bars of different diameter typically have different coefficients of variation. Spatial variability in steel yield strength is therefore considered through the treatment of beam and columns steel yield strengths, (f_{yb} and f_{yc} , respectively), as random variables. However, an assumption of complete independence between f_{yb} and f_{yc} is unrealistic, as the same manufacturer typically provides all the reinforcement for a site. Therefore a correlation coefficient of 0.7 is assumed between the parameters.

6.3.1.3 Variation in Infill Properties

Variability in infill properties does not only affect the failure state prediction, but is also important in determining the rate of attainment of lesser damage states. The infill panel compressive strength (f_{wc}) can be used to completely define the stress-strain behaviour of the panel using an appropriate constitutive model (e.g. Panagiotakos and Fardis 1994). Hence, it is used herein to represent the uncertainty associated with the infill behaviour.

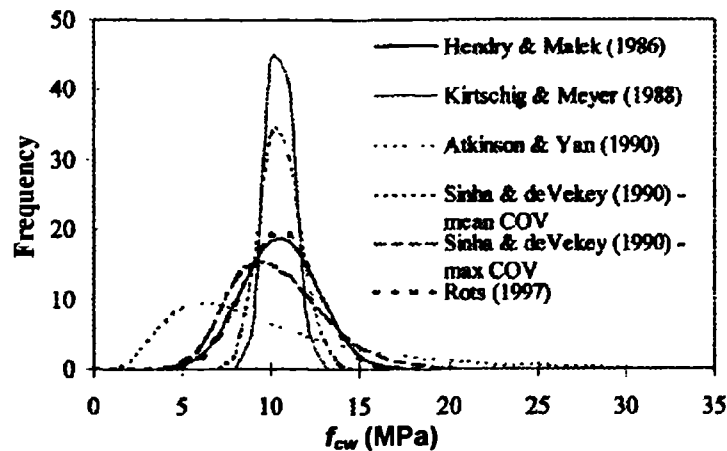


Figure 6. 13: Comparison of the experimental results obtained by various researchers for masonry infill panels with mean infill strength of 10MPa, (Dymiotis 2000).

Experimental values for the coefficient of variation of infill wall compressive strength, ($COVf_{wc}$), reported by different authors vary between 8% and 28%, (Figure 6.12). This variation in $COVf_{wc}$ is attributed to the wide range of materials used in infill panel construction and connection conditions with the surrounding frame. Due to the lack of comprehensive experimental test databases for infills, no empirical equation exists that relates $COVf_{wc}$ to the type of brick or tile material used. Poor control during construction adds to the parameter uncertainty and, in practice, a characteristic value of infill strength cannot be guaranteed. Hence, the mean infill panel compressive strength is chosen as the random variable. In accordance with published sources (e.g. Galambos et al. 1982, Sinha and De Vekey 1990, Fardis ed. 1996), variability in f_{wc} is modelled herein using a lognormal probability distribution, with an average $COVf_{wc}$ value of 20%.

Consideration of panel by panel variation of infill compressive strength introduces a large number of random variables and substantially increases the number of frames required to fully account for material uncertainty. The consequent increase in analysis time is not justified if account is taken of the fact that the construction practice and infill materials are typically the same at a single site, and that the analysed structure is only 3 storeys high. The large value of $COVf_{cw}$ adopted is, therefore, assumed to account for the variation in structural response due to uncertainty in infill panel characteristics.

6.3.2 Parameter Sampling and Combination

A set of frames is defined for the analysis based on the main building model (presented in Section 6.2.3), through consideration of material uncertainty. Table 6.8 summarises the random variables used to account for material variability and their associated probability distribution functions. Random values of the material parameters are sampled from these distributions and combined to define the material characteristics of each frame. Analysis of the population of frames, under increasing intensities of ground motion, results in exceedence probability values that can be used to define vulnerability curves for the system.

The Monte Carlo algorithm is often used for the random sampling of uncertain parameters from their associated probability distributions. This sampling process is completely random. Repetition of identical sampled values and combinations can occur and large sample sizes are required by the method to ensure adequate coverage of the entire range of parameter values. In the present case, this implies running an impractical number of structural analyses. Various methods exist for limiting the number of parameter values and combinations required. These variance reduction techniques, offer an increase in the efficiency of the simulation-based assessment of structural reliability for a relatively small number of simulation cycles, and expedite convergence. The Latin Hypercube Method is an example of such a technique. It divides the probability distributions into N equally spaced sub-divisions within which the sampling is carried out. A single value from each subdivision is selected, the location of which is determined from Monte Carlo sampling on a uniform distribution with limits 0 and 1, (assumed to coincide with the sub-division boundaries). A set of N random values are thus obtained for each of the material parameters. These are then combined using a shuffling algorithm, such as that of Law and Kelton (1991).

Material	Concrete	Steel	Masonry Infill
Random Variable	Mean f_c	COV f_y	Mean f_{cw}
Distribution	Normal	Log-normal	Log-normal
Mean	Nominal f_c	0.06	Nominal f_{cw}
COV	0.12	0.25	0.20
Design Variable	Mean f_c	Mean f_y (for f_{yk} =nominal f_y)	Mean f_{cw}

Table 6. 6: Summary of the material random variable probability distribution functions.

The 2ⁿ Factorial Composite Method (Hicks 1982), is another sampling and parameter combination procedure commonly used in the design of experiments, for response surface generation and the evaluation of uncertainty effects. This method allows a reduced number of analyses to be carried out, through the choice of key random variable values for the analyses. A total of $2^n + 2n + 1$ sampled values and combinations are required by the method to generate a second-order

response surface that will fully represent the uncertainty associated with n independent random variables. This yields an estimate of 25 frames in the present case. Unlike the Latin Hypercube method, a correlation coefficient cannot be introduced into the sampling directly. Therefore, a smaller number of random variable combinations than is predicted by the 2^n Factorial Composite method should be required for the representation of parameter uncertainty, if a degree of correlation exists amongst these. Due to the correlation between the beam and column reinforcement yield strengths, the Latin Hypercube method is regarded to be more adequate for use here.

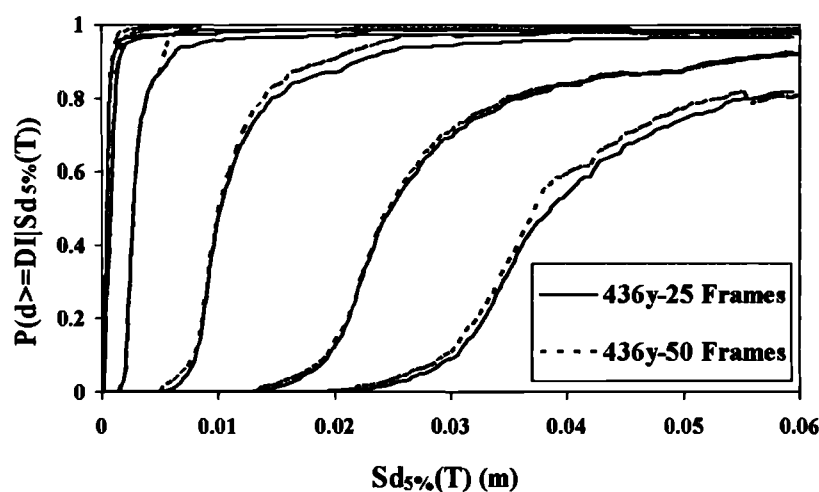


Figure 6. 14: Comparison of vulnerability curves obtained from analyses on populations consisting of 25 and 50 variations of the 3 storey infilled frame subjected to the 436y Killini earthquake record (Greece 1988).

Effective representation of the material uncertainty can only be ensured through comparison of vulnerability curves derived from the use of different population sizes. Figure 6.13 shows the vulnerability curves resulting from the assessment of populations consisting of 25 and 50 infilled frames under the scaled Kilini record, (Greece 1988, reference 436y in Table 4.4). The curves are seen to converge, hence 25 material combinations are adopted in the assessment of the system performance under the full set of earthquake records. The Latin Hypercube sampling method is used to select 25 values from each of the three material parameter probability distributions. The parameter value selection is carried out over the range defined by the lower and upper 5 percentile values of the material parameters. During the sampling, a correlation coefficient of 0.7, is maintained

between the column and beam steel yield strength values. The parameter values are shuffled and combined according to the algorithm adopted by Law and Kelton (1991). The resulting 25 sets of material property combinations defining the system population are summarised in Table 6.10.

Combination No.	f_c (MPa)	f_{yc} (MPa)	f_{yb} (MPa)	f_{cw} (MPa)
1	40.78	412.39	439.37	0.30
2	41.20	398.81	383.84	0.24
3	34.30	410.25	401.70	0.31
4	34.48	413.79	425.87	0.20
5	35.81	402.42	406.97	0.23
6	39.85	408.73	405.63	0.21
7	42.36	415.63	440.64	0.26
8	38.04	407.07	417.57	0.25
9	37.26	427.46	435.26	0.22
10	32.84	412.84	394.49	0.32
11	31.80	401.31	407.12	0.24
12	36.89	409.21	401.93	0.29
13	39.06	405.65	410.70	0.28
14	30.72	416.46	401.78	0.22
15	36.48	425.68	439.18	0.33
16	38.15	417.24	407.33	0.26
17	38.54	419.28	433.23	0.26
18	33.79	421.09	416.49	0.24
19	36.37	404.42	395.04	0.35
20	30.32	403.89	402.86	0.29
21	40.37	433.87	475.10	0.20
22	35.61	411.54	395.62	0.27
23	33.18	407.35	388.98	0.23
24	34.83	423.34	399.79	0.19
25	43.32	422.01	430.93	0.27

Table 6. 7: The material parameter combinations of the 25 infilled frames used in the analysis.

6.3.3 Assessment of Building Capacity Randomisation

The effect of material variability on the adaptive pushover response of the structure population is illustrated in Figure 6.14, for the 25 infilled frame models,

subjected to the Colfiorito record from the Umbria-Marche earthquake, (Italy 1997, reference 592y, Table 4.6). In order to evaluate the response uncertainty due to material variability, each adaptive pushover curve is idealised by a tri-linear plot, as shown in Figure 7.12c. The yield point is determined as the first deviation from linear behaviour. The location for the second change of slope is determined such that equal areas are maintained between the idealised and actual response curves (see Section 7.3.1). The variations in structural yield period, stiffness and top drift values defining the idealised curves, are evaluated for each ground motion record separately. The mean response values and coefficients of variation observed for the 30 sets of 25 curves, are presented in Table 6.8.

Random Variable	Present Study on Infilled Frames		Mosalam et al (1997) Infilled Frames	
	Mean	COV(%)	Mean	COV(%)
K1 (kN/mm)	720.610	14.38	9.86	30
K2 (kN/mm)	24.813	54.03	7.27	40
K3 (kN/mm)	5.983	36.96	1.87	30
U1 (mm)	0.565	36.73	1.02	40
U2 (mm)	26.504	52.80	3.28	30
T_y (s)	0.125	10.32	-	-

Table 6. 8: Comparison of the uncertainty in pushover response due to material variability assumed by Mosalam et al (1997) for the representative single degree of freedom structure, and measured as the average of the effect on the population response evaluated separately for each of the 30 earthquake records adopted herein.

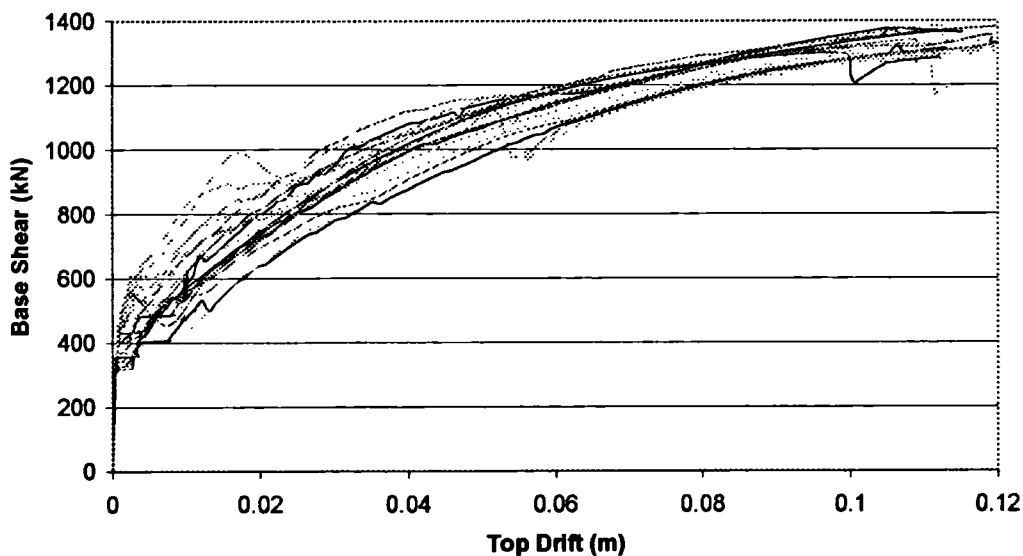


Figure 6. 15: Variation in the adaptive pushover response of the 25 infilled frames representing the system population, when subjected to the Colfiorito record from the Umbria-Marche earthquake (Italy 1997, reference 592y).

Mosalam et al (1997), consider model capacity uncertainty in the vulnerability evaluation of low-rise reinforced concrete infilled frames. The stiffness and displacement values describing the tri-linear response of the equivalent single degree of freedom systems used to model their structure, are varied directly according to the distribution characteristics described in Table 6.12. From comparison with the values obtained in the present work, the former tend to overestimate the variation in the yield point coordinates, but underestimate that of the post-yield stiffness and slope change drift values.

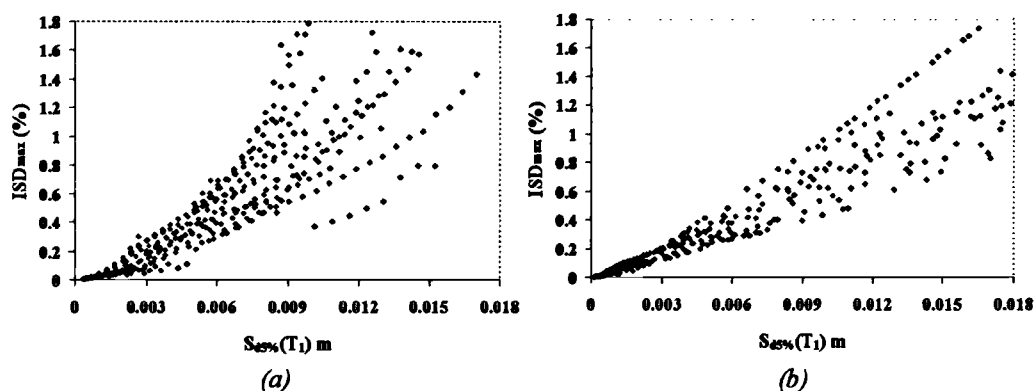


Figure 6. 16: The effect of material uncertainty on the maximum interstorey drift response (ISD_{max}) of the system when subjected to (a) record 142ya (aftershock of Friuli, Italy 1976) and (b) record 340xa (Lefkas, Greece 1981).

95YRP			475YRP			2475YRP		
Scaled pga (gm/s ²)	Mean ISD_{max} (%)	COV (%)	Scaled pga (gm/s ²)	Mean ISD_{max} (%)	COV (%)	Scaled pga (gm/s ²)	Mean ISD_{max} (%)	COV (%)
0.10	0.0275	25.17	0.10	0.0226	24.80	0.07	0.0167	32.60
0.25	0.1090	25.20	0.26	0.1378	20.55	0.29	0.1365	22.47
0.49	0.3730	17.60	0.51	0.4753	22.21	0.51	0.2623	16.84
0.74	0.6648	18.51	0.77	0.7646	16.04	0.73	0.4906	18.42

Table 6. 9: Evaluation of uncertainty in the maximum inter-storey drift response assessment values due to material variation. The values represent the average of those observed for the infilled-frame population for each earthquake, scaled to four pga values.

95YRP			475YRP			2475YRP		
Scaled pga (gm/s ²)	Mean S _d (T ₁) (m)	COV (%)	Scaled pga (gm/s ²)	Mean S _d (T ₁) (m)	COV (%)	Scaled pga (gm/s ²)	Mean S _d (T ₁) (m)	COV (%)
0.10	0.0009	23.30	0.10	0.0008	32.26	0.07	0.0007	35.19
0.25	0.0023	21.06	0.26	0.0019	24.93	0.29	0.0022	12.33
0.49	0.0047	18.35	0.51	0.0039	22.34	0.51	0.0036	12.37
0.74	0.0071	19.08	0.77	0.0058	23.14	0.73	0.0049	11.31

Table 6. 10: Evaluation of uncertainty in the spectral displacement values predicted for single earthquakes due to material variation. The values represent the average of those observed for the infilled-frame population in each earthquake.

The influence of material uncertainty on the system damage prediction can be evaluated from the final capacity assessment results, in terms of the variation in the predicted maximum interstorey-drift response (ISD_{max}) at different ground motion severities. Each earthquake record is considered separately and the mean assessed ISD_{max} response and its variation are evaluated for the corresponding 25 adaptive pushover curves for the records scaled to four pga values. The average values calculated over the ten earthquake records characterising each hazard level, are presented in Table 6.9. Similarly, the variation due to material uncertainty of the 5% damped elastic spectral displacement, is evaluated and reported in Table 6.10. The coefficient of variation of the predicted inter-storey drift, introduced by the material uncertainty, ranges between 16.8% and 32.6%. The variation tends to reduce slightly with increasing earthquake scale factor, but can be assumed to take an average value of 21.7% for all scenarios. Material variability introduces a coefficient of variation of 10.32% in the structural yield period. This results in an average COV of 21.3% for the ground motion response ($S_{d5\%}(T_1)$), which is comparable in size to that observed for ISD_{max} .

Chapter 7

Analytical Vulnerability Curves Part 2: Analysis, Assessment, Curve Derivation and Verification

7.1 The Analysis - Evaluation of the Models

In order to generate vulnerability curves, the building populations determined above must be assessed for damage under increasing levels of seismic load. The damage limit states of the structures are determined using the relationships derived in Chapter 3 between building damage and maximum inter-storey drift ratio ($ISD_{max\%}$). Use of $ISD_{max\%}$ as the damage measure, requires the analysis tool to reproduce the structure displacement response at the storey level with reasonable accuracy. Both the SEOAC Report “Vision 2000” (1995) and FEMA 273 (ATC 1997), suggest different elastic and inelastic, static and dynamic analysis methods for the assessment of the global structural behaviour under earthquake loading. The main methods used in practice, listed in order of increasing complexity are:

- Elastic Static – equivalent lateral force technique
- Elastic Dynamic – multi-modal response spectrum analysis
- Inelastic Static – pushover analysis
- Inelastic Dynamic – inelastic time history analysis

Elastic methods of assessment are seen to be inappropriate for use in this study due to their inability to represent the structural deformation at large displacements and consequent poor damage prediction. Non-linear static analyses are the most widely used seismic assessment tools in practice due to their simplicity and computational efficiency. These tools are, however, limited in their ability to predict storey displacements due to their use of constant force distributions and non consideration of second or higher mode response. Consequently, bad correlation with inter-storey drift ratios predicted by non-linear time-history

analyses has been reported by many sources (e.g. Seneviratna 1995, Elnashai 2001, Antoniou 2002). In the present case, the regularity of the building models and the general nature of the developed prediction tool may justify the use of this analysis procedure. However, the method has been seen to give very poor predictions for infilled structures due to the large period shifts that can occur after infill panel failure. An analysis methodology with improved modelling of stiffness degradation and of seismic demand effects on building capacity is therefore desirable.

Non-linear time history analyses require thorough knowledge of the assessed structure and of the probable ground motion. Time-history analyses consist of a step-by-step calculation of the structural response through the time domain. All aspects of the ground motion that can influence the building damage are accounted for in the analysis through the direct use of acceleration records as input. Geometric and material nonlinearities can be modelled into the analysis and more accurate building damage evaluations carried out. However, the method is more computationally demanding and time exhaustive than any other procedure. In the present study we deal with populations of structures for which the ground motion and structural characteristics are prone to variability. To account for these sources of uncertainty, multiple analyses are required wherein the characteristics of the input motion and building are altered via probabilistic means. The resulting number of repetitions and time of analysis are cumbersome. For example, in the present case study, where 25 infilled-frame models are to be analysed under 30 earthquake records scaled to 15 intensity levels, the required analysis time is approximately 4 years (assuming 3 hours per analysis). Considering current computational capabilities, it is not feasible to use an inelastic time-history analysis method for vulnerability curve generation.

It is concluded that none of the aforementioned conventional analysis techniques are adequate for achieving the goals of this research, either due to their inability to predict structural displacements or due to time and cost considerations. Innovative analysis methodologies are therefore considered for use in the vulnerability prediction. Recently, much effort has been dedicated to the development of a technique for structural analysis, which has the visual clarity of

the conventional pushover technique but incorporates effects of stiffness degradation and ground motion on the response. The principles underlying the Non-linear Adaptive Pushover Analysis method developed by Elnashai (2001) and Antoniou (2002), are described in the following section. In order to assess the suitability of the method for use in vulnerability curve generation, a feasibility study into its damage prediction abilities is carried out, and also presented below.

7.1.1 Non-Linear Adaptive Push-over Analysis Procedure

Pushover analyses are associated with analysis times that are a fraction of those required for full non-linear dynamic time-history analyses. Therefore, they are amenable for use in cases where numerous analyses are required. In conventional pushover procedures, a constant distribution of forces is applied to a structural model incorporating inelastic material properties, and is incremented to structural failure. At each applied load increment, iterations are carried out for the solution of the static equilibrium equations, wherein the out-of-balance forces are re-applied and the resistance of the structure and stiffness matrix are recalculated until convergence. Typically, the lateral forces applied at each floor of the building are assumed to be proportional to the floor masses (Uniform distribution), proportional to the storey heights (Triangular distribution) or are calculated from modal analysis using an appropriate spectral shape and number of modes (Modal distribution). In theory, the applied lateral force distribution should approximate that of the inertia forces experienced under an earthquake. However, seismic demand changes with the earthquake severity and degradation in structural dynamic properties, (as is shown, for example, in Gupta and Kunnath 2000). A single force distribution shape cannot properly represent the structure response over its entire displacement capacity. Moreover, large differences are observed between the pushover curves obtained using different force distributions (e.g. Mwafy and Elnashai 2001). This is recognised by many authorities, amongst which is the Federal Emergency Management Agency (ATC 1997), who recommend the use of an average response curve, obtained from the repetition of pushover analyses for more than one lateral load distribution.

In the Adaptive Pushover Method (APO), progressive updating of the lateral load distribution at each load increment is carried out, in order to take into account the stiffness degradation in the structure, changes in modal properties and consequent ground motion demand. The first and only instance in which an adaptive pushover method has been used within a vulnerability curve generation methodology is in Mosalam et al. (1997). The adaptive pushover method they use, takes a step towards the inclusion of stiffness degradation effects in pushover analyses by updating the applied force distribution according to the fundamental mode shape. An Eigen-value analysis is first carried out and the initial distribution of equivalent lateral loads is calculated according to Equation 7.1.

$$f_i^1 = \frac{m_i \phi_i^0 \{1\}^T [M] \{\phi^0\}}{\{\phi^0\}^T [M] \{\phi^0\}} S_a^0 \quad [7.1]$$

In this equation, the superscripts T and 0 refer to the transpose and initial state, respectively. f_i^1 and m_i are the initial equivalent static load and the lumped mass at the i^{th} floor, respectively. ϕ_i^0 is the component of the current Eigen-vector corresponding to the fundamental mode calculated at the i^{th} floor. S_a^0 is the initial response acceleration normalised to 1. $[M]$ is lumped mass matrix and $\{1\}$ is a unit column vector. The force vector is increased monotonically until “significant changes” in the dynamic properties of the structure are detected. The response spectral acceleration is then recalculated using the current force value at any floor i , using Equation 7.2, where n refers to the current state of the structure.

$$S_a^n = \frac{f_i^n \{\phi^n\}^T [M] \{\phi^n\}}{m_i \phi_i^n \{1\}^T [M] \{\phi^n\}} \quad [7.2]$$

The Eigen-value analysis is repeated on the damaged structure and a new force distribution is calculated according to Equation 7.1. The magnitude of the applied forces is scaled such that their sum equals the current base shear, and then re-applied to the damaged structure. The force incrementation and force distribution updating procedure are then repeated until structural failure. Mosalam et al. (1997) adopt the adaptive pushover procedure to analyse non-linear structural models,

and use the resulting curves to calibrate equivalent SDOF systems. The latter are then subjected to dynamic time-history analyses, under several earthquake records, to generate vulnerability curves. The procedure yields improved models for the SDOF response but does not eliminate the need to carry out numerous time-history analyses. The analysis method does not include multiple modes and is only applicable to structures responding in the fundamental mode. It also does not account for changes in the demand spectral acceleration with stiffness degradation.

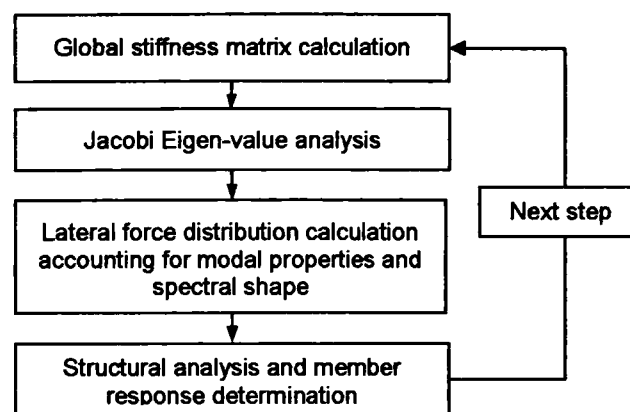


Figure 7. 1: Flowchart of the procedure followed within each incremental step of the Non-linear Adaptive Push-over analysis procedure of Antoniou (2002).

Paret et al. (1996), Bracci et al. (1997), Sasaki et al (1998) and Gupta and Kunnath (2000), amongst others, propose pushover methodologies wherein higher mode effects are taken into account. These authors also include spectral scaling of the applied force distributions to account for changes in demand within the analysis. However, the most complete methodology is the non-linear adaptive pushover analysis method developed by Antoniou (2002). It is multi-modal and accounts for period elongation and spectral amplification. Furthermore, it is implemented within the framework of the INDYAS finite element package (Elnashai et al. 2000), which allows sophisticated structural modelling and accounts for both spread inelasticity and member geometric non-linearity. This method is therefore used for the analysis of the structure population, in the here proposed analytical vulnerability curve derivation methodology.

The Adaptive Pushover (APO) analysis by Antoniou (2002) follows the same procedure as conventional pushover analysis, but at each load increment, the

lateral applied load distribution is updated to take into account the instantaneous stiffness, modal properties and consequent ground motion demand, as prefigure 7.1. The lateral force distribution at each load step, for each mode, is calculated from Equation 7.3, where i is the storey number, j is the mode number and n is the total number of modes considered. Γ_j is the modal participation factor for the j^{th} mode. ϕ_{ij} is the mass normalised mode shape value for the i^{th} storey and the j^{th} mode. M_i is the mass of the i^{th} storey and $S_a(j)$ the spectral amplification of the j^{th} mode.

$$F_{ij} = \Gamma_j \phi_{ij} M_i S_a(j) \quad [7.3]$$

The force distributions associated with each mode, are combined using the Complete Quadratic Combination (CQC) or Square-Root Sum of Squares (SRSS) methods, in order to obtain the force distribution along the building height. The forces are updated in a “total” fashion, which means that the forces applied at each increment are applied as an entirely new set of forces, accounting for the current state of the system resistance. An alternative “incremental” approach may be used, where only the increment of applied force is modified to account for the current state of the structure, and added to the force distribution of the previous step. The latter force combination method gives rise to a more stable solution in the case of sudden or large changes in structure stiffness. However, the total scaling is conceptually superior as there exists no rationale to the modulation of force increments. The result of the adaptive pushover analysis is a base-shear versus top displacement curve, which describes the building capacity under the considered seismic event. These pushover curves, once converted to Spectral Acceleration - Spectral Displacement (SASD) space, can be used directly within a Capacity Spectrum Method (CSM) assessment framework (see Section 7.2), to generate the damage statistics necessary for the vulnerability curve definition.

The APO methodology provides improved modelling of stiffness degradation and seismic demand effects on the building supply compared to conventional push-over techniques. However, it must be appreciated that the method is theoretically flawed and that there are several limitations to its application. Firstly,

modification of the force distribution is not possible past the peak of the push-over curve. The method assumes that the force distribution observed at the peak is continued into the high inelastic response range, which is often not the case for highly irregular structures. This implies that the method may not be accurate at large deformations, where high degrees of damage are expected to occur. Furthermore, although accounting to some degree for higher mode response, the analysis method is unable to deal with structures where second or higher modes dominate the response from the start of the analysis (Antoniou 2002). The procedure followed for the combination of spectral amplification factors has no theoretical substantiation, as it assumes that the structure simultaneously exhibits maximum response in all modes, and uses superposition concepts in conjunction with inelastic analysis (Elnashai 2001). The level of damping of the spectra is assumed by the method to be constant throughout the analysis, irrespective of the level of ductility. Structural response to earthquake records with acceleration spectra of peculiar shape, (i.e. with abnormal amplification characteristics) is also seen to be badly predicted by the method (Antoniou 2002). Notwithstanding, in many cases of regular or slightly irregular structures subjected to normal acceleration spectra, the adaptive pushover method is observed to closely replicate the results of time-history analyses in terms of total drift (Antoniou 2002, Vamvatsikos and Cornell 2002, Mwafy and Elnashai 2001, Rovithakis 2001). An example of the correlation between the adaptive technique and a dynamic pushover curve obtained for a 12 storey regular RC structure, is shown in Figure 7.2. The latter curve is obtained by plotting the maximum value of base-shear against the maximum total drift observed during inelastic dynamic analyses of the structure, under increasingly scaled ground motion records. Comparison of the curves shows that a single adaptive pushover curve is sufficient for the evaluation of the structure vulnerability, over all ground motion severities, for any earthquake of a given spectral shape. The repetition of structural analyses for each ground motion scale factor increment is eliminated and the number of analyses required for the determination of the vulnerability curves is considerably reduced (750 analyses, compared to 11250 for time-history analyses). The consequent reduction in analysis time and cost, render the adaptive pushover technique a highly desirable tool for application in vulnerability studies.

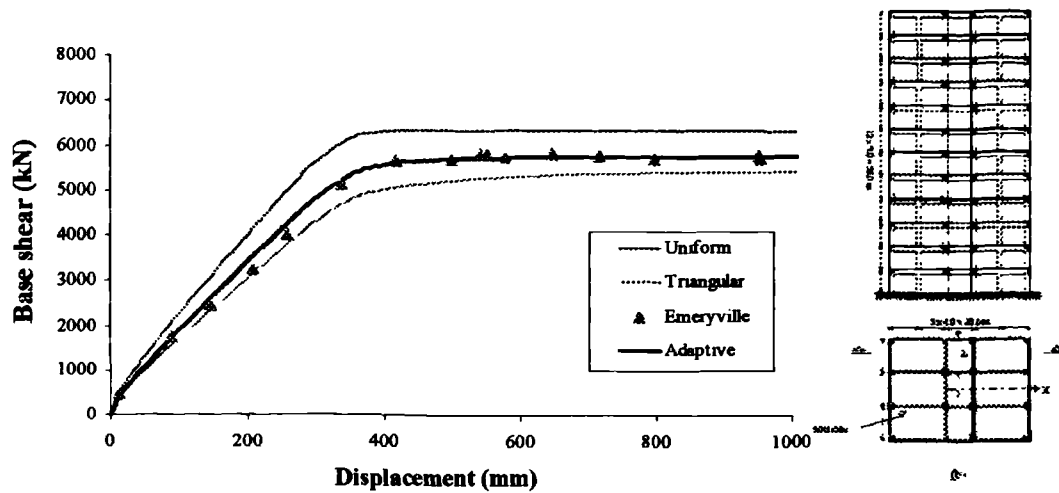


Figure 7. 2: Comparison of adaptive and conventional pushover analyses (using triangular and uniform distributions of force) with dynamic analyses carried out using the Emeryville record for a 12-storey regular RC frame, Rovithakis (2001).

However, the accurate prediction of structure total drift is insufficient proof of the damage prediction ability of the method. A good level of correspondence must be shown to exist between the predictions of maximum inter-storey drift ratio (ISD_{max}) resulting from time-historey analyses, and those made by the adaptive pushover technique at the corresponding total drift values. Antoniou (2002) assesses the interstorey drift response of 5, 10, 15 and 20 degree of freedom idealised structural models with stiffness irregularities, at a total drift value of 3%. A good agreement is generally observed between the values predicted by the two analysis methods. Gupta and Kunnath (2000) use a similar adaptive pushover method in the analysis of five reinforced concrete frame buildings varying in height between 4 and 20 storeys. A reasonable correlation between the ISD_{max} values obtained from pushover curves and time-history analyses is observed and the sequence of formation of local and global collapse mechanisms is satisfactorily predicted. Conventional (constant load distribution) pushover methods are observed by Gupta and Kunnath (2000), to give poor ISD_{max} predictions in the case of tall structures where higher mode effects are important. For the purposes of this study, it must be proven that the adaptive pushover analysis technique of Antoniou (2002), is equally capable of predicting the interstorey drift response of frame structures.

7.1.2 Feasibility Study

A feasibility study is carried out to assess the capabilities and limitations of the non-linear adaptive pushover method (APO) in predicting building damage, when the parameter of maximum inter-storey drift ratio (ISD_{max}) is used as damage indicator. In order to assess the performance of the adaptive pushover method for a variety of seismic design conditions and failure modes, the method is applied in the analysis of four regular 8-storey bare frames designed for different pga values and ductilities according to Eurocode 8 (Fardis 1994), and a non-seismically designed irregular 4-storey bare frame, tested pseudo-dynamically as part of the ICONS European Network Program (Pinto et al. 1999).

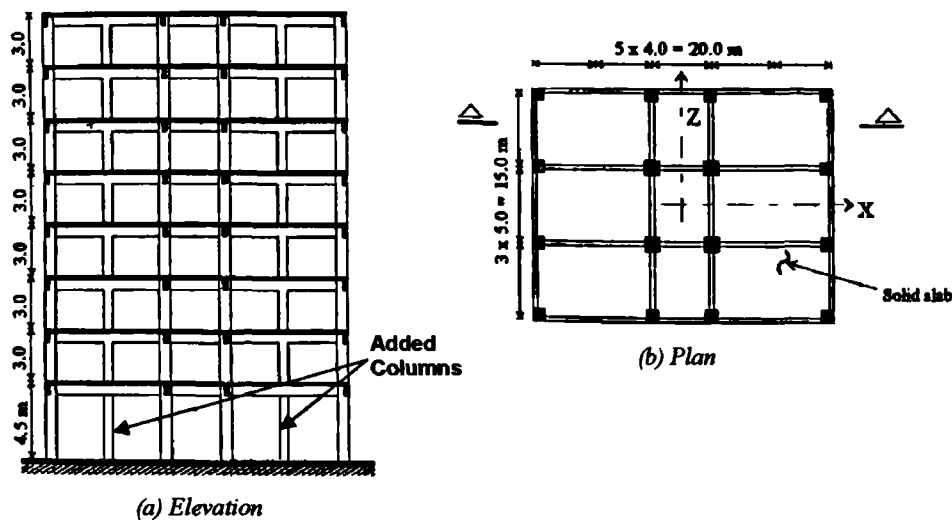


Figure 7. 3: Configuration of the regularised 8-storey frame designed by Fardis (1994).

The 8-storey frames, originally designed to be irregular in elevation by Fardis (1994), are here regularised through the addition of two columns at the ground storey (see Figure 7.3). Although the ground storey is of greater height than those above it, the extra strength provided by the added columns is sufficient to avoid the formation of a soft-storey at ground level and ensure a ductile mode of failure. The peak ground acceleration values considered in the design of the 8-storey frames are 0.15g and 0.3g, respectively corresponding to the low and high acceleration notation herein. The frames are designed for a live load of 2kN/m² and are assumed to be founded on firm soil, Type B in EC8. Equal configurations in plan and elevation are maintained Concrete of characteristic cylinder strength

25 N/mm² and steel with characteristic yield strength of 500N/mm², are adopted in all frames. Details of the member cross-sections are reported in Fardis (1994). The detailing provisions and force reduction factors in Eurocode 8 (CEN 1996), for low, medium and high ductility class structures are considered in the design of the 4 frames. Within this study, the 8-storey frames are referred to according to their design specifications, for example LALD refers to the frame designed for the low acceleration and low ductility levels whilst HAMD corresponds to the high acceleration, medium ductility case.

The ICONS frame (see Figure 3.6) is a 4 storey, 3 bay structure designed to resist gravity loads and a nominal lateral load of approximately 8% of its weight (Carvalho et al. 1999). The frame is representative of the construction practice used in Mediterranean countries in the 1950s. It is irregular in height and plan, reinforced with smooth steel bars and is not provided with seismic detailing. Further details of the design are presented in Section 3.4.2.2.

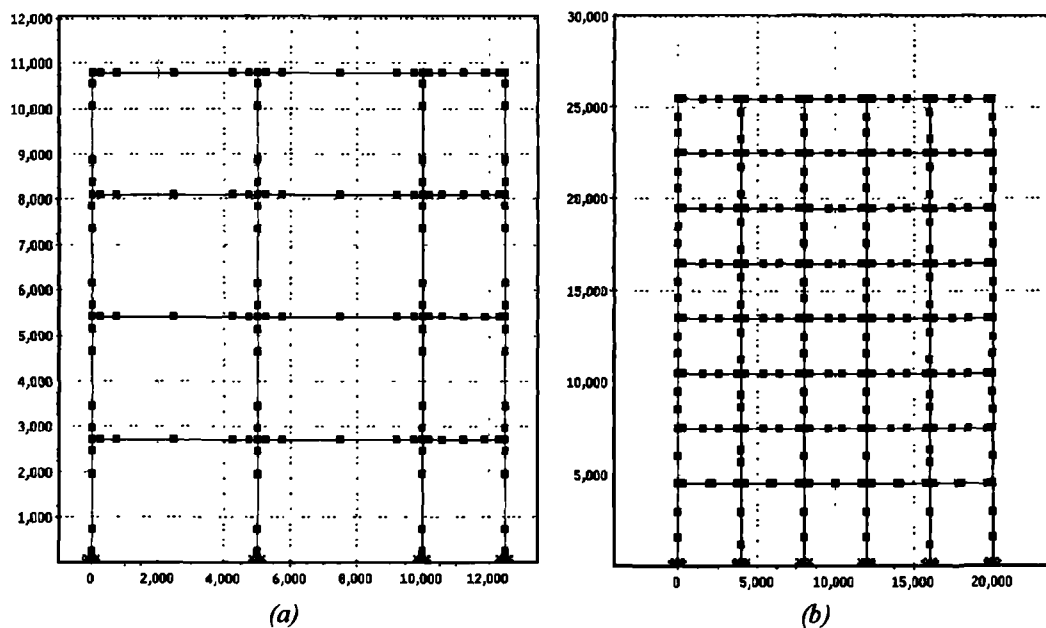


Figure 7. 4: Element and node pattern of the bare frame finite element modes used within INDYAS (Elnashai et al. 2000) for the APO feasibility analyses: (a) ICONS frame (Pinto et al. 1999), (b) 8-storey MRFs designed to EC8 (Fardis 1994).

The structures are modelled in finite elements and all analyses are carried out using the inelastic dynamic analysis program INDYAS (Elnashai et al. 2000). A single frame is used for modelling the ICONS frame. The 8-storey structures are

instead modelled via four frames, linked at the beam column connections by joints like those described in Section 6.2.2 for the designed 3-storey infilled frame model. These joints are representative of the slab behaviour. They have zero length orthogonal to the loading direction but show infinite stiffness in the direction normal to it. The gravity loads are modelled as lumped masses at the beam-column connections. The node and element patterns for the frames are presented in Figure 7.4. The structural members are sub-divided into a number of elements, with lengths which are determined considering the probable spread of plasticity and the member Gauss point locations, as for the infilled frame model described in Section 6.2.2.

Frame	LALD					LAMd				
Analysis	APO	Tri-PO	Uni-PO	APO1	Dyn-PO	APO	Tri-PO	Uni-PO	APO1	Dyn-PO
	Rating					Rating				
BS ⁽¹⁾	2	1	4	3	-	3	2	4	1	-
Top Drift	3	1	4	2	-	3	2	4	1	-
ISD _{max}	3	5	1	4	2	1	5	2	4	3
Storey ⁽²⁾	4	3	4	2	1	2	3	2	4	1
	Average Error (%)					Average Error (%)				
BS	11.24	7.00	16.83	7.19	-	9.82	6.14	13.02	6.53	-
ISD _{max}	5.72	9.35	3.19	8.07	6.30	3.03	5.73	3.27	4.67	4.22
Frame	HAMd					HAHD				
Analysis	APO	Tri-PO	Uni-PO	APO1	Dyn-PO	APO	Tri-PO	Uni-PO	APO1	Dyn-PO
	Rating					Rating				
BS ⁽¹⁾	1	1	2	2	-	2	3	4	1	-
Top Drift	1	1	3	2	-	2	2	3	1	-
ISD _{max}	4	3	5	1	2	4	5	3	2	1
Storey ⁽²⁾	2	1	2	1	1	2	3	4	2	1
	Average Error (%)					Average Error (%)				
BS	10.20	10.47	14.82	11.73	-	16.39	17.60	22.87	9.64	-
ISD _{max}	7.03	5.45	9.40	4.97	6.10	6.04	7.19	6.30	5.86	5.61

⁽¹⁾BS = Base shear, ⁽²⁾Ability to predict the correct storey of maximum inter-storey drift occurrence.

Table 7. 1: Summary of the rating and average errors in the prediction of the dynamic response of the 8-storey frames designed by Fardis (1994). Rating values 1 and 5 are assigned to the pushover analysis methods giving the best and worst response parameter estimates, respectively.

As stated in Section 3.4.2.2, the ICONS frame is tested under two earthquake records that are artificially generated to represent earthquakes with 475 and 975 year return periods (Carvalho et al. 1999). In the time-history analyses, a 15 second interval is provided between applications of the two records, to allow for complete attenuation of structural motion. Both records have durations of 15 seconds, but the 975 year return period (95YRP) earthquake record is truncated at 7.5 seconds, corresponding to the end of the experimental testing. The 475YRP record is not used in the analysis of the 8-storey frames, which are only tested under the full 975YRP record. The Eigen-value analysis and time-history response of the ICONS frame is verified with respect to the experimental measurements and a close correlation is observed (Appendix F). This demonstrates the modelling accuracy and strength of the INDYAS program.

The results of the adaptive pushover analysis (APO) are compared to predictions of ISD_{max} made by conventional push-over analyses with triangular (Tri-PO) and uniform rectangular (Uni-PO) lateral force distributions. These are also compared to the ISD_{max} values measured over the entire record (DYN), and up to the time of occurrence of the maximum drift in the dynamic analyses (i.e. corresponding to the points forming the incremental dynamic analysis pushover curve, Dyn-PO). In Figure 7.5 it is observed that a significant difference in the capacity curves results from the different choices made for the analyses. The capacity curve predicted by the adaptive pushover procedure typically lies between the Tri-PO and Uni-PO curves. The difference in the curves is observed in Figure 7.5 to be significant, and comparable to that obtained when extreme combinations of material properties, (as defined in Table F.2 in Appendix F), are used.

The results of the pushover analyses are compared to the dynamic time-history analysis predictions of base-shear, total drift and inter-storey drift. The analysis results are presented in full in Appendix F, and are summarised for the two frame sets in Table 7.1 and 7.2. Within these tables, ratings are given to the different methods according to their relative ability in predicting the response parameters and the storey of occurrence of maximum damage. The average error in base shear and ISD_{max} prediction, compared to the dynamic analysis results and experimental observations, are also presented in these tables. The analysis method

that yields the best estimate of structural damage, is that associated with the smallest combined error in the prediction of all three response parameters. Furthermore, similarity in the displaced shape of the structure with that seen in the dynamic analysis, must be observed. For example, in Table 7.2, Uni-PO gives the best estimate of the ISD_{max} values for the ICONS frame in numerical terms. However, the structure displaced shapes observed in the Uni-PO and dynamic analyses are completely different, resulting in the ISD_{max} value being predicted to occur at the wrong storey. The coincidence of the ISD_{max} values predicted by the two analysis methods is therefore not based on a systematic replication by Uni-PO of the dynamic results, but can almost be attributed to chance.

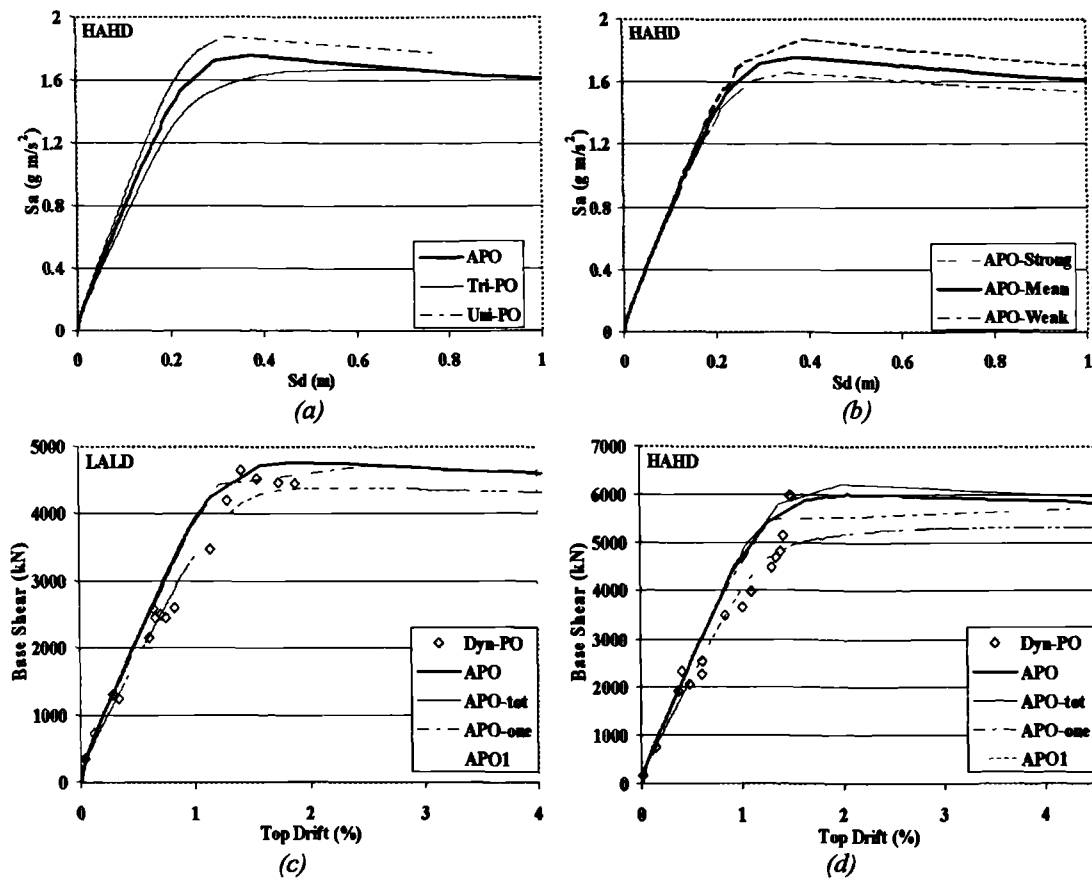


Figure 7. 5: Graphs of the pushover curves (expressed in SASD space) obtained for the 8-storey frames designed by Fardis (1994) with different assumptions: (a) Comparison of APO with conventional pushover analyses for HAHD, (b) APO curves for HAHD obtained using extreme material parameter value combinations (see Table F.2 in Appendix F), (c) and (d) Comparison, with incremental dynamic analysis, of the APO curves obtained for LALD and HAHD using different assumptions for the force profile updating.

From simple consideration of the derivation method of the pushover curves, it is expected that Dyn-PO will yield the smallest response prediction errors. However, the non-coincidence of the maximum total drift and maximum inter-storey drift, causes discrepancies in the displaced shape compared to the full time-history analyses (e.g. Table 7.1). In the case of the 8-storey frames, this often results in APO yielding better predictions of ISD_{max} than Dyn-PO. None of the methods used is observed to systematically under or over-predict either ISD_{max} or the maximum base shear for these frames. However, all methods show an increase in ISD_{max} prediction error with increasing non-linearity. The results obtained for the LALD frame suggest that Tri-PO provides the optimum analysis tool. However, for all other acceleration and ductility level design combinations, the analytical tool of preference is APO, as this gives the best combined prediction of base shear, top drift, maximum inter-storey drift and displaced structure profile.

Frame	ICONS 475-5s				ICONS 475				ICONS 475+(975-7.5s)			
Analysis	APO	Tri-PO	Uni-PO	DYN	APO	Tri-PO	Uni-PO	DYN	APO	Tri-PO	Uni-PO	DYN
	Rating				Rating				Rating			
BS ⁽¹⁾	1	1	2	-	1	2	1	-	1	2	3	-
Top Drift	1	1	2	-	2	3	1	-	2	1	3	-
ISD_{max}	2	3	1	-	3	2	1	-	3	2	1	-
Storey ⁽²⁾	3	3	3	-	2	1	1	-	2	1	1	-
	Average Error wrt Dynamic Analysis (%)				Average Error wrt Dynamic Analysis (%)				Average Error wrt Dynamic Analysis (%)			
BS	17.46	14.68	22.36	N/A	17.46	24.49	19.00	N/A	17.46	23.21	40.71	N/A
ISD_{max}	17.63	22.00	16.82	N/A	17.63	20.77	15.15	N/A	17.63	15.48	9.86	N/A
	Average Error wrt Experiment (%)				Average Error wrt Experiment (%)				Average Error wrt Experiment (%)			
BS	-	-	-	-	-8.56	5.36	-13.13	-15.24	-17.59	-6.75	-20.49	-6.03
ISD_{max}	-	-	-	-	5.86	10.74	5.24	14.27	40.36	44.02	37.53	50.52

⁽¹⁾BS = Base shear, ⁽²⁾Ability to predict the correct storey of maximum inter-storey drift occurrence.

Table 7. 2: Summary of the rating and average errors in prediction of the pushover procedures with respect to the results of dynamic analysis (DYN) and experimental observations made on the ICONS frame (Pinto et al. 1999), at three times during the analysis. Rating values 1 and 4 are assigned to the pushover analysis methods giving the best and worst response parameter estimates, respectively.

During experimental testing, the ICONS frame was observed to fail via the formation of a soft storey mechanism at the third floor. This is a highly irregular failure mode and extremely difficult for a conventional pushover technique to predict and replicate. Comparisons between experimental and analytical observations are made at 5 seconds from the start, and at the end of the two applied records. The dynamic time-history analysis is the only technique that correctly identifies the weak storey. A reasonable, tendentiously conservative, prediction of the experimentally recorded base shear is given by the dynamic analysis. However, errors in the predicted ISD_{max} are very large and reach values of 50% at severe states of structural damage. When the ICONS frame pushover analyses are compared to the dynamic analysis results, there is no observable trend as to their over- or under-prediction of the base shear. A general under-prediction is observed in the case of ISD_{max} , which increases in magnitude with increasing non-linearity. In Figures F.9 and F.10 (Appendix F), APO is identified as the method giving the optimum combined prediction of base shear and inter-storey drift. Figure F.11 (Appendix F) compares the displaced structure profile predicted by the analysis methods. In the figure, Tri-PO is seen to give the best correlation with the experimental results and the dynamic analysis displaced profiles at low levels of deformation. At higher deformations, Uni-PO is instead observed to provide better profile predictions. The displaced APO profile gives the best compromise at low and high deformations, and is therefore preferred for the evaluation of a wide range of damage states.

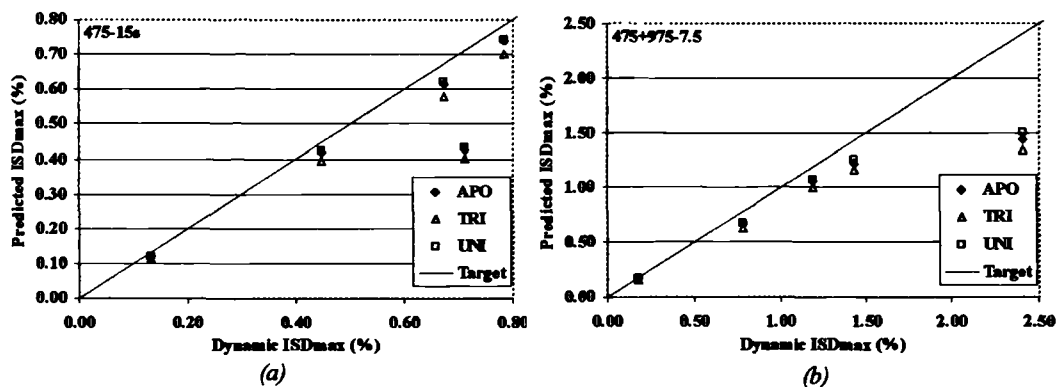


Figure 7. 6: Comparison of the maximum base shear and inter-storey drift predicted by inelastic dynamic analysis for the ICONS frame under the increasingly scaled 475YRP record and (475 + 975YRP) record sequence, with that obtained using different pushover methods.

It is concluded that of the pushover techniques, APO gives the optimum combination of accuracy for top-drift, base-shear and inter-storey drift prediction. The method is seen to perform well in the case of tall, regular structures and gives comparable results to dynamic analyses in the case of an irregular structure. However, it is evident from the feasibility analysis that APO cannot be declared the resounding “winner”, as the analyses also demonstrate that the results of APO are not hugely better than those obtained using conventional push-over techniques.

In carrying out adaptive pushover analyses, decisions must be taken regarding the frequency of updating of the force distribution, and whether this is to be done via total or incremental methods. In order to assess the optimum force combination method, the effect of using the different options in the APO analyses, are investigated for the case of the 8-storey frames. APO and APO1 refer to incremental force updating, where all modes of interest, or the first mode of vibration only, are considered in the force distribution updating at each incremental step, respectively. APO-tot refers to adaptive pushover analyses carried out using the total updating option for the force distribution. APO-one instead refers to analyses where a single force distribution updating is carried out at the first load step. Figure 7.5 shows the APO results are similar for the incremental and total updating analysis options. However, if only one mode shape is used for the force profile modification, a discrepancy in the non-linear range is observed between the results. This is expected as higher modes will gain importance after structural yielding. In the case where force distribution shape updating is carried out only in the first step of the analysis, a significant change in the curve shape and predicted capacity is observed. In this case, the pushover curve is seen to closely approximate that obtained using the triangular force distribution (Figure F.2, Appendix F). It is concluded that the incremental combination method should be used herein, for the adaptive pushover analyses of the structure population. This choice gives increased stability and allows all modes of interest to be taken into account in the force updating at each increment.

7.2 The Assessment - Evaluation of the Models

Capacity Spectrum Methods, (CSM), are proposed in the literature for the seismic assessment of building performance using the results of static pushover analyses. CSM was first proposed by Freeman et al. (1975) and later modified by Freeman (1992), Chopra and Goel (1999), Reinhorn (1997) and Fajfar (1999) amongst others. It is incorporated in codes of practice such as ATC-40 (ATC 1996) and FEMA 273 (ATC 1997), and is seen by many to be the future in seismic assessment of buildings, as it is of simple visual interpretation and involves elements of both acceleration and displacement response. A direct comparison is made between earthquake spectra representing the seismic demand, and a transformed force-displacement curve representing the seismic capacity of the structure. Both demand and capacity curves are plotted in spectral displacement - spectral acceleration (SASD) space, with the “performance point” (*PP*), defining the maximum response of the structure under the given earthquake, being determined from the location of their intersection.

Transformation of the pushover curve to SASD space is done using the modal participation factors and effective modal weight ratios, determined from the fundamental mode of the structure, using the following equations:

$$S_a = \frac{V_b}{M^*} ; \quad S_d = \frac{u_N}{\Gamma \phi_N} ; \quad \Gamma = \frac{\sum_{i=1}^N m_i \phi_i}{\sum_{i=1}^N m_i \phi_i^2} ; \quad M^* = \frac{\left(\sum_{i=1}^N m_i \phi_i \right)^2}{\sum_{i=1}^N m_i \phi_i^2} \quad [7.4]$$

In Equation 7.4, N is the total number of floors, u_N is the top floor displacement and V_b is the base shear force. ϕ_i is the i^{th} floor element of the fundamental mode shape (Φ_1). m_i is the lumped mass at the i^{th} floor level and M^* is the effective modal mass for the fundamental mode of vibration.

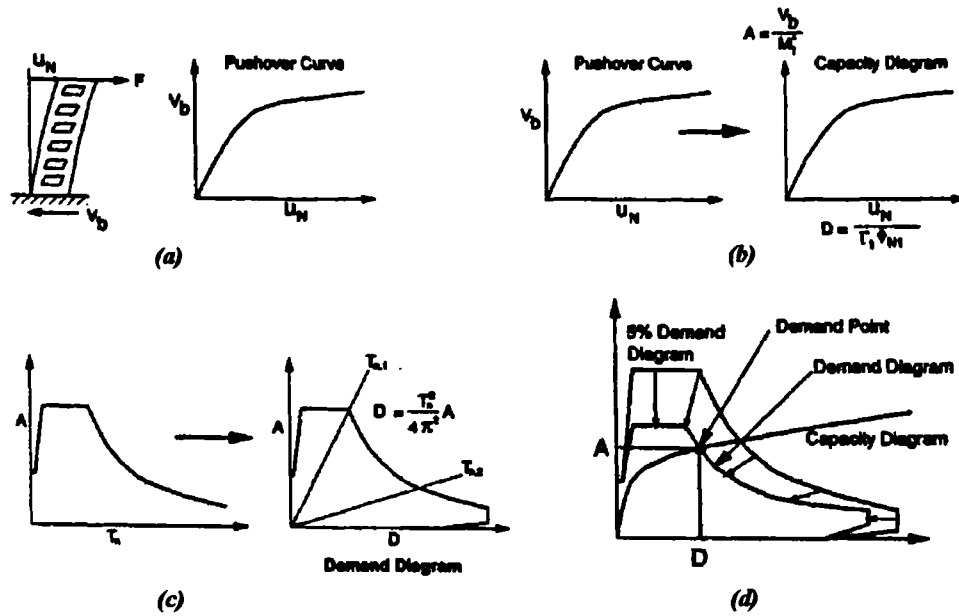


Figure 7. 7: Illustration of the steps of the ATC-40 (ATC 1996) capacity spectrum method taken from Chopra and Goel (1999): (a) development of pushover curve, (b) conversion of pushover curve to capacity diagram, (c) conversion of elastic response spectrum to SASD space, (d) determination of the performance point ("demand point") and displacement demand.

For elastic systems, the point of intersection between the capacity curve and elastic demand spectrum directly defines the performance point. In the case of inelastic systems, identification of this point is iterative and the evolution in damping of the structure with increasing ductility (i.e. damage) must be accounted for. Several 'equivalently damped' elastic spectra are required for comparison with the capacity curve. The performance point is defined by the crossing point of the curves at which the ductility of the structure is consistent with the damping value of the spectrum. The main steps of the ATC-40 CSM procedure (ATC 1996) are illustrated in Figure 7.7, as an example of a capacity spectrum method adopting equivalently damped spectra. The use of heavily damped spectra in CSM is, however, seen to lead to large errors in the predicted displacement response of structures. Chopra and Goel (1999) assessed the ATC-40 (ATC 1996) CSM and observed underestimation of structural deformations of up to 50%, (compared to those obtained using inelastic spectra), over wide ranges of periods and ductility factors. No physical principle justifies the existence of a stable relationship between the hysteretic energy dissipation of the maximum excursion and equivalent viscous damping, especially for highly inelastic systems (Krawinkler and Nassar 1990). Only approximate relationships exist to relate damping with ductility (e.g. Priestley et al. 1996), which differ for displacement and acceleration

spectra (e.g. Borzi et al. 1998). Furthermore, the period associated with the intersection of the capacity curve and highly damped spectrum may have little to do with the dynamic response of the inelastic system.

Reinhorn (1997) and Fajfar (1999) adopt modified CSM procedures, which use inelastic demand spectra instead of equivalently damped elastic spectra. Within these methods, the pushover curve is idealised to a bi-linear curve, which is compared to inelastic spectra deriving from the analysis of single degree of freedom (SDOF) systems, showing elastic perfectly-plastic behaviour. The performance point is defined by the point of intersection between the capacity and demand curves at which both structure and spectrum are associated with the same ductility value. The method of assessment is graphical, with the performance point determined through trial and error. A good agreement between the displacement and force response predicted by this capacity assessment method and dynamic time-history analysis is reported by Reinhorn (1997), for both single and multiple degree of freedom systems. Although the procedure results in an improved representation of the demand, the superposition of bi-linear capacity curves showing strain-hardening, with spectra determined from SDOF systems characterised by elastic-perfectly plastic behaviour, is not strictly correct. This inconsistency may lead to errors in the performance point prediction, especially at high levels of ductility.

The implicit assumption of a fundamental mode of response for the system, is the main impediment to the adoption of any of the aforementioned capacity spectrum methods, in the assessment of structural damage using the results of adaptive pushover analyses (APO). In the case of APO, the base shear-top displacement curve must be transformed into SASD space in a way that includes the combined effect of multiple response modes. A single transformation cannot be applied to the adaptive pushover curve as the relative contribution of each mode changes with each applied load increment. Hence, an approximate method for the transformation is proposed here, where the instantaneous displaced shape and storey forces at each increment step of the APO, are used to transform the force displacement curves into SASD space. The same expressions as for the SDOF transformation are adopted (Equations 7.4), with the current displaced shape of

the structure normalised to the top displacement (Φ_n), replacing the fundamental mode shape (Φ_1). Therefore, ϕ_i is now the component of Φ_n corresponding to the i^{th} storey. Consequently, u_N and V_b are the top displacement and base shear at the current load increment. The reasoning behind this transformation method is that the force distribution and resulting displacement distribution implicitly incorporate the modal combinations. This assumption may not be theoretically justified, but is observed to provide reasonable assessment results (see Figure 7.11). The proposed transformation algorithm was added to the program designed to run multiple INDYAS adaptive pushover analyses by Antoniou (2002), for the specific purposes of this study. Values of Γ and M^* are, therefore, output for each load increment as part of the pushover analysis results.

A further drawback of the capacity assessment method is that it is an iterative, graphical method of assessment. Due to the sheer number of assessments necessary for the simulation of damage statistics in vulnerability curve generation, the adoption of such a procedure is impractical. Below, a modified approach to capacity spectrum assessment using inelastic earthquake spectra is adopted in the creation of a program for the seismic evaluation of building populations.

7.3 The Capacity Assessment Program (CAsP)

The Capacity Assessment Program (CAsP) is created to assess structural performance under earthquakes of increasing severity, using the results of adaptive pushover analyses. CAsP is coded in Visual Basic 5.0 (Microsoft 1997). A brief description of the subroutines constituting the program is provided in Appendix G, together with a flowchart of their calling sequence (Figure G.1). The program code used, is specific to the configuration of the 3-storey infilled frames designed in Section 6.2 and the output files of the INDYAS analysis program (Elnashai et al. 2000), but can be easily altered to any building configuration and input/output file format.

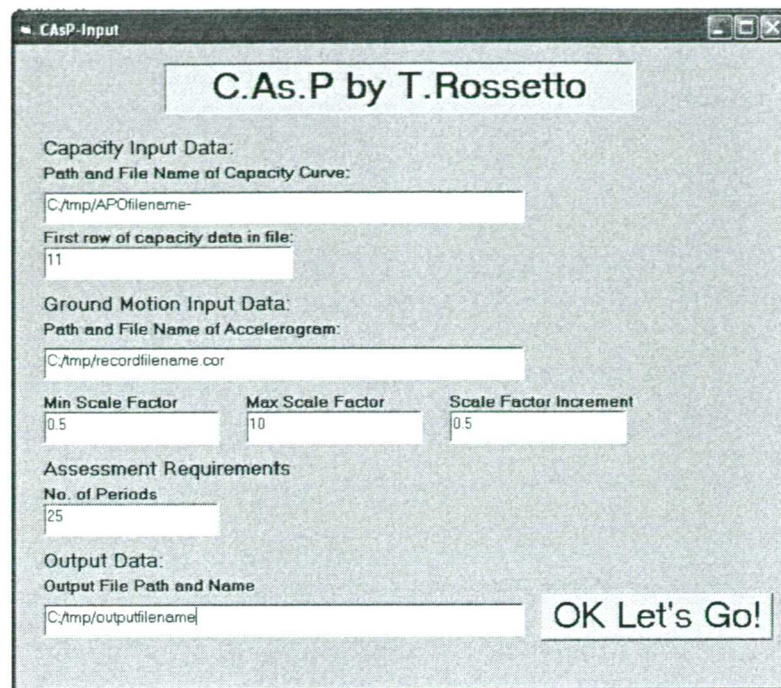


Figure 7. 8: Picture of the user interface of the Capacity Assessment Program (CAsP).

CAsP adopts a different approach to the visualisation of the Capacity Spectrum Method (CSM) of assessment described previously. This approach is summarised in Figure 7.10, and is seen both to aid in the automation of the assessment procedure and increase its efficiency. On commencing the assessment, details of the pushover analysis file, accelerogram file and range of record scaling factors is input via the program user interface, shown in Figure 7.8. The program reads the base shear (BS), storey drifts, Γ and M^* response values, at each load increment of the APO analysis. A filtering module is used to remove readings that do not correspond to points of equilibrium, (i.e. values of the parameters resulting from intermediate iteration steps in the analysis). The structure capacity curve is constructed from the input APO analysis results and the procedure set out in Section 7.2, is used to transform the force-displacement curve to spectral acceleration-spectral displacement space. The transformed pushover curve is then idealised as a multi-linear curve. In the evaluation of populations of building models, a large number of capacity curves of differing shape must be dealt with, (750 in the case of the infilled frame population assessed in this study). CAsP is designed to automatically detect the location of yield and the ultimate point locations that best describe the curve shape, according to the behavioural model

and yield criteria specified by the user (see Section 7.3.1). A series of points are selected along the idealised curve, at which capacity to demand checks will be carried out (the Capacity – Demand Checking Points, CDCPs). The idealised curve shape up to the chosen CDCP locations defines the elastic period, ductility and non-linear response curve characteristics of a corresponding set of equivalent single degree of freedom systems (SDOF). The SDOFs are analysed under the applied scaled earthquake records within an inelastic dynamic analysis module (discussed in Section 7.3.2). A series of inelastic spectral displacement and acceleration demand values are thus determined. These demand values may be visualised as corresponding to the structure capacity, along the radii of the SASD plot that intersect the CDCPs. A single inelastic demand curve can be drawn and the Performance Point is defined directly from the crossing of this curve with the capacity curve (Figure 7.9).

The effective periods of the equivalent SDOFs correspond to the radii of the plot at the CDCPs, and can be determined from their spectral coordinates as $T = 2\pi\sqrt{S_d/S_a}$. Elastic demand-to-capacity checks are carried out at the yield period (T_l). The number of post-yield points (N) at which the inelastic demand-to-capacity checks are carried out, is defined by the user. The location of the latter points along the strain hardening curve is determined from consideration of an equal period spacing between the yield period, (T_l), and that corresponding to the last spectral coordinates of the pushover curve, (T_n). Due to the rapid increase in inter-storey drift at large ductilities, a large number of assessment points should be selected when the structure is assessed under very large earthquakes, (25 post-yield assessment points are seen to be sufficient for the infilled frames analysed in this study). The damage state of the structure is identified from consideration of the maximum inter-storey drift, recorded during the loading process up to the Performance Point occurrence. The spectral displacement value evaluated from the scaled record at the structure elastic period ($S_{d5\%}(T_l)$), defines the seismic demand. Hence, an array of (T_l , $S_{d5\%}(T_l)$, ISD_{max}) parameter values, corresponding to different material combinations, is output by the program for use in the construction of response surfaces (Section 7.4.1). Two additional text files, containing the yield and hardening point characteristics used in the idealisation of

the pushover curve, and the capacity and demand point characteristics for each of the CDCPs, are also output by CAsP.

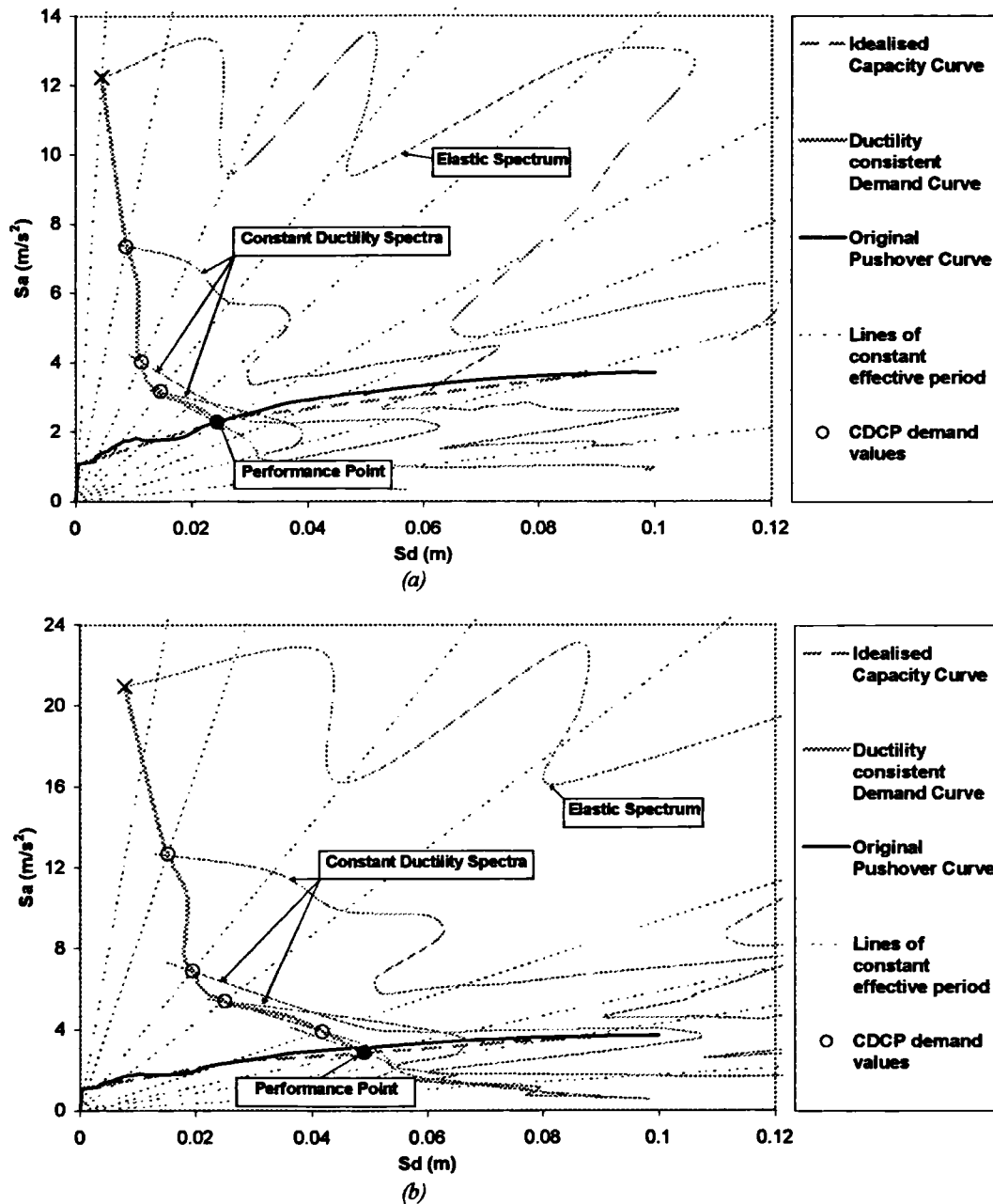


Figure 7. 9: Illustration of the performance point solution method and ductility constant demand curve used by CAsP for the 3 storey infilled frame designed in Section 6.2.1, with material combination 1, assessed under the Killini earthquake record, (Greece 1988, reference 436ya), scaled by a factor of (a) 3.5 and (b) 6.0.

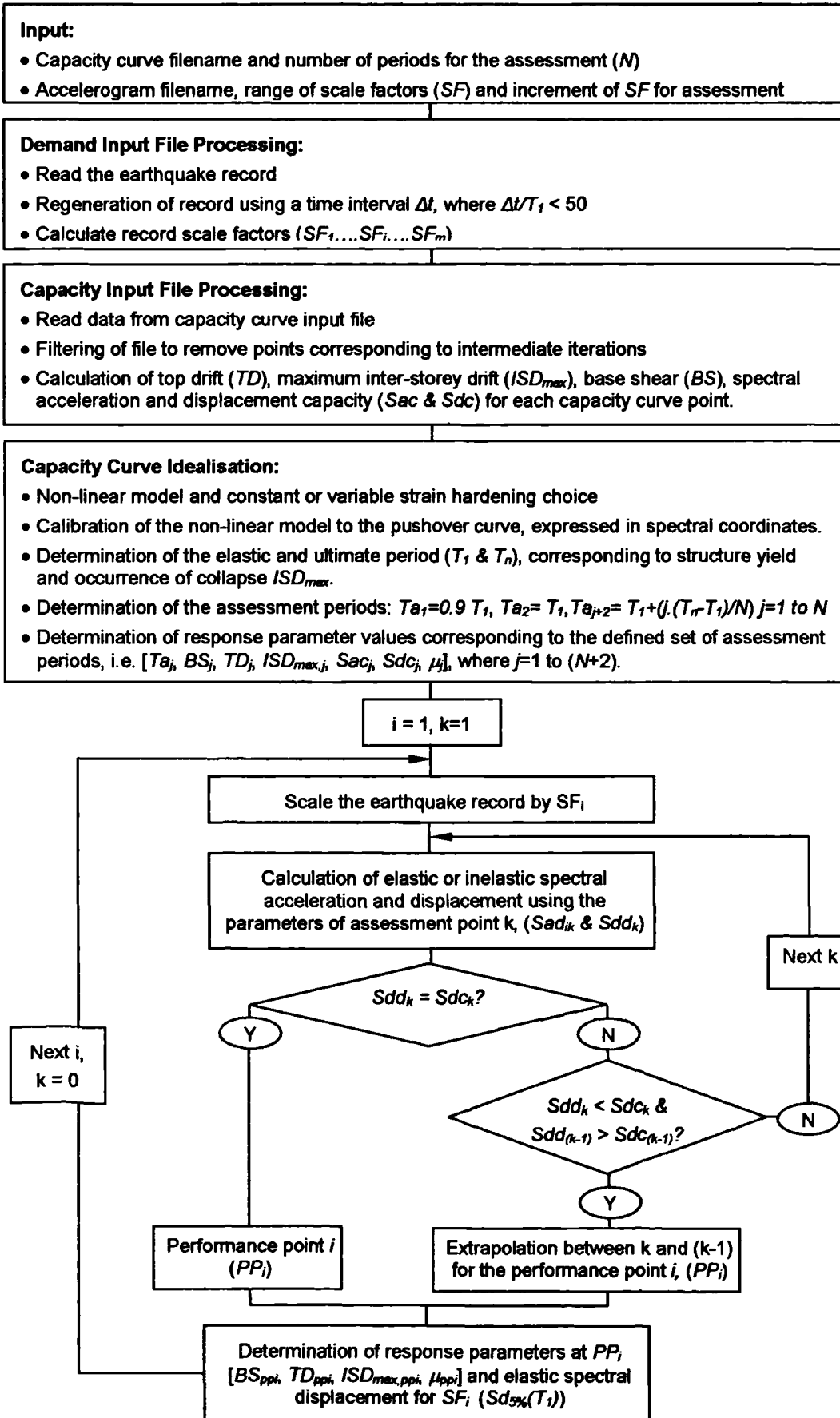


Figure 7. 10: Flowchart of the capacity spectrum assessment procedure adopted by the CASP Program in the evaluation of adaptive pushover curves under earthquakes of increasing severity.

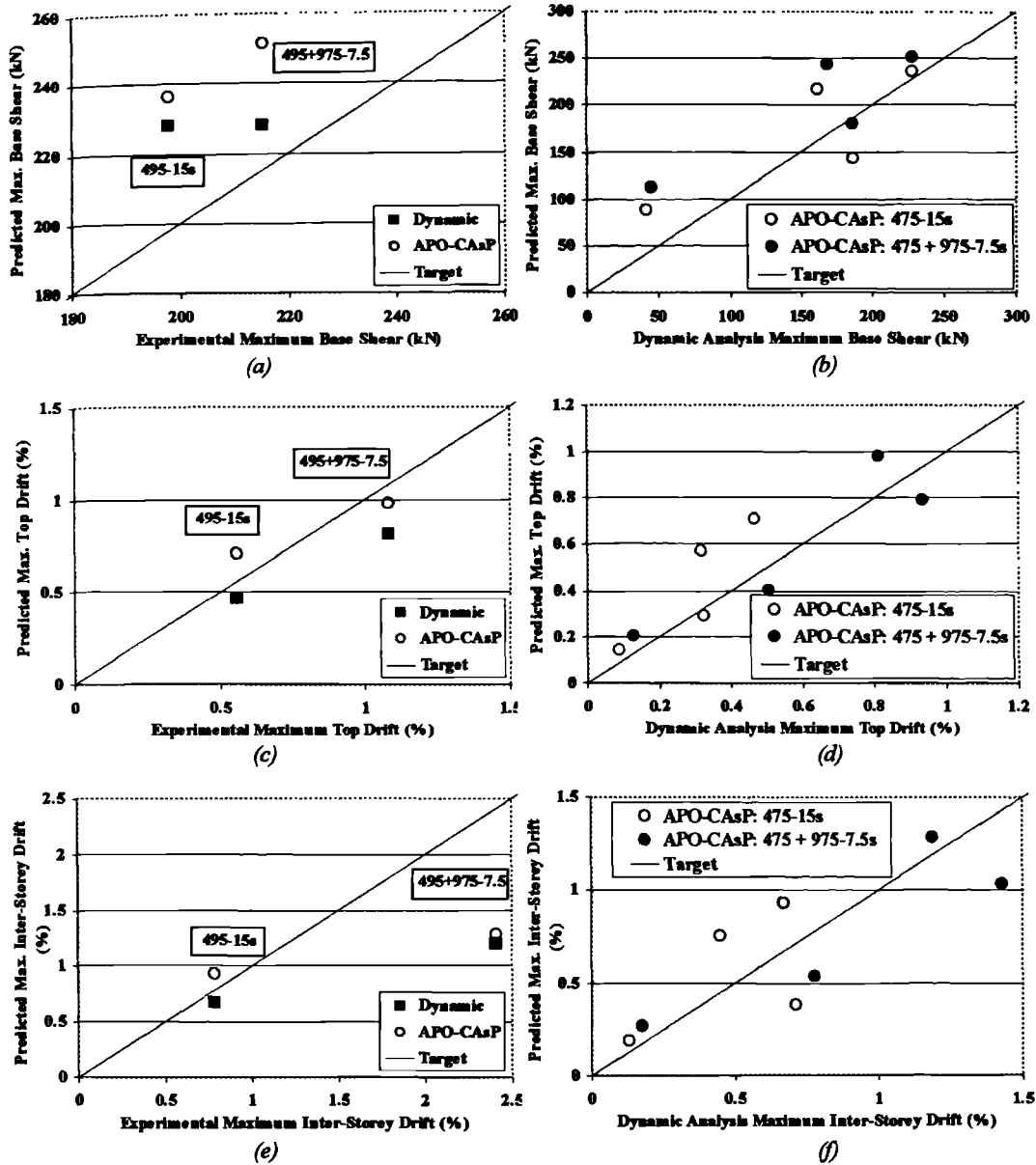


Figure 7. 11: Comparison of the maximum response values predicted by time-history analysis and from the capacity spectrum method using inelastic demand spectra and adaptive pushover curves idealised via the tri-linear model (Figure 7.12c), for the ICONS frame (Pinto et al. 1999); (a), (c) and (e) comparison of the analytical predictions with experimentally observed values; (b), (d) and (f) comparison of the analysis predictions for the 475YRP record and (475 + 975YRP)sequence of records (Carvalho et al. 1999) scaled to different intensities.

All adaptive pushover files relating to the specified earthquake should be assigned the same filename with a sequential integer suffix added to distinguish the system material combination (e.g. *filename-1*, *filename-2*, ..., *filename- nn*). This allows the program to read each file in sequence and as the curve shape idealisation is automated, a rapid evaluation of the population performance under each earthquake scale factor is achieved. In case study presented, CAsP automatically repeats the assessment procedure for the 25 APO curves defining the population

response under the selected ground motion record. Each pushover curve is assessed approximately twenty times under the increasingly scaled accelerogram, until an inter-storey drift value is obtained which exceeds that of the most severe damage state of interest. The program is then run for a new earthquake and corresponding set of APO analysis results.

Figure 7.11, shows the maximum response parameters predicted by CAsP for the ICONS bare frame (Pinto et al. 1999), used in the APO feasibility study (Section 7.1.2). In the figure, these values are compared to the results of dynamic time-history analyses. The TLM model with variable strain hardening (see Section 7.3.1), is used to describe the non-linear behaviour of the equivalent SDOF systems. As stated in Section 7.1.2, the ICONS structure deforms in a highly irregular manner. Comparison with experimental observations shows that CAsP yields equal, if not better, estimates of the maximum top and inter-storey drift ratios, than dynamic time-history analysis. However, much more conservative base shear estimates are predicted by CAsP, compared to the latter analysis method. In Figure 7.11 (b), (d) and (f), the CAsP response parameter predictions for the ICONS structure, are also compared to the results of incremental dynamic time history analyses. No bias in the predicted response compared to the dynamic analyses is seen, and a satisfactory correlation between the analysis methods is observed.

CAsP is seen to provide the optimum tool for the accurate and efficient seismic assessment of structural populations. Through use of the proposed assessment approach, only a single inelastic dynamic analysis of the equivalent SDOF system is required at each considered period, compared to the series of analyses for different ductility values implied by a graphical solution. Consequently, a significantly lower computation time is achieved. Adaptive non-linear modelling capabilities for the pushover curve and SDOF inelastic behaviour are incorporated in the program for the accurate representation of different system responses. Consistent curve shapes are used to determine both the demand and capacity, unlike in previous studies (e.g. Reinhorn 1997 and Fajfar 1999). Several yield criteria can furthermore be defined, and tri-linear behavioural models adopted in the assessment. To the author's knowledge, the latter model has never been used

within a CSM assessment framework. It is seen to have a significant effect on the performance point predictions for infilled frames, when a plateau in the pushover response exists, which is caused by sudden infill panel failure. The effect of various modelling options on the performance prediction is discussed below.

7.3.1 Capacity Curve Modelling

The influence of different curve modelling assumptions is investigated, with the aim of determining the optimum choices for use in the capacity spectrum assessment. Within CAsP, three curve shapes can be used to model the pushover response of structures: elastic-perfectly plastic (EPP), linear strain hardening (EST) or tri-linear model (TLM). These are illustrated in Figure 7.12. The aim is to calibrate the parameters of the models such that they reproduce the pushover curve as closely as possible. Several options exist for definition of the global yield point and strain hardening slope in each case and are represented by the parameters *yopt*, *yopt1* and *var* in the program.

In the case of TLM, yield is always defined by the first deviation from linear response (*yopt* = 1). The point of the second change in slope (ultimate point) is found considering equivalence of energy concepts (*yopt1*=1), (e.g. Albanesi, Nuti and Vanzi 2000). Its location is selected such that the areas under the idealised and actual curves (in SASD space) are approximately equal, and the spectral acceleration value lies within 15% of the corresponding original pushover curve value. In the cases of EPP and EST, the yield point coordinates completely define the model behaviour. In the case of EST, the yield point can either be assigned by the user or determined from the first deviation from linearity of the curve (*yopt* = 1). The corresponding spectral acceleration value can be determined either from the corresponding curve coordinates (*yopt1* = 0), or from consideration of equal energies (*yopt1*=1). Alternative yield definitions can be used in the case of EPP, which are defined by the intersection of the initial stiffness with the horizontal line representing maximum strength (*yopt* = 2), and the intersection of the line passing through the point of first deviation from linear response with the latter horizontal (*yopt* = 3).

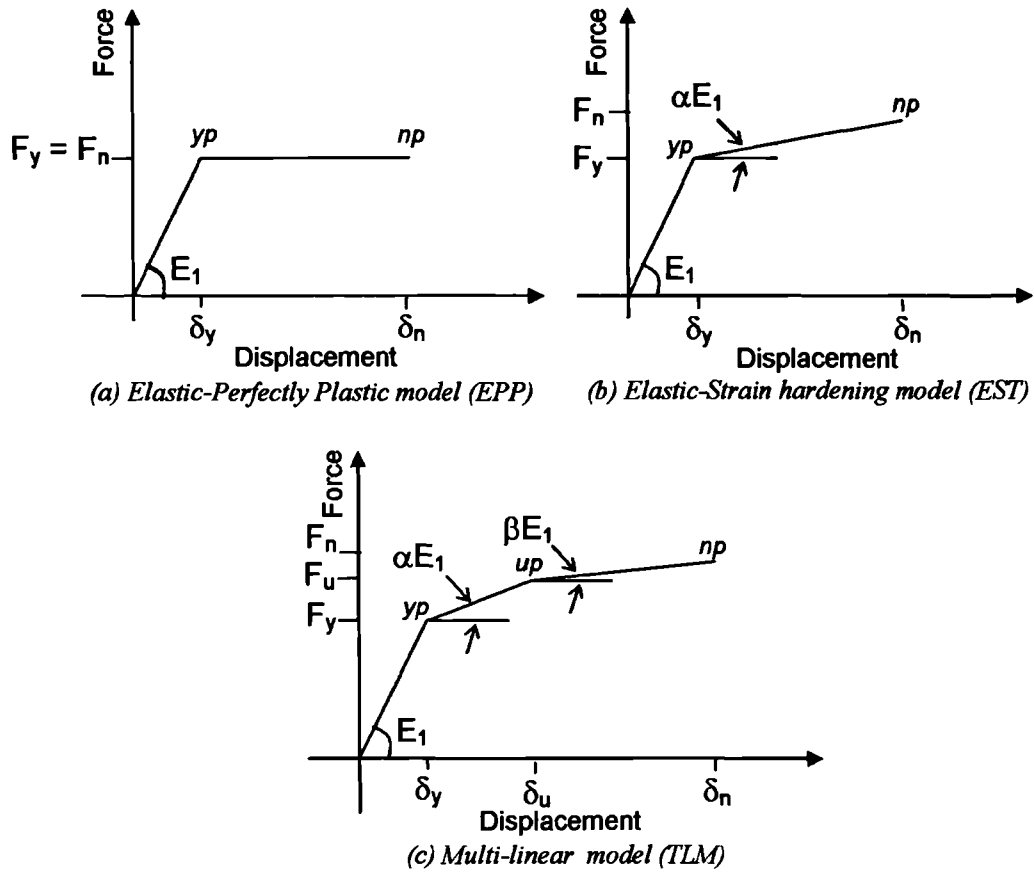


Figure 7. 12: The non-linear models for APO curve idealisation and SDOF inelastic behaviour characterisation. *yp*, *up* and *np* indicate the positions of the yield, ultimate and final points, respectively, as adopted by the CAsP program.

For the TLM and EST models, the option of considering the strain-hardening as either constant or variable, (*var* = 0 or 1, respectively), is given by CAsP. The constant strain hardening slopes are determined from the locations of the yield (*yp*), ultimate (*up*) and final (*np*) points, defined for the selected modelling criteria. In the variable slope case for EST, the strain hardening is defined by the location of the checked point (CDCP), relative to the yield point position. In the variable slope case for TLM, if the CDCP lies between the yield and ultimate points, bi-linear behaviour of the SDOF is assumed with the strain-hardening slope defined as per EPP. When the CDCP lies between the ultimate and final points, the first two slopes of the model are assumed constant (defined by the locations of *yp* and *up*), whilst the third slope is defined by the location of the CDCP relative to *up*. The deviation of the variable slope is, however, limited to a maximum of 50%

from the idealised constant strain hardening slopes. This is necessary in order to ensure stability of the solution and reduce the influence of local “bumps” in the curves. Use of the variable strain hardening option is seen to result in a closer match between the idealised non-linear behaviour of the SDOFs corresponding to the CDCP, and the shape of the capacity curve to that point (see Figure 7.13).

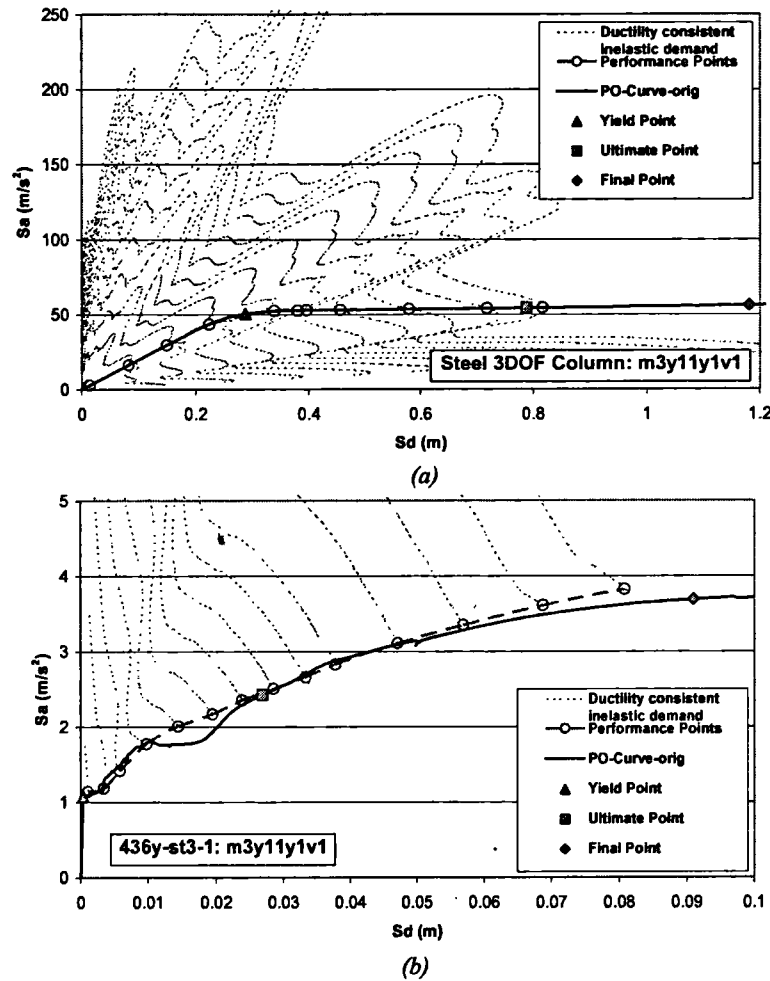


Figure 7. 13: Illustration of the performance point locations predicted by CAsP using the TLM model with variable strain hardening for the adaptive pushover curves of (a) the steel3DOF structure and (b) the 3 storey infilled frame with material combination 1, subjected to the increasingly scaled Killini earthquake record (Greece 1988, reference 436ya).

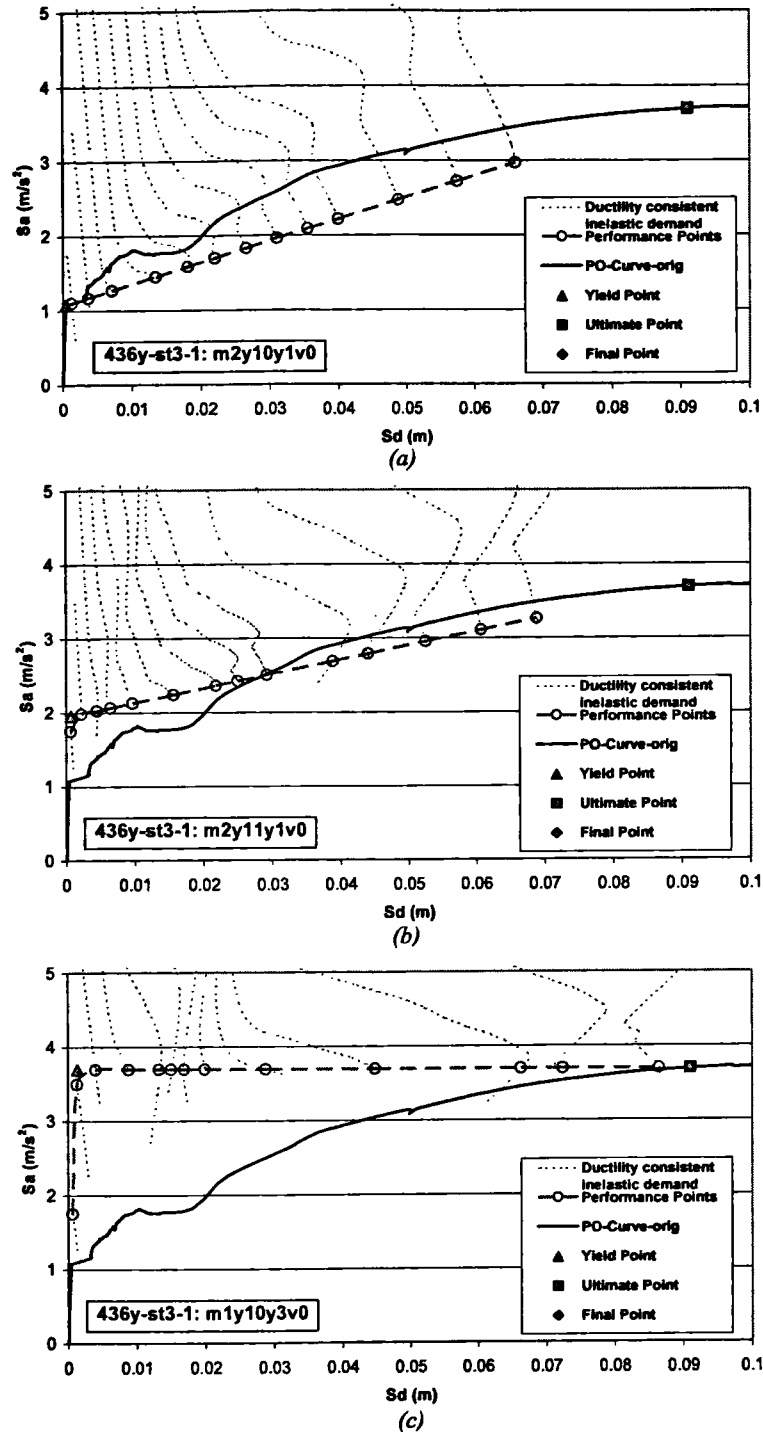


Figure 7. 14: Illustration of the performance points predicted by CASP for the 3 storey infilled frame subjected to the increasingly scaled Killini earthquake record (Greece 1988, reference 436ya), using different capacity curve idealisation models and yield criteria: (a) and (b) EST with constant strain hardening and yield point determined from the first deviation from linear response (y10y1v0) and using equal energy concepts (y11y1v0), respectively, (c) EPP with yield defined by the intersection of the maximum strength with a line passing through the point of first deviation from linear (y10y3v0).

The sensitivity of the assessment results to the choice of model, yield and strain-hardening criteria is investigated for the case of a 3 degree of freedom steel

column structure (that exhibits a smooth APO curve), and for the 3 storey infilled structure model with material combination 1 (Table 6.7). Both are analysed and assessed under increasing earthquake scale factors for the Kilini earthquake (Greece 1988, reference 436ya) record. The column structure has a square plan of 0.4m side, is 9m in height and sustains lumped vertical loads of 82.6kN, 82.6kN and 69.3kN at third intervals along its height. The steel material has yield strength of 413.6MPa and follows the STL1 non-linear behaviour (Figure 6.8). The variation in predicted maximum inter-storey drift ratio resulting from both these assessments is shown in Figure 7.15 and summarised in Tables 7.3 and 7.4, in terms of error of prediction with respect to the TLM model with variable strain hardening.

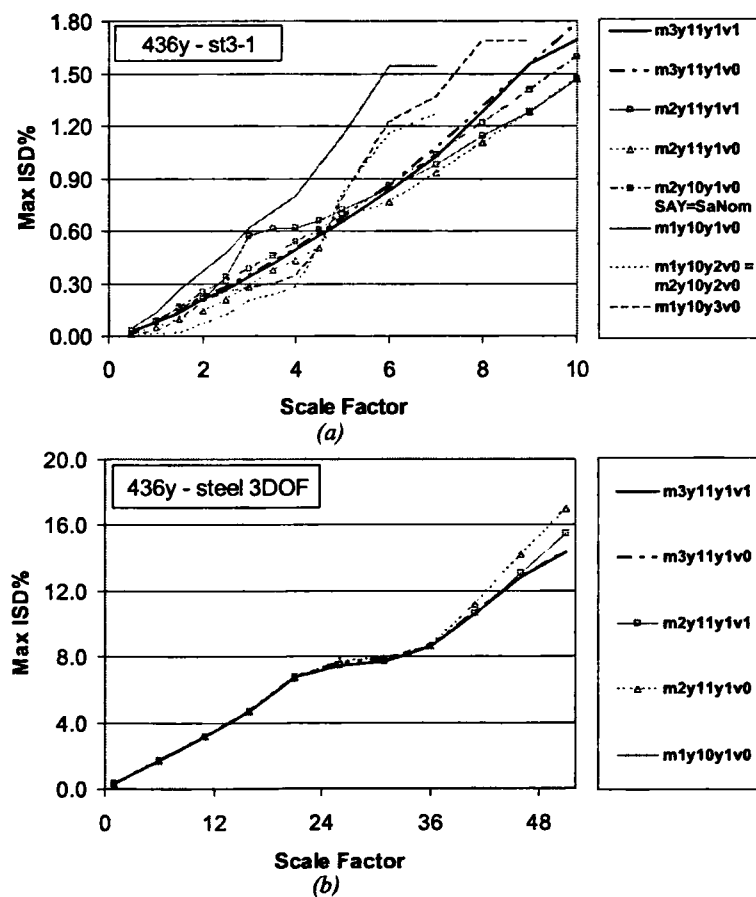


Figure 7. 15: The variation in maximum inter-storey drift ratio predicted by CAsP under increasing scale factors of the applied Killini earthquake record (Greece 1988, reference 436ya), using different capacity curve idealisations for (a) the 3DOF steel structure and (b) the 3 storey infilled frame with material combination 1. (n.b. m1, m2 and m3 represent the EPP, EST and TLM models, respectively. y11 stands for $y_{opt1}=1$, y1 stands for $y_{opt}=1$ and v1 stands for $var=1$. Similar meanings apply to y10, y0 and v0.)

In the case of the 3 storey infilled frame, the position of the Performance Points predicted for all earthquake scale factors, is seen to be affected strongly by the choice of idealisation model. For the 3DOF steel structure, a significant influence on the inter-storey drift predictions due to model variation is only observed at large ductility values. The use of constant rather than variable strain hardening in the case of TLM introduces errors of less than 10% in both cases and is regarded to be appropriate for use in its stead. In the case of EST, use of either variable strain hardening or a yield point defined using equal energy concepts, results in larger discrepancies than if constant strain hardening and a yield criterion of first deviation from linear are used. However, even in the latter case errors of around 20% are observed. Therefore, it is clear that use of this model is only appropriate in the idealisation of smooth pushover response curves, where sudden variations in slope do not occur. Use of the EPP model results in large errors, ($\approx 45\text{-}70\%$), for all earthquake scale factors and yield options. This model is not regarded to be suitable for use in the context of capacity spectrum assessments.

EQ. SF ⁽¹⁾	ISD _{max} (%)	Error in ISD _{max} wrt TLM (y11y1v1) (%) ⁽³⁾						
	TLM (y11y1v1) ⁽²⁾	TLM (y11y1v0)	EST (y11y1v1)	EST (y11y1v0)	EST (y10y1v0)	EPP (y10y1v0)	EPP (y10y2v0)	EPP (y10y3v0)
0.5	0.0235	-7.76	47.93	47.96	-14.62	-87.37	77.87	47.92
1	0.0800	9.39	-7.89	37.37	-1.06	-66.01	86.88	65.08
1.5	0.1300	-5.81	-26.51	24.68	-21.60	-104.17	85.12	31.56
2	0.2092	-7.96	-0.31	30.86	-21.15	-78.42	67.84	6.24
3	0.3363	-3.56	-70.22	17.08	-13.58	-83.41	40.46	19.09
4	0.4923	-1.11	-24.93	12.47	-8.94	-62.88	42.79	30.00
5	0.6492	-3.44	-11.01	-3.15	-6.30	-76.47	-25.32	-20.53
6	0.8264	-4.62	-1.33	7.13	-3.98	-86.63	-39.84	-48.27
7	1.0240	-4.98	4.31	9.06	-1.17	-50.61	-24.07	-33.10
8	1.2801	-2.13	10.83	13.74	4.81	-	-	-31.59
9	1.5526	0.09	17.43	17.45	9.36	-	-	-8.49
10	1.6920	-5.73	13.15	13.23	5.66	-	-	-

⁽¹⁾Earthquake scale factor; ⁽²⁾y11y1v1 represents the yield options: yopt1=1, yopt = 1 and var = 1, similar notation is used to represent the yield options in the other cases; ⁽³⁾Error is defined by $(100 \times (\text{TLM ISD value} - \text{model ISD value}) / \text{TLM ISD value})$

Table 7. 3: Summary of the variation in predicted maximum interstorey drift values for the 3 storey infilled frame (material combination 1, Table 6.7) assessed under the increasingly scaled 436ya earthquake record due to the use of different pushover curve idealisation models

EQ. SF ⁽¹⁾	ISD _{max} (%)	Error in ISD _{max} wrt TLM (y11y1v1) (%) ⁽³⁾			
	TLM (y11y1v1) ⁽²⁾	TLM (y11y1v0)	EST (y11y1v1)	EST (y11y1v0)	EPP (y10y1v0)
16	4.646	0.00	0.00	0.00	0.00
21	6.748	0.31	0.00	0.46	-0.01
26	7.43	-2.49	0.00	-3.27	-4.35
31	7.688	-2.30	0.00	-2.95	-5.72
36	8.608	0.19	0.00	-0.28	-2.99
41	10.594	-0.55	0.00	-5.22	-23.48
46	12.794	0.00	-2.09	-11.18	-27.58
51	14.34	0.04	-7.71	-18.21	-29.41

⁽¹⁾Earthquake scale factor, ⁽²⁾y11y1v1 represents the yield options: yopt1=1, yopt = 1 and var = 1, similar notation is used to represent the yield options in the other cases; ⁽³⁾Error is defined by (100*(TLM ISD value – model ISD value)/ TLM ISD value))

Table 7. 4: Summary of the variation in predicted maximum interstorey drift values for the 3 DOF steel structure assessed under the increasingly scaled Killini earthquake record (Greece 1988, reference 436ya), due to the use of different pushover curve idealisation models

All current capacity spectrum assessment procedures adopt EPP or EST type curves in the determination of the capacity and demand of structures. The above observations therefore raise concern over the adequacy of these methods for the assessment of infilled frames, and promote the use of a program such as CAsP that allow more complicated capacity curve modelling. It is concluded that TLM is the only model suitable for the idealisation of the pushover response of infilled frames. TLM with variable strain hardening is therefore adopted in the assessment of the low-rise infilled structural population used as an example herein.

7.3.2 Inelastic Demand Calculation

For a given earthquake record and scale factor, CAsP calculates the inelastic seismic demand corresponding to each capacity-demand-checking point (CDCP) of the structure pushover curve. The idealised shape of the capacity curve up to the CDCP location, characterises the elastic period, ductility and inelastic response of the single degree of freedom system used to determine the demand at that point. The earthquake record used in the analysis is scaled and discretised into time increments less than $(1/50)^{\text{th}}$ of the smallest vibration period of interest. The acceleration record is applied in these time steps to the SDOF and a full Newton-

Raphson iterative scheme, similar to that used by the program InSpect (Antoniou and Papazoglou 1997), is used to solve the dynamic non-linear equilibrium equation (Equation 7.5) for the evaluation of the SDOF response.

$$u''_{t+\Delta t} + 2\xi\omega u'_{t+\Delta t} + R_{t+\Delta t} = -(\ddot{u}_g)_{t+\Delta t} \quad [7. 5]$$

In Equation 7.5, t and $(t+\Delta t)$ are the times at the end of the previous and current applied time step (Δt). $\ddot{u}$, $\dot{u}$ and u are the SDOF acceleration, velocity and displacement response, respectively, whilst $\ddot{u}_g$ is the applied ground motion acceleration. R is the non-linear restoring force of the SDOF, which is calculated in accordance to the inelastic response curve characterising the SDOF behaviour and is expressed in acceleration units.

At the beginning of each time step (t_i), the acceleration ($\ddot{u}_i^{(0)}$), velocity ($\dot{u}_i^{(0)}$) and displacement ($u_i^{(0)}$) values are known either from the initial conditions or from the previous analysis step. A first estimate of the acceleration and response values at the end of the time step ($t_i+\Delta t$) is made using the Newmark-Raphson relationships:

$$u'_{t_i+\Delta t}^{(1)} = \left(1 - \frac{\gamma}{\beta}\right) u'_{t_i}^{(0)} + \left(1 - \frac{\gamma}{2\beta}\right) \ddot{u}_i^{(0)} \Delta t \quad [7. 6]$$

$$u''_{t_i+\Delta t}^{(1)} = -\frac{1}{\beta\Delta t} u'_{t_i}^{(0)} + \left(1 - \frac{\gamma}{2\beta}\right) \ddot{u}_i^{(0)} \quad [7. 7]$$

In these equations, γ and β are the parameters describing the algorithmic damping and assumed variation in acceleration within each time step, respectively. A value of 0.5 for γ implies no logarithmic damping. $\beta = 1/6$ and $1/4$ respectively represent linear and constant variations in acceleration within time-steps. In the analysis of the 3-storey infilled frames, parameter values of $\beta = 1/6$ and $\gamma = 1/2$ are chosen to ensure accuracy. Stability is ensured through the use of small time intervals for the integration, (defined by $1/50^{\text{th}}$ of the minimum period of interest).

The initial estimate of velocity and previous step displacement, are used to calculate the tangent stiffness ($K_T^{(l)}$) and non-linear restoring force ($R_{t+\Delta t}^{(l)}$), according to the model chosen to characterise the SDOF inelastic behaviour. The dynamic out of balance force ($\Delta F^{(l)}$) and corresponding displacement increment for the iteration ($\Delta u^{(l)}$), are calculated using Equations 7.8 and 7.9, (the denominator of the latter, being the dynamic tangent stiffness).

$$\Delta F^{(l)} = -\left(u''_{g,t}\right)_{t_i+\Delta t} - u''_{t_i+\Delta t}^{(0)} - 2\xi\omega u'_{t_i+\Delta t}{}^{(0)} - R_{t_i+\Delta t}^{(0)} \quad [7.8]$$

$$\Delta u^{(l)} = \frac{\Delta F^{(l)}}{\frac{1}{\beta\Delta t^2} + 2\xi\omega \frac{\gamma}{\beta\Delta t} + K_T} \quad [7.9]$$

The values of Δu and ΔF are checked against a pre-established tolerance level. If the tolerance is not satisfied, further iteration is required. The displacement is incremented (i.e. $u_i^{(l)} = u_i^{(0)} + \Delta u^{(l)}$). The process of $\dot{u}_i$, $\ddot{u}_i$, K_T , $R_{t+\Delta t}$, Δu and ΔF estimation is then repeated by substituting (1) and (0) in Equations 7.6 to 7.9, by the current (q) and previous ($q-1$) iteration numbers, respectively. Iterations continue until tolerance is satisfied. The final displacement, velocity and acceleration estimates are then used as the initial values for the iterative SDOF response solution under the next applied time step. The maximum response under the entire record, defines the spectral displacement value used to characterise the demand at the CDCP. This value is checked against the corresponding capacity value in the performance point evaluation.

The x-axis of the vulnerability curves derived herein, are characterised by the parameter of elastic spectral displacement at 5% critical damping ($S_{d5\%}(T)$). This demand value is also calculated within CAsP. The effects of material variability and structure inelastic behaviour, on both the building capacity and seismic demand, are included in the proposed method for structure performance assessment. Hence, the influence of these factors on the population vulnerability is implicitly included in the y-axis of the developed fragility curves. Use of either a variable elastic period or an effective period for the calculation of the demand parameter characterising the x-axis of the vulnerability curves, is superfluous and

would significantly complicate the curve application. A SDOF system with linear elastic behaviour, characterised by a mean natural period of vibration for the structure population, is therefore used to determine the $S_{ds\%}(T)$ value associated with the assessed building damage statistics. A mean elastic period of 0.125s is observed considering all system material combinations, and a yield criterion of first deviation from linearity. Therefore, a set of spectral displacement values corresponding to $T = 0.125s$ is calculated for each structural assessment by CAsP. All structures tested for a given earthquake and scale factor, are assigned the same seismic demand value for use in vulnerability curve plotting. This allows the construction of vulnerability curves wherein the material variability is not included in the ground motion axis.

7.4 Statistical Analysis of the Response

Existing analytical vulnerability curves are generated directly from observations of structural response under increasing ground motions. Such methods require an extremely large number of analyses to fully represent the structural and ground motion uncertainties involved in the seismic assessment of building populations. For example, Singhal and Kiremidjian (1997) carry out nonlinear time-history analyses on finite-element bare reinforced concrete frame models of varied material properties, using 100 artificial acceleration records, scaled to 40 seismic intensity levels (4000 analyses in total). The repetition of such a computationally intensive study for many different structure classes, is impractical due to the time involved and specialised analysis tools required. Therefore, it is general practice in existing analytical vulnerability curve derivation methods, to make compromises, either on the number of structural variations and earthquake records used, or on the accuracy of the structural modelling, analysis and assessment technique. For example, Mosalam et al (1997) adopt 800 earthquake records covering a wide range of intensities to test 200 material variations of their structural model, but simplify the structure to a single degree of freedom system for the analyses. These compromises may affect the reliability of the final vulnerability curves.

Within the method proposed here, a two stage process is adopted in the generation of the population damage statistics, which allows both a reduced number of analyses and a reliable representation of the population response uncertainty to be achieved. In the first stage, the results of the structural population assessment are used to derive relationships between the observed maximum inter-storey drift response, structural properties and ground motion parameters. Structural analyses resulting from suites of hazard scenario consistent acceleration records, are considered separately in the definition of relationships, (response surfaces), that are specific to the scenario. Experiment design methods are used to select the minimum number of random variable values and their combinations (Section 6.3.2), required for an appropriate representation of the population response in the construction of the surfaces. The smaller number of analyses required for the construction of response surfaces, results in a reduced analysis time and allows accurate models and analysis tools to be used (Section 7.1.1). In the second stage of the statistical analysis process, numerous structural parameter values are randomly sampled from their relative probability distributions. These sampled values are input to the surface equations. Hence, histograms of maximum inter-storey drift response are generated for a series of ground motion parameter values. Damage state exceedence probabilities are determined from the histograms, using the HRC damage state threshold values of inter-storey drift defined in Section 3.4.3. When plotted against the corresponding ground motion parameter values, the exceedence probabilities define the vulnerability plot used for the final curve shape regression. Uncertainty in the damage state prediction, and its variation with increasing ground motion intensity, is accounted for through vulnerability curve confidence bounds. These are determined from consideration of the fit of the analytical observations of maximum inter-storey drift to the response surfaces, within the damage histogram generation process.

A description of the response surface determination, damage statistics generation and vulnerability curve derivation methodologies are given in the following sections and illustrated for the case of the 3 storey infilled frame population. The vulnerability curves generated for this building category are then compared in Section 7.5.1, to observations of post-earthquake damage and the empirical relationships derived in Chapter 5.

7.4.1 Construction of Response Surfaces

The capacity spectrum assessment for each frame of defined properties (f_c, f_{cw}, f_y, f_{yb}), yields values of maximum inter-storey drift response (ISD_{max}) for increasing values of ground motion intensity. Response surfaces, relating the system ISD_{max} response to the material and ground motion parameters, are required by the proposed method for the generation of population damage statistics. Response surfaces are derived for each hazard scenario, through separate consideration of the analysis results corresponding to the scenario record suites. These are assigned names according to the return period of the earthquakes associated with the scenario, (i.e. 95YRP, 475YRP and 2475YRP). For comparison purposes, an additional response surface is derived considering all analysis results grouped together (ALLYRP). The elastic spectral displacement values corresponding to the mean of the yield periods for the population ($T = 0.125s$), are considered in the definition of the scenario response surfaces. This spectral displacement is evaluated at 5% critical damping within the CasP program (see Section 7.3.2). It is emphasised that the developed response surfaces may only be used in the assessment of structures with configurations similar to that of the three storey infilled frame, and with material strengths lying within the 90% occurrence probability bounds of the distributions presented in Section 6.3.2. An additional response surface is also derived for the prediction of the yield period (T_I) of the individual structures within the population, from knowledge of the material parameters.

From consideration of the 2ⁿ Factorial Composite design of experiment method discussed in Section 6.3.2, 25 and 43 parameter values are required to fully represent the uncertainty associated with 4 and 5 uncorrelated random variables, respectively. 25 combinations of the 4 material parameters are used herein for the low-rise infilled frame population definition. Each of these is tested under 10 records at approximately 15 ground motion intensity values, for each of the three earthquake scenarios. The number of records and procedure adopted for the selection of the accelerograms is discussed in Chapter 4, where attempts are made to fully account for ground motion variability. These considerations result in an

average of 3750 points for the regression of each earthquake scenario response surface. This number exceeds the minimum requirement for the derivation of a second order response surface from the 5 random variables involved, and is deemed sufficient for the detection of severe discontinuities or singularities.

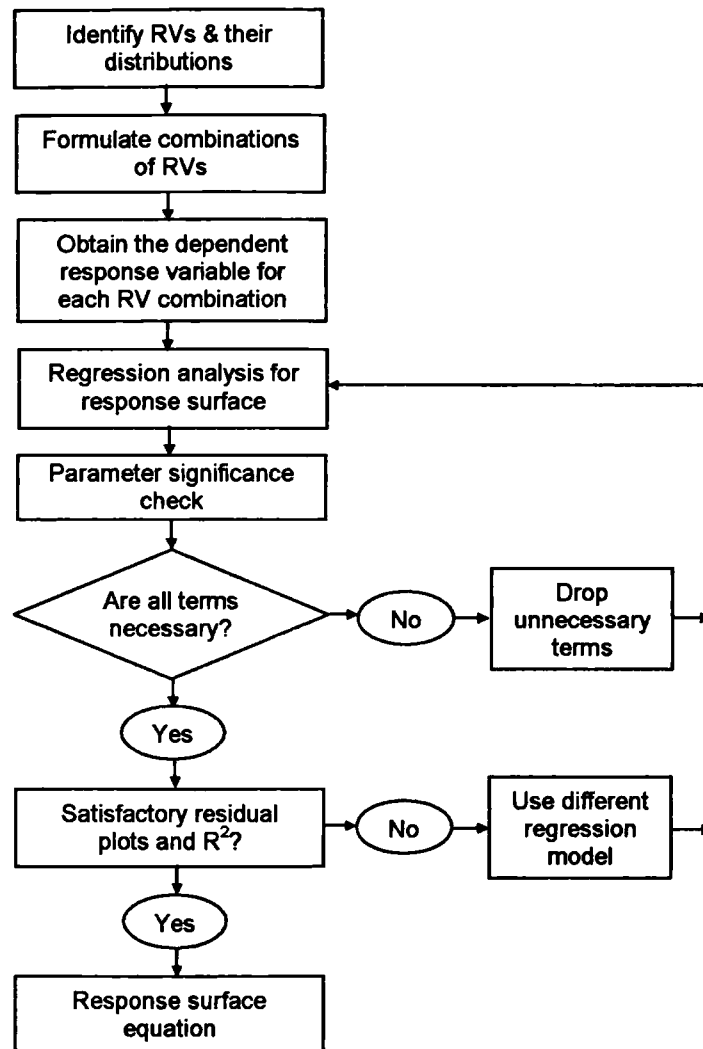


Figure 7. 16: Flowchart of the response surface derivation methodology from Dymiotis (2000).

Second order response surfaces of the form of Equation 7.10 are fit to the analysis results using the non-linear regression module of STATISTICA (StatSoft Inc. 1995), following the procedure outlined in Figure 7.16. Except for the case of beam and column steel yield strengths, which are intentionally correlated, the material parameters and their combinations do not have a strong inter-correlation. All considered parameters can therefore be included together in the response surface equations. Following each regression, the significance of the different

factors is assessed and those observed not to contribute significantly to the maximum inter-storey drift value prediction are eliminated. A new regression analysis is then carried out on the reduced parameter equation, and the procedure is repeated until all parameters are of significance.

$$T_1 \text{ or } (ISD_{\max} / S_d(T)) = \left\{ \begin{aligned} &a_1 f_c^2 + a_2 f_{cw}^2 + a_3 f_{yc}^2 + a_4 f_{yb}^2 + a_5 f_c + \\ &a_6 f_{cw} + a_7 f_{yc} + a_8 f_{yb} + a_9 f_c f_{cw} + a_{10} f_c f_{yc} + \\ &a_{11} f_c f_{yb} + a_{12} f_{cw} f_{yc} + a_{13} f_{cw} f_{yb} + a_{14} f_{yc} f_{yb} + C \end{aligned} \right\} \quad [7.10]$$

Response Parameter	YRP Data ⁽¹⁾	f_c^2	f_{cw}^2	f_{yc}^2	f_{yb}^2	f_c
T_1	ALL	-0.0002	2.209	4.16E-05	-8.97E-06	0.011
$ISD_{\max}/S_{d5\%}(T)$	ALL	0	2354.56	-0.09645	-0.00862	-34.46
	2475	-0.2578	4987.76	-0.10133	-0.01266	-37.04
	475	0	4618.25	-0.00477	0	-17.35
	95	-0.3081	196.69	-0.12184	-0.02209	-36.47
Response Parameter	YRP Data ⁽¹⁾	f_{cw}	f_{yc}	f_{yb}	$f_c f_{cw}$	$f_c f_{yc}$
T_1	ALL	0	-0.0293	0.0057	-0.023	-6.0E-05
$ISD_{\max}/S_{d5\%}(T)$	ALL	3693.5	61.108	-8.295	0	0.1242
	2475	-5658.9	44.642	-22.554	-64.293	0.1725
	475	11824.4	29.413	19.560	35.957	0.0448
	95	4093.1	63.683	-23.133	0	0.0794
Response Parameter	YRP Data ⁽¹⁾	$f_c f_{yb}$	$f_{cw} f_{yc}$	$f_{cw} f_{yb}$	$f_{yc} f_{yb}$	C
T_1	ALL	8.63E-05	-0.00326	0.00225	-5E-06	4.808
$ISD_{\max}/S_{d5\%}(T)$	ALL	-0.0420	-13.583	1.949	0.0405	-10555.17
	2475	0.0000	4.537	8.765	0.0748	-2935.30
	475	-0.0239	-37.065	0	-0.0446	-11145.23
	95	0.0629	-17.180	6.960	0.0910	-7994.82

All compressive and yield strengths are expressed in MPa; ⁽¹⁾ Return period associated with the set of analysis data used in the derivation of the response surface.

Table 7. 5: Inter-storey drift and yield period response surface equation (Equation 7.10) coefficients for the 3 storey infilled frame population.

The response surface equation coefficients derived for the 3-storey infilled frame are summarised in Table 7.5. The fit of the analysis data to the derived response curves is reported in Table 7.6, in terms of the correlation coefficient (R^2), mean error ($\mu(\epsilon)$) and the upper and lower 5th percentile values of the ratio of observed to predicted $ISD_{\max}$. A good fit of the data to the curves is indicated by small

$\mu(\varepsilon)$ values, and R^2 , 95th and 5th percentile ratio values approaching unity. A reasonable fit is achieved in the case of the ISD_{max} response surfaces (see Figure 7.17). This response value is prone to a large scatter due to the combined effects of material and ground motion variability. A very good fit is achieved in the case of the T_1 response surface (see Figure 7.20), which response is affected only by the former source of uncertainty. Bias in the residuals is minimised for all surfaces through the use of a non-linear regression procedure, which aims to maintain a constant percentage error with respect to the size of the observed analytical response value. This is shown in Figure 7.17, where an approximately constant value of normalised prediction error is observed over most of the considered range of ISD_{max} , for all three scenario response surfaces.

Response Surface		No. of obs. ⁽²⁾	$\mu(\varepsilon)$ (%) ⁽³⁾	R^2 ⁽⁴⁾	5% (O/P) ⁽⁵⁾	95% (O/P) ⁽⁶⁾
Response Parameter	YRP Data ⁽¹⁾					
T_1	ALL	25	0.00	0.90	0.97	1.05
$ISD_{max}/S_{d5\%}(T)$	ALL	11260	99.61	0.66	0.20	2.15
	2475	3669	82.89	0.59	0.23	2.77
	475	3878	123.96	0.71	0.17	1.91
	95	3713	93.97	0.69	0.22	1.69

All compressive and yield strengths are expressed in MPa; ⁽¹⁾Return period associated with the set of analysis data used in the derivation of the response surface; ⁽²⁾Number of analytical observations of ISD_{max} ; ⁽³⁾Mean absolute error in prediction (i.e. $100 \times (\text{pred-obs})/\text{obs}$); ⁽⁴⁾Correlation coefficient of data used in the derivation with the response surface; ⁽⁵⁾5th percentile (observed/predicted) inter-storey drift ratio value; ⁽⁶⁾95th percentile (observed/predicted) inter-storey drift ratio value.

Table 7. 6: Summary of the fit of the structural yield period and inter-storey drift ratio response surfaces with the analysis data used in the nonlinear regression, (n.b. $S_{d5\%}(T)$ is evaluated at a period of 0.125s).

Illustrations of the variation in the response with respect to the material parameters, as predicted by the surface equations for $ISD_{max\%}$ and T_1 are shown in Figure 7.18, Figure 7.19 and Figure 7.21. Although the main trends in the observed response with material parameter variation are similar in all cases, significant differences exist in the actual response values predicted by the different earthquake scenarios. This is expected to lead to the generation of significantly different vulnerability curve shapes for the scenarios in Section 7.5.

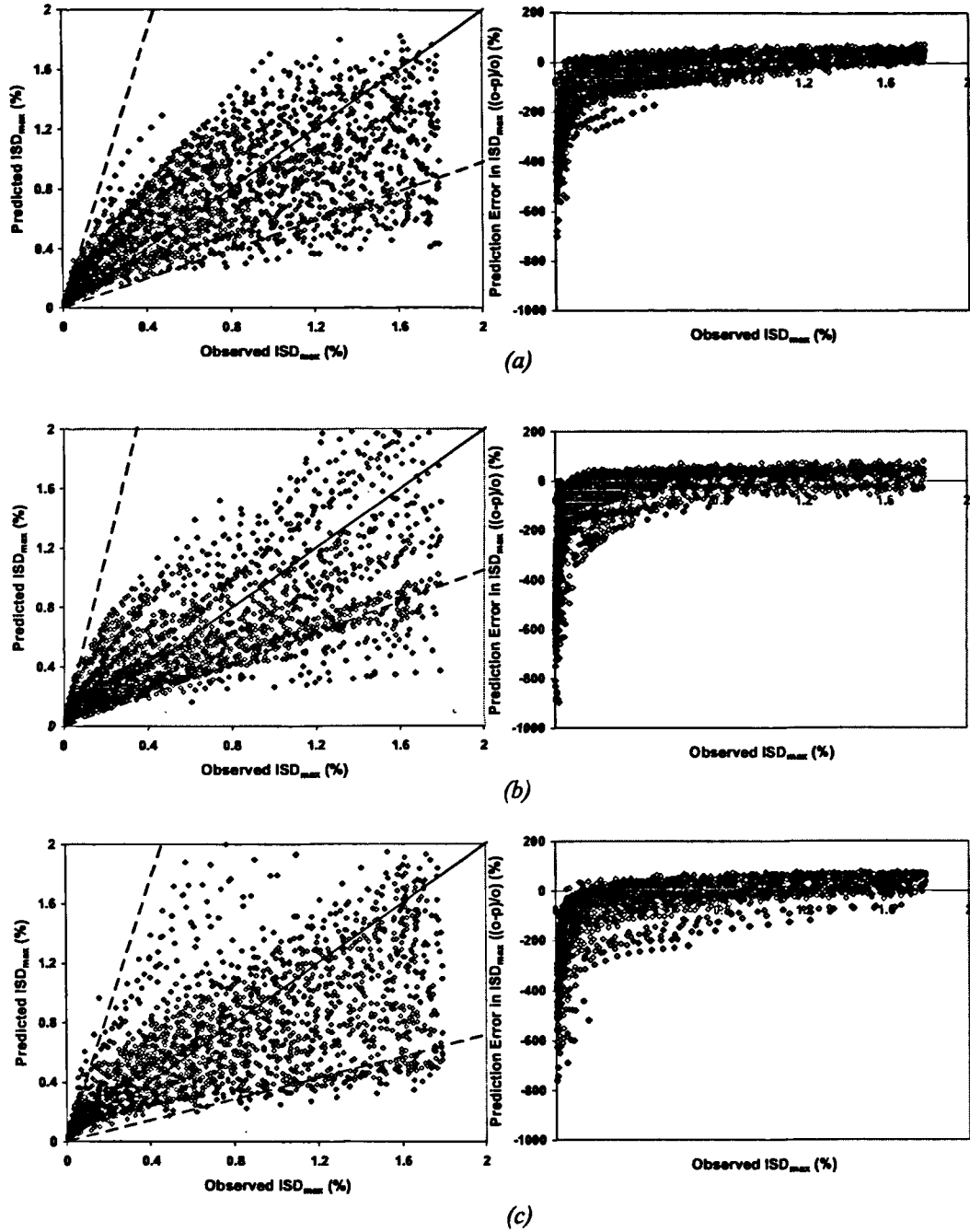


Figure 7. 17: Plots of the predicted values and prediction error in maximum inter-storey drift ratio with the analytically observed response values for the 3-storey infilled frame different scenario response surfaces derived using $S_d(T=0.125s)$: (a) 95YRP, (b) 475YRP and (c) 2475YRP. The dashed lines in the left hand side plots illustrate the 5th and 95th percentile observed/predicted ISD_{max} bounds.

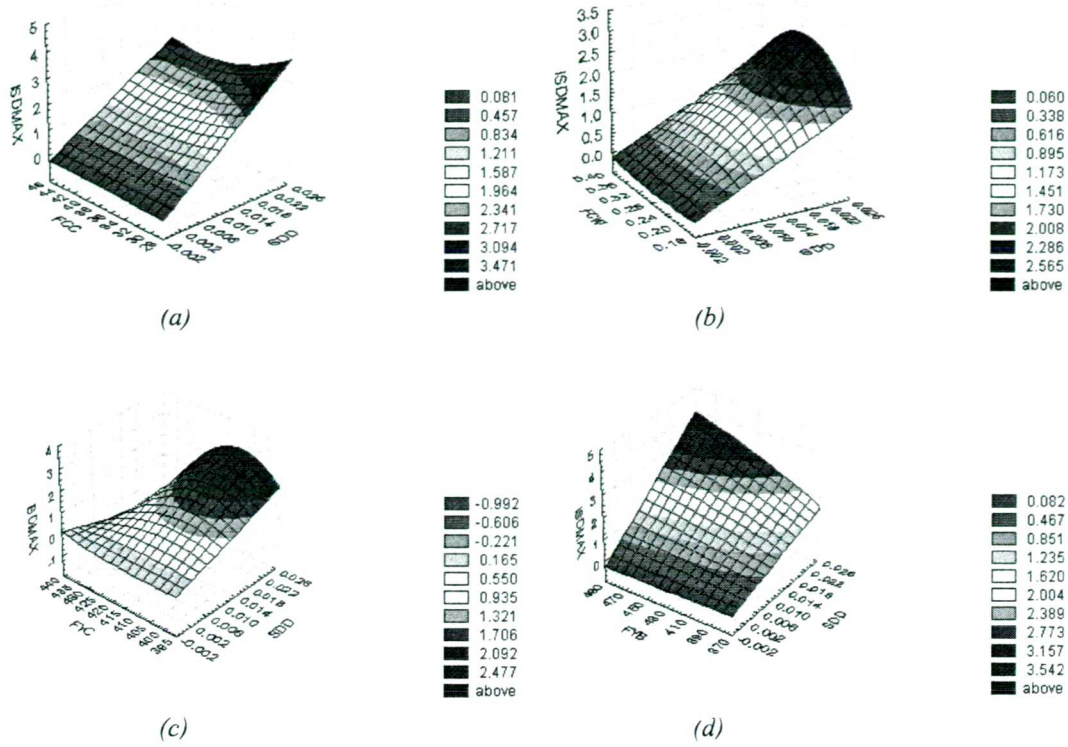


Figure 7. 18: The 3-storey infilled frame 95YRP response surface for ISD_{max} : Variation in response with increasing values of (a) f_c , (b) f_{cw} , (c) f_{yc} and (d) f_{yb} when all other material parameters take their constant mean values.

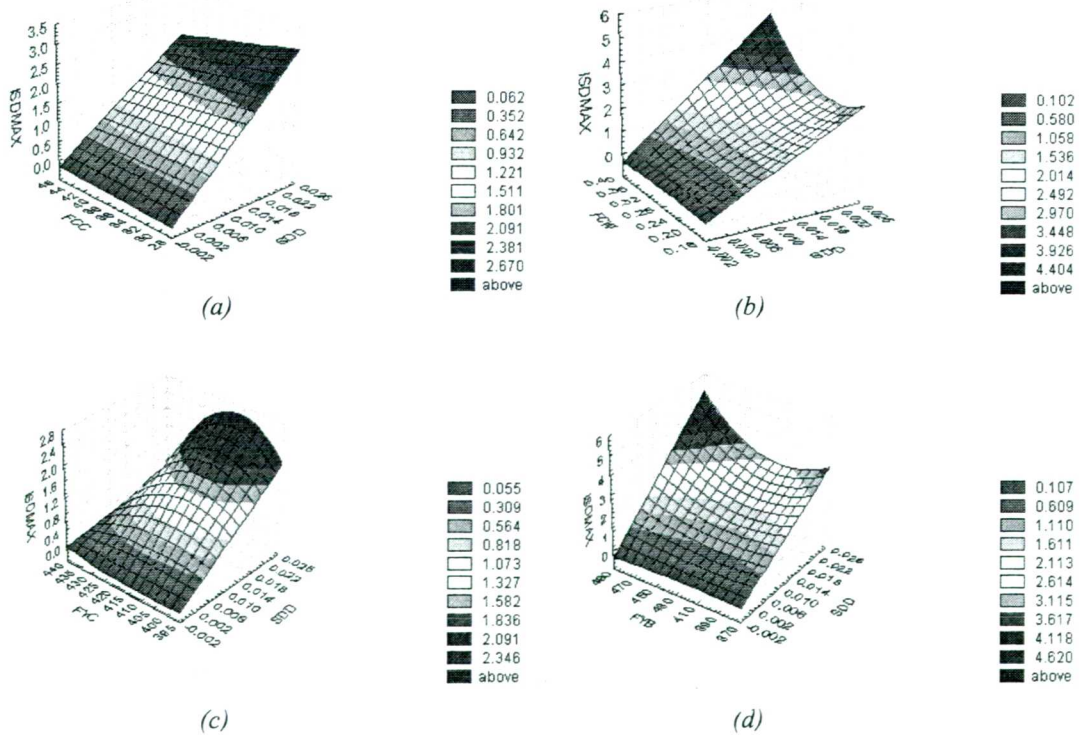


Figure 7. 19: The 3-storey infilled frame 2475YRP response surface for ISD_{max} : Variation in response with increasing values of (a) f_c , (b) f_{cw} , (c) f_{yc} and (d) f_{yb} when all other material parameters take their constant mean values.

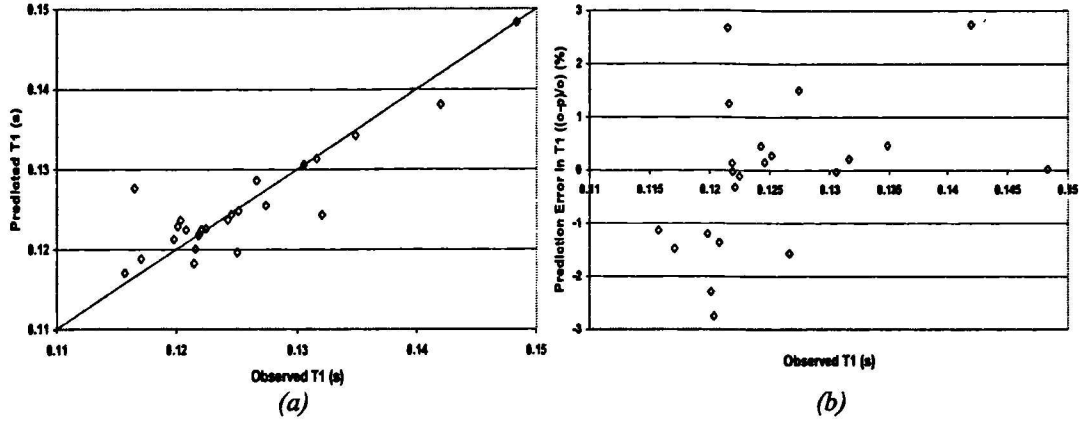


Figure 7. 20: Plots of (a) the predicted values and (b) prediction error in yield period, (T_1), with the analytically observed response values for the 3-storey infilled frame response surface.

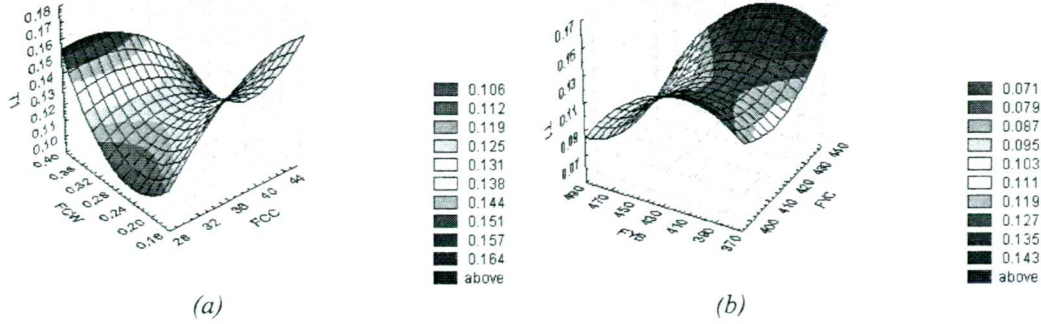


Figure 7. 21: The 3-storey infilled frame response surface for T_1 : (a) Variation in response with increasing values of f_c and f_{cw} , for constant mean values of f_{yc} and f_{yb} (413MPa), (b) Variation in response with increasing f_{yc} and f_{yb} values, for constant mean values of f_{cw} and f_c (0.26MPa and 36.64MPa respectively)

7.4.2 Generation of Damage Statistics and Vulnerability Curves

Damage statistics are generated for a system through the multiple selection of material parameter value combinations for input into the response surfaces. Values of ISD_{max} response are evaluated at a series of spectral displacements, for each material parameter combination. Frequency plots, such as those shown in Figure 7.23, are obtained for each spectral displacement value and can be used to define the shape parameters of probability distribution functions for ISD_{max} . Damage state exceedence probabilities are calculated from the latter functions considering the threshold ISD_{max} values for the HRC damage states presented in

Section 3.4.3. Alternatively, if a large number of system variations are adopted, the damage state exceedence probabilities may be determined directly from the proportion of the population with responses exceeding the threshold ISD_{max} limits. In the case of the 3-storey infilled frame, use of 650 material parameter combinations is found to be sufficient for the direct evaluation of the damage state exceedence probabilities. This is shown in Figure 7.22, where consistent vulnerability curve shapes are obtained using 500 or more system variations.

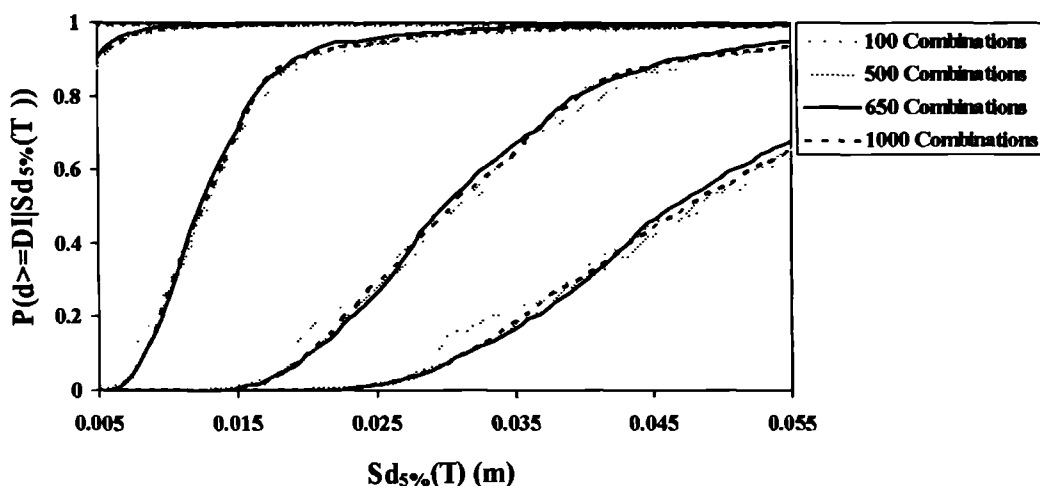


Figure 7. 22: Comparison of the vulnerability curves generated using different numbers of material combinations to generate damage statistics from the 475YRP response curves, (only the extensive to collapse limit state curves are shown).

A transformation method is used to convert random deviates with a uniform distribution to normally distributed deviates with zero mean and unit variance. The normally distributed random variables are then used to select material parameter values from their relative probability distribution functions. In the case of the log-normally distributed material parameters, equivalent normal mean and standard deviation values for the parameter natural logarithm are calculated and used in the selection of the normal deviates. The subroutine, written in Excel (Microsoft® Excel 2001), to select the normally distributed deviates and convert them to material parameter values, is adapted from Numerical Recipes (Press et al. (2001), p280). It uses the Box-Muller transformation method and the Rnd function of Excel as the source for uniform deviates. Correlation between the sampled beam and column steel yield strengths is achieved through the selection of COV_{fb} values, from uniform distributions centred about the sampled COV_{fc} values. The

uniform variate takes values between $-0.6COV_{fy_c}$ and $+0.6COV_{fy_c}$, which range is observed to give a correlation of 0.7 between the resulting beam and column yield strengths.

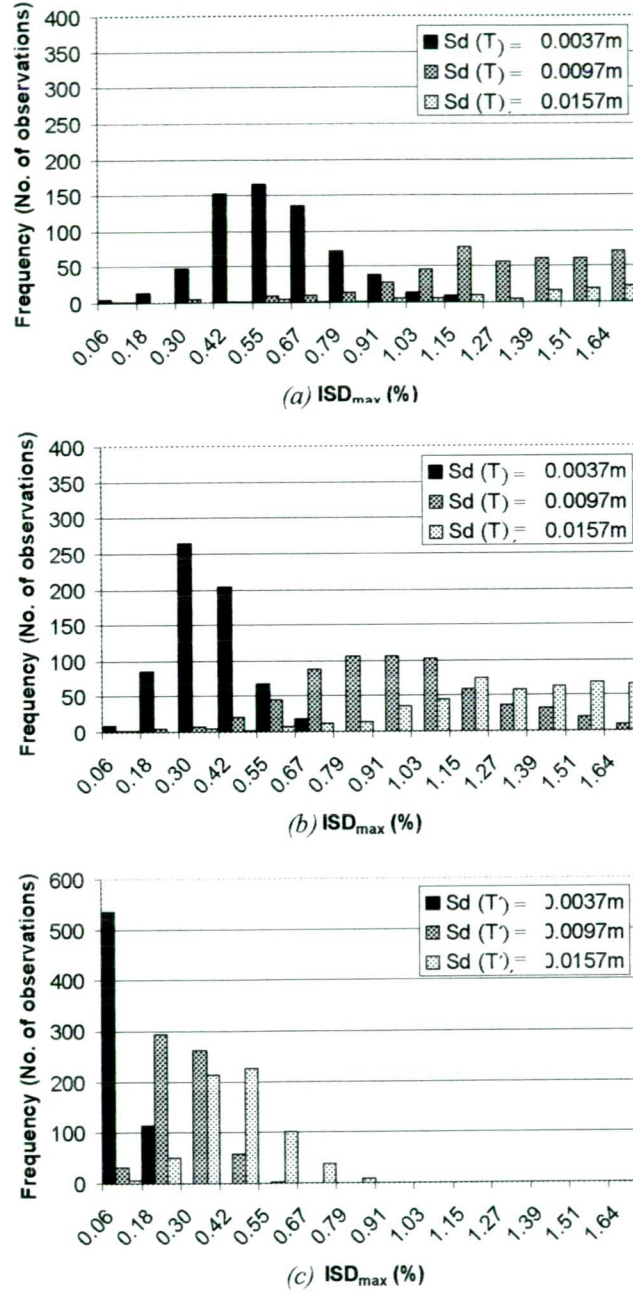


Figure 7. 23: Histograms of maximum inter-storey drift response at three levels of spectral displacement generated from the 475YRP response surface for $T=T_1$: (a) Mean, (b) Upper bound (95th percentile) and (c) Lower bound (5th percentile) inter-storey drift response histograms .

Each response surface for ISD_{max} in Table 7.5, is considered separately in the response evaluation, at approximately 100 spectral displacement values, of a population consisting of 650 material variations of the assessed system. The

proportion of buildings exceeding each damage state is calculated at each ground motion level and plotted against the corresponding spectral displacement value for the vulnerability curve shape regression. The range and interval size of the spectral displacements used for the damage statistics generation are varied according to the damage state assessed, in order to obtain a better representation of the curve shapes. The fit of the analysis data to the response surfaces is a direct indication of the uncertainty in response prediction due to input parameter variability. The 5th and 95th percentile values of the ratio of analytically observed to predicted ISD_{max} response are therefore applied to the mean ISD_{max} predicted by the response surface for each system variation. New exceedence probability statistics are generated using the extreme ISD_{max} estimates. These are plotted against the corresponding ground motion values and used in the regression of 90% confidence bounds for the vulnerability curves.

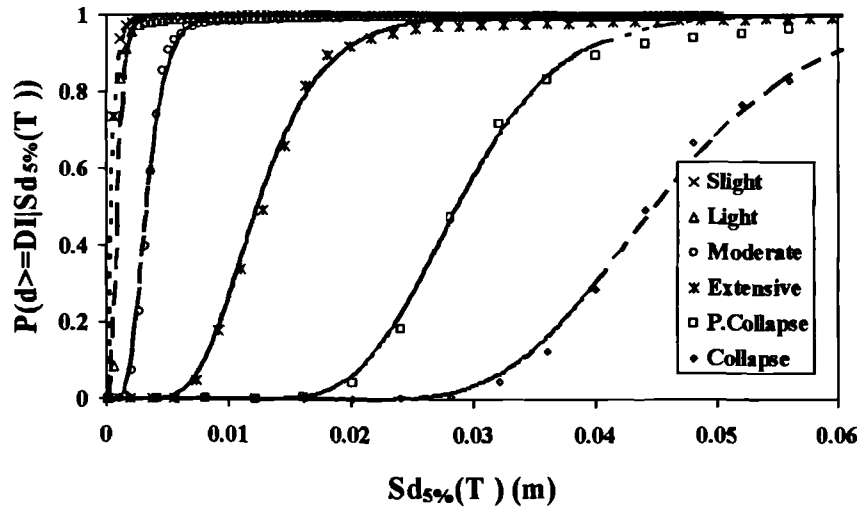


Figure 7. 24: The fit of the regressed mean analytical vulnerability curves for the 3-storey frame, (derived using mixed earthquake scenarios, HRC damage scale and $Sd_{5\%}(T)$) with the vulnerability plot data derived from the generated damage statistics.

As in the case of the empirical vulnerability curves derived in Chapter 5, exponential and lognormal cumulative distribution functions, (of the forms of Equation 5.3 and 5.2, respectively), are considered for the vulnerability curve shapes. In the empirical case, little difference was observed in the ability of the two functions to represent the observed damage statistics due to the large scatter in these values. In the present case the log-normal distribution is instead observed to give a far superior representation of the analytical vulnerability points. This

function is therefore used in the derivation of all analytical vulnerability curves for the three-storey infilled frame population. The non-linear regression tool of STATISTICA (StatSoft Inc. 1995) is used to regress for the parameters of the lognormal cumulative probability functions. A summary of the resulting vulnerability relationships for all response surface and damage states, is presented in Table H.1 (Appendix H). No vulnerability curves for the prediction of damage in populations of infilled reinforced concrete frames exist in FEMA 273 (ATC 1997). Vulnerability curves corresponding to the FEMA 273 damage scale are therefore also derived from the response surfaces for $S_d(T_1)$. The damage state conversions of Table 3.4 are used to determine ISD_{max} threshold values for the FEMA 273 limit states. Using these values, vulnerability plots are generated, to which log-normal curves are fit via non-linear regression. The resulting curves are presented in Table H.3, (Appendix H). A very close fit of the regressed curves to the analytical data is achieved in all cases, with R^2 values above 0.99 being observed. An example of the fit of the analytical data to one of the HRC damage scale regressed curves is shown in Figure 7.24.

7.5 Performance-Consistent Analytical Vulnerability Curves

Discrepancies between the vulnerability curve shapes derived from the different earthquake scenario response surfaces, is observed in Figure 7.25. The disparity is attributed to changes in the spectral ordinate amplification within the period range of interest, caused the different accelerogram characteristics of the events defining each earthquake scenario. Thus, in order to obtain a realistic representation of the expected damage, it is important that a scenario is selected that is consistent with the performance objective associated with each damage state. A set of performance-consistent vulnerability curves is proposed, where the “Slight” and “Light” damage state curves are derived from the 95YRP response surface, the “Moderate” and “Extensive” damage state curves are derived from the 475YRP response surface and “Partial Collapse” and “Collapse” state curves are derived from the 2475YRP response surface.

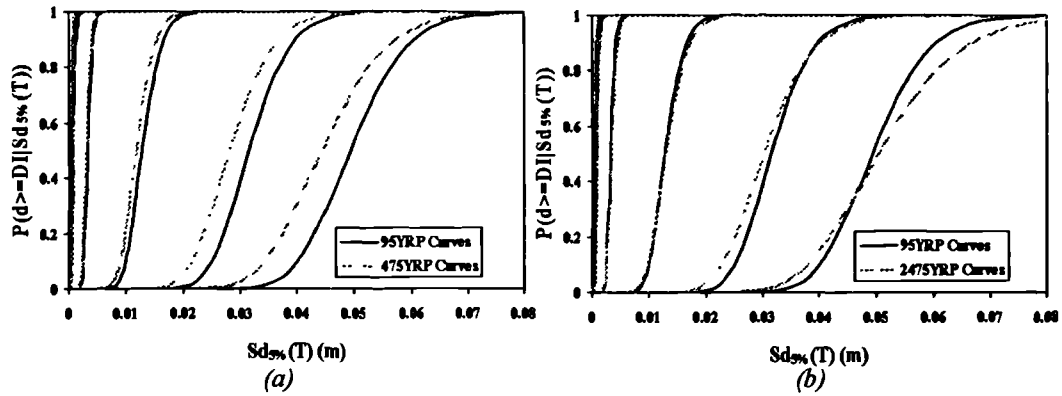


Figure 7. 25: Comparison of the HRC damage scale analytical vulnerability curves for the 3-storey frame, derived using the 95YRP, 475YRP and 2475YRP response surfaces.

The mean performance-consistent curves are shown in Figure 7.26a, and the parameters describing the log-normal curve functions are summarised in Table 7.7. A set of performance-consistent vulnerability curves is similarly defined for the FEMA 273 damage scale using the 95YRP response surface for the “Serviceability” damage state curve, the 475YRP response surface for the “Damage control” curve and the 2475YRP response surface for the “Limited safety” and “Collapse prevention” curves. A summary of the curve parameters is presented in table 7.8, and the mean curves illustrated in Figure 7.26b. Due to the different limit state definitions for infilled structures in the HRC and FEMA scales, the latter are seen to give a much more conservative prediction of collapse.

Damage State	Mean			95% Upper Bound			5% Lower Bound		
	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$
Slight	-7.80	0.60	0.996	-8.25	0.60	0.998	-6.30	0.42	0.997
Light	-7.15	0.40	0.999	-7.82	0.60	0.998	-5.76	0.34	0.998
Moderate	-5.78	0.21	0.998	-6.32	0.20	0.997	-4.51	0.22	0.999
Extensive	-4.44	0.21	0.997	-4.92	0.21	0.996	-3.12	0.22	0.996
P.C.	-3.49	0.22	0.998	-3.98	0.22	1.000	-2.12	0.27	0.995
Collapse	-2.99	0.22	0.997	-3.50	0.22	0.998	-1.67	0.24	0.997

⁽¹⁾Mean; ⁽²⁾Standard deviation; ⁽³⁾Correlation coefficient of data used in the derivation with the vulnerability curve.

Table 7. 7: Summary of the performance consistent analytical HRC damage state vulnerability curve parameters for the three storey infilled frame system, using elastic spectral displacement at 5% critical damping to characterise the seismic demand.

Damage State	Mean			95% Upper Bound			5% Lower Bound		
	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$	$\mu^{(1)}$	$\sigma^{(2)}$	$R^2^{(3)}$
Serviceability	-7.85	0.60	0.998	-7.81	0.40	1.000	-6.50	0.40	0.998
Damage Control	-6.25	0.25	0.999	-7.40	0.25	1.000	-5.27	0.25	0.999
Limited Safety	-4.35	0.22	0.997	-5.37	0.22	0.998	-3.36	0.22	0.997
Collapse Prevention	-3.90	0.22	0.998	-4.92	0.20	0.999	-2.95	0.20	0.999

⁽¹⁾Mean; ⁽²⁾Standard deviation; ⁽³⁾Correlation coefficient of data used in the derivation with the vulnerability curve.

Table 7. 8: Summary of the analytical mixed scenario FEMA damage state vulnerability curve parameters for the three storey infilled frame system, , using elastic spectral displacement at 5% critical damping to characterise the seismic demand.

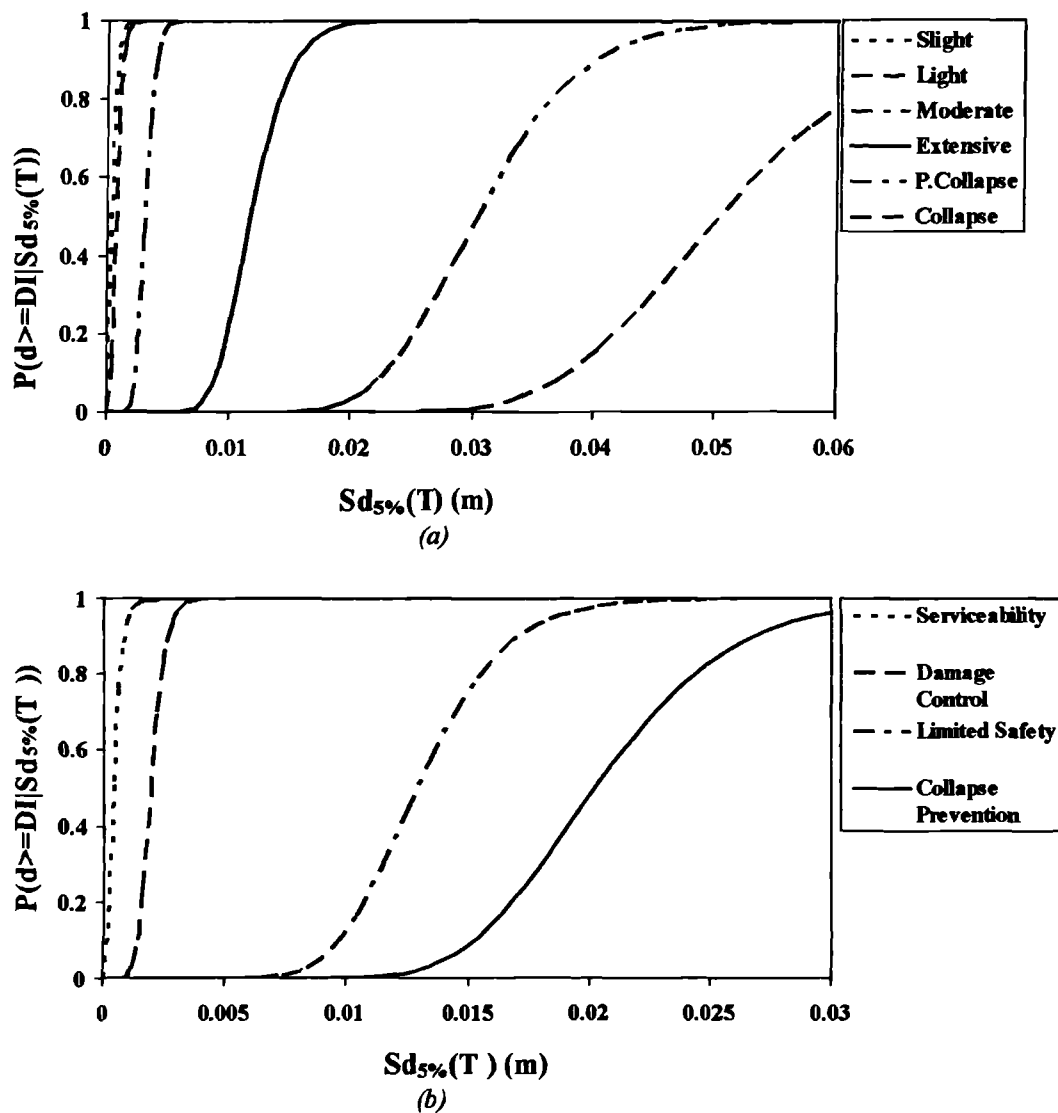


Figure 7. 26: Mean performance consistent analytical curves for a population of three storey infilled frames designed to old seismic codes, using spectral displacement at 5% critical damping used to characterise the seismic demand: (a) Vulnerability curves for the HRC damage scale, (b) Vulnerability curves for the FEMA 273 (ATC 1997) damage scale.

The proposed vulnerability curve derivation method implicitly includes the effects of non-linear structural response and material variability, in the performance assessment of the structures. The derived curves therefore allow vulnerability predictions for the structure class, to be obtained directly from elastic spectral displacement demand values, (with 5% critical damping), evaluated at an estimated mean natural period of vibration for the buildings.

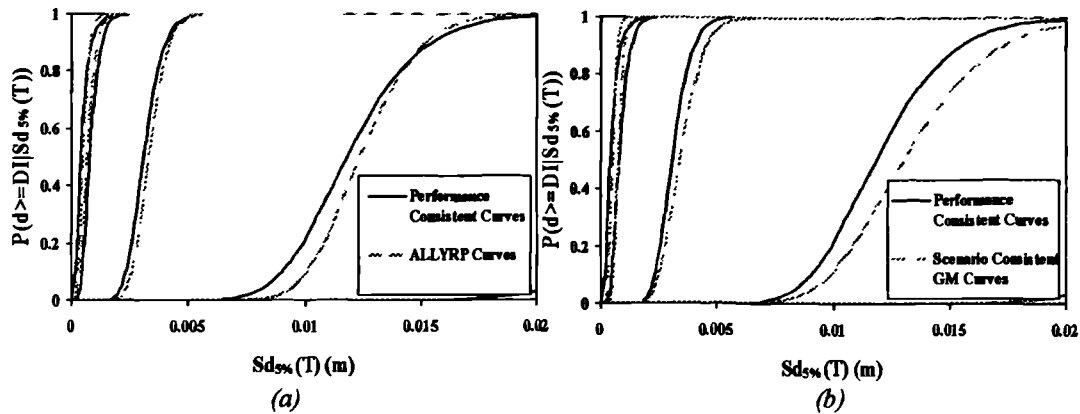


Figure 7. 27: Comparison of the performance consistent analytical vulnerability curves for the 3-storey frame with those, derived using (a) the ALLYRP response surfaces and (b) Scenario consistent ground motion intervals ("Slight" to "Extensive" damage curves shown only).

The performance-consistent curves are compared in Figure 7.27a, to vulnerability curves derived using the analytical results for all earthquake scenarios, (ALLYRP). Unconservative estimates of the exceedence probability for higher damage states result from the latter curve, due to the influence of the frequent earthquake characteristics on the structural response. These observations highlight the importance of considering different earthquake scenarios in the definition of different damage state vulnerability curves.

The performance-consistent curves adopt a single scenario for the generation of each curve, over the entire range ground motion parameter values. In order to reach values of spectral displacement that are large enough to cause the entire population to reach collapse, the 95YRP, 475YRP and 2475YRP acceleration records need to be scaled by up to 50, 30 and 16 times their original pga , respectively. No guidelines for the definition of an acceptable degree of acceleration record scaling exist in the literature, and concern is raised in Chapter

4 regarding the effect of excessive scaling on the vulnerability prediction. An alternative definition for the vulnerability curves is therefore investigated, wherein each scenario response surface is used to derive the damage statistics for all damage states, over the range of ground motions that characterise the scenario. In this manner, less scaling of the more frequent event scenario records is required to define the curve shapes. The mean values of magnitude and site distances defined in Table 4.3, are used within the attenuation relationships of Borzi et al. (1998) to calculate approximate mean spectral displacement values (corresponding to the mean structural period of 0.125s), for each hazard scenario. The range of $S_{ds\%}(T=0.125s)$ values over which each scenario response surface is used, are summarised in Table 7.9. In this table, the boundary values of spectral displacements are defined by the values lying midway between the calculated mean $S_{ds\%}(T=0.125s)$ values of successive scenarios. The resulting vulnerability curves are summarised in Table H.2 of Appendix H, and are compared to the performance consistent curves in Figure 7.27b. Only the “Slight” to “Extensive” damage state curves are shown in the figure, as the higher damage state curves remain unaltered, being totally defined by the 2475YRP response surface. The “Moderate” and “Extensive” damage state curves are also practically identical to those defined for the 2475YRP scenario with only the “Slight” and “Light” damage state curves being significantly affected by more than one scenario. A less conservative damage estimate and less rapid progression between damage states, compared to the performance-consistent curves, results from this new curve definition. The record scaling for both the 95YRP and 475YRP accelerogram suites is reduced to a factor of 3, whilst the 2475YRP records are still required to be scaled by a factor of 16.

Earthquake scenario return period	Earthquake scenario $S_{ds\%}(T=0.125s)$ limits
95 years	0 - 0.000725m
475 years	0.000725m – 0.00123m
2475 years	0.00123m – 0.00182m

Table 7. 9: Spectral displacement ranges assigned to the different seismic hazard scenarios and evaluated at a period of 0.125s using the attenuation relationships of Borzi et al. (1998) and the mean earthquake magnitudes and fault distances reported in Table 4.3.

At any one ground motion value, the curves derived using scenario consistent ground motion intervals give a theoretically more correct representation of the distribution in damage within the population, than the performance-consistent curves. However, the limits of spectral displacement defining the earthquake scenarios change according to the seismicity and typical dynamic properties of structures in the assessed area. This introduces an additional source of uncertainty. Furthermore, use of different response surfaces for the definition of a single damage state, results in curve shapes with a worse fit to the vulnerability plot points over the range of ground motions associated with the corresponding performance objective. Hence, where the satisfaction of multiple performance objectives under different ground motion intensities needs to be checked, the performance-consistent curves are regarded to provide a more reliable vulnerability prediction within the range of relevant spectral displacements. Thus, despite the increased level of record scaling, the performance-consistent method of vulnerability curve definition is recommended.

7.5.1 Comparison of Analytical Curves with Observational Damage Data

The analytical performance consistent vulnerability curves for the 3-storey infilled RC frame designed to the Italian standards of 1982, are compared to the observational post-earthquake survey data presented in Chapter 5. The analytical curves are compared to the general structure empirical curves, and to all the weighted observational data in Figure 7.28. A poorer fit to the data is observed compared to the general structure empirical curves. This is expected as the latter curves are derived directly from the observational data, and the analytical curves relate only to a sub-set of the structures covered by the general structure category.

A summary of the observational damage data fit with the analytical curves is given in Tables H.4 to H.6 of Appendix H, for all structural categories concerning the assessed system. In Figure 7.29a, the analytical curves are compared to the empirical curves and observational data specifically relating to infilled frames. The analytical curves give much more conservative estimates of damage than the empirical curves for all ground motions. In Figure B.5, (Appendix B), a slower

attenuation with distance (for a constant earthquake magnitude), of the lighter damage state curves is seen compared to the infilled frame empirical vulnerability relationships. A more rapid attenuation with distance of the higher damage state curves is instead observed. The vulnerability curves for low-rise infilled frames proposed by Mosalam et al. (1997) are presented in Figures B.1d and B.7c, and although named “Heavy” and “Moderate”, correspond only to the light damage states of the HRC-Scale. The derived analytical curves are also observed to give more conservative damage estimates over a larger distance than the latter curves, (for any given earthquake magnitude).

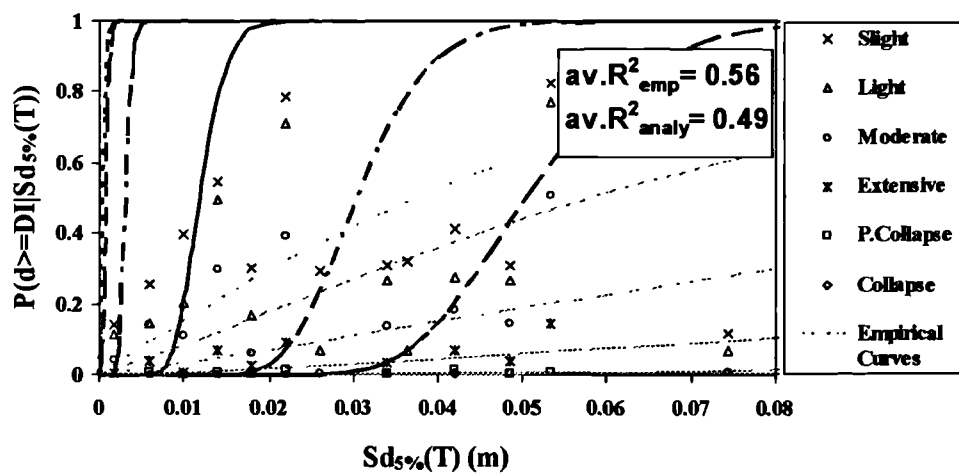


Figure 7.28: Comparison of the mean performance consistent HRC damage state analytical curves for 3 storey infilled frames designed to old seismic codes with all weighted observational data and the general structure empirical vulnerability curves presented in Chapter 5.

Although the observational damage state exceedence probabilities are over-predicted by the analytical curves, a better fit to the data is achieved than in the case of the empirical curves. This is possible due to the scarcity and highly scattered nature of the post-earthquake damage surveys available for infilled RC frames, which result in the derivation of unreliable curve shapes for the empirical curves (see Section 5.4). Very little data is available for comparison with the analytical curves for the structural sub-categories of low-rise infilled frames, infilled frames designed to old seismic codes and low-rise infilled frames designed to old seismic codes. However, for this limited quantity of data, the analytical curves are seen to give very good predictions of the observed damage. In Figure B.13 in Appendix B, the analytical curves are also compared to damage data for low and mid-rise infilled frames observed after the Campania (Italy 1980)

earthquake. They are observed to perform better than the derived empirical curves and existing curves, which are compared to the same data in Figures B.7, B.8, B.10 and B.11 (Appendix B). Based on these comparisons, it may be concluded that the analytical curves give a reasonable, though conservative, fit to the observational data for the assessed structure class.

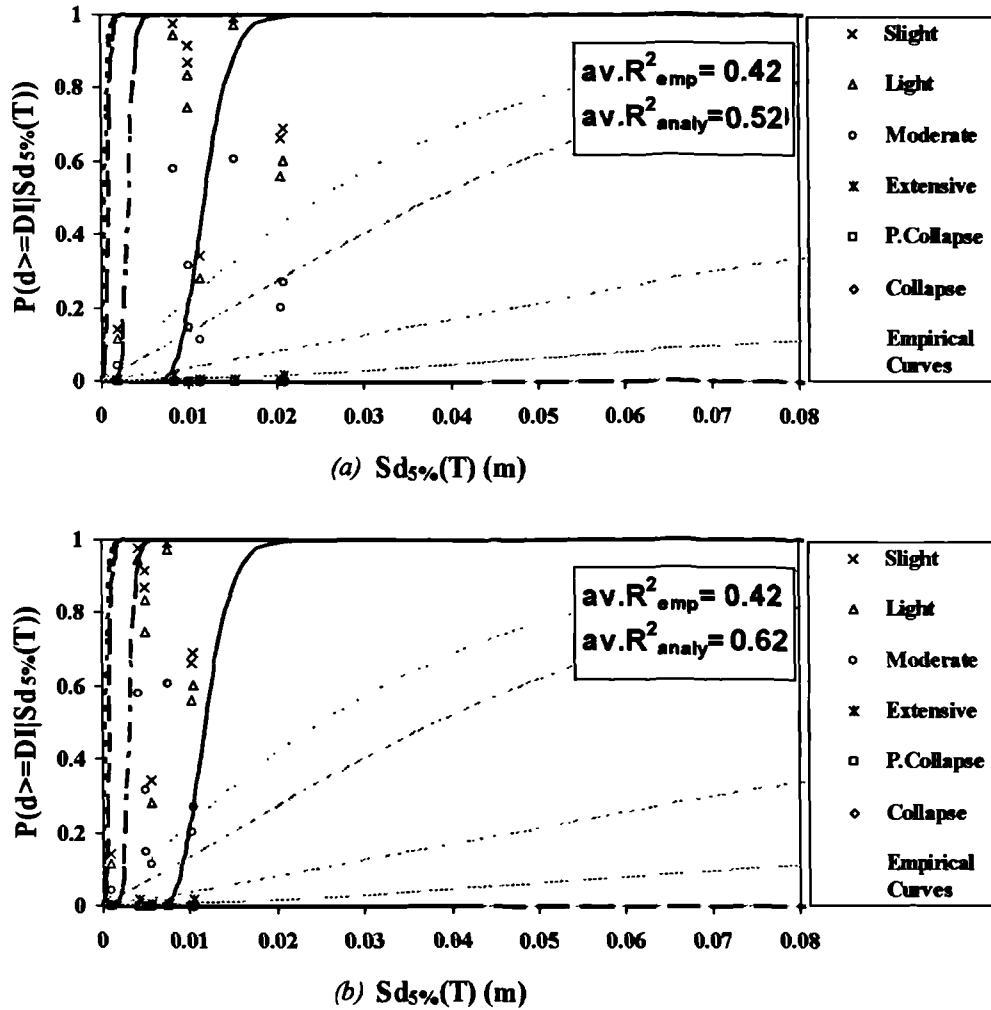


Figure 7.29: Comparison of the HRC damage state analytical curves for 3 storey infilled frames designed to old seismic codes with un-weighted observational data for infilled frames and the infilled frame empirical vulnerability curves presented in Chapter 5: (a) Comparison of analytical curves derived using $S_{d5\%}(T=0.125s)$, with observational data with $S_{d5\%}(T)$ evaluated using the natural period estimate of EC8 (Eq. 4.4); (b) Comparison of analytical curves derived using $S_{d5\%}(T=0.125s)$, with observational data with $S_{d5\%}(T)$ evaluated using half the natural period estimate of EC8 (Eq. 4.4). (n.b. The comparison is made only for the "Slight" to "Extensive" damage state curves, as no data is available for higher damage states).

Damage State	No. of Bldgs ⁽¹⁾	Data-sets ⁽²⁾	Re-Derived Empirical Infilled Frame Vulnerability Curve				
			$\alpha^{(3)}$	$\beta^{(4)}$	$\mu(\varepsilon)^{(7)}$	$COV(\varepsilon)^{(8)}$	$R^2^{(9)}$
Slight	2720	8	472	1.1	0.216	0.612	0.472
Light	2720	8	351	1.1	0.238	0.568	0.419
Moderate	2720	8	135	1.2	0.173	0.751	0.276
Extensive	2460	7	5.1	1.3	0.005	0.772	0.329
Damage State	No. of Bldgs ⁽¹⁾	Data-sets ⁽²⁾	Analytical Infilled Frame Vulnerability Curve				
			$\mu^{(5)}$	$\sigma^{(6)}$	$\mu(\varepsilon)^{(7)}$	$COV(\varepsilon)^{(8)}$	$R^2^{(9)}$
Slight	2720	8	-7.80	0.60	0.179	1.097	0.624
Light	2720	8	-7.15	0.40	0.217	0.686	0.665
Moderate	2720	8	-5.78	0.21	0.180	1.023	0.559
Extensive	2460	7	-4.44	0.21	0.009	0.776	0.397

⁽¹⁾ Number of buildings; ⁽²⁾ Number of observational datasets; ⁽³⁾ and ⁽⁴⁾ Shape parameters of the re-derived empirical vulnerability curves; ⁽⁵⁾ and ⁽⁶⁾ Shape parameters of the analytical vulnerability curves; ⁽⁷⁾ Mean absolute error in the observational exceedence probability prediction ($\varepsilon = \text{Abs}(p-o)$); ⁽⁸⁾ Coefficient of variation of the prediction error; ⁽⁹⁾ Correlation coefficient of the analytical predictions with the observational data.

Table 7. 10: Summary of the empirical infilled frame vulnerability curve parameters, re-derived using $S_{d5\%}$ ($T=0.125s$) to characterise the seismic demand, and their fit to the observational data for this system. Comparison with the correlation obtained for the same data using the analytically derived vulnerability curves

The degree of conservatism associated with the analytical curve estimates is expected to be less in reality, than is observed in the reported comparison. The observational damage distributions relating to infilled frames are assigned a spectral displacement value by assuming that their natural period can be estimated from the EC8 (Eurocode 8, CEN 1996) natural period equation ($T = 0.05H^{3/4}$). The latter equation predicts an elastic period of 0.26s for a 3-storey infilled frame, which is approximately double the mean period value obtained from the population analysis. If half the EC8 predicted period is used in the ground motion parameter evaluation for the observational data, a significant improvement in the data fit with the analytical curves is observed and a conservative, yet realistic damage prediction obtained (Figure 7.29b and Table H.4 in Appendix H). The use of a more realistic period for the re-derivation of the empirical curves for the infilled frame structural category, results in steeper curves for the slight to moderate damage states, which approach those obtained from analysis (Figure 7.30), but yield less conservative results. However, no significant change is seen

in the correlation coefficient of the re-derived curves with the data (Table 7.10). These observations highlight the problems that may occur from the adoption of an inappropriate relationship for the prediction of the structural period in the derivation of empirical curves, and further emphasises the importance of an analytical approach for the curve generation.

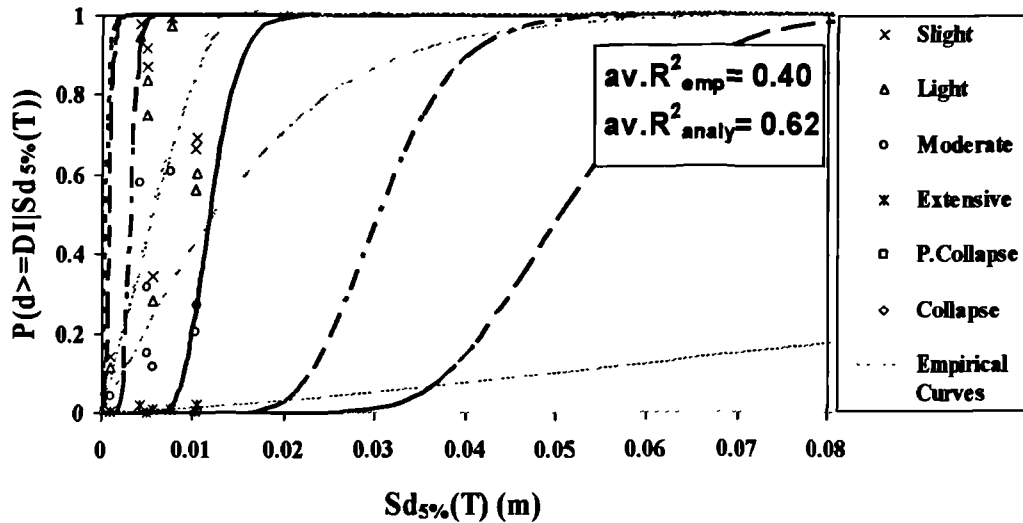


Figure 7. 30: Comparison of the HRC damage state analytical curves for 3 storey infilled frames designed to old seismic codes (bold lines), with empirical curves (faint lines) re-derived from the un-weighted observational data evaluated using half the natural period estimate of EC8 (Eq.4.4) to calculate $S_{d5\%}(T)$. (n.b. The comparison is made only for the “Slight” to “Extensive” damage state curves, as no data is available for higher damage states).

7.5.2 Analytical Curve Confidence Bound Derivation

Log-normal functions representing the 90% confidence bounds of the proposed performance consistent curves, are presented in Table 7.7 and illustrated in Figure 7.31. The vulnerability data used in the regression of the bound curve shapes is generated together with damage statistics defining the mean vulnerability curves, and considers the fit of the analysis data to the response surfaces (see Section 7.4.2). The confidence bounds are wide and increase in width for higher damage states. It is observed that similar sized confidence bounds are generated for cases where the response surfaces are derived from data regarding a single scenario, as when they are derived from the combined data from all scenarios (ALLYRP, see

Figure H.4in Appendix H). This indicates that the variation in material and ground motion is fully represented in the analyses for each earthquake scenario.

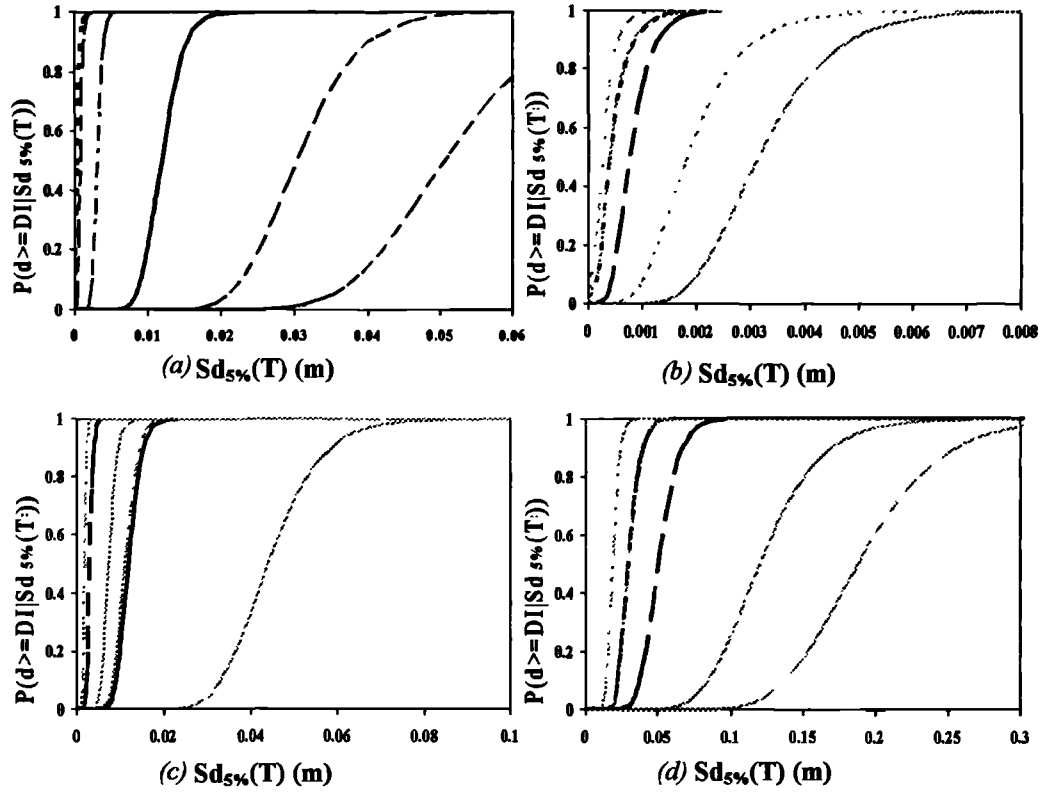


Figure 7.31: The proposed performance consistent analytical vulnerability curves (dark lines) and 90% confidence bounds (light lines) for the 3-storey infilled frame system using the HRC damage scale and $S_d(T=T_1)$: (a) Mean curves; (b) Slight and Light damage states; (c) Moderate and Extensive damage states; (d) Partial Collapse and Collapse damage states

The width of the confidence bounds in terms of maximum exceedence probability interval, (for constant spectral displacement), is comparable to that observed for the infilled frame empirical curves. The width in terms of maximum spectral displacement interval, (for constant damage state exceedence probability), is instead a fraction of that observed for the infilled frame empirical curves. The difference is due to the scarcity of observational data for the empirical curve derivation, especially at high levels of ground motion, which results in large uncertainties. This observation confirms the importance of analytical methods for the generation of vulnerability curves.

Within the analytical curves the difference between the mean and lower bound tends to be much larger than that for the mean and upper bound curves. This indicates that the mean curves provide a reasonably conservative estimate of the

damage distribution for the analysed population. This observation is confirmed by comparisons made between the mean analytical curves and observational data for infilled frames in Section 7.5.1. It is therefore concluded that the analytical curves derived using the proposed methodology, provide a reliable tool for the vulnerability assessment of populations of reinforced concrete structures. In cases where a large uncertainty is associated with the material properties of the structural population or seismic hazard prediction, it is however recommended that the upper bound vulnerability curves be used in the population damage assessment.

Chapter 8

Summary and Conclusions

The need for new vulnerability curves for the assessment of reinforced concrete building populations in Europe is established following a review and critique of existing worldwide vulnerability relationships. None of the latter is deemed adequate, due to problems associated with their choice of building classes, damage scale, ground-motion parameter or derivation methodology. Furthermore, many do not use a sufficient range of damage data for a reliable definition of the curves at high-ground motions or for severe damage states. The influence of the choices made for damage data source, ground motion and damage characterisation, on the reliability and range of application of vulnerability curves, is described. A comparison is made between the existing curves most relevant to European reinforced concrete buildings, and observed post-earthquake building damage statistics. A poor fit of the observational data to the curves is obtained in all cases. The uncertainty in prediction associated with the application of the curves to building types specific to the country of their origin, is shown to be comparable to that obtained from their application to European buildings of similar structural system. This implies that it is possible to combine observational damage data deriving from different countries and earthquakes, in the derivation of new empirical relationships, given appropriate consideration of structural similitude.

In light of these observations two new procedures are proposed for the derivation of vulnerability curves for the seismic assessment of populations of reinforced concrete structures. The first of these consists of a methodology for the interpretation and combination of observational post-earthquake building damage statistics to produce vulnerability curves. The second methodology outlines a new procedure for the generation and processing of building population damage statistics from analysis. Both procedures adopt the same damage scale and ground motion parameter, for the performance and demand evaluation, respectively. The choice of these parameters is discussed in detail in Chapters 3 and 4.

Existing damage scales are reviewed with a view to their use for the representation of damage in European type RC structures. Scales developed in the United States and Japan, are observed not to cover all types of European RC constructions. Problems with existing damage scales are also identified. Amongst these is the adoption of ambiguous wording for limit state descriptions, use of too few limit states, non-consideration of non-structural damage or the choice of criteria other than structural damage to characterise limit states. Existing scales are therefore not deemed suitable for use in the derivation of vulnerability curves for European structures. A new damage scale called the Homogenised Reinforced Concrete Scale (HRC-Scale) is proposed for this purpose. Following a review of existing damage indices, the HRC-scale is calibrated to the parameter of maximum inter-storey drift ratio ($ISD_{max\%}$). $ISD_{max\%}$ is chosen to represent the global damage state of the structures, due to its ease of evaluation and ability to detect both ductile and soft-storey modes of failure. The calibration of the HRC-scale is carried out using published results of dynamic and pseudo-dynamic experimental tests on RC structures, many of which being for full-scale models. Separate calibration equations are derived for structures of different lateral load resisting system. The HRC-scale is also approximately correlated to other national and internationally adopted visual damage scales, to allow its use in the re-interpretation of heterogeneous observational data in the vulnerability curve generation. With the possible exception of the Park and Ang (1985a) damage index, this is a unique example of the calibration of an observational damage scale with numerous, reliable experimental results.

A review of ground motion parameters for the characterisation of earthquake damage potential is carried out in Chapter 4. The correlation of peak and spectral, acceleration and displacement values, with observed earthquake damage is investigated. Of the considered parameters, spectral displacement, evaluated at the elastic period of vibration for 5% critical damping, ($S_{d5\%}(T)$), is found to best represent the damage potential of earthquake records. A method for the selection of performance compatible earthquake records, for use in the generation of analytical vulnerability curves is presented. The method consists in deriving “target” spectra that are characteristic of an event with a return period for which the structural damage state defining the curve is acceptable or is expected. Three

target spectra are determined for events with response periods of 95, 475 and 2475 years, using the Italian hazard maps of Albarello et al. (1999), associated data and attenuation rules. Uncertainty in ground motion is accounted for through the use of suites of ten accelerograms in each limit-state vulnerability curve derivation. The records are selected for different site soil conditions, such that the average spectral shape of the selected suite of records closely approximates the target spectrum. The records thus selected, have peak, frequency and duration characteristics that correspond to scenario events, which are representative of the hazard level associated with the assessed performance objectives. Although derived using Italian hazard maps, the records selected derive from various European countries and may be used to carry out vulnerability analyses in any area with similar seismic activity and fault mechanisms to those considered (i.e. medium European seismicity, shallow crustal faulting). For areas with very different seismicity or characterised by subduction earthquakes, the same procedure for record selection may be applied, however appropriate ranges of peak ground accelerations, earthquake surface magnitudes and fault distances should be found from historical earthquake catalogues or from local hazard maps.

In Chapter 5 the first procedure for the derivation of vulnerability curves, that based on observational building population post-earthquake damage distributions, is presented. The procedure is followed in the derivation of empirical curves from a data bank of 99 post-earthquake damage distributions, observed in 19 earthquakes and concerning a total of 340,000 RC structures. The databank, specifically assembled for the purposes of this study, contains observational data for structures with varied lateral-load resisting system, presented in terms of different damage scales. The statistics are therefore re-interpreted in terms of the HRC scale with the aim of producing a single set of “homogeneous” vulnerability relationships, which are applicable to all lateral-load resisting systems. The resulting empirical curves are observed to give a better correlation with observed post-earthquake building damage statistics, than existing empirical relationships. This is despite their inclusion of a greater number of structural classes. It is concluded that a substantial improvement can be achieved, within the limits imposed by the quality and quantity of data available, through use of the proposed vulnerability curve generation process. A considerable success is observed in the

removal of the influence of lateral load resisting system on the vulnerability curve shape, through the use of the HRC damage scale. However, a sensitivity analysis of the homogeneous curves to different building characteristics shows that other building characteristics which affect the lateral load resistance can preclude the “homogeneous” curve use in the vulnerability assessment of high-rise buildings and buildings pre-dating seismic code implementation. It is concluded that a greater quantity and variety of large-scale dynamic experiments are required for the calibration of the HRC-damage scale, if the effects of construction quality and seismic design are to be accounted for in the “homogeneous” empirical vulnerability curves. Until this calibration data is available, the latter effect may only be accounted for through the use of separate vulnerability curve sets, generated for structures of predominantly different heights and seismic design codes. Curves for pre- and old seismic code, low- and medium-rise frame categories are derived from subsets of the observational databank and are presented in Chapter 5. However, as these curves are based on few observations, it is recommended that they be used only with due consideration of the confidence bounds. A greater quantity of empirical data is needed in order to generate reliable curves for infilled frames, shear-wall, high-rise and new seismic code structures. The empirical curves presented herein for the later structural systems should not be used for vulnerability assessments.

From the application of the proposed empirical curve derivation methodology, it is evident that the use of empirical data alone is not sufficient for the realisation of a risk assessment tool that can be applied with a high degree of confidence to populations of reinforced concrete buildings. The lack of data for high damage states and ground motion severities means that the vulnerability curves could be extrapolated in a number of different shapes, and still maintain the same fit to the observational data. Furthermore, large scatter results from the inherent heterogeneity of the building survey sample and erroneous assessments of building damage in the original reported statistics. The influence of these factors is significant and can not be removed through data manipulation after the survey completion. It is concluded that empirical observations must be replaced or supplemented by analytically-simulated damage statistics for reliable vulnerability predictions to be obtained.

In Chapters 6 and 7, a new procedure is proposed for the derivation of analytical displacement-based vulnerability curves for the seismic assessment of populations of reinforced concrete structures. The methodology represents an optimum solution between reliability and computational efficiency. Adaptive pushover analyses are adopted within a capacity spectrum framework of assessment, to determine the performance of a population of building models under increasing ground motions. Adaptive pushover analyses can account for the effect of ground motion characteristics on structural response and require reduced computational effort compared to time-history analyses. Furthermore, the chosen assessment method avoids the repetition of analyses under increasing ground motions, thus reducing the analysis number from several thousands to a few hundred.

In order to carry out the vulnerability assessment of a population of RC buildings, different vulnerability curves must be derived for each predominant structural system within the assessed area. Building design codes and construction materials should be considered in the determination of these structural systems, and local seismic activity should be considered in the earthquake record selection for the analyses. In order to illustrate the proposed methodology, the procedure is applied in the derivation of vulnerability curves for typical existing Italian low-rise, infilled RC frames. For this purpose a 3-storey building is designed and detailed according to the 1982 Italian codes of practice for gravity and earthquake loads. The building configuration and material properties used for the design are selected to be representative of the construction period assumed. The 3-storey building is modelled in 3D finite elements and analysed within the INDYAS inelastic dynamic analysis program (Elnashai et al. 2000). The sub-division of the frames into finite elements is carried out considering the spread of plasticity in members. The Mander et al. (1988) model and a bi-linear curve are adopted to represent the concrete and steel stress-strain behaviour, respectively. The stress-strain curve proposed by Panagiotakos and Fardis (1994), is modified to account for the effect of openings using the equations of Mosalam (1996), and is used to represent the infill panel behaviour. Within the analyses, a combination of diagonal compression struts, with different bi-linear stress-strain curves, is adopted to model the latter behaviour.

The building model population is generated from a single design through consideration of material parameter uncertainty. A Latin Hyper-cube method is adopted for the sampling of material parameter values from their probability distributions. These values are then shuffled and combined, to obtain a set of frames with the same nominal design, but different material characteristics. In the proposed method, the concrete unconfined compressive strength, the infill compressive strength and the beam and column reinforcing bar steel yield strength are chosen to be randomly varied between frames. In the latter case, a correlation coefficient of 0.7 is assumed to exist between the sampled parameter values within the elements of single frames. Probability distribution functions are assigned to each material property, which are consistent with tests carried out on European construction materials. Design of experiment techniques are used to optimise the population size and ensure convergence of the results. Material variability is seen to introduce an average coefficient of variation of 21.7% in the predicted maximum inter-storey drift ratio of the low-rise infilled frames analysed for the analytical curve example derivation.

The generated building population is analysed using the Adaptive Pushover Method proposed by Elnashai (2001) and Antoniou (2002). The validity of the adaptive pushover technique, is verified with respect to the dynamic time-history analysis of four 8-storey RC bare frames (Fardis 1994) designed according to the seismic provisions of Eurocode 8 (CEN 1996), for different ductility levels and *pga* values. A further verification is carried out for the 4-storey ICONS frame (Pinto et al. 1999), designed for gravity loads only and tested at the ELSA Laboratories in Ispira, Italy. Both conventional pushover and dynamic pushover analyses are compared to the adaptive pushover method. The latter is, however, seen to give the optimum combination of accuracy, in the prediction of the structural top-drift, base-shear and inter-storey drift responses. The method is seen to perform well in the case of the regular, seismically designed 8 storey structures and provides comparable results to dynamic analyses in the case of the irregular, non-seismically designed ICONS frame.

A program is written for the assessment of the population capacity curves under the selected earthquake records. A different approach to the visualisation of the conventional Capacity Spectrum Method of assessment is used, which both aids the automation and increases the efficiency of the assessment procedure. The first stage of the assessment process involves the transformation of the structure base shear – top drift response curves, obtained from the Adaptive Pushover analysis, to spectral acceleration-spectral displacement (SASD) space. As multiple degree of freedom systems are dealt with, a methodology is proposed whereby this transformation is carried out using the instantaneous displaced profile of the structure at each load increment of the adaptive pushover analysis, instead of the structure fundamental mode shape. In the second stage of the assessment process, the resulting capacity curves are idealised as bi-linear or tri-linear curves, (EPP, EST and TLM models, see Figure 7.12). A variety of global yield point definitions and strain hardening options can be used in the curve idealisation. However, from a comparison of the different options the tri-linear (TLM) model with adaptive strain hardening (i.e. that varies according to the point on the curve being assessed), is seen to give the best representation of the infilled structure non-linear pushover response. The idealised capacity curve is next discretised into a series of points, denominated the capacity-demand checking points (CDCP). The idealised curve shape to each CDCP, defines the non-linear behaviour of a corresponding single degree of freedom system (SDOF). The resulting series of SDOFs are analysed dynamically under the earthquake records, to obtain a set of inelastic spectral acceleration and displacement values. These demand values may be visualised as corresponding to the structure capacity along the radii of the SASD plot that intersect the CDCPs. Using these results a single inelastic demand curve may be drawn, and the Performance Point is defined directly from the crossing of this curve with the capacity curve. The maximum inter-storey drift ratio ($ISD_{max\%}$) response is determined at the performance point, from the results of the adaptive pushover analysis. This value is stored together with the corresponding elastic spectral displacement demand at 5% critical damping, evaluated at the average natural period of vibration for the structure population. The same capacity curve is used to assess a structure, over the full range of ground motions required for the vulnerability curve generation, through record scaling.

The results of the population assessment are used to construct second order response surfaces, which relate $ISD_{max\%}$ to the material parameters and elastic spectral displacement demand ($S_{d5\%}(T)$). A different response surface is generated for each damage state scenario, using the results of the population assessment under the corresponding “performance-compatible” record suite. The damage statistics forming the basis of the vulnerability curves, are generated from the response surfaces by re-sampling material parameter values via a Monte Carlo technique. Probability distribution functions (pdfs) are constructed from the frequency of sampled $ISD_{max\%}$ values at fixed values of $S_{d5\%}(T)$. Mean vulnerability curves are plotted for each damage state, from the $ISD_{max\%}$ pdfs. The vulnerability relationships for the “Slight” and “Light”, “Moderate” and “Extensive”, and “Partial Collapse” and “Collapse” damage states are selected from the curve sets derived from the 95YRP, 475YRP and 2475YRP response surfaces, respectively. These relationships are combined to form a “performance-consistent” set of vulnerability curves. The uncertainty bounds for the curves, are determined from the variation in the standard deviation of the $ISD_{max\%}$ pdfs with increasing ground motion.

The analytical vulnerability curves derived for low-rise, old seismic code, infilled RC frames, are compared to the derived empirical curves and observed post-earthquake damage data for infilled frames. The analytical curves are seen to give a better correlation to the observed damage data than the empirical curves. This further casts doubts as to the reliability of the latter. The analytical curves also show improved prediction of observed damage compared to other existing curves for infilled RC frames. It is emphasised that the analytical curves derived herein, should only be applied in the assessment of populations with a predominant structural design and system similar to that considered. It is however concluded that the proposed derivation methodology is capable of producing analytical vulnerability curves that give reasonable predictions of observed post-earthquake building damage with the added benefit of being versatile, scalable and transferable to different structural types and seismo-tectonic environments.

The main outcome of the study is considered to be the proposal of clear methodologies for the derivation of vulnerability curves using empirical and analytical damage statistics, which address problems of uncertainty in the input ground motion and damage state identification. Both the proposed empirical and analytical curves are adequate for use within a displacement-based assessment framework as they relate a deformation based damage index to spectral displacement. The proposed methodologies are regarded to represent the best possible solutions given current knowledge and analysis capabilities.

Future development of the empirical vulnerability curve derivation methodology would involve a more extensive calibration of the HRC-scale for greater reliability of the damage state assessment. A series of full-scale experiments might be carried out specifically for this purpose, and would include monitoring damage in buildings with differing lateral load resisting systems, degrees of seismic design and varying material properties. Data from instrumented buildings damaged by earthquakes might also be used to enrich the calibration data, when such recordings become available. Research currently being carried out for the development of GIS and digital photography based methods for building inventory data collection could be extended to the assembly of accurate post-earthquake building damage distribution data for large areas. Appropriate programs for the interpretation of the photographic evidence and building damage identification would have to be developed for this purpose and validated with on-site damage survey observations.

Further validation of the analytical vulnerability curve derivation methodology is required. The proposed method should be applied to a greater number of structural systems and the resulting curves verified with large quantities of reliable observational data for the assessed structural system. The analytical curve derivation methodology could easily be incorporated into a program which automatically carries out the analysis and assessment of a specified structural system. Databases of structural models, covering different seismic and gravity design codes, structural configurations and heights might be included in the program, together with a corresponding list of characteristic material parameter variation values. The record selection technique could also be automated and

included in the program to allow the generation of the seismic input from a chosen Hazard map. Else suites of standard records included for use in different tectonic environments. In this manner, different vulnerability curves could automatically be derived for all building types in an assessed area and the overall damage exceedence probabilities for a population of given building composition could be easily predicted. The entire procedure could be further enhanced through the incorporation of a spatial building inventory database, which coupled with an adequate visualisation program, would allow an aerial distribution of the probable post-earthquake building damage to be seen. Such a tool would aid in identifying areas where seismic strengthening is required. Furthermore, with advancement of computer technology and the development of faster processors, such a tool could potentially be used for a rapid damage assessment in the event of an earthquake and aid in the distribution of resources for disaster management.

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Appendix A

Review of Existing Vulnerability Relationships

The curves reviewed here are presented in order to illustrate the range of different damage data sources, derivation methodologies, damage and ground motion parameters that have been used in existing curves to date. The equations for the curves are presented in Table 2.2 and their salient characteristics are summarised in Table 2.1.

A.1 Spence, Coburn and Pomonis (1992)

Empirical vulnerability curves for reinforced concrete pre-seismic code moment resisting frames are derived by Spence et al (1992). The building damage data used in the curve generation derives from observational post-earthquake surveys of 13 worldwide seismic events (Figure A. 1). A number of surveyed damage distributions (datasets) between 20 and 100 can be inferred from the source to have been used in the curve regression. Normal cumulative distributions are used for the vulnerability curves; the Parameterless Scale of Intensity (PSI, Pomonis et al. 1992) is used to represent ground motion severity and the Medvev Sponheuer Karnik (MSK) scale for the damage state evaluation.

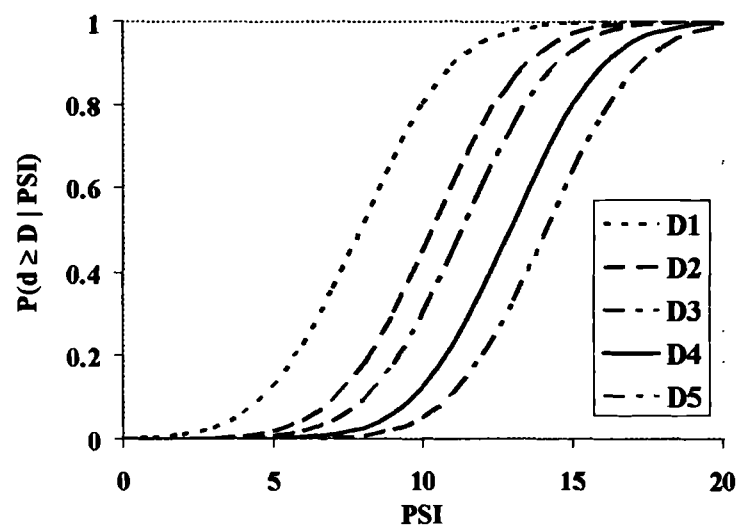


Figure A. 1: Vulnerability curves for non-seismically designed reinforced concrete moment resisting frames by Spence et al. (1992).

The PSI scale is proposed by Pomonis, Coburn and Spence 1992, as a continuous alternative to the Modified Mercalli Intensity (MMI) scale for use in the characterisation of ground motion damage potential in vulnerability curves. The

PSI scale is evaluated from observation of the damage distribution in buildings and their relative performance with respect to brick masonry constructions in the same area. As the intensity value is evaluated directly from the damage distribution observed in buildings, it does not purely represent the ground motion damage potential, but rather the comprehensive effects of ground motion, soil conditions and structural vulnerability. Although no attenuation relationships exist for the prediction of this intensity value, the authors propose simple linear relationships between PSI and acceleration based ground motion parameters, (for which attenuation relationships exist). The relationships are derived from regressions on observational data obtained from European sites where post-earthquake damage evaluations have been carried out in close proximity to ground-motion recording instruments. A very limited number of such observations, (a maximum of 15), are used. This PSI calibration data forms a subset of the data used in the construction of the empirical vulnerability curves. The relatively small number of observations adopted in the curve derivation, leads to a significant improvement in the correlation of the regression data with the proposed vulnerability curves, compared to cases where ground motion parameters that are not derived from damage distributions are used.

MSK Damage State	D1	D2	D3	D4	D5
Confidence	0.7	0.9	0.5	0.8	1.0

Table A. 1: The confidence level associated with 1 standard deviation (of value 2.5) from the mean vulnerability curves of Spence et al. (1992).

The confidence limits for the curves, in terms of the percentage of points that lay within one standard deviation either side of the predicted mean curves, are reported in Table A. 1. The standard deviation of the probabilities of exceedence is assigned a value of 2.5 for all the curves. A good degree of confidence is seen for the curves associated with the D2, D4 and D5 damage state curves. However, a large scatter is observed for the other damage states, where a significant portion of the data lies outside the limits defined by the standard deviation.

A.2 Orsini (1999)

Orsini (1999), uses building damage data collected after the Irpinia earthquake (Italy, 1980), to construct vulnerability curves for different types of structures. In the case of reinforced concrete construction, the curves are based on observations of damage made in 50,000 apartments at 41 survey sites (Figure A. 2). The PSI and MSK scales are used to represent the ground motion and damage, respectively. Hence, the same problems regarding the inter-dependency of vulnerability curve axes as in Spence et al. (1992), apply to the curves of Orsini (1999). The use of apartments as the structural damage unit, rather than buildings, is a drawback of the curves. No consideration of the difference in damage accumulation rate of disparate structural systems is made, and as inventory data is not generally categorised in terms of apartments, the curves are of limited applicability in pre-earthquake assessments. The use of a single event for the source of damage data further limits the application of the curves to situations with similar magnitude events and geotechnical environments.

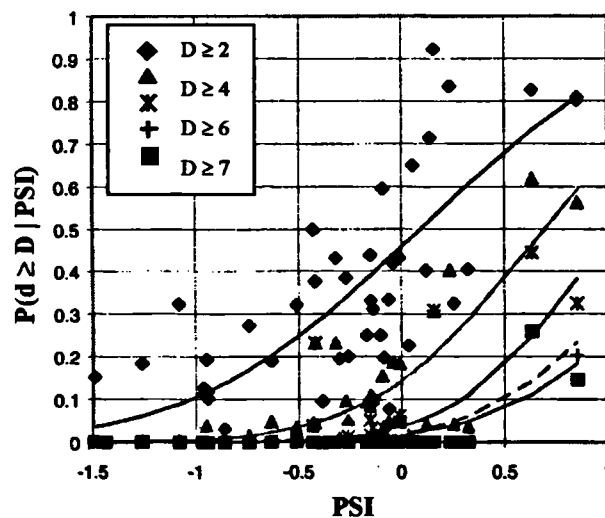


Figure A. 2: Vulnerability curves for reinforced concrete moment resisting frames by Orsini (1999) derived from observations of damage after the Irpinia earthquake (Italy, 1980).

Damage survey data collected over a limited area following a single event can be assumed to represent a homogenous building sample. Nevertheless, the scatter associated with the empirical curves of Orsini (1999), is very large. Correlation coefficients (R^2 values) between 0.41 and 0.49 are reported, and are attributed by the author to the use of badly defined damage limit states in the original survey.

A.3 Yamazaki and Murao (2000)

Yamazaki et al. (2000), use building damage data collected after the Kobe earthquake (Japan, 1995), to derive empirical vulnerability curves for reinforced concrete structures. Cumulative lognormal distributions describe the vulnerability curve shapes, the Architectural Institute of Japan (AIJ) classification of damage is adopted and peak ground velocity (pgv) is used as the ground motion parameter. The damage statistics adopted are of high quality and derive from a detailed survey of all the districts within the Nada Ward, carried out by the AIJ. The surveyed buildings are categorised according to material, age and height. However, the damage statistics for reinforced concrete structures are reported at only 16 locations and refer to a total sample size of 3,814 buildings. Yamazaki and Murao (2000), derive vulnerability curves for reinforced concrete structures built according to different codes of practice (Figure A. 3). However, no distinction is made between RC structures with different lateral load resisting systems. Only the two damage states of “Moderate” and “Heavy” are considered by the authors, which are insufficient for a complete assessment of structural performance.

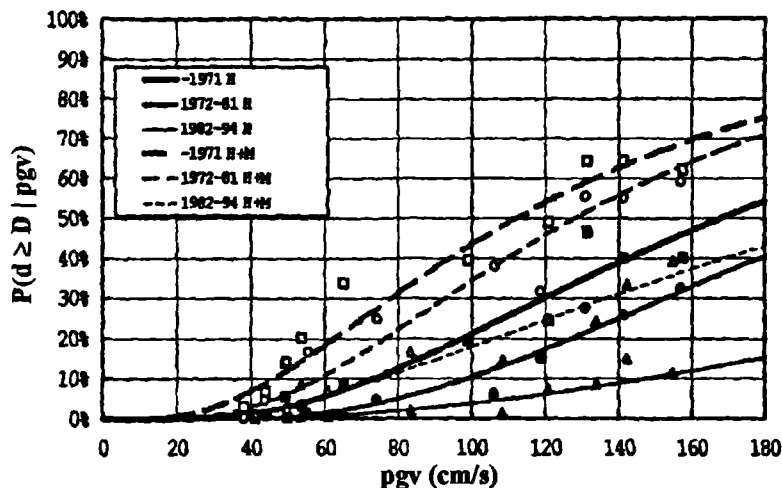


Figure A. 3: Vulnerability curves for reinforced concrete structures with different ages by Yamazaki and Murao (2000).

Although the aerial distribution of building damage is known in detail, an equal degree of accuracy in the distribution of the ground motion parameter is not available. To overcome this, the authors adopt the iterative procedure of Figure A.

4 to derive the vulnerability curves. Within the iterative process, the *pgv* distribution proposed by Yamaguchi and Yamazaki (1999) is re-estimated using preliminary vulnerability functions obtained from observed distributions of damage in wood-frame buildings. The resulting ground motion parameter distribution is used to draw new vulnerability curves, and the process of *pgv* re-estimation is repeated.

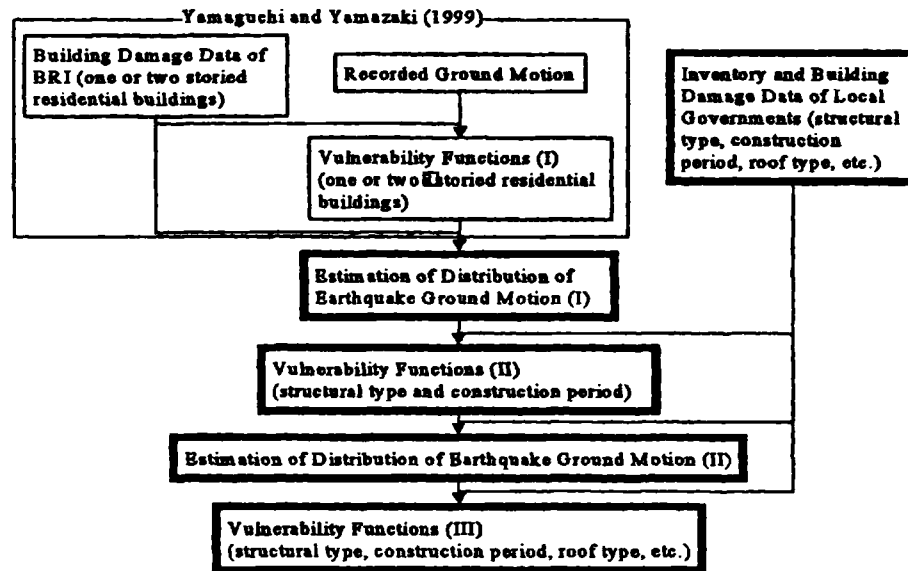


Figure A. 4: Flowchart of the iterative vulnerability curve derivation procedure used by Yamazaki and Murao (2000).

The aerial distribution of *pgv* obtained after application of the iterative process, differs significantly from the original, (Figure A. 5). The derived *pgv* distribution may, therefore, no longer be considered as representative of the actual ground motion, but more akin to an intensity distribution. The correlation coefficients reported by Yamazaki and Murao (2000) range between 0.84 and 0.97. These values are more than double those reported by Orsini (1999), which also relate to a single earthquake event. This is partly due to the increased quality of the survey data, but also due to the iterative method used for the curve prediction. Though the two curve sets are not strictly comparable, an idea of the effect on observational data scatter, of the vulnerability derivation methodology used by Yamazaki and Murao (2000), can be obtained from comparison of their high damage state curves. As these damage states are easily identifiable, the corresponding buildings are less prone to misclassification. The influence of data

quality on the data scatter is thus removed. Comparison of the high damage state correlation coefficients for the two sources shows that the iterative method adopted by Yamazaki and Murao (2000) leads to misleadingly high confidence in the risk assessment results.

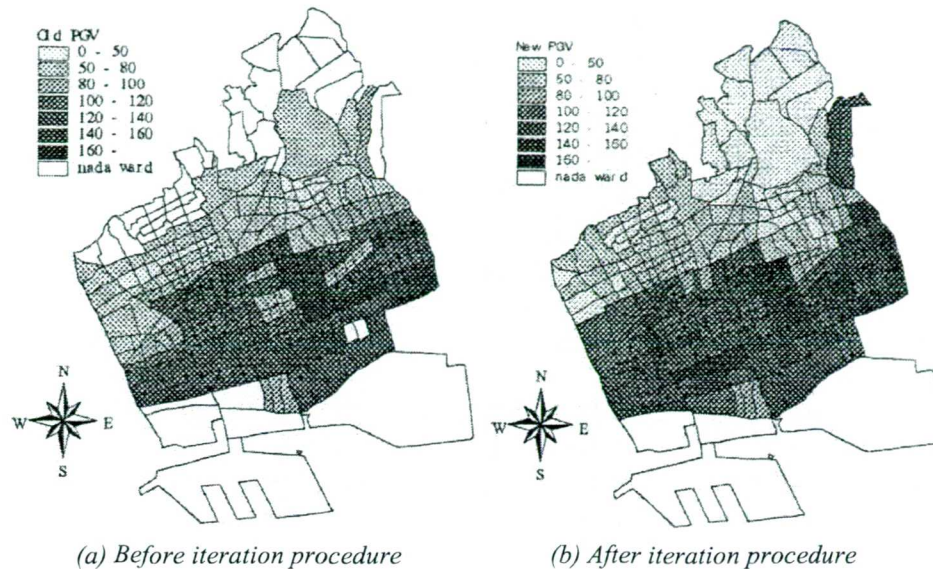


Figure A. 5: The predicted pgv distribution for the Nada Ward during the Kobe earthquake (Japan, 1995); (a) from Yamaguchi and Yamazaki (1999) and (b) following the iterative process adopted by Yamazaki and Murao(2000) in their vulnerability curve generation.

A.4 Miyakoshi, Hayashi, Tamura and Fukuwa (1997)

Miyakoshi et al. (1997), develop vulnerability curves for reinforced concrete buildings based on damage data from the Kobe earthquake, (Japan, 1995). Observations of building damage collected throughout the Hanshin area are used in the curve construction. These observations are more numerous and cover a wider range of ground motion than those used by Yamazaki and Murao (2000). The quality of the survey data enables the authors to derive curves for reinforced concrete structures built to different codes. However, only the severe and moderate damage limit states are considered. These are reported to have average correlation coefficients with the derivation data, of 0.87 and 0.96, respectively. The high correlations are attributed to the high quality of the observational damage data and to the inter-correlation between the vulnerability curve axes. The latter arises from their use of the *pgv* distribution of Hayashi (1997) in the ground motion determination, which is derived from observations of structural damage.

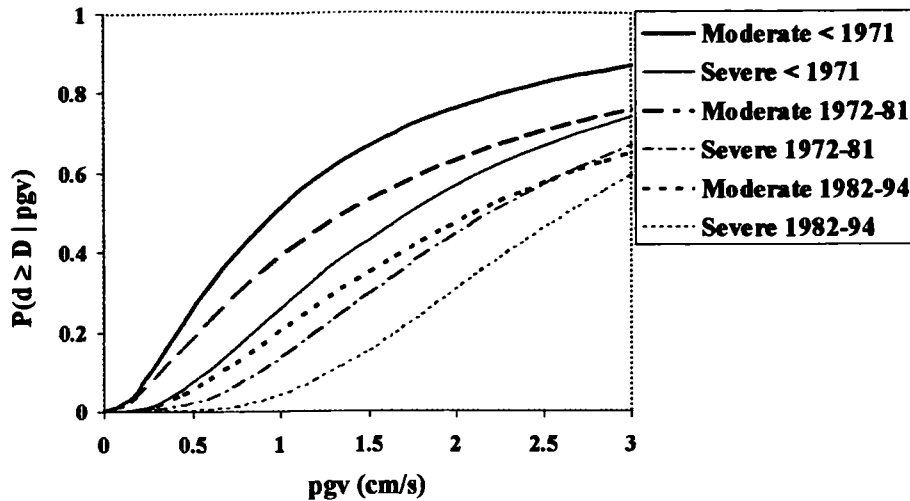


Figure A. 6: Empirical vulnerability curves for Japanese RC structures of different ages derived by Miyakoshi et al. (1997).

The main problem posed by these curves stems from their use of a parameter other than structural damage for building performance determination. Losses in property values of -20% and -50%, are used to represent the moderate and severe damage states, respectively. However, the use of economic criteria is seen by Miyakoshi et al. (1997) to result in a consistent overestimation of the observed structural damage in buildings affected by the Kobe earthquake, (Japan, 1995).

A.5 ATC-13, (ATC 1985)

The ATC-13 document was developed by the Applied Technology Council (ATC 1985) for the estimation of economic seismic risk in California, USA. The document presents damage probability matrices (DPM), which report the damage factor, (DF), expected in 36 classes of buildings, for modified mercalli intensities (MMI) between VI to XII. DF is defined as the ratio of dollar loss to replacement value, expressed in percentage. The DF values in the DPMs are determined from estimates of expected damage made by over 70 experts in the field of earthquake engineering. Each expert is required to provide best, low and high estimates of the DF at each MMI value. These are assumed to correspond to the mean, lower and upper 90% confidence bounds, and are combined to define Beta probability distribution functions (pdf) for each MMI. The DPMs are hence determined

through the discretisation of each PDF, according to the ranges of DF shown in Table A. 2. Singhal and Kiremidjian (1997), converted the DPMs into vulnerability curves assuming a lognormal distribution of damage. Examples of the curves resulting for different classes of reinforced concrete structures are shown in Figure 2.1.

Damage State	DF Range (%)	Central DF (%)	Mid-rise moment resisting non-ductile RC frame Probability of damage in %						
			VI	VII	VIII	IX	X	XI	XII
None	0	0	0.3						
Slight	0-1	0.5	30.9	0.3					
Light	1-10	5	68.8	96.9	33.6	1.9	0.2		
Moderate	10-30	20		2.8	65.7	65.1	30.8	3.6	0.5
Heavy	30-60	45			0.7	33.0	67.7	70.0	27.9
Major	60-100	80					1.3	26.4	71.2
Destroyed	100	100							0.4
MDF ⁽¹⁾			3.6	5.4	15.1	28.0	37.7	53.3	70.0

⁽¹⁾ Mean Damage Factor, calculated as the average of the central DFs, weighted according to the probabilities of each central DF.

Table A. 2: Example of a DPM in ATC-13 (ATC 1985)

ATC-13 provides DPMs for structural classes that are typical of the building stock in California. The most common European reinforced concrete structural category of infilled moment resisting frames is not included in ATC-13. Furthermore, knowledge of local building configurations, material performance and construction practice are implicitly included in the expert estimates used for the DPMs. No method exists for predicting the degree of conservatism associated with the original DF estimates, and confidence bounds derived from the DPMs may be of little practical significance. The use of an economic damage index also introduces temporal and aerial specificity. Therefore, the application of these curves in the seismic assessment of other areas is compromised.

A.6 Onose (1982)

Onose (1982), uses a reliability-based method for the assessment of reinforced concrete buildings. The damage state exceedence probabilities used for the vulnerability curve definition, are obtained from a comparison of the statistical distributions of building seismic resistance and ground motion demand variables within a survey area. The Shiga (1977), shear strength coefficient (C_{ys}) and a seismic force parameter based on peak ground acceleration (pga), are used as the resistance and demand variables, respectively. C_{ys} is evaluated from the ratio of wall and column cross-sectional areas in a building with respect to the total floor space, and is given by equation A.1.

$$C_{ys} = \frac{12A_c + 33A_w}{1300 \sum A_f} \quad [A. 1]$$

In Eq.A.1, A_c is the column area of the first storey (cm^2) and $\sum A_f$ is the total floor area above the first floor (m^2). A_w is the wall area of the first storey (cm^2), taken as the smaller of the two values for the longitudinal and transverse directions, and includes the contribution of non-structural walls. The factors of 12 and 33 describe the ultimate shear strength (kg/cm^2) of the columns and walls respectively, whereas the value of 1300 represents the unit floor weight (kg/m^2). The distribution of C_{ys} is evaluated from observations of its variation within a sample of 405 low and medium sized reinforced concrete frames built in Japan between 1965 and 1970. Log-normal curves with a constant coefficient of variation of 0.45 are found to fit the resistance variable distribution of the buildings. Different structural heights are considered separately, through the use of a mean value of C_{ys} , (μ_{Cys}), for the resistance parameter distribution, which depends on the number of stories (N):

$$\mu_{Cys} = 4.9899 \left(\frac{1}{N^2} \right) - 0.9645 \left(\frac{1}{N} \right) + 0.9768 \quad [A. 2]$$

The demand variable distribution is determined from observations of the variation in acceleration magnification factor made from 206 Japanese earthquake records with pga values greater than 0.3g. An approximately constant coefficient of variation of 0.4 is observed in the distributions obtained for natural vibration periods below 0.4s (corresponding to low-rise frames). Over this range of natural periods, the mean value of acceleration magnification factor is also seen by the authors to be approximately constant and takes a value of 2.0. The variation in the seismic force is, therefore, modelled by a log-normal distribution with COV of 0.4 and mean of $(2.pga)$.

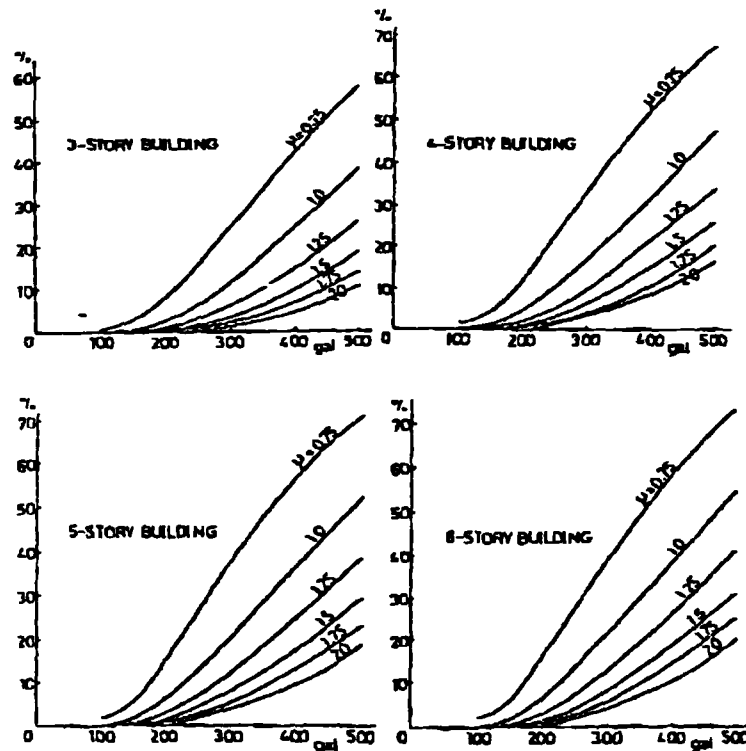


Figure A. 7: Vulnerability curves for different height reinforced concrete structures by Onose (1982)

Ductility factor values, assigned from judgement, are used to indicate the building damage level. The forces representing the thresholds of each damage state are obtained from the product of the building resistance and ductility factors. The probability of exceedence of each damage limit state is found from the area of overlap between the resistance and demand curves, for forces exceeding the threshold value. The resulting vulnerability curves are shown in Figure A. 7. Although the damage index is not calibrated to experimental observations of damage, the vulnerability predictions resulting from the curves are shown by the

authors to give reasonable results when compared to damage observations made in three districts of Sendai city (Japan) after the Miyagi-ken Oki earthquake, (Japan, 1978).

Even though the methodology is applicable to any region, the derived curves are specific to the constructions and seismicity in Japan. Prior knowledge of the distribution of building resistance in the assessed area is required by the method. Furthermore, the use of a ductility factor as a damage indicator is not deemed suitable, as it is unable to account for failures caused by damage concentration at the storey level. Use of the same acceleration amplification factor for all structures is also regarded to be too conservative for the case of the higher period structures.

A.7 Mosalam, Ayala, White and Roth (1997)

Mosalam et al. (1997), derive analytical vulnerability curves for low-rise reinforced concrete frames, with and without infill walls (Figure A. 8). The buildings are representative of those found in Memphis (USA), are of 2-bay 2-storey configuration and comprise many of the detailing deficiencies typical of gravity-load designs. Pushover analyses are carried out on the models in order to define the dynamic properties of equivalent single degree of freedom (SDOF) systems. Uncertainty in the response due to material property variation is taken into account through the definition of 200 SDOFs with differing dynamic properties. Peak ground acceleration (*pga*), is used as the ground motion parameter and uncertainty in ground motion is accounted for through the use of an extensive number of earthquake records in the time-history analysis of the SDOFs. Two sets of artificial earthquake records are used, denoted by CERI (Hwang and Huo 1994) and LDEO (Horton et al. 1996), consisting of 600 and 200 earthquake records, respectively. All the records contained in these databases have *pga* values less than 0.5g. This type of earthquake excitation is expected to cause significant damage to infill walls, but produce small relative storey displacements in the low-rise infilled frames considered. Maximum inter-storey drift ratio is used as the damage state indicator. The parameter values defining the damage limit state boundaries are based solely on judgement, with one tenth of the values used for

bare frames being adopted in the infilled frame assessment. The values of inter-storey drift used to define the damage limit states suggest that, despite being assigned names such as “Heavy” and “Major”, the vulnerability curves actually correspond to significantly lower damage limit states.

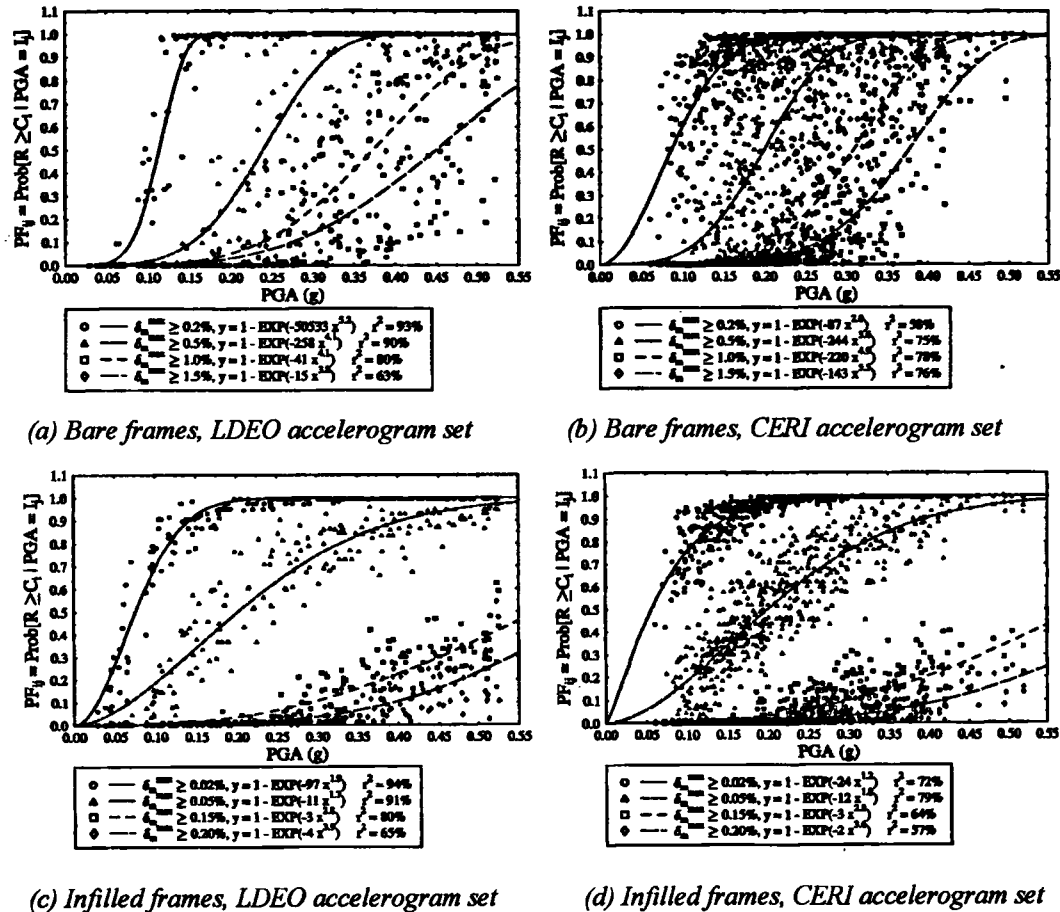


Figure A. 8: Vulnerability curves for low and medium rise RC buildings by Mosalam et al. (1997)

Damage State	Insignificant	Moderate	Heavy	Major
Bare frames (CERI)	0.58	0.75	0.78	0.76
Infilled frames (CERI)	0.72	0.79	0.64	0.57
Bare frames (LDEO)	0.93	0.90	0.80	0.63
Infilled Frames (LDEO)	0.94	0.91	0.80	0.65

Table A. 3: Summary of the correlation coefficients (R^2) of the different damage state vulnerability curves of Mosalam et al (1997), with their derivation data.

Despite the fact that the vulnerability curves are derived from analysis, a significant scatter is observable in the exceedence probabilities around the

regressed curves, (Table A. 3). This is due to the uncertainty included for material properties and ground motion.

A.8 Singhal and Kiremidjian (1997)

Singhal and Kiremidjian (1997), derive separate analytical vulnerability curves for low-, medium- and high-rise reinforced concrete bare frames in California, USA. A rigorous, computationally intensive, analytical procedure that includes both ground motion and structure capacity uncertainty is adopted. Thus, the curves can be regarded as the most reliable analytical vulnerability curves in published literature to date.

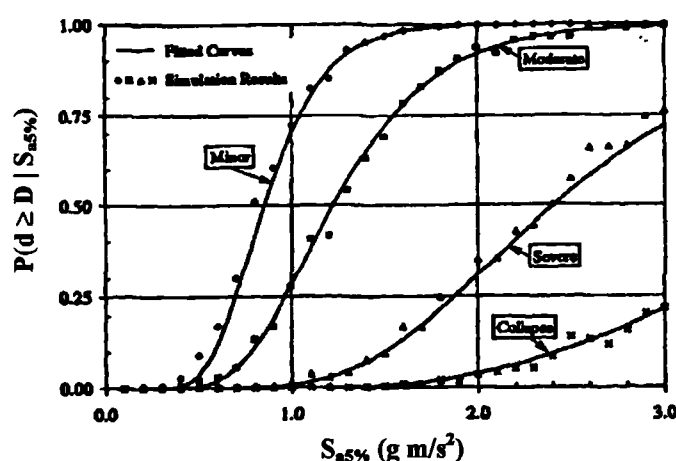


Figure A. 9: Analytical vulnerability curves for mid-rise bare reinforced concrete frames by Singhal and Kiremidjian (1997)

In the damage statistics generation, numerous non-linear time-history analyses are carried out on three 5-bay model frames of 2, 5 and 12 storeys height, respectively. The frames are designed according to the SEOAC (1990) recommendations for special moment resisting frames and the Park and Ang (1985a) damage index is used to assess the damage limit state of the buildings. The average spectral acceleration ordinate values, in the 0.1-0.5s, 0.5-0.9s and 0.9-2.5s period ranges, are used to represent the ground motion for the three structural classes, respectively. Non-linear time-history analyses are carried out on the model structures using 100 artificial time-histories. For each spectral acceleration value chosen, the records are scaled to have the same average spectral acceleration value across the period band corresponding to the class of frame being analysed.

Lognormal cumulative distributions are used to describe the vulnerability curve shapes, (Figure A. 9). Uncertainty in the capacity parameters is taken into account through treatment of the concrete compressive strength and steel yield strength, as normally and log-normally distributed random variables, respectively. The resulting variation in the Park and Ang (1985a) global damage index , at each ground motion level, is used to define a set of confidence bounds (Figure A. 10).

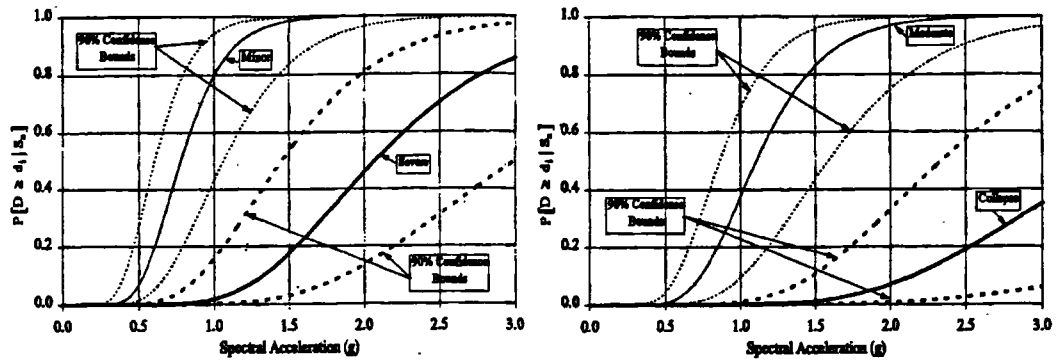


Figure A. 10: The 90% confidence bounds of the analytical vulnerability curves for mid-rise bare reinforced concrete frames by Singhal and Kiremidjian (1997)

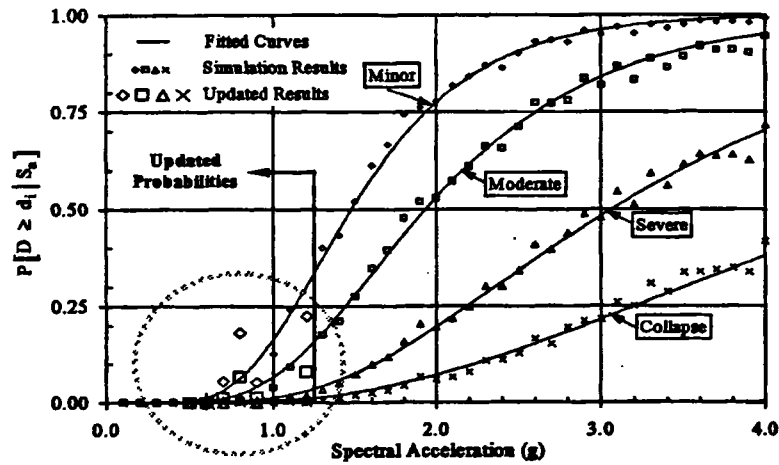


Figure A. 11: Hybrid vulnerability curves for low-rise bare reinforced concrete bare frames derived by Singhal and Kiremidjian (1997). The analytical exceedence probabilities that have been updated with observational data are shown in the dashed circle.

The vulnerability curves derived for low-rise frames are verified against damage observations made in 84 buildings following the Northridge earthquake (USA, 1994). From Figure A. 11, it can be seen that the data covers only a very limited range of ground motion values. Furthermore, the refinement of the damage statistics used is very low, as they derive from colour tagging surveys. The analytical curves for the low-rise frames are updated with the observational

damage statistics using a Bayesian updating technique. The updated statistics are represented by the larger point markers within the circled area of the vulnerability curves of Figure A. 11. It is observed that the inclusion of observational data significantly influences the vulnerability plot points and introduces scatter.

Appendix B

Comparison of Existing and Derived Vulnerability Relationships for Different Earthquake Magnitudes and Site-to-Source Distances, and their Prediction of Observed Damage Distributions for Single Events.

The pga and $S_{a5\%}(T)$ attenuation relationships of Ambrassey et al. (1996), Joyner and Boore (1988) and Youngs et al. (1997) are used to plot the vulnerability curves for Europe and the Middle East, America and Japan, respectively. Borzi et al. (1998) is used for all $S_{d5\%}(T)$ curves. The vertical axis represents the damage state exceedence probability in all plotted curves.

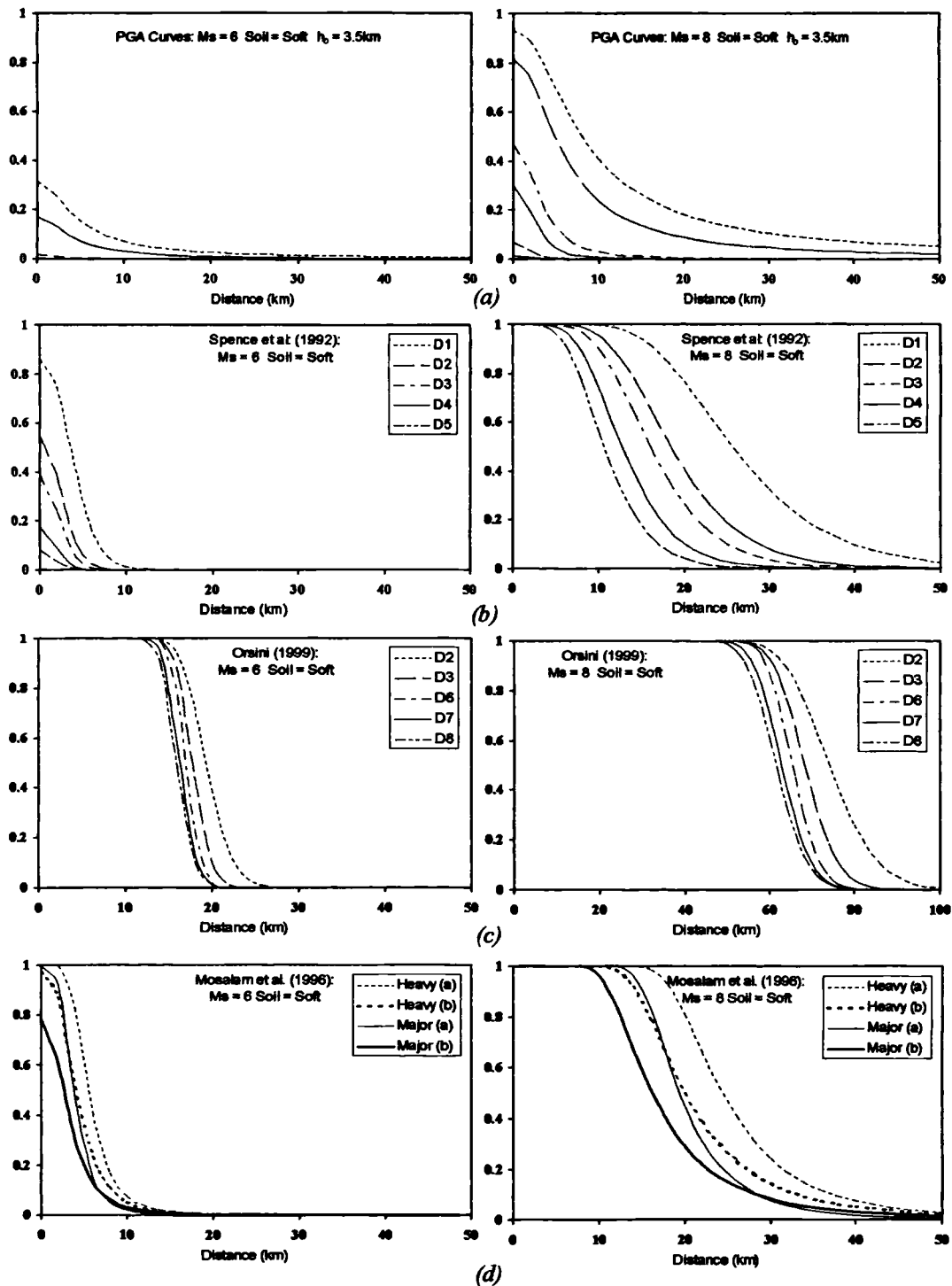


Figure B. 1: The variation in the vulnerability prediction with distance from the source, given by various relationships based on pga , for the surface magnitudes of 6 and 8, assuming soft soil conditions. (a) Homogeneous empirical relationships derived herein; (b) Spence et al. (1992); (c) Orsini (1999) ; (d) Mosalam et al. (1997).

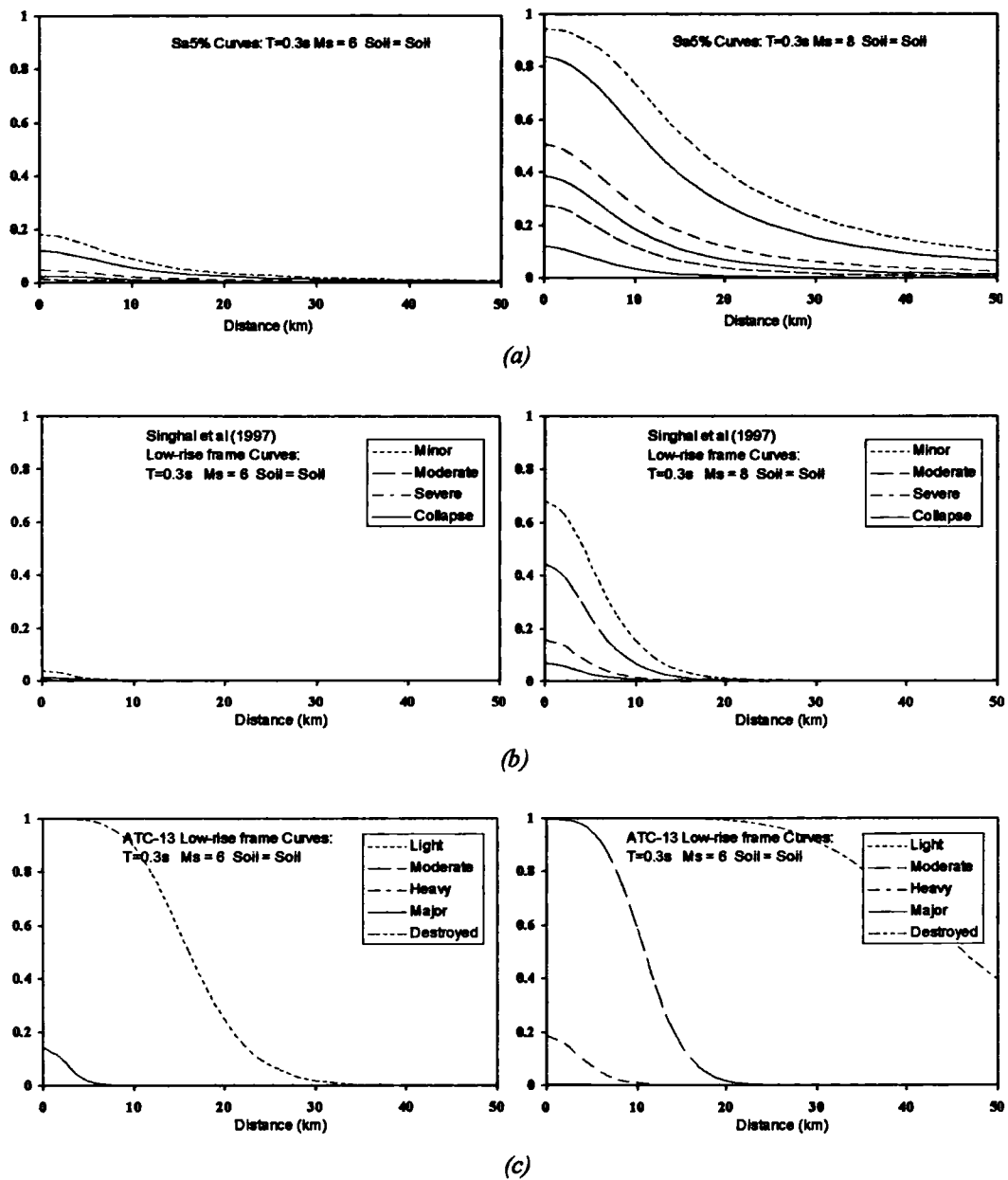


Figure B. 2: The variation in the vulnerability prediction with distance from the source, given by various relationships for low-rise RC structures, based on $S_{a5\%}(T=0.3s)$. Surface magnitudes of 6 and 8, and soft soil conditions are assumed. (a) Homogenised empirical relationships derived herein; (b) Singhal and Kiremidjian low-rise MRF curves (1997); (c) ATC-13 low-rise MRF curves (ATC, 1985).

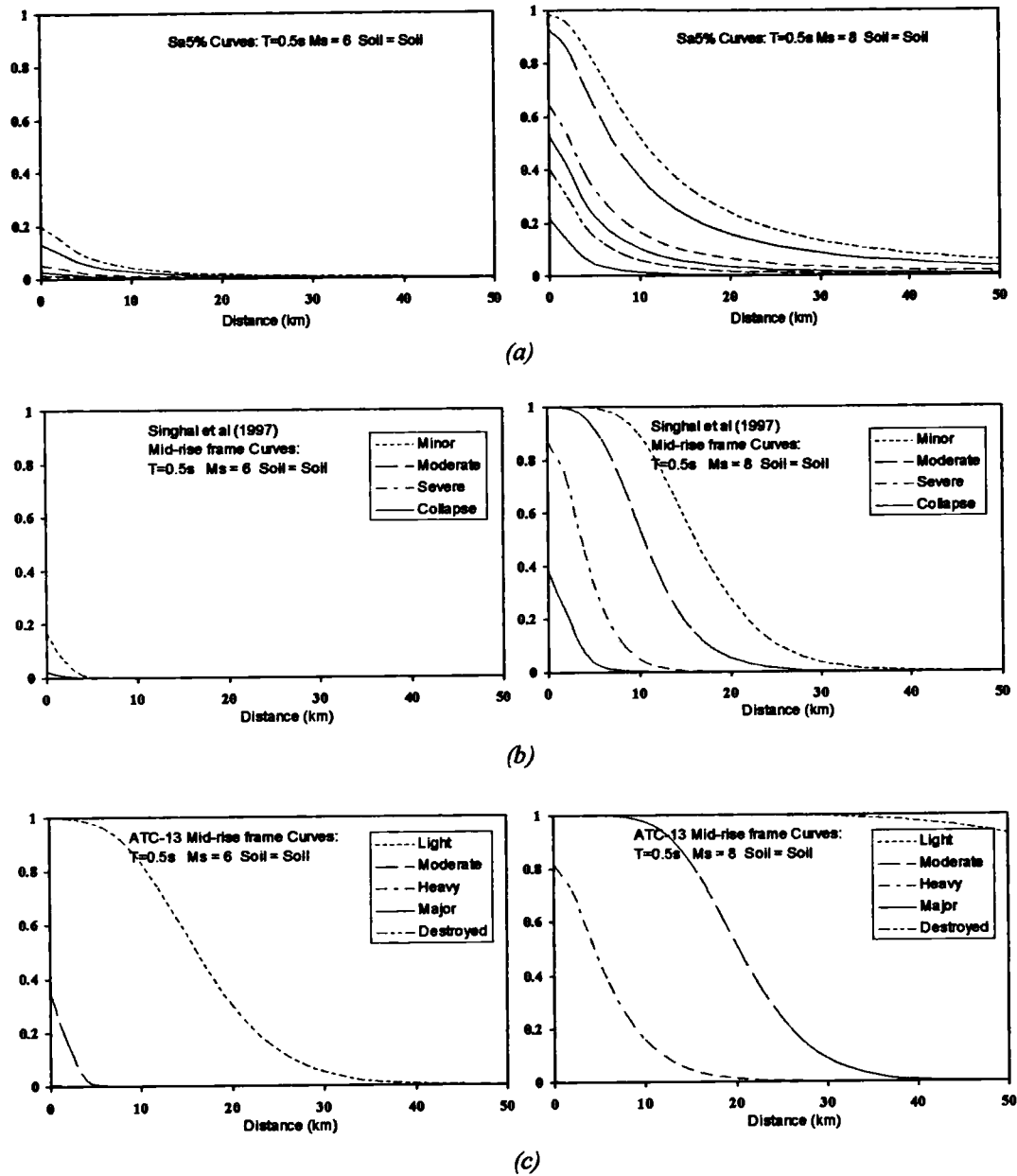


Figure B. 3: The variation in the vulnerability prediction with distance from the source, given by various relationships for medium-rise RC structures, based on $S_a 5\%$ ($T=0.5s$). Surface magnitudes of 6 and 8, and soft soil conditions are assumed. (a) Homogenised empirical relationships derived herein; (b) Singhal and Kiremidjian mid-rise MRF curves (1997); (c) ATC-13 mid-rise MRF curves (ATC, 1985).

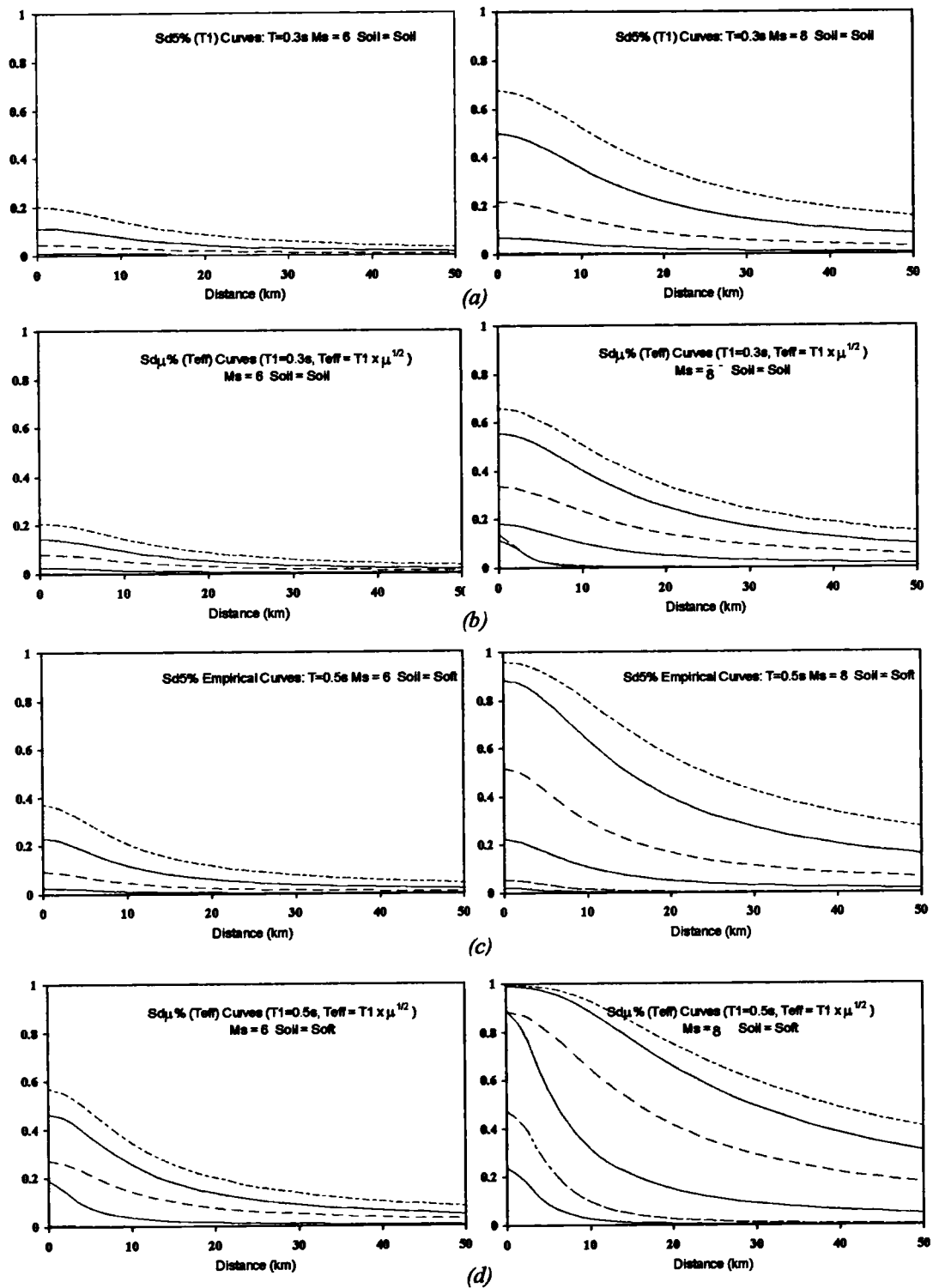


Figure B. 4: The variation in the vulnerability prediction with distance from the source, predicted by the derived homogenised empirical curves for general RC buildings. Surface magnitudes of 6 and 8, and soft soil conditions are assumed. (a) Empirical vulnerability curves for $S_{d5\%}(T=0.3s)$; (b) Empirical vulnerability curves for $S_{d_{\mu}\%}(T=0.3s)$; (c) Empirical vulnerability curves for $S_{d5\%}(T=0.5s)$; (d) Empirical vulnerability curves for $S_{d_{\mu}\%}(T=0.5s)$.

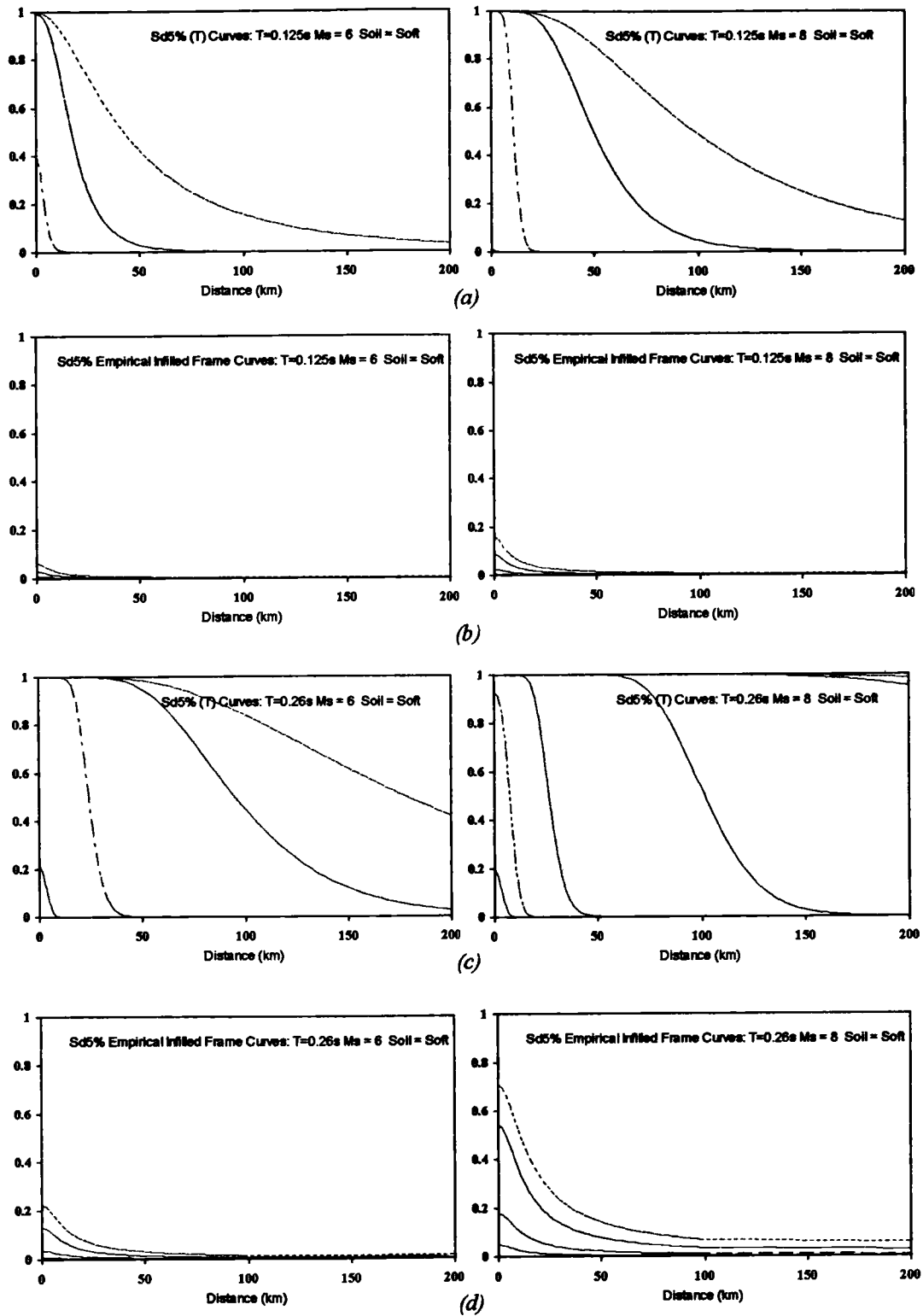


Figure B. 5: The variation in the vulnerability prediction with distance from the source, predicted by the derived infilled frame vulnerability curves. Surface magnitudes of 6 and 8, and soft soil conditions are assumed. (a) Analytical vulnerability curves with $S_{d5\%}(T)$ evaluated at $T=0.125s$; (b) Empirical vulnerability curves with $S_{d5\%}(T)$ evaluated at $T=0.125s$ (c) Analytical vulnerability curves with $S_{d5\%}(T)$ evaluated at $T=0.26s$; (d) Empirical vulnerability curves with $S_{d5\%}(T)$ evaluated at $T=0.26s$.

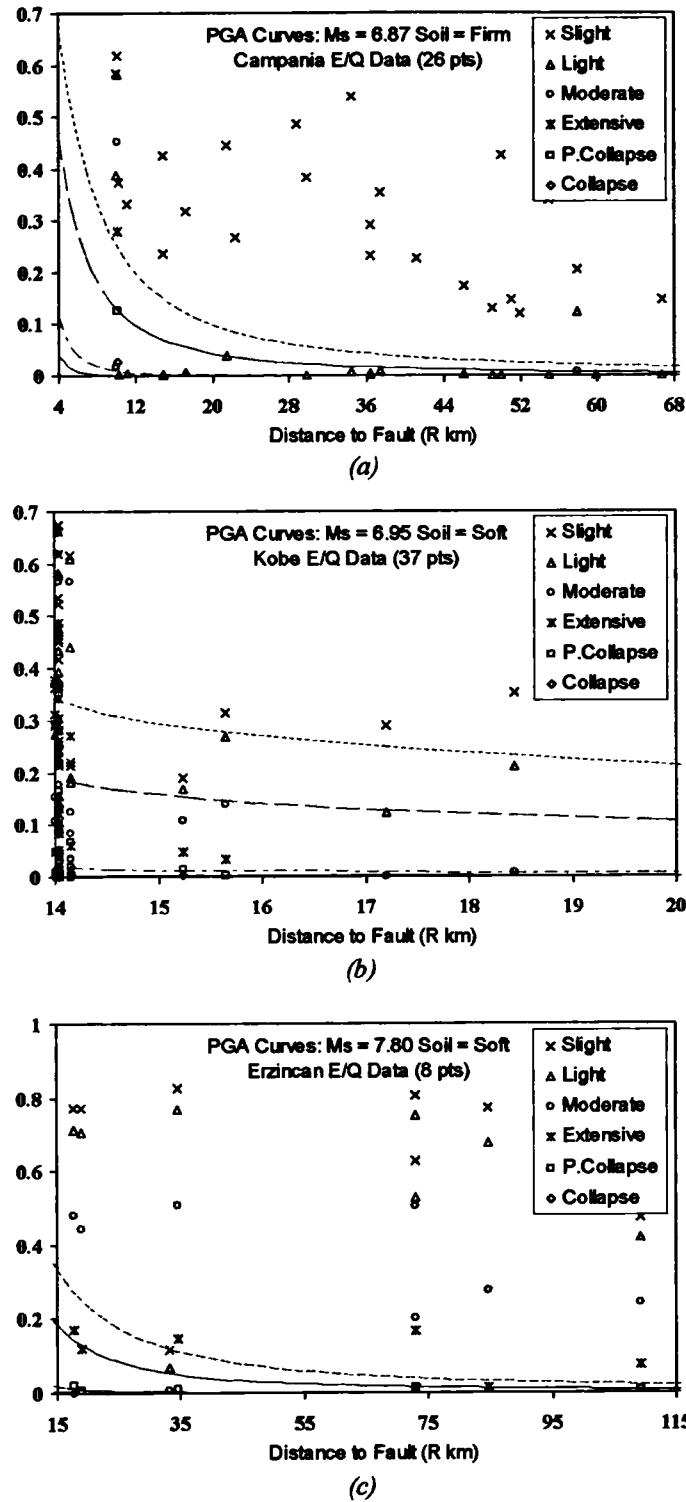


Figure B. 6: Comparison of the vulnerability prediction of the derived empirical pga curves at increasing distance from the earthquake source, with observational data for single events: Post-earthquake RC building damage observed after the (a) Campania (Italy, 1980) earthquake; (b) Kobe (Japan, 1995) earthquake; (c) Erzincan (Turkey, 1992) earthquake.

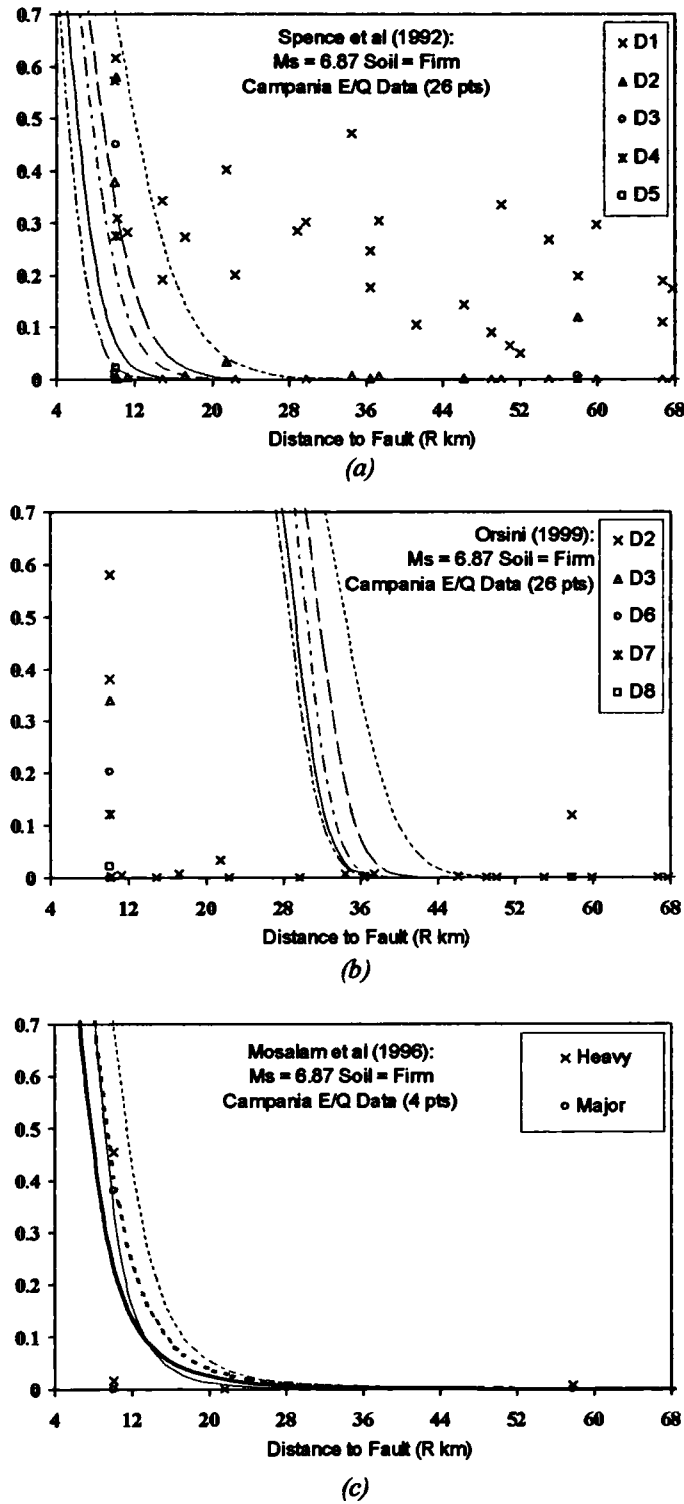


Figure B. 7: Comparison of the vulnerability prediction of existing curves based on p_{ga} , at increasing distance from the earthquake source, with observational data from the Campania (Italy, 1980) earthquake: (a) Spence et al. (1992) curves for RC frames; (b) Orsini (1999) curves for apartments; (c) Mosalam et al. (1997) curves for low-rise infilled RC frames.

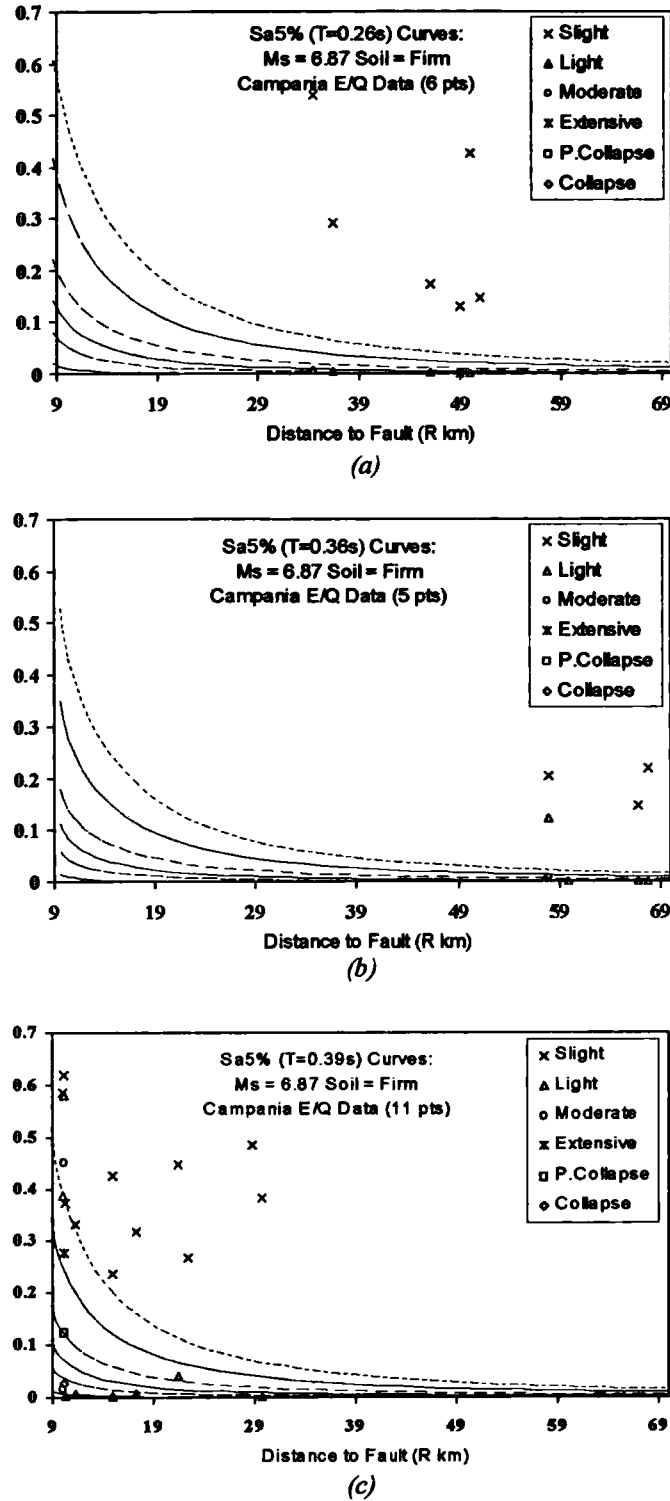


Figure B. 8: Comparison of the vulnerability prediction of the derived empirical curves based on $S_{a5\%}(T)$, at increasing distance from the earthquake source, with observational data from the Campania (Italy, 1980) earthquake: (a) Data for low-rise RC buildings with periods of 0.26s; (b) Data for mid-rise RC buildings with periods of 0.36s; (c) Data for mid-rise RC buildings with periods of 0.39s.

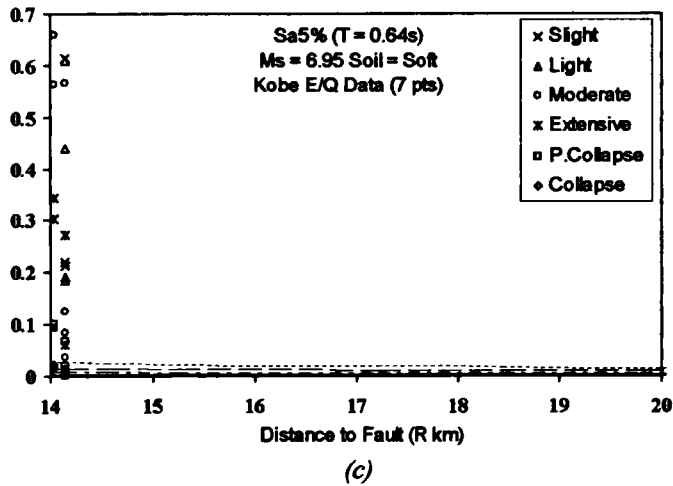
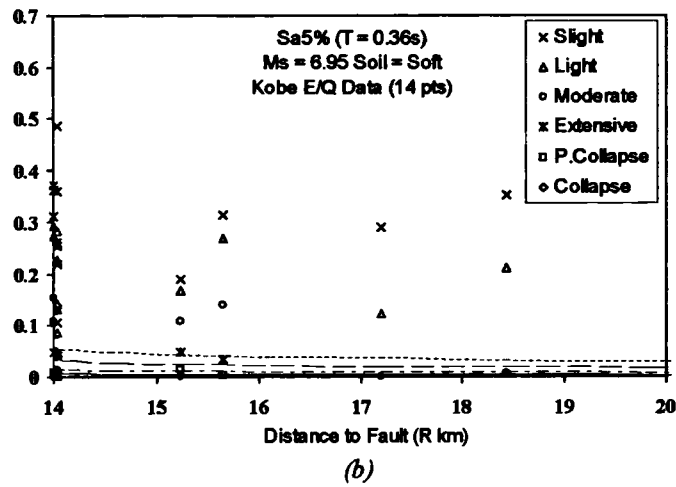
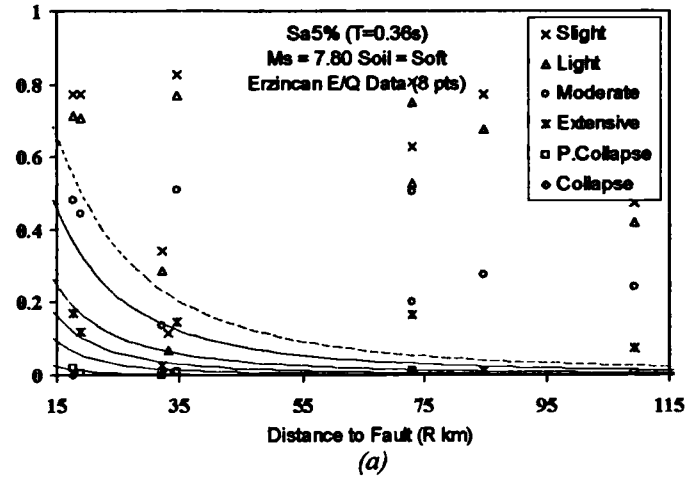
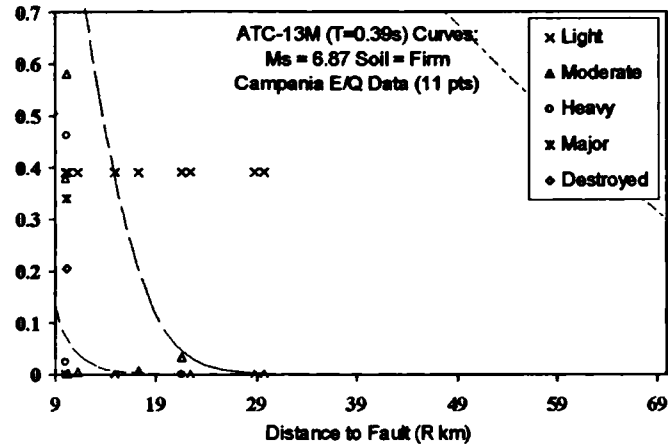
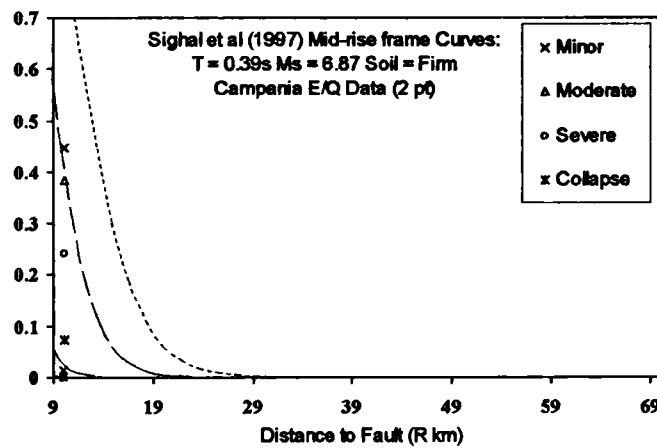


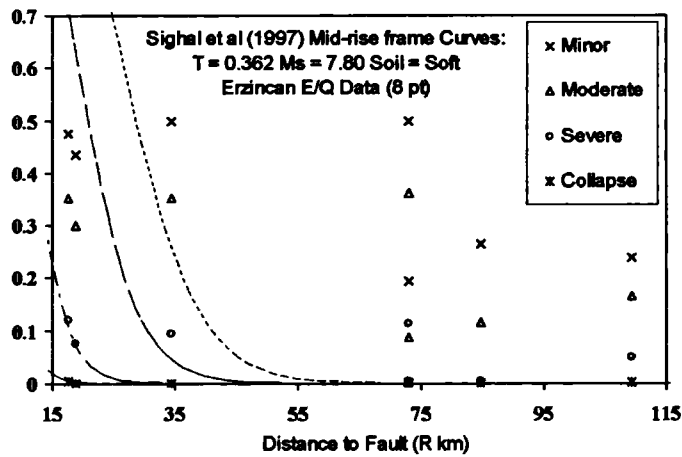
Figure B. 9: Comparison of the vulnerability prediction of the derived general structure empirical curves based on $S_{a5\%}(T)$, at increasing distance from the earthquake source, with observational data from different events: (a) Data for mid-rise RC buildings with periods of 0.36s affected by the Erzincan (Turkey, 1992) earthquake; (b) Data for mid-rise RC buildings with periods of 0.36s affected by the Kobe (Japan, 1995) earthquake; (c) Data for high-rise RC buildings with periods of 0.64s affected by the Kobe (Japan, 1995) earthquake.



(a)



(b)



(c)

Figure B. 10: Comparison of the vulnerability prediction of existing curves based on $S_{avg}(T)$, at increasing distance from the earthquake source, with observational data from different events: (a) ATC-13 (ATC, 1985) curves for mid-rise MRFs, with data for the latter structures, with periods of 0.39s, affected by the Campania (Italy, 1980) earthquake; (b) Singhal and Kiremidjian (1997) curves for mid-rise bare frames, with data for the latter structures, with periods of 0.39s, affected by the Campania (Italy, 1980) earthquake; (c) Singhal and Kiremidjian (1997) curves for mid-rise bare frames, with data for the latter structures, with periods of 0.36s, affected by the Erzincan (Turkey, 1992) earthquake.

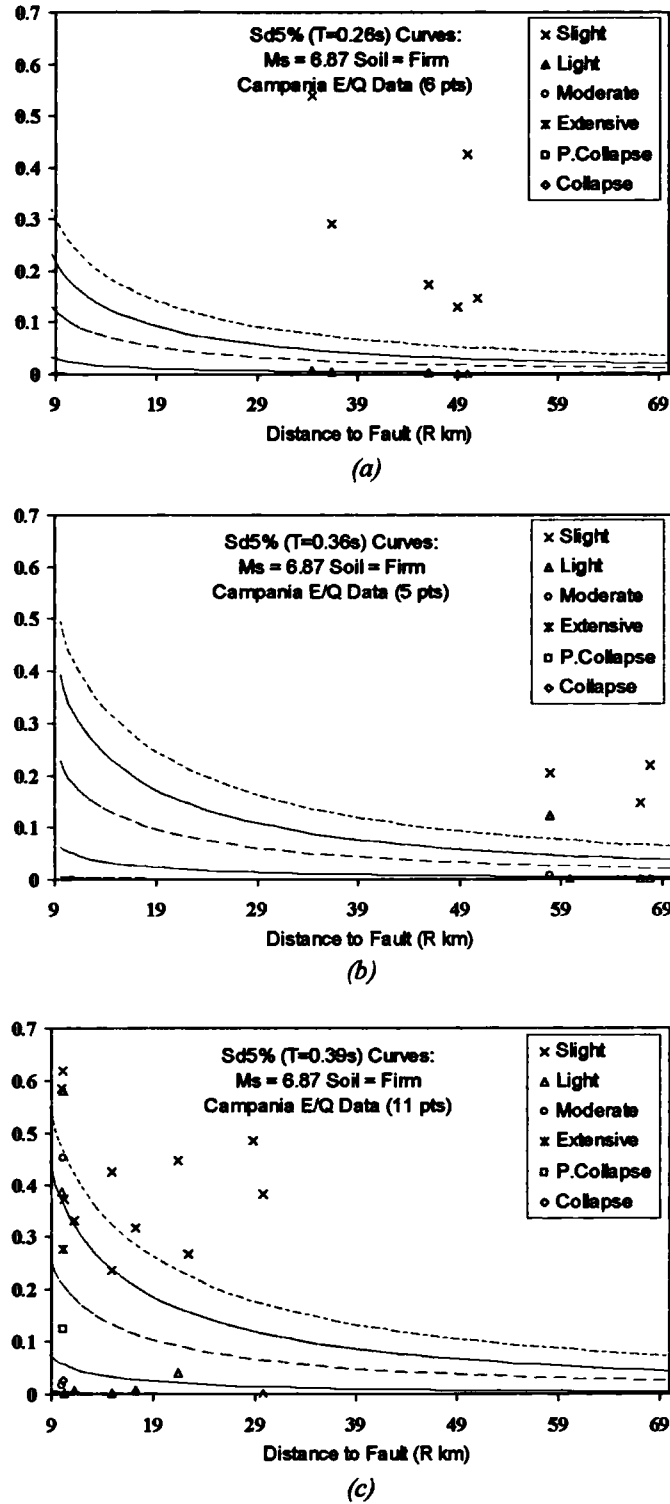
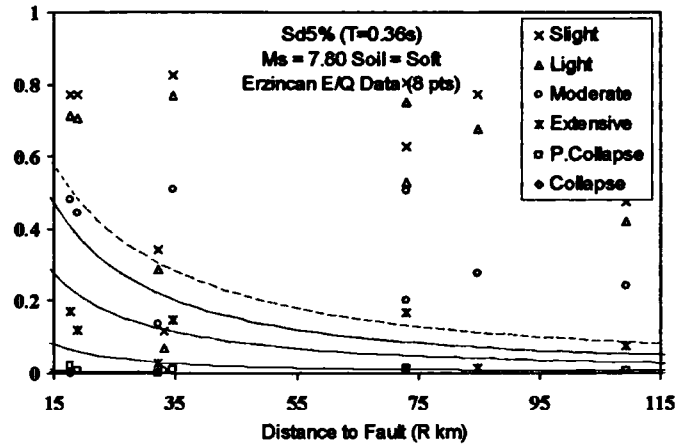
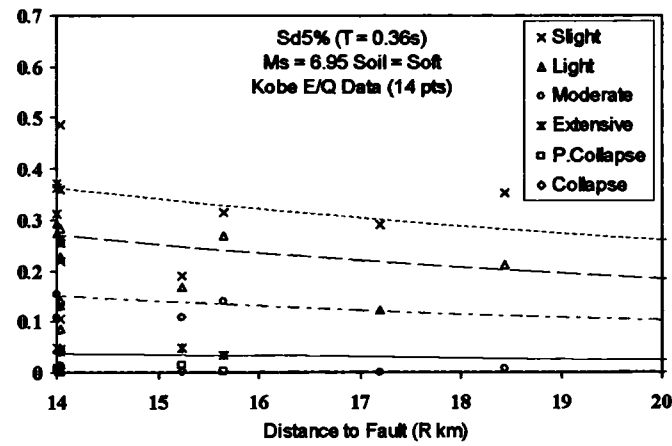


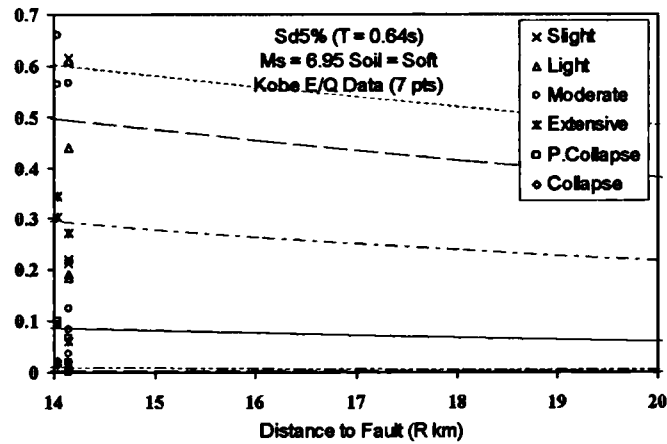
Figure B. 11: Comparison of the vulnerability prediction of the derived empirical general structure curves based on $S_{d5\%}(T)$, at increasing distance from the earthquake source, with observational data from the Campania (Italy, 1980) earthquake: (a) Data for low-rise buildings with periods of 0.26s; (b) Data for mid-rise buildings with periods of 0.36s; (c) Data for mid-rise buildings with periods of 0.39s.



(a)

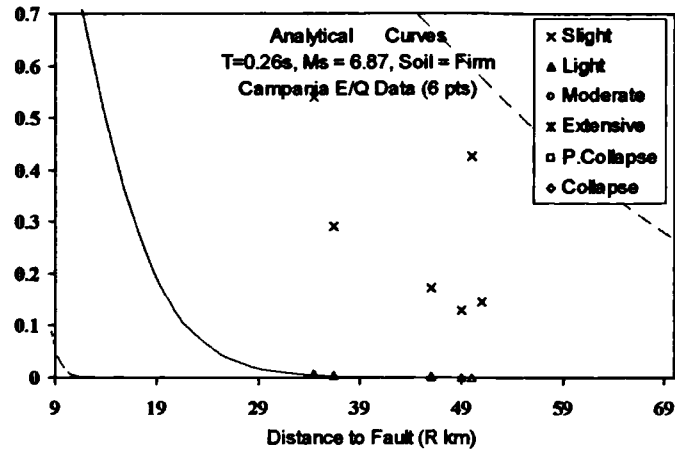


(b)

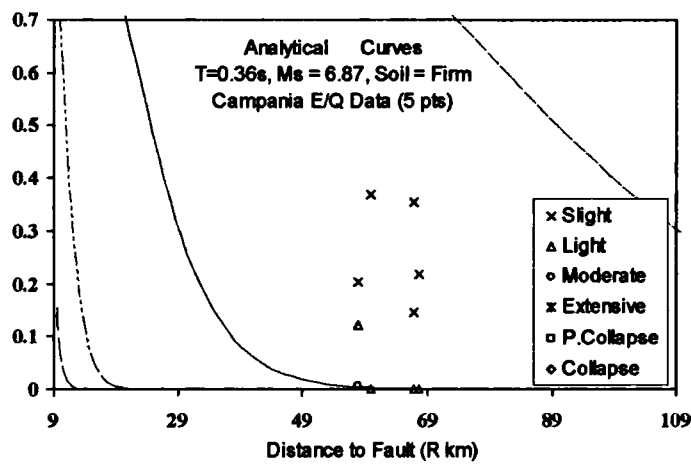


(c)

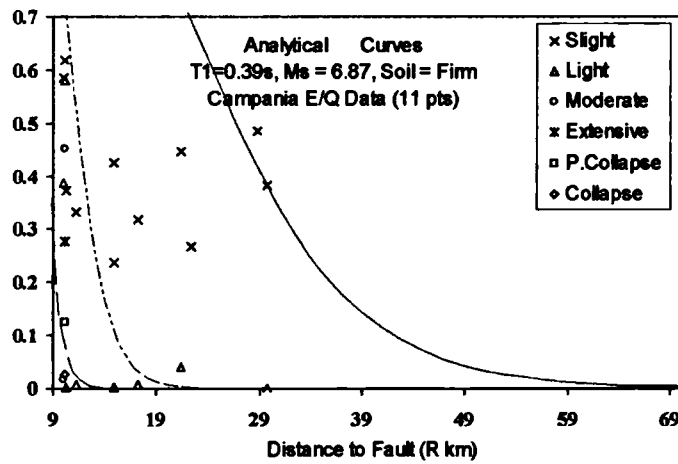
Figure B. 12: Comparison of the vulnerability prediction of the derived general structure empirical curves based on $S_{d5\%}(T)$, at increasing distance from the earthquake source, with observational data from different events: (a) Data for mid-rise RC buildings with periods of 0.36s affected by the Erzincan (Turkey, 1992) earthquake; (b) Data for mid-rise RC buildings with periods of 0.36s affected by the Kobe (Japan, 1995) earthquake; (c) Data for high-rise RC buildings with periods of 0.64s affected by the Kobe (Japan, 1995) earthquake.



(a)



(b)



(c)

Figure B. 13: Comparison of the vulnerability prediction of the derived analytical low-rise infilled RC frame curves based on $S_{a5\%}(T)$, at increasing distance from the earthquake source, with observational data from the Campania (Italy, 1980) earthquake: (a) Data for low-rise buildings with periods of 0.26s; (b) Data for mid-rise buildings with periods of 0.36s; (c) Data for mid-rise buildings with periods of 0.39s.

Appendix C

Summary of Existing Local and Global Damage Indices

Name	Evaluation	Damage state definition	Type ⁽¹⁾	δ_{max} ? ⁽²⁾	CD? ⁽³⁾	Comments
Rotation Ductility Ratio (θ_{DR})	$\mu_{\theta} = \theta_{max} / \theta_y$ Where θ_{max} and θ_y are the maximum and yield rotations.	Dependent on member and section geometry, and material properties.	D	Yes	No	Does not account for cumulative damage. Difficulty in evaluating the yield rotation, i.e. in taking into account bond slip and shear contributions.
Curvature Ductility Ratio (ϕ_{DR})	$\mu_{\phi} = \phi_{max} / \phi_y$ Where ϕ_{max} and ϕ_y are the maximum and yield curvatures.	Dependent on member and section geometry, and material properties.	D	Yes	No	Same problems as for θ_{DR} , but easier to evaluate compared to the rotation ductility. Applies only to the most damage section along the member length. It is unable of representing damage spread along members.
Normalised Cumulative Rotation (NCR) by Banon (1980)	$NCR = \Sigma \theta_p / \theta_y$ Where θ_p is the sum of the inelastic rotations to the point of interest. θ_y is the yield rotation.		D	No	Yes	Depends on the duration and intensity of the earthquake. The index values correlate with flexural member tests but the index values at failure show significant scatter (Williams and Sexsmith 1995).
Krawinkler and Zohrei (1983)	$D = C \sum_{i=1}^N (\Delta \delta p_i)^q$ where C & q are structural performance parameters, N is the number of applied inelastic cycles, and $\Delta \delta p_i$ is the plastic deformation range of excursion i.		D	Yes	Yes	Difficulty in determining C and q. Many experimental results are required for the calibration to different damage states.

⁽¹⁾ The generic type of the damage index, (D = Deformation-based, E = Energy-based and F = Force-based); ⁽²⁾ δ_{max} ?, stands for the question: " Does the damage index take account of the maximum applied deformation?"; ⁽³⁾ CD ?, stands for the question: " Does the damage index take account of the effect of cumulative damage?".

Table C. 1: Review of the most commonly used local damage indices for reinforced concrete building assessment.

Name	Evaluation	Damage state definition	Type ⁽¹⁾	δ_{max} ⁽²⁾	CD? ⁽³⁾	Comments
Wang and Shah D.I. (1987)	$D = \frac{\exp(sb)-1}{\exp(s)-1}, b = c \sum_i \frac{\delta_{m,i}}{\delta_f}$ s is a measure of the strength degradation in each cycle and b is a scaled cumulative displacement ductility. Both s and c are constants. Suggested values are: c=0.1, $\delta_f=5\delta_u$ for frames or $\delta_f=6\delta_u$ for shear wall structures. s=1 for heavily reinforced joint and s=-1 for poorly reinforced joints.		D	Yes	Yes	This is a strength degradation measure. The damage is dependent on the maximum deformation, with the accumulation of damage being assumed proportional to that incurred in previous cycles. The result is not very sensitive to the value of s.
Dymiotis et al. (1999)	$DI = \max \left\{ \frac{\theta_{p,req}}{\theta_{p,avail}}, \frac{V_{R2,req}}{V_{R2,avail}}, \frac{V_{R3,req}}{V_{R3,avail}} \right\}$ where V_{R2} & V_{R3} are shear forces evaluated as per Penelis and Kappos (1997)	DI = 1, failure	F/D	Yes	Yes	Evaluated at each step of a non-linear dynamic analysis. Tries to account for the possibility of failure by shear, as well as that by flexure.
Roufaei and Meyer (1981) Local DI	$D = \frac{k_{full} (k_{max} - k_o)}{k_{max} (k_{max} - k_o)}$ k_{full}, k_{max} and k_o are the ultimate, maximum response and initial stiffness.		F	Yes	Yes	This is a softening index using stiffness degradation. Good correlation with the residual strength and stiffness in members tested under flexure (Williams and Sexsmith 1995).
Chung, Meyer and Shinozuka (1988) Local DI	Uses modifiers to account for the effects of loading history, i.e. stiffness degradation and bending moment. See text, Section 3.3.2.	Failure moment envelope defined. Failure occurs at intersection of this envelope with the degraded M- ϕ curve.	F	Implicit	Yes	Considers the difference in the response of members to positive and negative bending. Good correlation with hysteretic curves of tested flexural members by very limited calibration with observed damage (Williams and Sexsmith 1995)

Table C.1: Continued.

Name	Evaluation	Damage state definition	Type ⁽¹⁾	$\delta_{max}^{(2)}$	CD? ⁽³⁾	Comments
Normalised Dissipated Energy (NDE) by Banon et al. (1980)	$E_n(t) = \frac{\int_0^t [M(\tau)\theta_d(\tau)]}{0.5M_y\theta_y}$ Where θ_d is the rotation increment of the inelastic spring at one member end. θ_y and M_y are the yield curvature and moment, respectively.	Large scatter in values defining failure.	E	Yes	Yes	Ratio of energy dissipated by inelastic rotation at one end of the member, to 0.5 x maximum elastic energy stored in the member in anti-symmetric bending. The index correlates closely to NCR. The values at failure for 32 flexurally tested members are seen by the authors to give large scatter.
Park and Ang (1985a) Local Damage Index (PA)	$D = \frac{\theta_{max}}{\theta_{u,monotonic}} + \beta \frac{\int dE_h}{M_y\theta_{u,monotonic}}$ (see text, Section 3.3.1, for variable definition)	D<0.4 repairable D≥0.4 irreparable D≥1.0 collapse	E	Yes	Yes	Assumes that cumulative damage and damage due to maximum response can be added linearly. Uncertainty in the evaluation of the factor β . The value of PA is dependent on the hysteretic model used for the materials. Uncertainty in the definition of θ_u .
Bracci et al. (1989) Damage Potential (Dp)	$D_p = D_M + D_\theta - D_p D_M; D_M = \frac{\Delta M}{M_y}; D_\theta = \frac{\phi_m - \phi_y}{\phi_f - \phi_y}$ See Section 3.3.1.	D≤0.33 Serviceable D≤0.66 Repairable D≤1 Irreparable D=1 Collapse	E	Yes	No	Calibration with damage observed in tests on columns and model, scaled, 3 and 6 storey frames. See Section 3.3.1.
Fardis (1995)	$D = \frac{\max E_d + \beta \int dE_h}{E_{du}}$ See Section 3.3.1.	D=1, failure	E	Yes	Yes	Variant of Park and Ang (1985a). Uses θ_u and β values as calculated by Park and Ang (1987). Compared with cyclic and monotonic tests on 169 members.
Kunnath et al. (1990) Local Damage Index	$D = \frac{\phi_m - \phi_y}{\phi_u - \phi_y} + \beta \frac{\int dE}{M_y\phi_u}$ Where ϕ_m, ϕ_y and ϕ_u are the maximum response, yield and ultimate curvatures, respectively. See section 3.3.1.	D≤0.11 No damage D≤0.4 Repairable D≤0.77 Irreparable D>0.77 Collapse	E	Yes	Yes	Based on the Park and Ang (1985a) damage index, but adopts curvatures instead of rotations.

Table C.1: Continued 2.

Name	Evaluation	Damage state definition	Type ⁽¹⁾	δ_{max} ⁽²⁾	CD ⁽³⁾	Comments
Ductility Ratio (DR)	$DR = \delta_{max} / \delta_{yield}$ δ_{max} and δ_{yield} are the maximum and yield total drift.	Failure occurs when DR = ultimate structural ductility	D	Yes	No	Unsatisfactory especially where there is shear distortion in joints or bar slip, (i.e. in older buildings). Does not account for the cumulative damage. The deformation capacity is assumed to equal that under monotonic load, (not conservative for old structures)
Maximum Inter-storey Drift (ISD _{max})	$ISD_{max} = \max(\delta_i / h_i)$ δ_i = the relative displacement between 2 adjacent floors and h_i is the inter-storey height.	Different limit values apply to different structural types	D	Yes	No	The inter-storey drift limits can be affected by the configuration and detailing of the structure.
Roufaei and Meyer (1981) Softening Damage Index	$D = \frac{\delta_r - \delta_y}{\delta_f - \delta_y} = \frac{14.2\delta_r \left(\sqrt{\omega_r / \omega} - 1 \right)}{\delta_f - \delta_y}$ δ_r & δ_y are the maximum and failure roof deflections, δ_f is the roof deflection when the first member reaches yield, ω_r & ω are the fundamental frequency before and after the earthquake.		D	Yes	Implicit	Strictly softening indices don't take into account the hysteretic energy dissipation.
Slope Ratio (SR)	$SR = K_l / K_{unl}$ K_l & K_{unl} are the slopes of the loading and unloading branches of the P- δ diagram respectively	From tests (Ghoborah et al. 1999): SR = 1.0, safe SR = 0.2, critical damage	F	Yes	No	A measure of stiffness degradation with cycling.

⁽¹⁾ The generic type of the damage index, (D = Deformation-based, E = Energy-based and F = Force-based); ⁽²⁾ δ_{max} ? , stands for the question: " Does the damage index take account of the maximum applied deformation?"; ⁽³⁾ CD ? , stands for the question: " Does the damage index take account of the effect of cumulative damage?".

Table C. 2: Review of the most commonly used global damage indices for reinforced concrete building assessment.

Name	Evaluation	Damage state definition	Type ⁽¹⁾	δ_{max} $\gamma^{(2)}$	CD $\gamma^{(3)}$	Comments
Flexural Damage Ratio (FDR) by Banon et al.(1980)	$FDR = K_d / K_{eff}$ K_d & K_{eff} are the slopes of the P- δ diagram at yield and maximum displacement respectively		F	Yes	No	A measure of stiffness degradation with cycling. Similar to SR. Not seen to give consistent indication of failure. Difficulties in calibration from influence of earthquake characteristics on the response.
Chung, Meyer and Shinozuka (1988)	Combination of the local index on a storey level. The storey level indices are weighted and combined to give a global damage state index. See Section 3.3.2		F	Implicit	Yes	See Section 3.3.2.
Maximum Softening Index, by Di Pasquale & Cakmak (1987)	Based on the period change of the equivalent SDOF system during the seismic loading (see Section 3.3.2) $\delta_m = 1 - \frac{T_0}{T_{max}}$ T_0 is the initial fundamental period, T_{max} is the maximum period during loading.	Vulnerability curves relating δ_m to different damage states is provided by the authors.	F	Yes	Implicit	A global index providing no insight as to local damage. It combines the effects of stiffness degradation and plastic deformation. The period change will also be affected by damage to non-structural components. Earthquake record required for index evaluation. Assumes the fundamental frequency is changed but that the mode shape remains the same.
Final Softening Index, by Di Pasquale & Cakmak (1987)	Based on the period change of the equivalent SDOF system during the seismic loading. $\delta_f = 1 - \frac{T_0^2}{T_{final}^2}$ T_0 is the initial fundamental period, T_{final} is the 1st mode period after the earthquake.	Relationship between δ_m and δ_f are given, for use with the vulnerability curves for δ_m .	F	Yes	Implicit	Is similar to the degradation in stiffness of the structure. Don't need to know the particulars of the ground motion record, as it is evaluated from parameters, pre and post- earthquake. Assumes no change in the mode shape after loading, this is incorrect for structures undergoing large inelastic deformations.

Table C.2: Continued.

Name	Evaluation	Damage state definition	Type ⁽¹⁾	δ_{max} ? ⁽²⁾	CD? ⁽³⁾	Comments
Maximum Softening Index, by Mork (1992)	Based on equivalent SDOF: $\delta_n = 1 - \sqrt{\frac{k_{n,max}}{k_{n,0}}}$ where $n=mode=1,2$ K_0 is the initial stiffness and K_{max} is the maximum response stiffness.		F	Yes	Implicit	Based on Di Pasquale and Cakmak (1987) index, but tries to account for the second mode. δ_1 & δ_2 represent damage to the lower and upper parts of the structure respectively. This is better than the preceding 2 softening indices at correlating with damage.
Stiffness Damage Index (DI) _K , Ghobarah et al. (1999).	$(DI)_K = 1 - (K_{FINAL} / K_{INITIAL})$ Where the K-values are evaluated from analytical pushover tests before and after the earthquake. The index can be evaluated at the storey level also.	DI = 0 No damage DI ≤ 0.15 Minor DI ≤ 0.3 Moderate DI ≤ 0.8 Severe DI > 0.8 Collapse	F	Yes	Implicit	The method is based on analytical modelling of the structure. The scale is calibrated with respect to other parameters and not to any observation of damage.
Park and Ang (1985a) Global Damage Index	$D_T = \sum_i \lambda_i D_i; \lambda_i = \frac{E_i}{\sum E_i}$ Combination of the local Park and Ang (1985a) damage indices. See Section 3.3.1.	0.01 < DT ≤ 0.1 Non-structural damage DT ≤ 0.2 Slight structural damage DT ≤ 0.5 Moderate structural damage DT ≤ 0.85 Severe DT > 0.85 Collapse (Hwang & Huo 1994)	E	Yes	Yes	See Section 3.3.1.

Table C.2: Continued 2.

Appendix D

Experimental Data for the HRC Damage Index calibration

Source	Structure Type	Damage State	Parameter	Value	Comments
Duffey et al. (1984)	Low-rise shear walls	Serviceability (threshold) Life safety (threshold)	ISD _{max}	0.8% 1.8%	Experiment
Gulkan and Sozen (1974)	MRF	Non-structural damage Repairable damage Irreparable damage Collapse	ISD _{max}	0.0-1.0% 1.0-4.0% 4.0-6.0% >6.0%	Experiment
Bracci et al. (1995)	Non-ductile MRF	No damage Slight damage (threshold) Minor damage (threshold) Moderate damage (threshold)	ISD _{max}	0.28, 0.24, 0.24% 0.89, 1.07% 1.33% 2.24, 2.03%	Shake-table testing of three 1/3 scale models of MRF designed to gravity loads only.
Rzhevsky and Avanesov (1978)	MRF	Collapse (threshold)	ISD _{max}	7-8%	Dynamic tests on 3 scaled model frames
Wang and Shah (1987)	Infilled MRF	Non-structural damage (threshold) Non-structural damage Collapse of non-structural components	ISD _{max}	0.004% 0.004-0.02% >0.02%	Tests on models of infilled frames.
Kircher et al. (1997)	Low-rise shear-walls: High seismic code Moderate code Low seismic code Gravity designed	Slight/Moderate/Extensive/Collapse Slight/Moderate/Extensive/Collapse Slight/Moderate/Extensive/Collapse Slight/Moderate/Extensive/Collapse	Drift ratio	0.004/ 0.01/ 0.03/ 0.08 0.004/ 0.008/ 0.023/ 0.06 0.003/ 0.006/ 0.015/ 0.035 0.002/ 0.005/ 0.012/ 0.028	Assembled as the average of published experimental results by several different authors.
Ghoborah et al. (1999)	MRF	Non-structural damage Repairable damage Irreparable damage/partial collapse Collapse	ISD _{max}	<1.0% 1.0-4.0% 4.0-6.0% >6.0%	Based on experimental work of Roufaei and Meyer (1981).

Table D. 1: Experimental values of maximum inter-storey drift ratio (or total drift ratios for low-rise buildings) characterising observed damage limit states in RC structures.

Source	Structure Type	Damage State	Parameter	Value	Comments
Schultz (1990)	Non-ductile MRF	Yield (threshold) Soft-storey collapse	ISD _{max}	2.0% 7.0%	Shake-table tests on model, 9-storey, 3-bay, non-ductile bare frames.
Zhong, Tian and Ren (1982)	Ductile MRF	Yield (threshold) Collapse (threshold)	ISD _{max}	0.9, 1.86% 7.92, 4.32%	Dynamic tests on 2 seismically designed 2-bay, 2-storey model frames.
Valiasis and Stylianidis (1989)	Infilled frames	Yield (threshold) Collapse (threshold)	ISD _{max}	0.2-0.5% 1.7-4.3%	16 Dynamic tests on 1-storey, 1-bay, 1/3 scale models.
Ersoy et al. (1999)	Ductile (old code) bare MRF	Heavy Damage	ISD _{max}	Average=1.47%	7 Dynamic tests on 2-storey, 1-bay, 1/3 scale models
Chiou et al. (2000)	Bare frame	Collapse (threshold)	ISD _{max}	4%	Experimental tests
	Partially infilled MRF	Collapse (threshold)	ISD _{max}	4.2%	Experimental tests

Table D.1: Continued

Source	Structure Type	Damage State	Parameter	Value	Comments
Freeman (1977)	Partitions	Limiting damage (threshold) Severe damage (threshold)	ISD _{max}	0.005 0.015	Tests on partitions
Calvi and Magenes (1994)	Masonry infills	No damage Slight damage Light damage Repairable non-structural damage	ISD _{max}	0.0-0.1% 0.1-0.3% 0.3-0.5% 0.5-1.5%	Dynamic tests on masonry structures failing by soft-storey mechanism.
Kircher et al. (1997)	Infill panel	Slight Moderate Extensive Collapse	Drift ratio	0.0-0.004 0.004-0.008 0.008-0.025 0.025-0.05	Based on experiments by Ferrito (1983) and data from the California DSA Report (1996)

Table D. 2: Experimental values of maximum inter-storey drift ratio (or total drift ratios for low-rise buildings) characterising observed damage limit states in masonry infill panels.

Source	Structure Type	Damage State	Parameter	Value	Comments
Mosalam et al. (1997)	Low-rise bare frames	No damage Insignificant damage Yielding/moderate damage General yielding/extensive damage	ISD _{max}	0.0-0.2% 0.2-0.5% 0.5-1.0% >1.0%	Judgement
	Low-rise infilled frames	No damage Insignificant damage Yielding/moderate damage General yielding/extensive damage	ISD _{max}	0.0-0.02% 0.02-0.05% 0.05-0.10% >0.10%	Judgement

Table D. 3: Judgement/code values of maximum inter-storey drift ratio (or drift ratios for low-rise buildings) characterising damage limit states in RC structures.

Source	Structure Type	Damage State	Parameter	Value	Comments
HAZUS99 (FEMA 1999)	Low-rise, High seismic code level: Bare MRF Shear-wall	Slight/Moderate/Extensive/Collapse	Total drift ratio	0.005/ 0.01/ 0.03/ 0.08% 0.004/ 0.01/ 0.03/ 0.08%	Based on judgement but includes some experimental data.
			Total drift ratio	0.005/ 0.0087/ 0.0233/ 0.06% 0.004/ 0.0084/ 0.0232/ 0.06%	
	Low-rise, Low seismic code level: Bare MRF Shear-wall Infilled MRF	Slight/Moderate/Extensive/Collapse	Total drift ratio	0.005/ 0.008/ 0.02/ 0.05% 0.004/ 0.0076/ 0.0197/ 0.05% 0.003/ 0.006/ 0.015/ 0.035%	
		Slight/Moderate/Extensive/Collapse	Total drift ratio	0.004/ 0.0064/ 0.016/ 0.04% 0.0032/ 0.0061/ 0.0158/ 0.04% 0.0024/ 0.0048/ 0.012/ 0.028%	
		Slight/Moderate/Extensive/Collapse	Total drift ratio	0.004/ 0.0064/ 0.016/ 0.04% 0.0032/ 0.0061/ 0.0158/ 0.04% 0.0024/ 0.0048/ 0.012/ 0.028%	
	MRF	Fully operational Operational Life safety Near collapse Collapse	Transient/ Permanent Drift	0.0- 0.2 / 0.0% 0.2- 0.5 / 0.0% 0.5- 1.5 / 0.0-0.5% 1.5- 2.5 / 0.5-2.5% > 2.5 / > 2.5%	
Vision 2000 (SEOAC 1995)					Judgement

Table D.3 Continued: Judgement/code values of maximum inter-storey drift ratio (or drift ratios for low-rise buildings) characterising damage limit states in RC structures.

Source	Structure Type	Damage State	Parameter	Value	Comments
FEMA273 (ATC 1997)	MRF	Immediate occupancy Life safety Collapse prevention	Transient/ Permanent Drift	0.0-1.0 / 0.0% 1.0-2.0 / 0.0-1.0% 2.0-4.0 / 1.0-4.0%	Judgement and some experimental data.
	Shear-walls	Immediate occupancy Life safety Collapse prevention	Transient/ Permanent Drift	0.0-0.5 / 0.0% 0.5-1.0 / 0.0-0.5% 1.0-2.0 / 0.5-2.0%	
FEMA273 (ATC 1997)	Masonry infill panel	Immediate occupancy Life safety Collapse prevention	Transient/ Permanent Drift	0.0-0.1 / 0.0% 0.1-0.5 / 0.0-0.3% 0.5-0.6 / 0.3-0.6%	Judgement, some experimental data.
UBC (1998)	Low-rise MRF (<19.8m)	Ultimate state	ISD _{max}	1.5%	Implicit in design code.
	High-rise MRF (≥19.5m)	Ultimate state	ISD _{max}	1.125%	
ATC 3 -06 (1978)	Essential facilities	Ultimate state	ISD _{max}	1.0%	Implicit in design code.
	Non-essential facilities	Ultimate state	ISD _{max}	1.5%	
	Structures of ordinary importance < 3 stories high	Ultimate state	ISD _{max}	2.0%	
Mexico DF (1987) (see Bertero et al. 1991)	MRF	Ultimate state	ISD _{max}	0.6 to 1.2%. Value depends on whether or not non-structural elements can be damaged.	Implicit in design code.
BSL (Japan) (see Bertero et al. 1991)	MRF	Ultimate state	ISD _{max}	1%	Value adopted in practice. None specified in code.
NZS (1984)	MRF	Ultimate state	ISD _{max}	1.0%	Value implicit in code.
CEB (1987)	MRF	Ultimate state	ISD _{max}	2.5%	Value implicit in code.

Table D.3: Continued 2.

Appendix E

**Summary of the Observational Post-Earthquake Survey RC
Building Damage Distributions used in the Empirical Curve
Derivation, Expressed as Continuous Functions and Re-
interpreted in terms of the HRC Damage Scale.**

The observational building damage distributions used in the empirical curve derivation in Chapter 5 are summarised in Tables G.2 and G.3. The latter table summarises the datasets where damage distributions are available, at a single site, for buildings of different height and age classes. The damage distributions are presented after re-interpretation in terms of the HRC-Scale according to the procedure of Section 5.3.1. The tables provide the shape parameters of the cumulative distribution functions (cdfs) fitted to each damage survey. α and β , and μ and σ , are the shape functions for the fitted Beta (equation 5.1) and Log-normal (equation 5.2) cdfs, respectively. The maximum difference between the observed and fitted damage distribution is presented for both relationships. These values are compared to the critical values of Table G.1, to check the satisfaction of the Kolmogorov-Smirnov goodness of fit tests at different levels of significance. The values of critical difference are selected for the comparison, according to the number of damage states (sample size) used in the original damage survey. In Tables G.2 and G.3, the fitted shapes highlighted in dark grey fail the 10% significance test, whereas those highlighted in light grey fail the test at 5% significance.

Sample Size	Critical difference values for different levels of significance		
	10%	5%	1%
6	0.470	0.521	0.618
5	0.510	0.565	0.669
4	0.564	0.624	0.733
3	0.642	0.708	0.828

Table E. 1: Kolmogorov-Smirnov Test critical values for checking the goodness of fit at three levels of significance, of distributions assigned to samples of different sizes.

Dset Ref. ⁽¹⁾	E/q Ref. ⁽²⁾	Site Location	Str. Type ⁽³⁾	SS ⁽⁴⁾	Fitted Beta CDF			Fitted Lognormal CDF		
					α ⁽⁵⁾	β ⁽⁶⁾	Max. Diff. ⁽⁷⁾	μ ⁽⁸⁾	σ ⁽⁹⁾	Max. Diff.
1	8	Anzi	1	6	515.790	9465.931	0.000	-2.898	0.053	0.509
2	8	Arienzo	1	6	0.813	23.949	0.057	-2.768	0.411	0.406
3	8	Baronissi	1	6	61.206	1066.940	0.032	-2.919	0.121	0.032
4	8	Buccino	1	6	40.248	759.909	0.023	-2.994	0.146	0.023
5	8	Calvanico	1	6	606.722	10780.154	0.000	-2.933	0.039	0.000
6	8	Casamarciano	1	6	464.985	8665.797	0.000	-2.933	0.044	0.538
7	8	Casola di Napoli	1	6	80.806	1498.015	0.000	-2.861	0.130	0.373
8	8	Castel Franci	1	6	97.344	1756.200	0.000	-2.949	0.095	0.000
9	8	Castiglione del Genovesi	1	6	99.795	1821.205	0.000	-2.959	0.094	0.000
10	8	Colliano	1	6	60.353	1112.413	0.025	-2.970	0.120	0.025
11	8	Comiziano	1	6	448.168	8410.609	0.000	-2.933	0.039	0.538
12	8	Conza della Campania	1	6	609.906	10898.244	0.000	-2.938	0.040	0.000
13	8	Contursi Terme	1	6	45.019	884.349	0.000	-2.872	0.134	0.429
14	8	Laurenzana	1	6	83.506	1532.974	0.000	-2.929	0.019	0.033
15	8	Lioni	1	6	2.917	40.353	0.105	-2.806	0.552	0.099
16	8	Liveri	1	6	120.565	2172.229	0.024	-2.947	0.086	0.024
17	8	Marsicovetere	1	6	60.519	1224.735	0.000	-2.898	0.088	0.487
18	8	Mercogliano	1	6	39.102	752.642	0.000	-2.851	0.168	0.394
19	8	Monteforte	1	6	36.006	675.576	0.000	-2.989	0.155	0.000
20	8	Paternopoli	1	6	34.308	654.967	0.000	-3.005	0.157	0.000
21	8	Pellezzano	1	6	78.469	1508.578	0.000	-2.884	0.098	0.444
22	8	S.Mauro Forte	1	6	551.719	9947.544	0.000	-2.883	0.064	0.459
23	8	S.Pietro al Tanagro	1	6	92.535	1747.930	0.000	-2.880	0.099	0.428
24	8	Sant'Agnello	1	6	50.100	987.664	0.005	-2.875	0.154	0.422
25	8	S.Angelo dei Lombardi	1	6	0.310	0.875	0.263	-2.239	1.596	0.215
26	8	Striano	1	6	86.816	1733.791	0.000	-2.903	0.061	0.503
27	8	Tramonti	1	6	28.601	598.343	0.011	-2.871	0.193	0.443
28	8	Volturara Irpina	1	6	25.194	454.029	0.042	-2.956	0.187	0.042
29	9	Nocera Umbra	11	6	4.024	93.942	0.059	-3.297	0.516	0.056
30	9	Macerata & Ascoli Piceno Counties	11	5	0.078	11.407	0.008	-3.208	0.319	0.528
31	7	Konitsa (Zone I)	10	3	1.880	14.566	0.000	-2.425	0.735	0.005
32	7	Konitsa (Zone II)	10	3	0.110	6.966	0.000	-5.101	1.566	0.002
33	7	Konitsa (Zone III)	10	3	1.219	61.293	0.000	-3.742	0.472	0.000
34	3	Korinthos	11	6	0.747	30.878	0.136	-4.291	1.197	0.133
35	5	Kalamata Center	11	6	2.852	126.874	0.118	-3.969	0.659	0.113
36	5	Kalamata Nisaki	11	6	3.166	183.330	0.181	-4.230	0.565	0.172
37	14	Erzincan	11	6	1.017	58.660	0.192	-4.617	0.192	0.260
38	4	Plataees	1	4	1.617	23.696	0.059	-2.958	0.814	0.047

⁽¹⁾Dataset reference number; ⁽²⁾Earthquake reference number (see Table 5.1); ⁽³⁾Predominant structural type (1 = bare MRF, 11 = infilled MRF; 12 = shearwall structures; 10 = mixed); ⁽⁴⁾Number of damage states adopted in the original damage survey; ⁽⁵⁾ and ⁽⁶⁾ α and β are the shape parameters of the fitted cumulative Beta distribution functions; ⁽⁷⁾Maximum error between the actual and fitted damage distributions; ⁽⁸⁾ and ⁽⁹⁾ μ and σ are the shape parameters of the fitted cumulative Log-normal distribution functions.

Table E. 2: The observational damage distributions reinterpreted in terms of the HRC Scale

Dset Ref. ⁽¹⁾	E/q Ref. ⁽²⁾	Site Location	Str. Type ⁽³⁾	SS ⁽⁴⁾	Fitted Beta CDF			Fitted Lognormal CDF		
					α ⁽⁵⁾	β ⁽⁶⁾	Max. Diff. ⁽⁷⁾	μ ⁽⁸⁾	σ ⁽⁹⁾	Max. Diff.
39	4	Erythres	1	4	1.280	35.539	0.000	-3.467	0.668	0.000
40	11	Ashiya City	10	4	0.068	1.710	0.018	-3.391	0.973	0.477
41	11	Hokudan-cho	10	4	25.815	1228.217	0.000	-3.573	0.338	0.427
42	11	Ichinomiya-cho	10	4	1.074	51.626	0.034	-3.389	0.760	0.366
43	11	Tsuna-cho	10	4	1.893	107.403	0.012	-3.553	0.521	0.412
44	11	Ohashi	10	3	0.075	1.555	0.024	-2.762	0.790	0.504
45	11	Nishinomiya City	10	3	0.176	4.108	0.005	-2.712	0.749	0.470
46	11	Nada and Higashi- Nada Wards	10	6	14.941	679.439	0.086	-3.859	0.247	0.086
47	11	Nada Ward	10	3	0.156	3.171	0.010	-2.683	0.791	0.467
48	10	Naga-machi	1	5	0.247	6.171	0.032	-3.987	1.400	0.025
49	10	Oroshi-machi	1	5	0.779	19.938	0.041	-3.666	1.048	0.030
50	10	Kamisugi	1	5	2.602	60.839	0.000	-3.247	0.493	0.000
51	11	AREA 2	10	4	0.125	3.035	0.012	-3.170	0.923	0.442
52	11	AREA 3	10	4	0.377	12.705	0.019	-3.191	0.730	0.415
53	11	AREA 4	10	4	0.314	9.260	0.034	-3.144	0.786	0.426
54	11	AREA 5	10	4	12.998	666.666	0.000	-3.568	0.206	0.533
55	11	AREA 7	10	4	0.415	28.562	0.001	-3.438	0.480	0.481
56	11	AREA 8	10	4	0.060	5.086	0.022	-3.476	0.520	0.535
57	11	AREA 9	10	4	0.132	5.180	0.055	-3.295	0.799	0.433
58	14	Fatih, Yunus, Aksemsettin & Yavus Selim Districts	1	3	0.214	2.763	0.003	-3.426	1.442	0.015
59	13	Commercial district of Baguio	10	3	0.218	5.288	0.001	-4.189	1.537	0.008
60	12	Mexico City Old Lakebed Zone	10	3	0.037	2.987	0.001	-6.856	2.450	0.005
61	12	Lazaro Cardenas	10	3	0.249	10.295	0.001	-4.254	1.189	0.000
62	12	Mexico City, Delegacion Cuauhtemac	10	3	0.316	1.464	0.092	-2.789	1.965	0.131
63	19	Los Angeles Area	1	3	0.029	1.731	0.002	-6.252	2.325	0.005
64	19	Los Angeles, Ventura & Orange counties.	1	3	0.288	2.035	0.012	-2.867	1.442	0.037
65	18	Long Beach	10	5	1.614	25.981	0.040	-3.082	0.799	0.020
66	17	Santa Barbara	10	5	0.883	5.520	0.073	-2.419	1.065	0.093
67	1	Chenoua Area	10	5	49.830	2184.096	0.034	-3.607	0.439	0.426
68	2	Vina Del Mar	12	4	0.575	31.293	0.015	-3.034	0.353	0.499
69	6	Aegion	11	3	0.204	11.078	0.000	-3.425	0.861	0.474
70	15	Cumhuriyet Mahallesi District	11	6	5.243	121.527	0.120	-3.274	0.449	0.115
71	16	Sakarya	1	3	0.784	2.866	0.045	-1.915	1.040	0.084
72	16	Kocaeli	1	3	0.902	3.856	0.021	-1.996	0.955	0.057
73	16	Yalova	1	3	1.051	3.883	0.026	-1.839	0.904	0.067
74	16	Istanbul(Avcilar)	1	3	1.539	10.410	0.000	-2.247	0.692	0.005
75	16	Bolu	1	3	0.918	3.278	0.038	-1.848	0.965	0.079
76	16	Bursa	1	3	1.011	8.671	0.000	-2.501	0.796	0.006
77	16	Eskisehir	1	3	0.358	2.731	0.018	-2.795	1.314	0.039

Table E.2: Continued.

Dset Ref. ⁽¹⁾	E/q Ref. ⁽²⁾	Site Location	Str. Type ⁽³⁾	SS ⁽⁴⁾	Fitted Beta CDF			Fitted Lognormal CDF		
					α ⁽⁵⁾	β ⁽⁶⁾	Max. Diff. ⁽⁷⁾	μ ⁽⁸⁾	σ ⁽⁹⁾	Max. Diff.
78	11	Dist.1 (Chuo Ward)	10	5	1.640	46.247	0.073	-3.593	0.819	0.062
79	11	Dist. 2 (Chuo Ward)	10	5	2.910	66.467	0.129	-3.296	0.658	0.120
80	11	Dist.3 (Chuo Ward)	10	5	2.498	100.866	0.045	-3.820	0.535	0.045
81	11	Dist.4 (Chuo Ward)	10	5	0.114	2.632	0.040	-5.325	2.455	0.054
82	11	Dist.5 (Chuo Ward)	10	5	1.110	100.157	0.005	-4.464	0.600	0.005
83	11	Dist.6 (Chuo Ward)	10	5	0.215	7.156	0.027	-4.616	1.650	0.019
84	11	Dist.7 (Chuo Ward)	10	5	1.796	46.682	0.078	-3.498	0.805	0.064
85	11	Dist.8 (Chuo Ward)	10	5	0.751	10.595	0.060	-3.201	1.130	0.034
86	11	Dist.9 (Chuo Ward)	10	5	0.207	1.702	0.047	-3.773	2.193	0.079
87	11	Dist.10 (Chuo Ward)	10	5	0.186	2.060	0.043	-4.056	2.091	0.040
88	11	Dist.11 (Chuo Ward)	10	5	0.277	1.307	0.065	-2.968	2.106	0.112
89	11	Dist.12 (Chuo Ward)	10	5	0.275	2.530	0.011	-3.536	1.862	0.046
90	11	Dist.13 (Chuo Ward)	10	5	0.553	14.892	0.040	-3.891	1.252	0.017
91	11	Dist.14 (Chuo Ward)	10	5	0.333	3.757	0.071	-3.499	1.625	0.047
92	11	Dist.16 (Chuo Ward)	10	5	0.264	16.597	0.005	-4.867	1.378	0.012
93	11	Dist.17 (Chuo Ward)	10	5	0.094	1.245	0.064	-5.197	2.769	0.051
94	11	Dist.18 (Chuo Ward)	10	5	0.281	4.336	0.067	-3.870	1.668	0.045
95	11	Dist.19 (Chuo Ward)	10	5	0.474	1.343	0.108	-2.049	1.691	0.166
96	11	Dist.20 (Chuo Ward)	10	5	0.491	1.859	0.163	-2.323	1.526	0.151
97	11	Hyogo Ward	10	3	1.484	4.591	0.056	-1.577	0.559	0.140
98	11	Nagata Ward	10	3	0.579	2.466	0.069	-2.241	1.362	0.114
99	11	Part of Suma ward	10	3	0.715	2.398	0.118	-1.935	1.278	0.168

Table E.2: Continued 2.

Dataset Ref. ⁽¹⁾	Structure Type ⁽²⁾	Bldg Class ⁽³⁾	Sample Size ⁽⁴⁾	Fitted Beta CDF			Fitted Lognormal CDF		
				α ⁽⁵⁾	β ⁽⁶⁾	Max. Diff. ⁽⁷⁾	μ ⁽⁸⁾	σ ⁽⁹⁾	Max. Diff.
31	10	H-L	3	0.308	4.842	0.002	-3.685	1.471	0.016
32	10	H-L	3	0.110	6.966	0.000	-5.101	1.566	0.002
33	10	H-L	3	0.555	1.696	0.141	-2.019	1.592	0.197
38	1	H-L	4	1.532	22.068	0.054	-2.952	0.835	0.039
45	10	A-H	3	0.076	5.043	0.001	-2.952	0.479	0.529
45	10	A-M	3	0.388	7.673	0.001	-2.591	0.773	0.421
45	10	A-O	3	0.251	2.183	0.043	-2.296	1.035	0.381

⁽¹⁾ Dataset reference number; ⁽²⁾ Predominant structural type (1 = bare MRF, 11 = infilled MRF; 12 = shearwall structures; 10 = mixed); ⁽³⁾ Building class (H-H, H-M, H-L, stand for High rise, mid-rise and low-rise building classes, respectively. A-H, A-M and A-O, stand for new seismic code, old-seismic code or Pre-seismic code structure classes, respectively); ⁽⁴⁾ Number of damage states adopted in the original damage survey; ⁽⁵⁾ and ⁽⁶⁾ α and β are the shape parameters of the fitted cumulative Beta distribution functions; ⁽⁷⁾ Maximum error between the actual and fitted damage distributions; ⁽⁸⁾ and ⁽⁹⁾ μ and σ are the shape parameters of the fitted cumulative Log-normal distribution functions.

Table E. 3: The observational damage distributions available for buildings of different height and seismic code classes, reinterpreted in terms of the HRC Scale

Dataset Ref. ⁽¹⁾	Structure Type ⁽²⁾	Bldg Class ⁽³⁾	Sample Size ⁽⁴⁾	Fitted Beta CDF			Fitted Lognormal CDF		
				α ⁽⁵⁾	β ⁽⁶⁾	Max. Diff. ⁽⁷⁾	μ ⁽⁸⁾	σ ⁽⁹⁾	Max. Diff.
46	10	A-H	6	17.048	819.823	0.041	-3.949	0.687	0.093
46	10	A-M	6	1.514	64.418	0.100	-3.940	0.222	0.032
46	10	A-O	6	1.195	43.207	0.110	-3.920	1.005	0.106
46	10	H-H	6	1.389	29.676	0.078	-3.908	0.226	0.041
46	10	H-L	6	17.271	861.174	0.032	-3.851	0.269	0.100
46	10	H-M	6	12.857	577.316	0.101	-3.362	0.955	0.075
48	1	H-L	5	0.353	12.634	0.031	-4.264	1.362	0.020
49	1	H-L	5	0.696	18.337	0.043	-3.780	1.154	0.029
49	1	H-M	5	2.431	44.270	0.000	-3.062	0.558	0.001
50	1	H-L	5	3.590	82.244	0.000	-3.254	0.478	0.000
50	1	H-M	5	2.731	64.863	0.000	-3.205	0.421	0.000
58	1	H-L	3	2.710	46.388	0.000	-2.786	0.362	0.000
58	1	H-M	3	0.477	2.678	0.007	-2.437	1.207	0.036
59	10	H-L	3	0.287	9.424	0.000	-4.260	1.799	0.019
59	10	H-M	3	0.179	3.357	0.004	-4.082	1.202	0.001
60	10	H-H	3	0.421	4.909	0.003	-7.543	3.085	0.011
60	10	H-L	3	0.029	4.262	0.000	-7.099	2.266	0.002
60	10	H-M	3	0.032	1.629	0.006	-3.277	1.368	0.022
61	10	H-L	3	0.136	6.301	0.000	-4.841	1.577	0.003
61	10	H-M	3	5.760	114.883	0.000	-2.351	0.744	0.341
69	11	A-H	3	0.545	80.768	0.000	-3.738	0.486	0.537
69	11	A-M	3	0.238	12.473	0.000	-3.451	0.843	0.481
69	11	A-O	3	0.261	6.201	0.001	-3.395	0.872	0.466
69	11	H-L	3	0.188	10.753	0.000	-3.104	0.990	0.403
69	11	H-M	3	0.370	11.635	0.010	-3.053	1.118	0.392
70	11	H-H	6	757.669	71821.670	0.000	-4.282	0.405	0.447
70	11	H-M	6	4.698	203.687	0.065	-3.889	0.517	0.064

Table E.3: Continued.

Appendix F

Adaptive Pushover (APO) Analysis Feasibility Study Results

Period (s)	ICONS Experiment	ICONS Analysis	LALD			HAHD		
			W	M	S	W	M	S
T1	0.630	0.680	0.597	0.577	0.560	0.553	0.535	0.521
T2	0.210	0.226	0.198	0.191	0.185	0.180	0.174	0.167
T3	0.130	0.134	0.114	0.109	0.106	0.100	0.097	0.094
T4	0.090	0.095	0.076	0.073	0.071	0.065	0.063	0.061
T5	-	-	0.065	0.062	0.060	0.058	0.056	0.054

Table F. 1: Comparison of experimentally measured modal periods with the results of Eigen-value analyses for the ICONS frame (Pinto et al. 1999). Results of Eigen-value analyses on the 8-storey frames of Fardis (1994), for the low acceleration low ductility (LALD), and high acceleration high ductility (HAHD) design cases, with the weak (W), medium (M) and strong (S) material parameter combinations.

Combination	f_c (MPa)	f_y (MPa)
Weak	26.49	564.17
Medium	33.00	585.00
Strong	39.51	619.50

Table F. 2: Summary of the material combinations used to assess the effect of material variability on the response of the 8-storey LALD and HAHD frames designed by Fardis (1994).

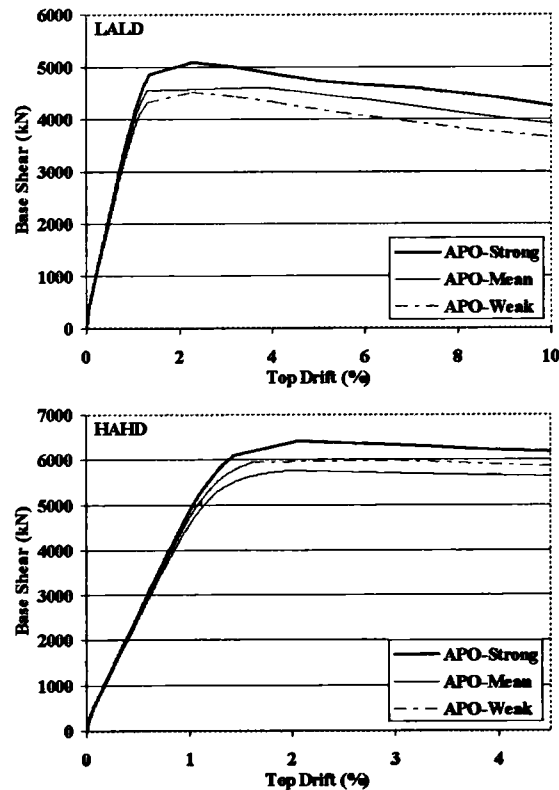


Figure F. 1: Comparison of the adaptive pushover curves for the LALD and HAHD 8-storey frames designed by Fardis (1994), with the material combinations of Table J.2.

Run	SF ⁽¹⁾	Error in Base Shear (%)				Error in ISD _{max} (%)						Error in Storey?					Best for BS ⁽⁷⁾	Best for ISD ⁽⁸⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	APO1 ⁽⁵⁾	APO	Tri-PO	Uni P-O	APO1	DYNPO ⁽⁶⁾	APO	Tri-PO	Uni PO	APO1	DYNPO			
1	0.1	8.44	16.15	-2.80	15.12	12.63	16.09	9.27	13.06	1.65	N	N	N	N	N	Uni PO	Uni PO	
2	0.3	5.38	13.77	-6.09	16.88	1.99	5.18	-1.14	4.493	1.57	Y	N	Y	N	Y	APO	Uni PO	
3	0.5	-22.69	-6.91	-32.74	-5.999	-4.94	-1.14	-8.36	-1.284	2.91	Y	Y	Y	N	Y	APO1	Tri-PO	
4	0.7	-1.39	11.04	-10.10	11.90	5.61	8.51	1.72	8.413	22.39	Y	Y	Y	Y	Y	APO	Uni PO	
5	0.9	-17.12	-1.61	-25.67	-1.277	-1.09	3.26	-3.29	2.024	0.00	N	N	N	N	N	APO1	APO	
6	1.1	-2.23	10.86	-10.37	10.74	4.69	8.85	2.62	7.543	7.05	Y	Y	Y	Y	Y	APO	Uni PO	
7	1.3	-11.33	3.57	-19.30	3.707	5.80	9.94	3.78	8.757	1.60	Y	Y	Y	Y	N	Tri-PO	Uni PO	
8	1.5	-14.70	-0.01	-23.54	-0.187	6.85	10.94	4.84	9.628	0.18	Y	Y	Y	Y	N	Tri-PO	Uni PO	
9	1.7	-23.93	-7.87	-32.96	-7.338	7.16	10.71	4.96	10.01	1.48	Y	Y	Y	Y	N	APO1	Uni PO	
10	1.9	-26.84	-10.50	-35.97	-10.03	4.48	7.92	2.21	7.225	4.57	Y	Y	Y	Y	N	APO1	Uni PO	
11	2.5	-21.61	-7.71	-28.05	-5.942	1.47	6.02	-0.43	4.086	3.52	Y	N	Y	N	N	APO1	Uni PO	
12	3	-4.92	4.27	-11.78	5.026	2.77	7.65	0.31	6.230	17.19	Y	N	Y	N	Y	Tri-PO	Uni PO	
13	3.5	2.66	10.34	-3.07	10.926	4.46	9.44	0.26	7.858	0.00	Y	Y	Y	Y	N	APO	Uni PO	
14	4	-3.61	4.88	-7.44	5.676	9.86	14.32	3.40	12.57	7.02	Y	Y	Y	Y	Y	APO	Uni PO	
15	4.5	-6.11	1.78	-9.56	2.586	10.58	16.11	3.20	14.33	13.97	Y	Y	Y	Y	Y	Tri-PO	Uni PO	
16	5	-6.94	0.84	-9.91	1.763	7.11	13.49	-1.29	11.61	15.68	Y	Y	Y	Y	Y	Tri-PO	Uni PO	
Av.		11.243	7.007	16.833	7.194	5.719	9.348	3.192	8.070	6.299	2	5	2	6	8	A/A1/T	Uni-PO	

⁽¹⁾ Scaling factor applied to accelerogram; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ APO with force updating for 1st mode only; ⁽⁶⁾ Dynamic pushover analysis; ⁽⁷⁾ & ⁽⁸⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 3: Comparison of the results of adaptive, dynamic and conventional pushover analyses with the results of full time-history analyses for the 8-storey LALD frame designed by Fardis (1994).

Run	SF ⁽¹⁾	Error In Base Shear (%)				Error In ISD _{max} (%)						Error In Storey?					Best for BS ⁽⁷⁾	Best for ISD ⁽⁸⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	APO1 ⁽⁵⁾	APO	Tri-PO	Uni P-O	APO1	DYNPO ⁽⁶⁾	APO	Tri-PO	Uni PO	APO1	DYNPO			
1	0.1	6.57	9.49	-9.07	15.48	5.33	10.97	4.13	7.646	2.25	N	N	Y	N	N	APO	Uni PO	
2	0.3	14.91	20.14	3.35	26.35	5.03	8.05	1.51	9.028	4.11	Y	Y	Y	N	N	Uni	Uni PO	
3	0.5	-11.58	2.23	-19.64	2.527	-4.55	-0.89	-7.08	-2.796	5.60	Y	Y	Y	Y	Y	Tri-PO	Tri-PO	
4	0.7	-11.67	3.14	-17.94	2.517	-5.00	-2.27	-8.63	-2.974	0.60	N	N	N	N	N	APO1	Tri-PO	
5	0.9	-25.47	-7.96	-31.73	-9.694	-2.88	0.66	-4.61	-1.105	2.76	Y	Y	Y	Y	Y	Tri-PO	Tri-PO	
6	1.1	-13.26	2.61	-18.93	0.966	-0.51	3.06	-1.97	1.303	8.48	Y	Y	Y	Y	Y	APO1	APO	
7	1.3	-10.23	5.08	-15.75	3.692	-0.22	2.09	-1.56	1.573	5.47	N	N	N	N	N	APO1	APO	
8	1.5	-3.40	11.27	-7.82	10.45	3.26	6.25	1.81	4.728	8.82	Y	Y	Y	Y	Y	APO	Uni PO	
9	1.7	-3.43	9.68	-8.64	9.012	3.58	7.41	2.73	5.828	7.97	Y	Y	Y	Y	Y	APO	Uni PO	
10	1.9	-5.66	6.49	-11.33	6.106	-1.22	3.68	-1.75	1.812	5.07	Y	Y	Y	Y	Y	APO	APO	
11	2.5	-8.25	4.35	-11.17	4.072	6.95	11.89	5.42	10.08	0.00	N	Y	Y	N	N	Tri-PO	Uni PO	
12	3	-9.44	2.86	-11.84	2.473	3.14	8.74	1.31	6.962	1.06	N	Y	Y	N	N	APO1	Uni PO	
13	3.5	-9.21	2.78	-11.48	2.345	2.67	8.58	0.97	6.739	5.42	Y	N	Y	Y	Y	APO1	Uni PO	
14	4	-8.13	3.52	-10.20	3.114	-2.66	3.86	-4.38	2.111	6.04	Y	N	Y	Y	Y	APO1	APO1	
15	4.5	-8.56	3.06	-10.64	2.635	0.33	6.76	-1.30	5.071	3.46	N	Y	Y	N	N	APO1	APO	
16	5	-7.33	3.49	-8.82	3.048	-1.11	6.58	-3.11	4.916	0.33	N	Y	Y	N	N	APO1	APO	
Av.		9.818	6.135	13.021	6.530	3.026	5.734	3.268	4.667	4.215	7	5	7	4	8	APO1	Uni-PO	

⁽¹⁾ Scaling factor applied to accelerogram; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ APO with force updating for 1st mode only; ⁽⁶⁾ Dynamic pushover analysis; ⁽⁷⁾ & ⁽⁸⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 4: Comparison of the results of adaptive, dynamic and conventional pushover analyses with the results of full time-history analyses for the 8-storey LAMD frame designed by Fardis (1994).

Run	SF ⁽¹⁾	Error In Base Shear (%)				Error In ISD _{max} (%)						Error In Storey?				Best for BS ⁽⁷⁾	Best for ISD ⁽⁸⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	APO1 ⁽⁵⁾	APO	Tri-PO	Uni P-O	APO1	DYNPO ⁽⁶⁾	APO	Tri-PO	Uni PO	APO1	DYNPO		
1	0.1	15.12	21.75	3.16	21.93	13.92	15.71	12.06	14.66	0.00	N	N	N	N	N	Uni PO	Uni PO
2	0.3	7.16	16.85	-3.80	18.30	1.15	5.29	-3.24	4.410	4.69	N	N	N	N	Y	Uni PO	APO
3	0.5	1.27	12.57	-8.82	14.03	-2.06	1.17	-5.10	1.889	6.90	N	N	N	N	Y	APO	Tri-PO
4	0.7	14.78	24.51	6.03	25.74	3.12	6.22	0.25	6.834	2.23	Y	Y	Y	Y	Y	Uni PO	Uni PO
5	0.9	-15.42	-2.39	-27.38	-1.061	-7.73	-3.93	-10.84	-4.40	8.79	Y	Y	Y	Y	Y	APO1	Tri-PO
6	1.1	-11.88	0.75	-23.70	2.014	-1.49	2.11	-5.42	1.654	0.00	N	N	N	N	N	Tri-PO	APO
7	1.3	15.48	24.97	6.57	25.93	4.88	8.26	1.60	7.824	1.65	Y	Y	Y	Y	Y	Uni PO	Uni PO
8	1.5	2.96	13.80	-7.29	14.91	14.29	17.19	11.50	16.897	13.35	Y	Y	Y	Y	Y	APO	Uni PO
9	1.7	-20.36	-6.79	-32.76	-5.702	-2.65	1.31	-5.60	0.681	0.00	Y	Y	Y	Y	N	APO1	APO1
10	1.9	-30.32	-15.75	-42.93	-14.51	-7.83	-3.73	-11.01	-4.456	5.00	Y	Y	Y	Y	Y	APO1	Tri-PO
11	2.5	-2.00	9.10	-10.63	10.83	7.01	13.19	3.84	12.697	9.99	N	N	N	N	Y	APO	Uni PO
12	3	-4.75	6.49	-12.52	8.736	-6.85	1.91	-12.58	0.18	13.75	Y	Y	Y	Y	Y	APO	APO1
13	3.5	-1.00	9.90	-8.74	11.98	-5.25	2.97	-10.52	1.569	6.77	Y	Y	Y	Y	Y	APO	APO1
14	4	-7.98	3.13	-15.43	5.801	-10.87	-0.46	-17.42	-2.551	4.83	Y	N	Y	N	N	Tri-PO	Tri-PO
15	4.5	-9.12	1.90	-15.91	4.837	-10.39	1.64	-16.67	-0.521	7.30	Y	Y	Y	Y	N	Tri-PO	APO1
16	5	-8.25	2.50	-14.58	5.400	-10.59	3.13	-16.72	0.989	8.98	Y	Y	Y	Y	N	Tri-PO	APO1
17	5.5	-5.57	4.90	-11.66	7.773	-9.43	4.47	-15.48	2.341	9.40	Y	Y	Y	Y	Y	Tri-PO	APO1
Av.		10.202	10.473	14.818	11.734	7.030	5.451	9.402	4.974	6.096	5	6	5	6	6	APO	APO1

⁽¹⁾ Scaling factor applied to accelerogram; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ APO with force updating for 1st mode only; ⁽⁶⁾ Dynamic pushover analysis; ⁽⁷⁾ & ⁽⁸⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 5: Comparison of the results of adaptive, dynamic and conventional pushover analyses with the results of full time-history analyses for the 8-storey HAMD frame designed by Fardis (1994).

Run	SF ⁽¹⁾	Error In Base Shear (%)			Error In ISD _{max} (%)						Error In Storey?					Best for BS ⁽⁷⁾	Best for ISD ⁽⁸⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	APO ⁽⁶⁾	APO	Tri-PO	Uni P-O	APO1	DYNPO ⁽⁶⁾	APO	Tri-PO	Uni PO	APO1	DYNPO		
1	0.1	20.56	37.41	9.32	27.25	8.30	13.01	6.24	9.179	3.41	N	N	N	N	N	Uni	Uni
2	0.3	-14.03	9.57	-30.24	1.452	-1.22	-0.84	-5.98	-1.696	2.63	Y	Y	Y	Y	Y	APO1	Tri-PO
3	0.5	-0.46	25.93	-10.60	13.47	2.72	8.93	1.46	6.127	6.96	Y	Y	Y	Y	N	APO	Uni
4	0.7	10.56	34.47	2.10	23.32	9.78	15.84	6.62	11.98	9.07	N	Y	N	Y	Y	Uni	Uni
5	0.9	-33.88	3.78	-44.97	-14.42	-6.91	1.07	-9.73	-4.273	2.85	Y	Y	Y	Y	Y	Tri-PO	Tri-PO
6	1.1	-19.91	12.77	-30.87	-3.206	9.80	15.73	6.48	11.71	10.41	Y	Y	Y	Y	Y	APO1	Uni
7	1.3	-20.68	13.32	-30.73	-3.148	-5.96	2.02	-8.80	-3.341	8.82	Y	Y	Y	N	Y	APO1	Tri-PO
8	1.5	-15.52	17.07	-25.44	1.022	-5.93	1.62	-8.02	-3.354	0.00	N	N	N	Y	N	APO1	Tri-PO
9	1.7	-28.47	6.21	-40.33	-11.97	-6.94	1.36	-9.74	-4.876	1.61	Y	Y	Y	N	N	Tri-PO	Tri-PO
10	1.9	-23.75	8.85	-35.15	-7.322	-8.76	-0.08	-11.87	-7.782	2.07	Y	Y	Y	Y	Y	APO1	Tri-PO
11	2.5	-22.45	9.91	-31.53	-5.223	4.83	10.93	2.36	4.369	0.00	Y	Y	Y	Y	N	APO1	Uni
12	3	-18.13	12.62	-27.10	-1.595	7.58	13.67	5.26	7.332	13.04	Y	Y	Y	Y	Y	APO1	Uni
13	3.5	-15.90	13.89	-24.70	0.233	5.94	12.35	3.77	5.870	9.26	Y	Y	Y	Y	Y	APO1	Uni
14	4	-9.47	18.52	-17.66	5.715	-0.24	6.61	-2.55	-0.177	10.04	N	N	Y	N	Y	APO1	APO1
15	4.5	4.03	28.46	-2.78	17.21	-8.51	-1.08	-10.59	-8.097	9.53	N	N	Y	N	Y	Uni	Tri-PO
16	5	4.48	28.79	-2.38	17.62	3.25	9.87	1.36	3.570	0.00	Y	Y	Y	Y	N	Uni	Uni
Av.		16.391	17.597	22.868	9.635	6.042	7.188	6.301	5.858	5.607	5	4	3	5	6	APO1	Tri/Uni

⁽¹⁾ Scaling factor applied to accelerogram; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ APO with force updating for 1st mode only; ⁽⁶⁾ Dynamic pushover analysis; ⁽⁷⁾ & ⁽⁸⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 6: Comparison of the results of adaptive, dynamic and conventional pushover analyses with the results of full time-history analyses for the 8-storey HAHD frame designed by Fardis (1994).

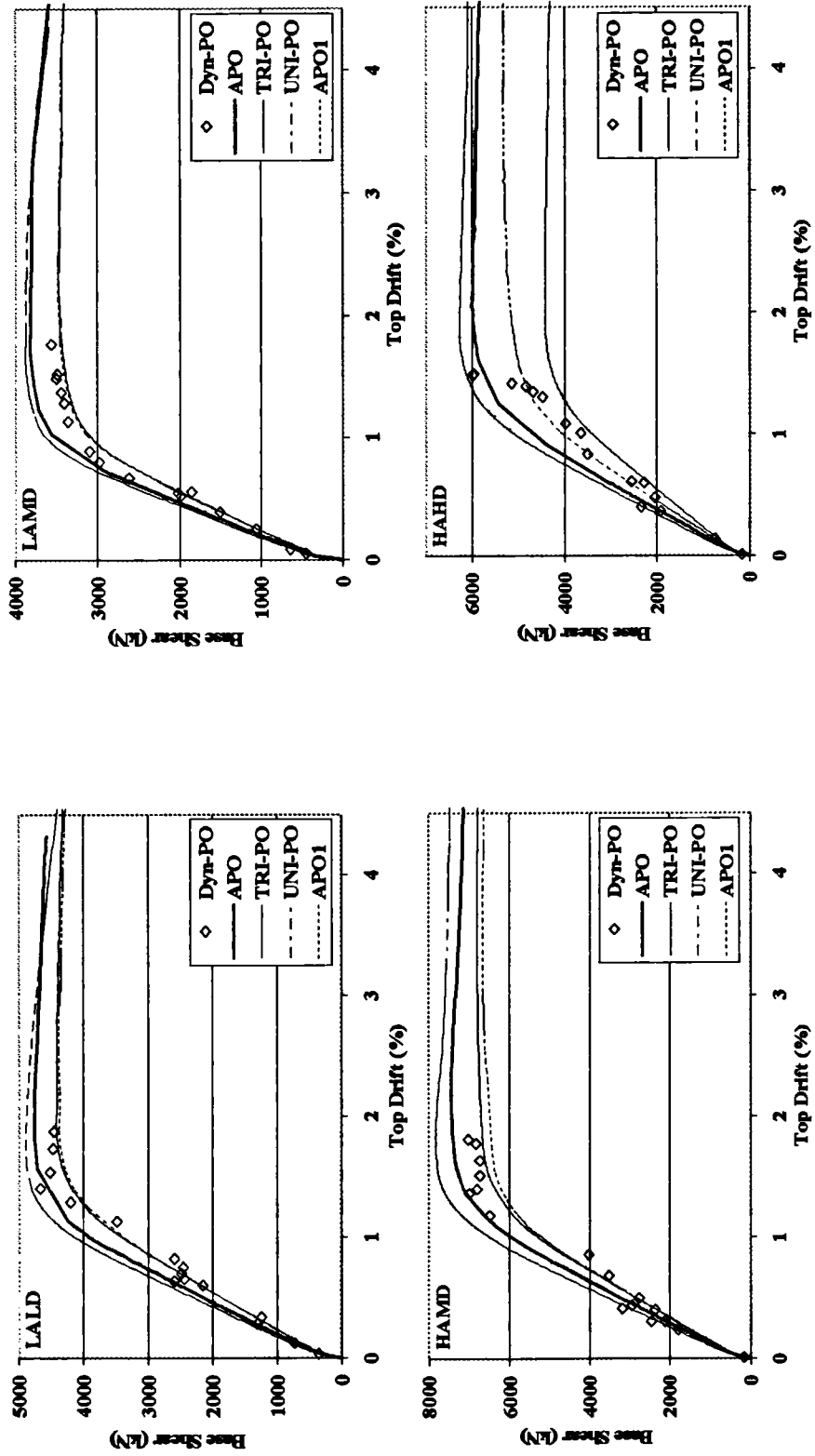


Figure F. 2: Comparison of the pushover curves for the adaptive and conventional analysis methods with the results of incremental dynamic analyses, for the 8-storey bare reinforced concrete frames designed by Fardis (1994).

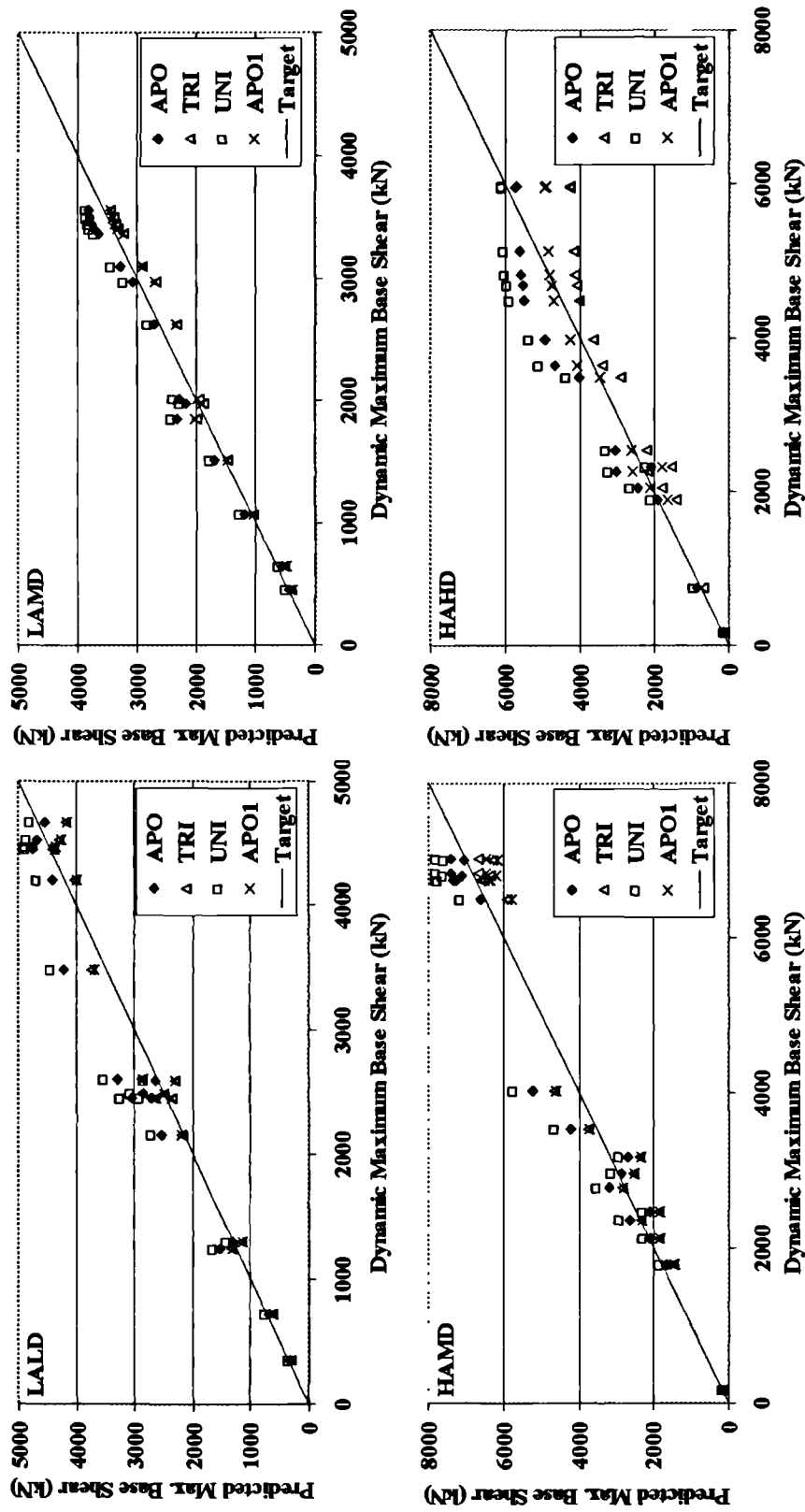


Figure F. 3: Comparison of the values of base shear and top drift predicted by the adaptive pushover analysis methods with those of the dynamic time-history analyses, for the 8-storey bare-frames designed by Fardis (1994).

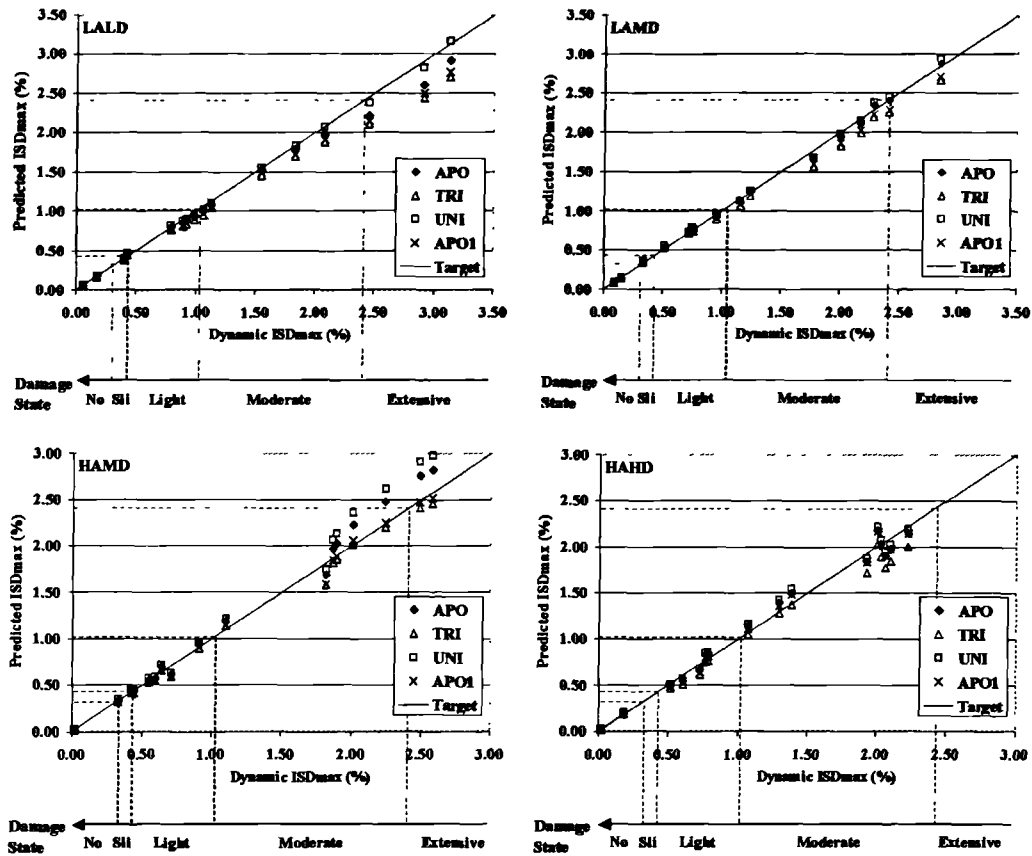


Figure F. 4: Comparison of the maximum inter-storey drift and HRC damage state predictions of pushover analysis procedures with the values estimated by dynamic time-history analysis, for the 8-storey frames designed by Fardis (1994).

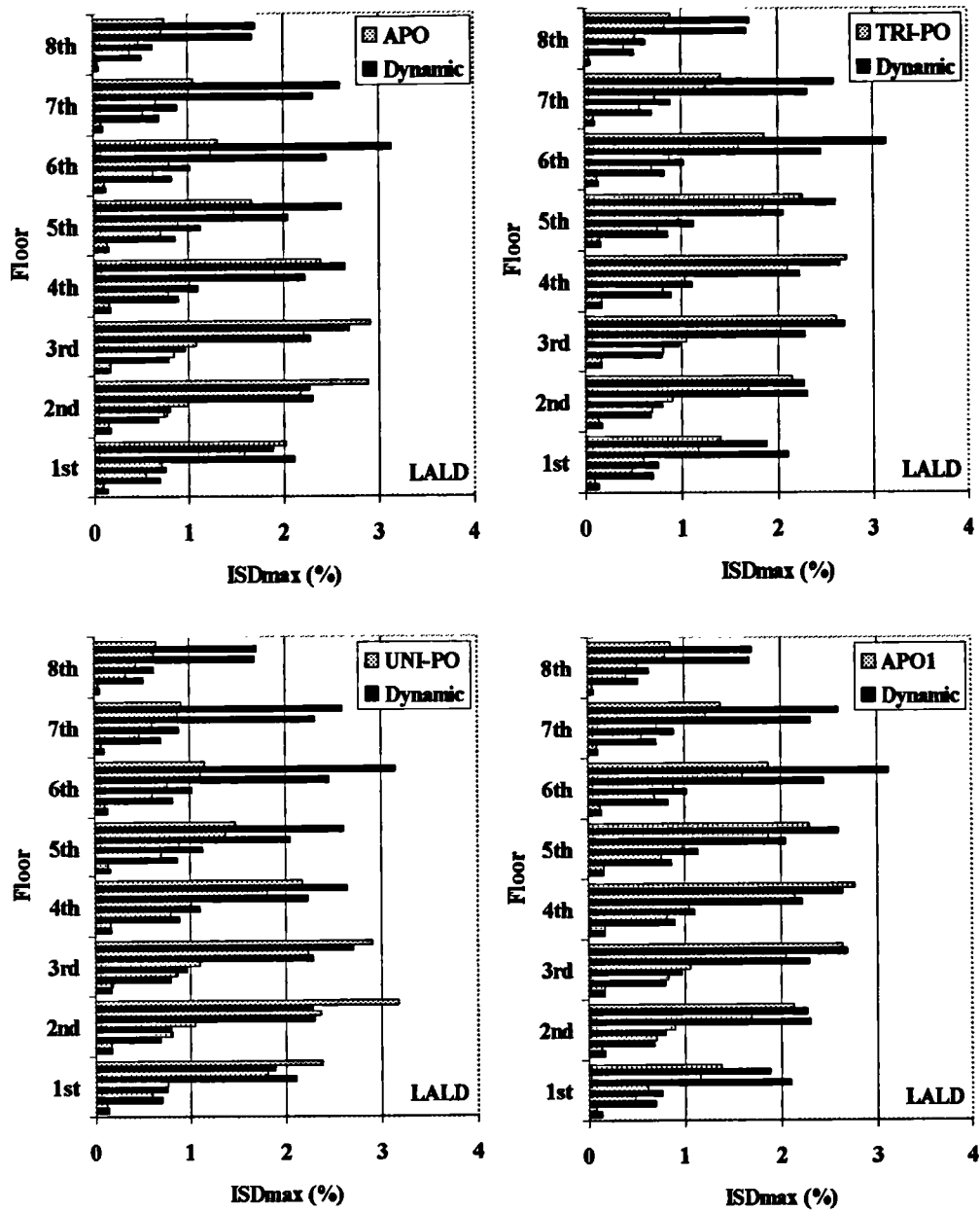


Figure F. 5: Comparison of the distribution with height of the maximum inter-storey drift estimated by the adaptive pushover analysis methods with those estimated from dynamic time-history analyses under increasing ground motion severities, for the 8-storey LALD frame designed by Fardis (1994).

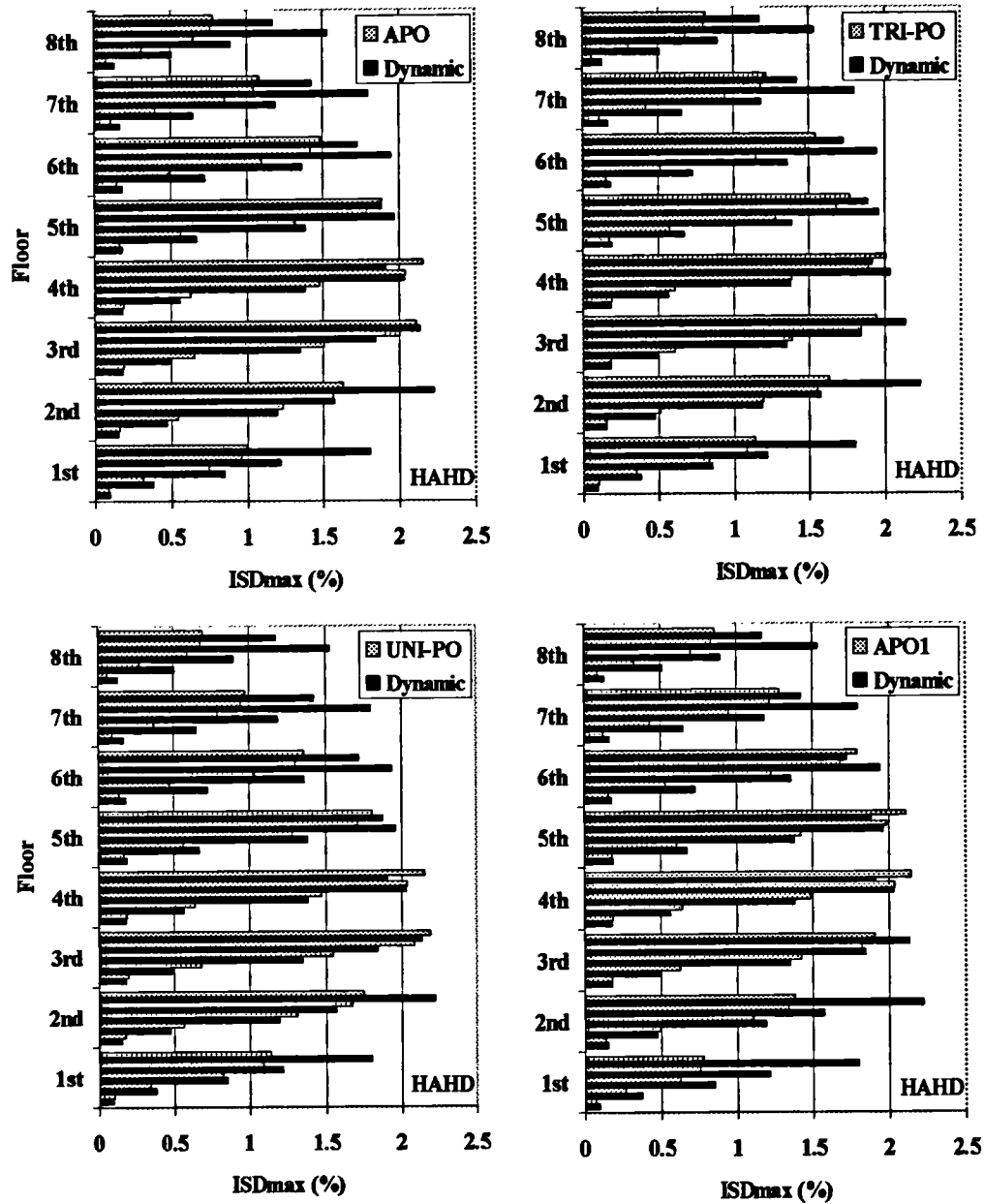


Figure F. 6: Comparison of the distribution with height of the maximum inter-storey drift estimated by the adaptive pushover analysis methods with those estimated from dynamic time-history analyses under increasing ground motion severities, for the 8-storey HAHD RC frame designed by Fardis (1994).

Run	SF ⁽¹⁾	Error In BS (%)				Error In ISD _{max} (%)				Error In Storey of Predicted ISD _{max} ?				Best for BS ⁽⁶⁾	Best for Isd ⁽⁷⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	DYN ⁽⁵⁾	APO	Tri-PO	Uni P-O	DYN	APO	Tri-PO	Uni P-O	DYN		
1	0.2	-15.58	0.22	-25.12	N/A	15.53	19.59	14.59	N/A	Y	Y	Y	N/A	Tri-PO	Uni P-O
3	0.6	-5.25	9.87	-12.12	N/A	19.92	24.06	19.20	N/A	Y	Y	Y	N/A	APO	Uni P-O
4	0.8	-13.34	3.37	-20.37	N/A	12.89	17.71	12.08	N/A	Y	Y	Y	N/A	Tri-PO	Uni P-O
5	1	35.64	45.25	31.83	N/A	22.17	26.62	21.40	N/A	Y	Y	Y	N/A	Uni P-O	Uni P-O
Av.		17.455	14.680	22.360	N/A	17.627	21.997	16.818	N/A	0	0	0	N/A	Tri-PO	Uni-PO

⁽¹⁾ Scaling factor applied to accelerometer; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ Dynamic time-history analysis; ⁽⁶⁾ & ⁽⁷⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 7: Comparison of the adaptive pushover analysis results with the dynamic time-history analysis predictions for the ICONS frame (Pinto et al. 1999) when the first 5 seconds of the 475 year return period earthquake accelerogram is applied.

Run	SF ⁽¹⁾	Error In BS (%)				Error In ISD _{max} (%)				Error In Storey of Predicted ISD _{max} ?				Best for BS ⁽⁶⁾	Best for Isd ⁽⁷⁾
		APO ⁽²⁾	Tri-PO ⁽³⁾	Uni PO ⁽⁴⁾	DYN ⁽⁵⁾	APO	Tri-PO	Uni P-O	DYN	APO	Tri-PO	Uni P-O	DYN		
1	0.2	-15.58	-21.90	-52.47	N/A	15.53	13.57	8.05	N/A	Y	N	N	N/A	APO	Uni P-O
3	0.6	-5.25	29.48	12.45	N/A	19.92	43.52	39.30	N/A	Y	Y	Y	N/A	APO	APO
4	0.8	-13.34	19.92	0.58	N/A	12.89	11.99	5.44	N/A	Y	Y	Y	N/A	Uni P-O	Uni P-O
5	1	35.64	26.66	10.46	N/A	22.17	13.97	7.79	N/A	Y	Y	Y	N/A	Uni P-O	Uni P-O
Av.		17.455	24.489	18.991	N/A	17.627	20.765	15.147	N/A	0	1	1	N/A	A/U	Uni-PO
Exp		-8.56	5.36	-13.13	-15.24	5.86	10.74	5.24	14.27	Y	Y	Y	N	Tri-PO	Uni P-O

⁽¹⁾ Scaling factor applied to accelerometer; ⁽²⁾ Adaptive pushover analysis; ⁽³⁾ Pushover analysis with triangular force distribution; ⁽⁴⁾ Pushover analysis with uniform force distribution; ⁽⁵⁾ Dynamic time-history analysis; ⁽⁶⁾ & ⁽⁷⁾ Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 8: Comparison of the adaptive pushover analysis results with the dynamic time-history analysis predictions and experimental observations for the ICONS frame (Pinto et al. 1999) when the entire (15seconds) 475 year return period earthquake accelerogram is applied.

Run	Scale	Error in BS (%)				Error in ISDmax (%)				Error in Storey of Predicted ISD _{max} ?				Best for BS	Best for ISD
		APO	Tri-PO	Uni P-O	DYN	APO	Tri-PO	Uni P-O	DYN	APO	Tri-PO	Uni P-O	DYN		
1	0.2	-15.58	-49.74	-86.31	N/A	15.53	9.00	3.12	N/A	Y	N	N	N/A	APO	Uni P-O
3	0.6	-5.25	5.53	-14.21	N/A	19.92	18.94	13.55	N/A	Y	Y	Y	N/A	APO	Uni P-O
4	0.8	-13.34	-33.90	-51.97	N/A	12.89	18.67	12.47	N/A	Y	Y	Y	N/A	APO	Uni P-O
5	1	35.64	3.67	-10.35	N/A	22.17	15.32	10.29	N/A	Y	Y	Y	N/A	Tri-PO	Uni P-O
Av		17.455	23.213	40.711	N/A	17.627	15.483	9.856	N/A	0	1	1	N/A	APO	Uni-PO
Exp		-17.59	-6.75	-20.49	-6.03	40.36	44.02	37.53	50.52	Y	Y	Y	N	DYN	Uni P-O

(¹) Scaling factor applied to accelerogram; (²) Adaptive pushover analysis; (³) Pushover analysis with triangular force distribution; (⁴) Pushover analysis with uniform force distribution; (⁵) Dynamic time-history analysis; (⁶) & (⁷) Best analysis method for base shear (BS) and maximum interstorey drift ratio (ISD_{max}) prediction.

Table F. 9: Comparison of the adaptive pushover analysis results with the dynamic time-history analysis predictions and experimental observations for the ICONS frame (Pinto et al. 1999) when the entire (15seconds) 475 year return period earthquake accelerogram is applied followed by the first 7.5 seconds of the 975 year return period record.

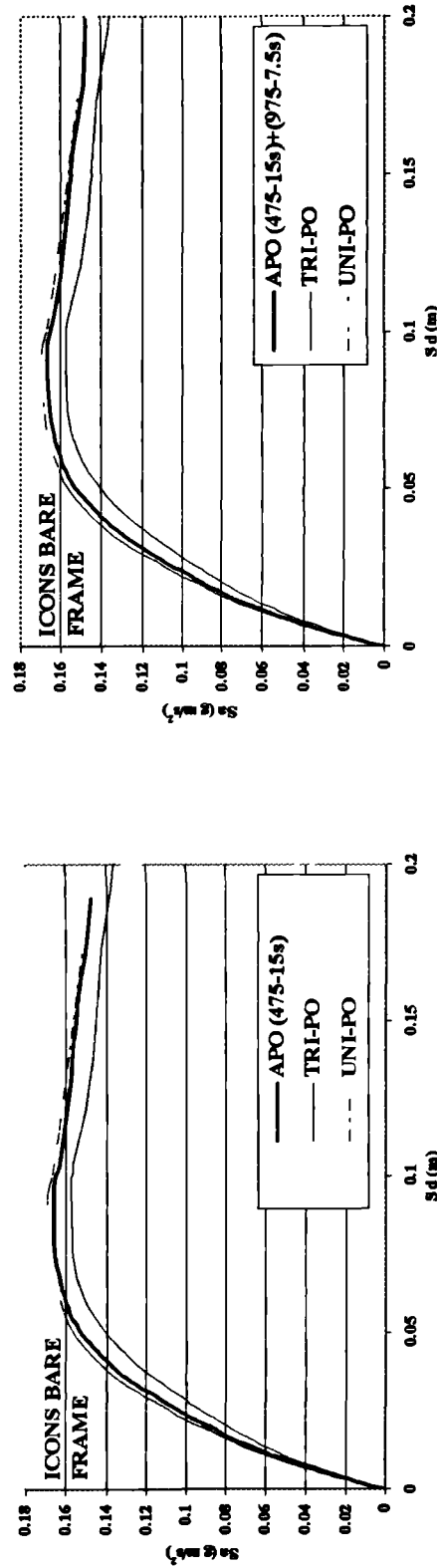


Figure F. 7: Comparison of the pushover curves (expressed in terms of spectral coordinates) considering different truncation times for the applied records for the ICONS frame. (Pinto et al. 1999)

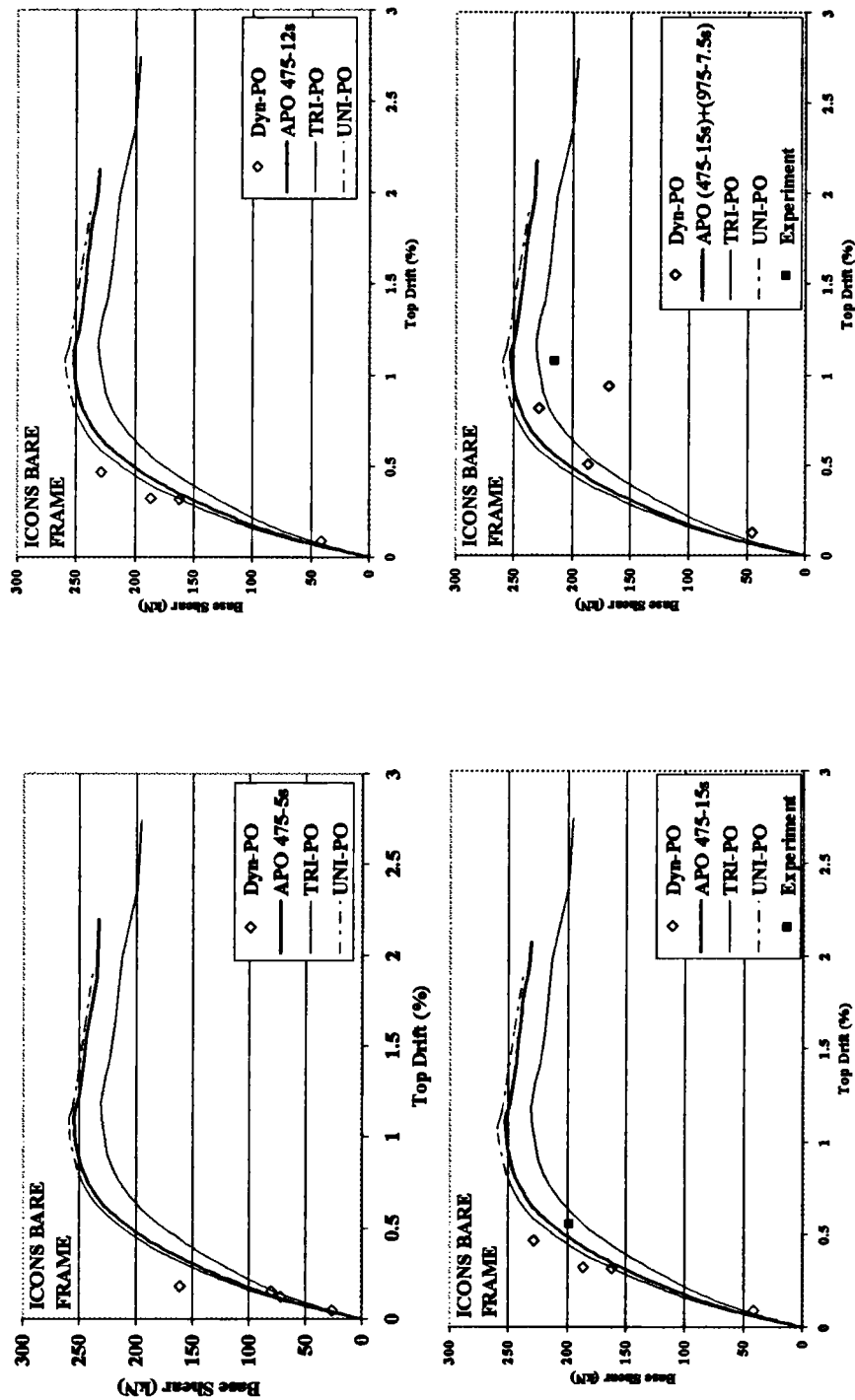


Figure F. 8: Comparison of the pushover curves with the results of incremental dynamic analyses for the ICONS (Pinto et al. 1999) frame.

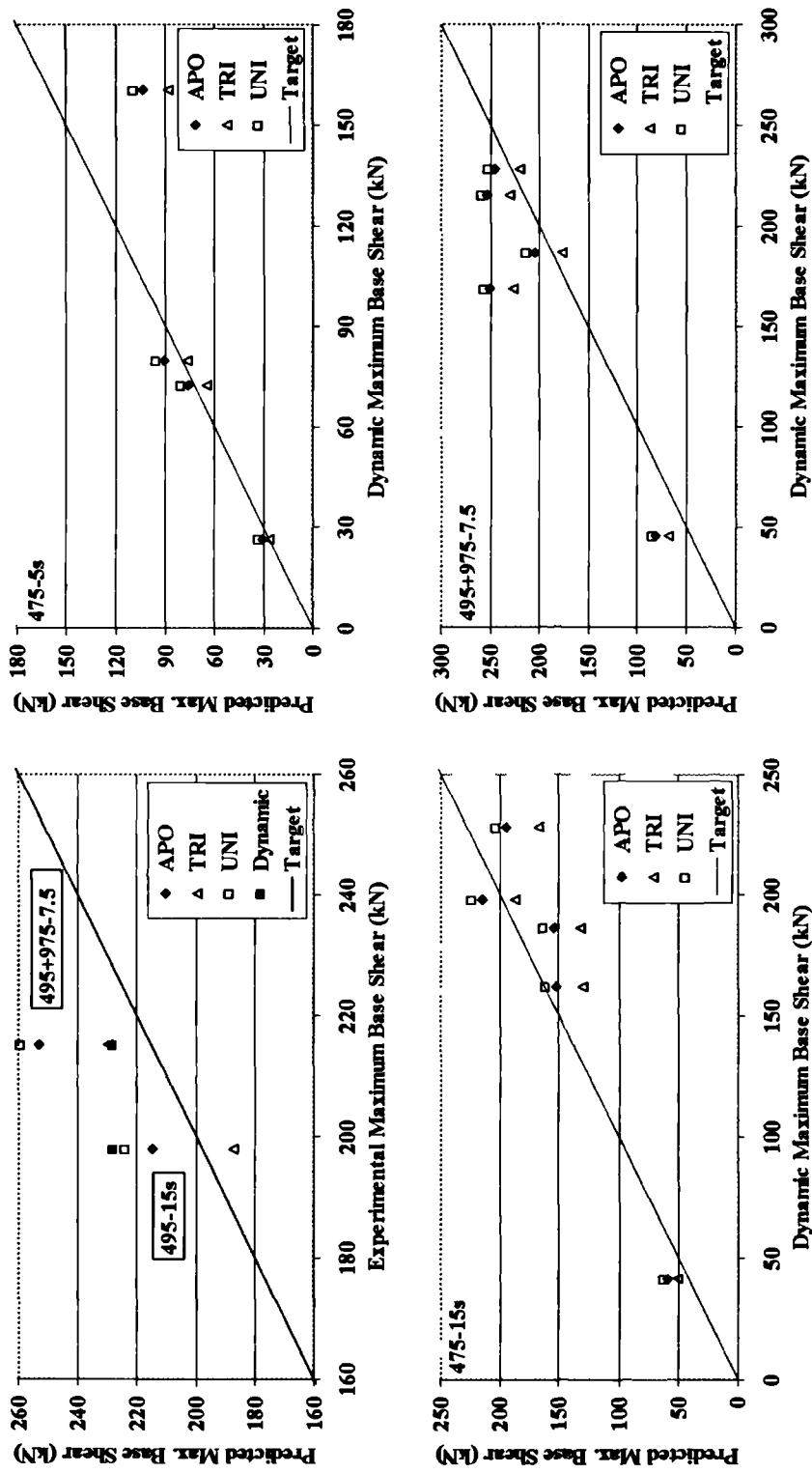


Figure F. 9: Comparison of the base shear predicted by the adaptive pushover methods with the results of dynamic time-history analyses and experimental observations, considering different record truncation times and the ICONS frame (Pinto et al. 1999).

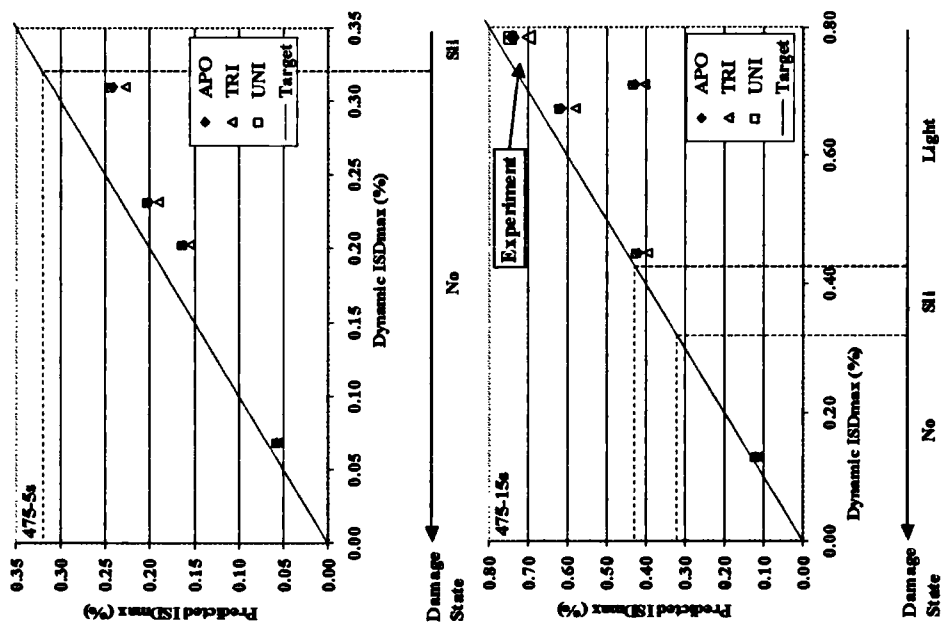


Figure F. 10: Comparison of the maximum inter-storey drift values predicted by the pushover analyses with those of dynamic time-history analyses and experimental observations for the ICONS frame (Pinto et al. 1999) assuming different record truncation times.

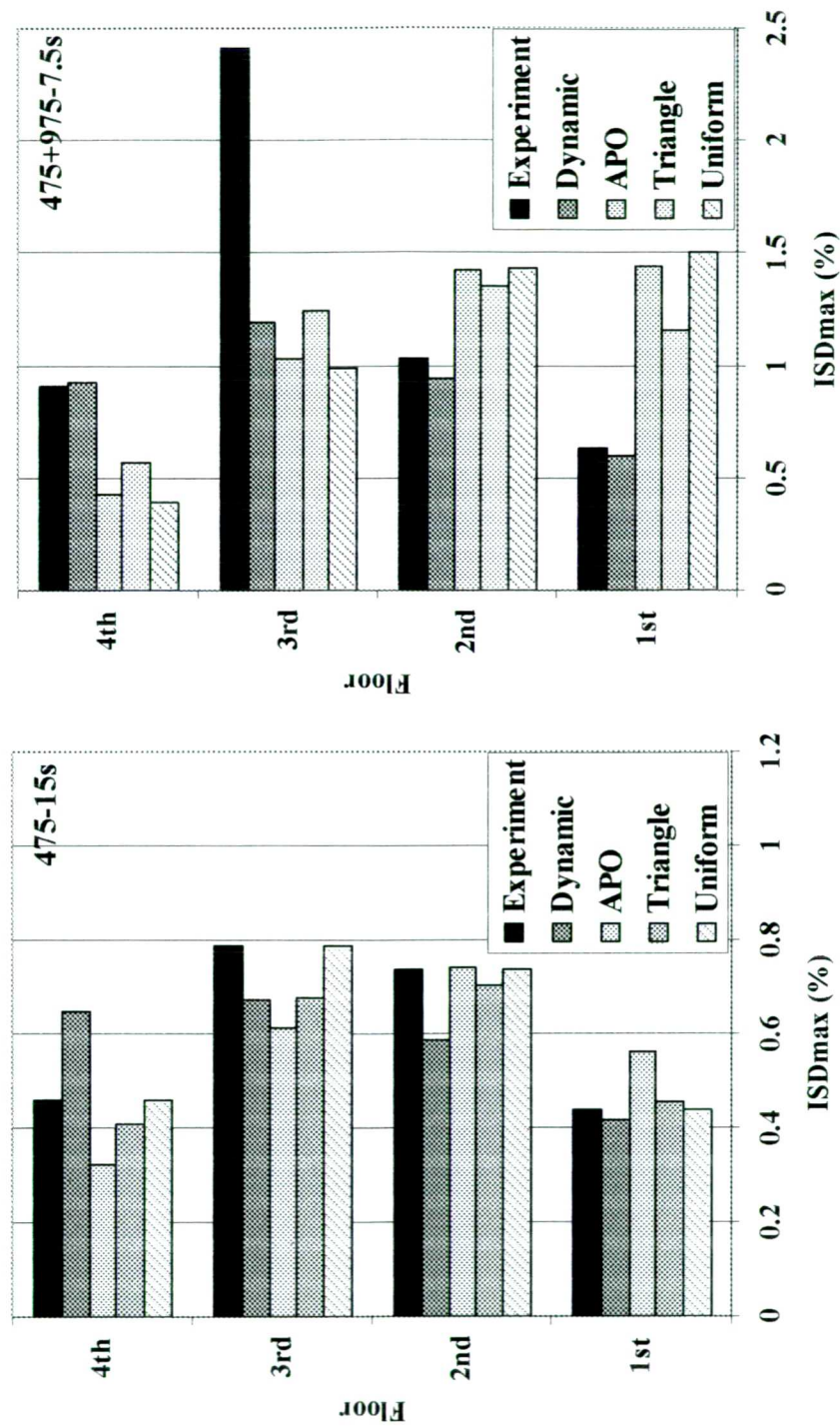


Figure F. 11: Comparison of the distribution of maximum inter-storey drift predicted by the pushover and dynamic time-history analyses with those observed at different times during the experiment on the ICONS frame (Pinto et al. 1999). The inter-storey drift values are calculated at the total drift value observed during the experiment.

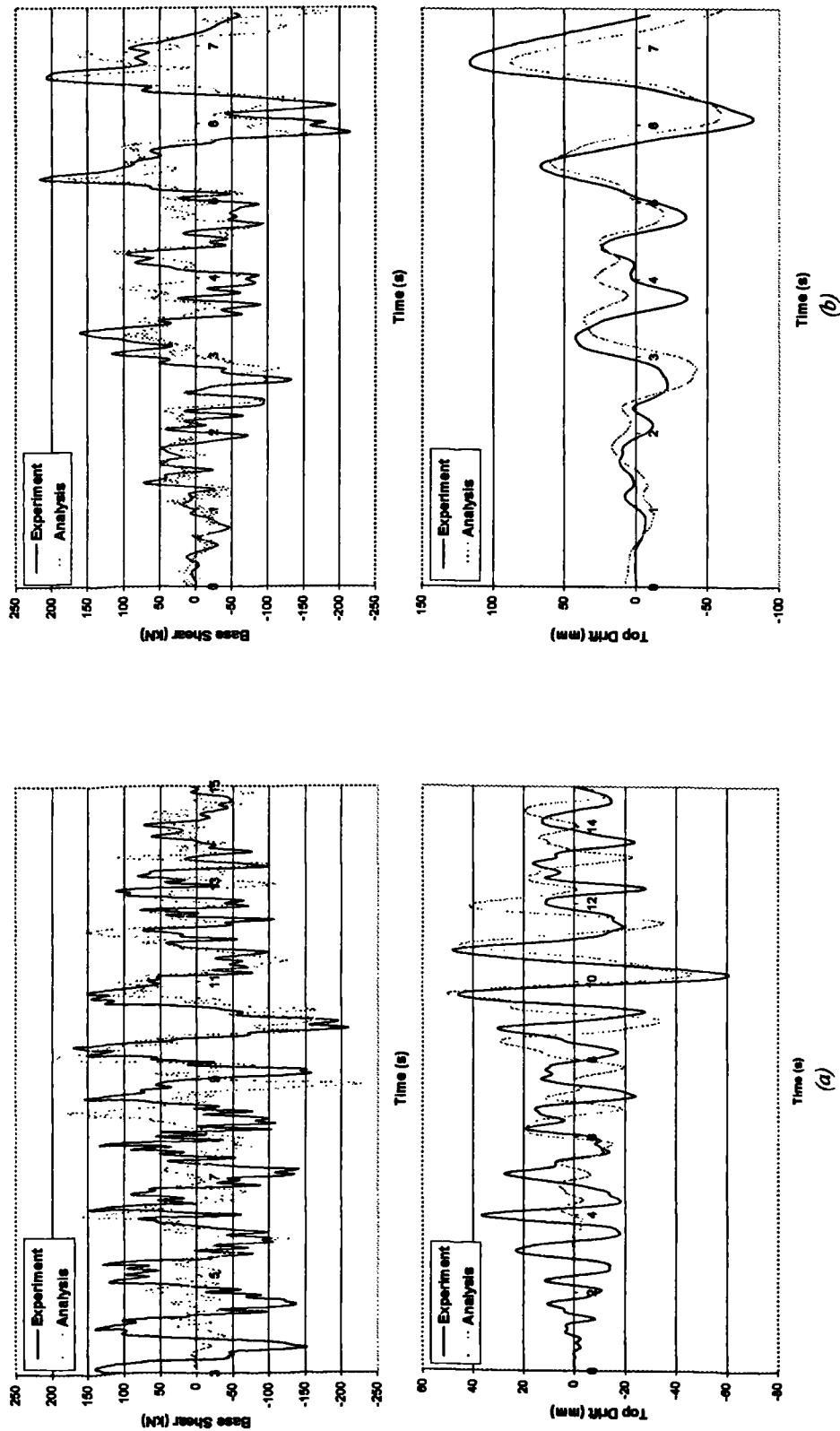
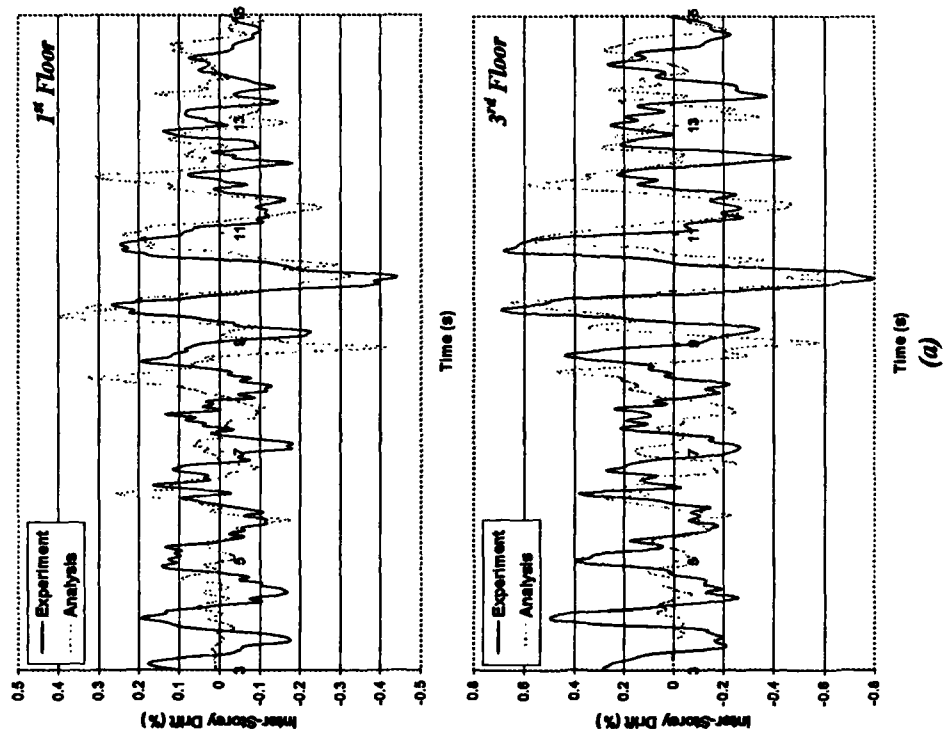
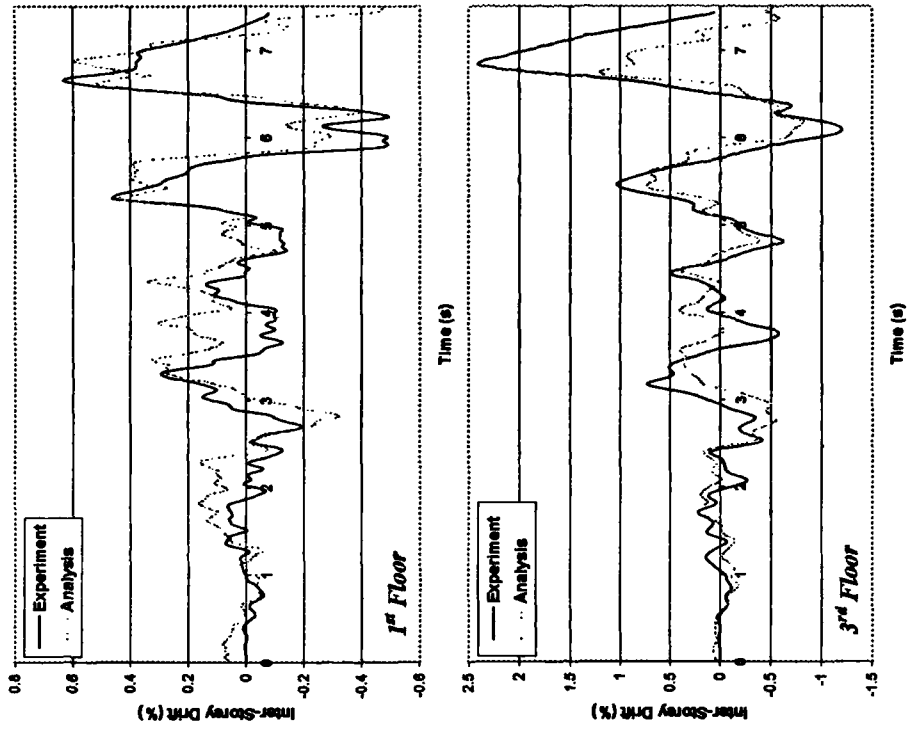


Figure F. 12: Comparison of analytical and experimental base shear and roof drift response of the ICONS (Pinto et al. 1999) frame when subjected to the (a) 475yrp and (b) 975yrp records.



(a)



(b)

Figure F. 13: Comparison of analytical and experimental inter-story drift response at the first and third floors of the ICONS (Pinto et al. 1999) frame, when subjected to (a) the 475yrp record and (b) the 975yrp record.

Appendix G

Capacity Assessment Program (CAsP): Brief Description of Program and Subroutines.

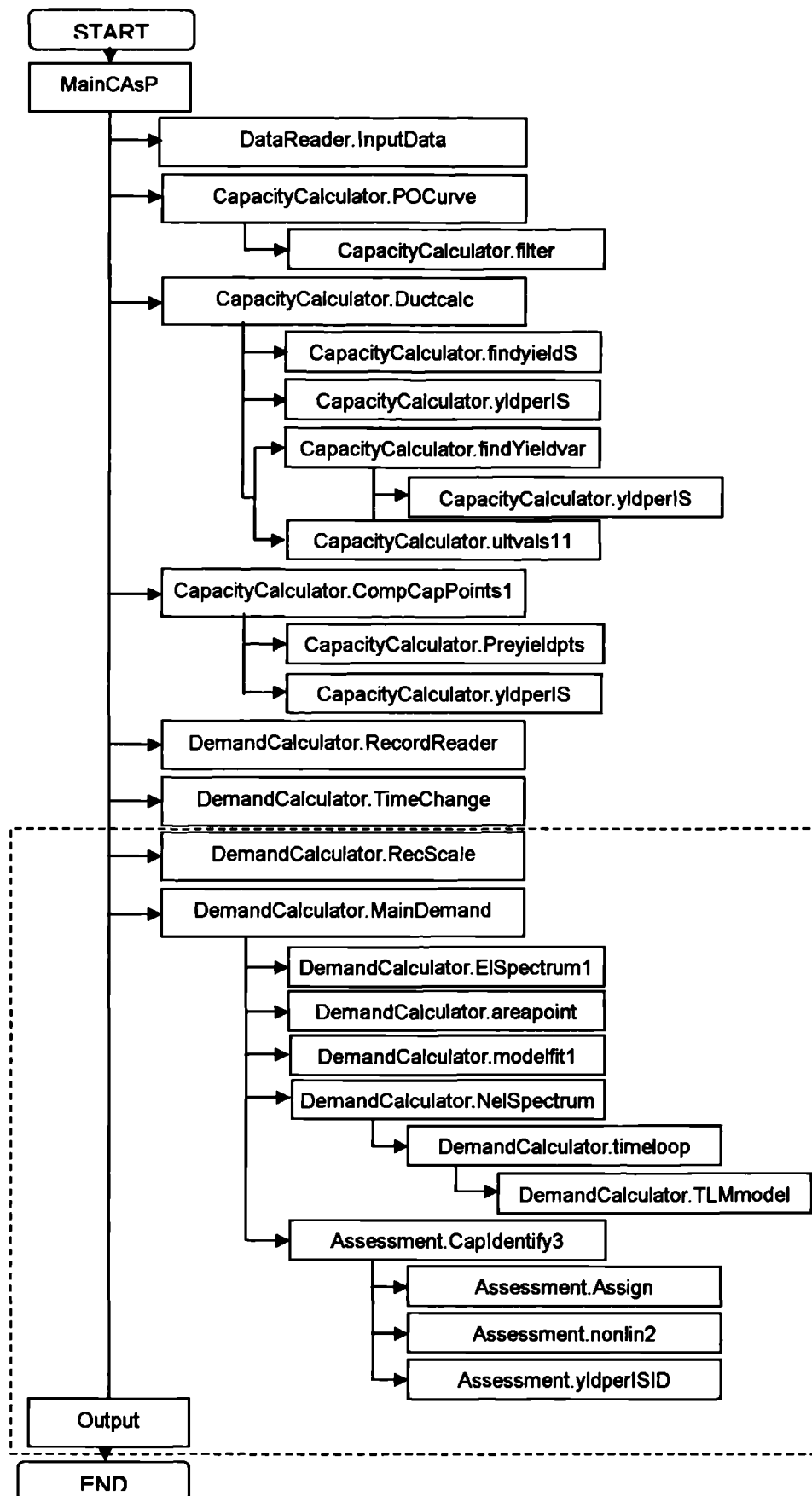


Figure G. 1: Subroutine calling sequence of the CAsP program for the analysis of an APO curve using the variable strain hardening option and TLM model for the curve idealisation. The sequence within the dashed box is repeated for each earthquake scale factor.

G.1 MainCasp

The main program:

- Calls the main subroutines
- Contains the capacity curve modelling options:
 - *model* = 3, signifies TLM model (see Fig.6.25c). *model* = 2, signifies EST model (see Fig.6.25b). *model* = 1, signifies EPP model (see Fig.6.25a).
 - *yopt* = 1, signifies the spectral displacement at yield (S_{dy}) is determined from the point of first deviation from linear behaviour, (obligatory option for TLM model). *yopt* = 2, S_{dy} defined by the intersection of the initial stiffness with a horizontal at maximum strength; *yopt* = 3, S_{dy} defined by line passing through point of first deviation from linear and the latter horizontal. *yopt* = 0, user defined S_{dy} .
 - *yopt1* = 0, spectral acceleration at yield (S_{ay}) found as coordinate corresponding to S_{dy} from the actual capacity curve. *yopt1*=1, S_{ay} value for EST model, and S_a and S_d at ultimate point for the TLM model, are found from equivalence of energy concepts, considering the entire capacity curve, (S_a of idealised model must lie within 15% of the original curve value).
 - *var* = 0, a constant strain hardening value for EST and for the 3rd branch of TLM is used throughout the assessment. *var* = 1, a variable strain hardening value for EST and for the 2nd and 3rd branches of TLM is used, which changes value for each capacity point checked.
- Contains the framework for the repetition of the assessments under increasingly scaled earthquakes and for different material combination APO curves.
- Outputs the response parameter values corresponding to the capacity points to an output file

G.2 DataReader.InputData

Subroutine to read the input filenames, locations and other input data from the program interface.

- Reads the input pushover curve (*POCurve*) and earthquake record (*Record*) filenames and locations
- Reads first line of *POCurve* file, the minimum and maximum scalefactor (*SF*) and *SF* increment for the assessment.
- Reads the number of post-yield periods (*NoPer*) used for the capacity-to-demand checks.
- Reads the output file name (*OutFile*) and location

G.3 CapacityCalculator.POCurve

Subroutine to read data from the *POCurve* file, calculate response values and create an array of the capacity curve data.

- Reads base-shear, storey drift data and spectral conversion coefficients from each row of the *POCurve* file corresponding to the converged iteration points of the pushover analysis and converts them to numerical values.
- Calculates the response parameters for each pushover curve point and harmonises the units.
- Creates an array of global response values (*CapCurve0*) and interstorey drift values (*ISDCurve0*)
- Calls the APO curve filtering subroutine *CapacityCalculator.filter*

G.4 CapacityCalculator.filter

Subroutine to filter the capacity curve data arrays (*CapCurve0* and *ISDCurve0*) and eliminate values pertaining to intermediate iterations between applied load increments.

- Reads subsequent capacity spectral displacement values (S_{dc}) from the capacity curve array *CapCurve0*.
- The S_{dc} value is maintained if the difference with respect to the last kept point value is positive and eliminated otherwise.
- The array is redimensioned. Curve points occurring after first incidence of the defined maximum ISD_{max} of interest are eliminated and maximum response values for the curve are calculated.
- The subroutine yields filtered arrays of capacity curve response parameters (*CapCurve* and *ISDCurve*).

G.5 CapacityCalculator.ductcalc

Subroutine to carry out the capacity curve idealisation and calculate the yield period according to the chosen non-linear model.

- Calls the subroutines for the yield point coordinate calculation (*CapacityCalculator.findyieldS* and *CapacityCalculator.yieldperIS*).
- Calls the subroutines for the slope change points coordinate calculation according to the chosen non-linear idealisation model and strain hardening options.

- Outputs arrays of the idealised capacity curve coordinates and strain hardening slopes corresponding to the points of *CapCurve* as well as the associated ductility and effective period values.
- The output file “C:/tmp/yultvals.txt” is created containing the yield period, the yield, ultimate and final point coordinates for the capacity curve idealisation model.

G.6 CapacityCalculator.findyieldS

Subroutine to find the yield point coordinates according to the chosen model and yield options.

- The gradient (S_{ac}/S_{dc}) of successive capacity curve points is calculated and the elastic gradient determined from the first 3 points
- The location of first deviation from linear is identified through comparison of successive gradients. Yield is assumed to occur if the change in successive gradients is greater than 30%. A further check is carried out to ascertain that a significant change from the initial period has occurred.
- The yield point coordinates (S_{ay} and S_{dy}) are determined from the pushover curve according to the chosen yield criterion.

G.7 CapacityCalculator.yldperIS

Subroutine to find the structure response parameters corresponding to target spectral displacement and acceleration coordinates (S_d and S_a) through extrapolation of points in the actual capacity curve array (*CapCurve*).

- The rows of the capacity curve and inter-storey drift arrays are scrolled through until the two points around the target S_d are located.
- Extrapolation of the response parameters between the points is carried out and the response parameters corresponding to the occurrence of S_d are output.

G.8 CapacityCalculator.findYieldvar

Subroutine to find the location of the yield point coordinates for the EST model from energy equivalence considerations.

- The total area under the pushover curve (expressed in spectral coordinates) is found.
- The location of the last curve point of interest is defined (i.e. the final point)

- Geometry is used to find the S_{ay} value that will result in equal areas under the idealised and real curves whilst maintaining the yield period as is defined by the chosen yield criterion.

G.9 CapacityCalculator.ultvals11

Subroutine to find the location of the ultimate point coordinates for the TLM model from energy equivalence considerations.

- The total area under the pushover curve (expressed in spectral coordinates) is found.
- Geometrical considerations are used to define a relationship between S_d and the ultimate spectral acceleration value (S_{au}) that will result in an equal area under the idealised and real capacity curve.
- The ultimate point is obtained if the capacity curves S_{ac} coordinate lies within 15% of the S_{au} value defined for S_{dc} .

G.10 CapacityCalculator.CompCapPoints1

Subroutine to set up arrays for the capacity curve that include the spectral coordinates and response parameters for the number of user defined assessment periods for use in the capacity-to-demand checks.

- All pre-yield capacity curve points are maintained.
- A series of S_{dc} values are defined at equal period increments along the post-yield curve. (n.b. If the TLM model is used 70% of the points are made to lie between the yield and ultimate points).
- Response parameter values and coordinates are determined for the selected periods from the actual capacity curve arrays (*CapCurve*, *IDCapCurve*, *ISDCurve* and *IDISDCurve*) through extrapolation.
- Arrays containing the capacity curve coordinates and response parameter values and for the selected periods are set up (*CapPoints*, *ISDPoints*).
- The idealised capacity curve coordinates and response parameter values are calculated corresponding to the S_d values defining the assessment points. Arrays of these values are set up (*IDCapPoints* and *IDISDPoints*).

G.11 CapacityCalculator.Preyieldpts

Subroutine to set up arrays for the capacity curve that includes the spectral coordinates and response parameters for pre-yield points.

- Values of effective period are assigned arbitrarily to each pre-yield point as a proportion of T_y (i.e. $T_{eff} = k \cdot T_y / K$, where K is the number of pre-yield

points and $k=1$ to $(K-1)$). These values are used to define SDOF characteristics for elastic analysis under the applied earthquake record and are included simply as an aid in the problem visualisation.

- The idealised capacity curve coordinates and response parameter values are calculated for each of the pre-yield points. Arrays of these values are set up in *IDCapPoints* and *IDISDPoints*.

G.12 DemandCalculator.RecordReader

Subroutine to read the earthquake record data from the user specified file (Earthquake).

- Reads time and acceleration data from each row of the *Earthquake* file and converts them to numerical values.
- Outputs an array of the acceleration record (*Record0*) with *mnmax* rows.

G.13 DemandCalculator.TimeChange

Subroutine to re-interpret the earthquake record in terms of a smaller time interval if $\Delta t > T_y/50$ (condition that must not be satisfied if accurate results from the Newmark Integration Scheme are required).

- Linear variation of the acceleration between successive times in the record are assumed and acceleration values corresponding to intermediate times are calculated through extrapolation.
- Outputs a regenerated array of the acceleration record (*Record*) with *mnmax1* rows.

G.13 DemandCalculator.RecScale

Subroutine to scale the earthquake record by the desired scale factor.

- The acceleration values of the array *Record* are multiplied by the scale factor (SF).
- Outputs a scaled acceleration record array (*Record*) with *mnmax1* rows.

G.14 DemandCalculator.MainDemand

Subroutine that sets up the framework for the seismic demand determination.

- The points of the idealised capacity curve (*IDCapPoints*) and corresponding model parameters are used to characterise SDOF systems.

- Elastic dynamic analyses are carried out for SDOFs corresponding to the pre-yield points assuming $T=T_{eff}$ and for $T=T_y$.
- Inelastic dynamic analyses are carried out for the SDOFs corresponding to the post-yield points, using the Newmark Integration Scheme with $\gamma = 0.5$ and $\beta = 1/6$.
- Subroutines for the determination of the capacity point are called.
- The capacity point coordinates and corresponding response parameter values are output to the *MainCasp* subroutine.
- The demand and capacity points for every checked point are output to a file “C:/tmp/graphstlm” , “C:/tmp/graphsest”, “C:/tmp/graphsepp” according to the chosen model.
- If no answer is found for the capacity point by the end of the capacity curve plot, the structure is assumed to collapse. Notwithstanding, a fictitious capacity point is found by extending the capacity plot and incrementing the response values according to the parameter variation pattern observed in the last curve point.

G.15 DemandCalculator.Areas

Subroutine to calculate the area under the actual capacity curve to the points corresponding to the idealisation model slope changes. The areas to the S_{dc} values defining the yield, ultimate and final points are calculated from the capacity curve data array (either *CapCurve* or *CapPoints* is used according to the refinement of the arrays).

G.16 DemandCalculator.areapoint

Subroutine to calculate the area under the actual capacity curve to the point being checked. The area to the S_{dc} value defining the point being checked is calculated from the capacity curve data array (either *CapCurve* or *CapPoints* is used according to the refinement of the arrays).

G.17 DemandCalculator.modelfit1

Subroutine to assign hardening values to the EST and TLM models when the option of variable strain hardening with increasing ductility is selected.

- The area under the capacity curve to the current assessment point is used to define the hardening parameters corresponding to the point, through

consideration of equivalence of area between the idealised model curve and the actual pushover curve.

- In the case of EST S_{dcy} and S_{acy} are assumed constant in the calculation and the variable $\tan A$ defines the slope of the strain hardening curve.
- In the case of TLM for $S_{dc} \leq S_{dcu}$: S_{dcy} and S_{acy} are assumed constant in the calculation and the variable $\tan A$ defines the slope of the strain hardening curve. For $S_{dc} > S_{dcu}$: S_{dcy} , S_{acy} , S_{dcu} , S_{acu} and $\tan A$ are assumed constant in the calculation and the variable $\tan D$ defines the slope of the strain hardening curve.

G.18 DemandCalculator.modelfitconst

Subroutine to assign hardening values to the EST and TLM models when the option of constant strain hardening with increasing ductility is selected.

- In the case of EST $\tan A$ is defined from the yield and final point spectral coordinates, and is constant for all the post-yield capacity points checked.
- In the case of TLM $\tan A$ and $\tan D$ are defined from the yield, ultimate and final point spectral coordinates. All post-yield capacity curve points are attributed the same $\tan A$ and $\tan D$ values.

G.19 DemandCalculator.ElSpectrum1

Subroutine to calculate the elastic response of a SDOF of period T to an applied earthquake record using the Newmark Integration Scheme.

- The natural frequency and damping of the SDOF are determined as $\omega = 2\pi/T$ and $\xi = 0.05$.
- Each time-step of the earthquake record (*Record*) is run through and the acceleration and displacement response are calculated using the Newmark Integration Scheme with $\gamma = 0.5$ and $\beta = 1/6$ (corresponding to a linear assumption for the variation in acceleration).
- The maximum acceleration and displacement response values define the elastic spectral response of the SDOF (S_{ad} and S_{dd}).

G.20 DemandCalculator.NelSpectrum

Subroutine to calculate the inelastic response under an applied earthquake record of a SDOF of period T_y , ductility μ and non-linear response characteristics defined by the chosen model, using the Newmark Integration Scheme.

- The natural frequency and damping of the SDOF are determined as $\omega = 2\pi/T_y$ and $\xi = 0.05$, and an initial value of the maximum restoring

force of $0.999S_{adp}$ (where S_{adp} is the elastic spectral response) is calculated to commence iterations.

- The maximum number of iterations for each time-step (*ittmax*) and maximum tolerance for the effective force increment (*tol*) are set.
- The initial exponent value (*expon*) is defined. This value is modified during analysis for more rapid convergence.
- Each time-step of the earthquake record (*Record*) is run through and the acceleration and displacement response are calculated iteratively using the Newmark Integration Scheme with $\gamma = 0.5$ and $\beta = 1/6$ (corresponding to a linear assumption for the variation in acceleration).
- The maximum acceleration and displacement response values define the inelastic spectral response of the SDOF (S_{ad} and S_{dd}).

G.21 DemandCalculator.timeloop

Subroutine to calculate the acceleration and displacement response increment for each time-step of an inelastic dynamic response analysis, using the Newmark Integration Scheme.

- Acceleration, velocity and displacement response values for the current time –step are estimated from the results of the previous time-step.
- The appropriate model idealisation subroutine is called for the calculation of the response acceleration resulting from the application of the estimated displacement.
- Force equilibrium is checked and if the tolerance is exceeded the process is repeated using updated estimates of the acceleration, velocity and displacement responses. If the tolerance is satisfied the iterations are stopped and the time-step response acceleration and displacement values determined.

G.22 DemandCalculator.EPPModel

Subroutine to calculate the acceleration and displacement response for a system exhibiting elastic-perfectly plastic behaviour (EPP). The acceleration value corresponding to the displacement value input into the subroutine is calculated from an EPP model curve characterised by the yield point coordinates of the capacity curve.

G.23 DemandCalculator.ESModel

Subroutine to calculate the acceleration and displacement response for a system exhibiting elastic-linear strain hardening behaviour (EST). The acceleration value

corresponding to the displacement value input into the subroutine is calculated from an EST model curve characterised by the yield point coordinates and $\tan A$ value assigned to the checked capacity curve point.

G24. DemandCalculator.TLMModel

Subroutine to calculate the acceleration and displacement response for a system exhibiting tri-linear behaviour (TLM). The acceleration value corresponding to the displacement value input into the subroutine is calculated from an TLM model curve characterised by the yield and ultimate point coordinates and $\tan A$ and $\tan D$ values assigned to the checked capacity curve point.

G.25 Assessment.Capidentify2 and 22

Subroutines to check if the capacity point is reached between the current checking point and that preceding it.

- The calculated demand spectral values for the current checking point (S_{ad} and S_{dd}) and that preceding it, are compared to the idealised capacity curve points.
- For each capacity point, the program goes through all the demand points. If the crossing point of the capacity and demand lines is not reached, the next capacity point is checked.
- If the crossing point is identified the subroutines for extrapolating the response parameters is called. (The idealised curve is used).

G.26 Assessment.Capidentify3 and 4

Subroutine to check if the capacity point is reached between the current checking point and that preceding it.

- This subroutine is called in the case that the variable strain hardening option is selected for the pushover curve modelling.
- The calculated demand spectral values for the current checking point (S_{ad} and S_{dd}) and that preceding it, are compared to the idealised capacity curve points.
- Account is taken of the fact that different hardening characteristics are associated with the two capacity points in assessing the occurrence of the crossing point.

- If the line joining the two demand points crosses the capacity curve, the performance point (*PP*) is reached, the response values corresponding to *PP* are determined from extrapolation.

G.27 Assessment.nonlin2

Subroutine to find the crossing point (*PP*) if this is in the elastic range of the pushover curve

G.28 Assessment.Assign

Subroutine to find the position of the Performance Point (*PP*) if this is in the non-linear portion of the pushover curve.

G.29 Assessment.YldperISID

Subroutine to find the response parameters associated with the structure at occurrence of the Performance Point (*PP*). The idealised curves are compared and the response parameters are found.

Appendix H

Analytical vulnerability Curves for 3 Storey Infilled Frames Designed to the Italian Seismic Codes of 1982, and their Comparison with Observational Post-Earthquake Damage Distributions.

Vulnerability Curve		Mean			95% Upper Bound			5% Lower Bound		
RTP ⁽¹⁾	Damage State	μ ⁽²⁾	σ ⁽³⁾	R^2 ⁽⁴⁾	μ	σ	R^2	μ	σ	R^2
95YRP	Slight	-7.80	0.60	0.996	-8.25	0.60	0.998	-6.30	0.42	0.997
	Light	-7.15	0.40	0.999	-7.82	0.60	0.998	-5.76	0.34	0.998
	Moderate	-5.72	0.20	0.954	-6.21	0.25	0.953	-4.37	0.15	0.979
	Extensive	-4.35	0.20	0.981	-4.84	0.19	0.984	-3.03	0.12	0.974
	P.C.	-3.45	0.18	0.986	-3.93	0.17	0.982	-2.16	0.11	0.992
	Collapse	-3.01	0.16	0.989	-3.50	0.17	0.990	-1.71	0.10	0.991
475YRP)	Slight	-7.65	0.22	0.996	-8.10	0.21	0.998	-6.30	0.22	0.997
	Light	-7.15	0.21	0.998	-7.60	0.21	0.999	-5.83	0.22	0.998
	Moderate	-5.78	0.21	0.998	-6.32	0.20	0.997	-4.51	0.22	0.999
	Extensive	-4.44	0.21	0.997	-4.92	0.21	0.996	-3.12	0.22	0.996
	P.C.	-3.56	0.21	0.999	-4.04	0.21	0.998	-2.22	0.22	0.997
	Collapse	-3.11	0.20	0.997	-3.60	0.21	0.998	-1.78	0.22	0.997
2475YRP	Slight	-7.60	0.40	0.996	-8.00	0.40	1.000	-6.15	0.28	0.999
	Light	-6.95	0.30	0.998	-7.60	0.40	1.000	-5.68	0.28	0.998
	Moderate	-5.68	0.23	0.997	-6.19	0.23	0.997	-4.30	0.28	1.000
	Extensive	-4.34	0.22	0.998	-4.83	0.22	0.998	-2.99	0.27	0.998
	P.C.	-3.49	0.22	0.998	-3.98	0.22	1.000	-2.12	0.27	0.995
	Collapse	-2.99	0.22	0.997	-3.50	0.22	0.998	-1.67	0.24	0.997
ALLYRP	Slight	-7.60	0.40	1.000	-7.80	0.40	0.999	-6.20	0.25	0.999
	Light	-7.30	0.40	1.000	-7.55	0.40	1.000	-5.71	0.20	0.997
	Moderate	-5.71	0.19	0.999	-6.24	0.19	0.998	-4.41	0.17	0.998
	Extensive	-4.39	0.16	0.998	-4.85	0.16	0.997	-3.10	0.17	0.999
	P.C.	-3.50	0.15	0.999	-3.98	0.15	0.999	-2.17	0.17	0.998
	Collapse	-3.05	0.14	0.996	-3.53	0.14	0.995	-1.72	0.15	0.998

⁽¹⁾ Return period of record suite used in the analyses for the response surface; ⁽²⁾ and ⁽³⁾ Mean and standard deviation of the fitted cumulative lognormal distribution; ⁽⁴⁾ Correlation coefficient of data used in the derivation with the vulnerability curve.

Table H. 1: Summary of the analytical 95, 475 and 2475YRP scenario (and ALL scenario) HRC damage state vulnerability curve parameters for the three storey infilled frame system, using $T=0.125s$ to evaluate the spectral displacement characterising the seismic demand.

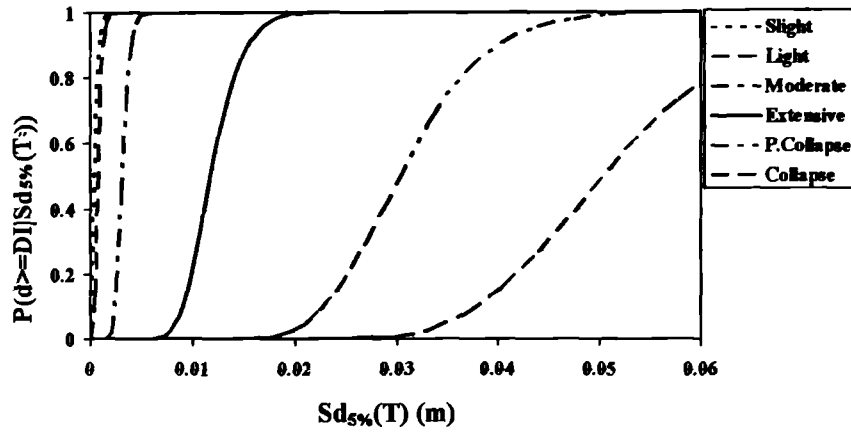


Figure H. 1: Mean performance consistent analytical curves for three storey infilled frames designed to old seismic codes. The HRC damage scale is used to derive vulnerability curves with $T=0.125$ s assumed for the spectral displacement evaluation.

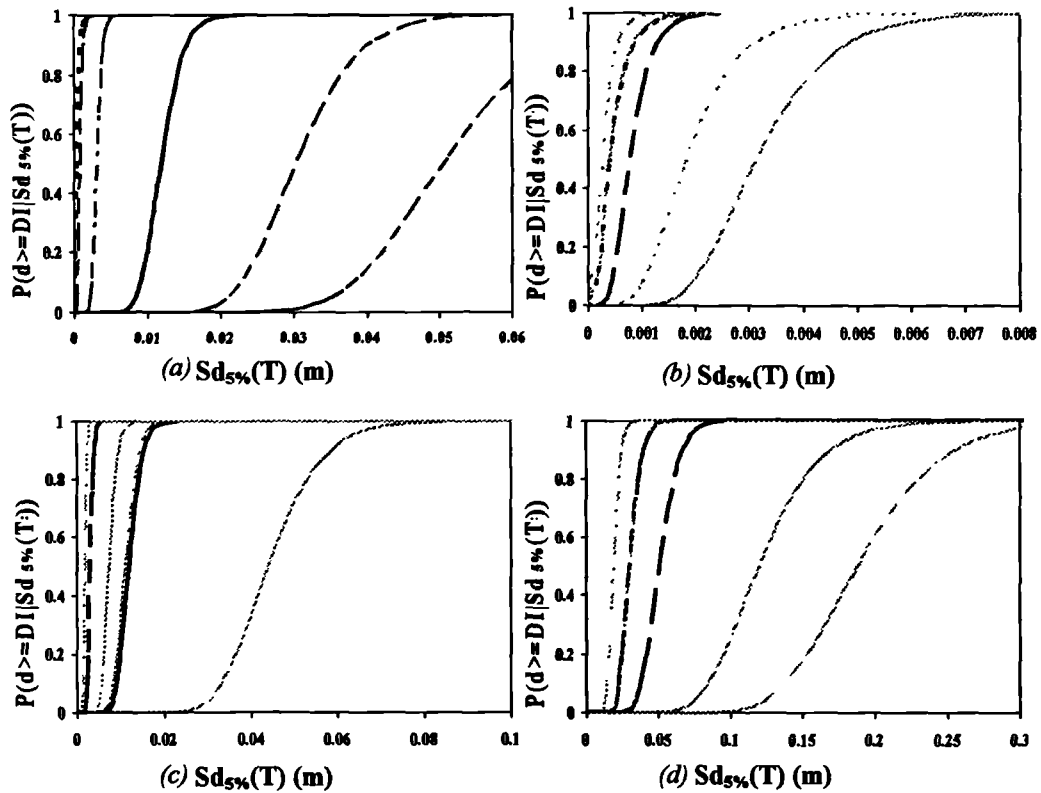


Figure H. 2: The proposed performance consistent analytical vulnerability curves (dark lines) and 90% confidence bounds (light lines) for the 3-storey infilled frame system using the HRC damage scale and $S_d(T=0.125s)$: (a) Mean curves; (b) Slight and Light damage states; (c) Moderate and Extensive damage states; (d) Partial Collapse and Collapse damage states

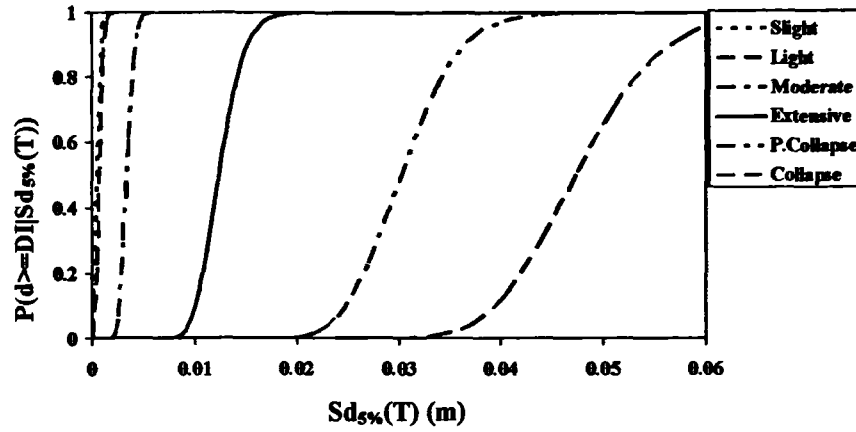


Figure H. 3: Mean analytical curves for a three storey infilled frame system designed to old seismic codes. ALLYRP generated damage data and the HRC damage scale are used to derive vulnerability curves with $T=0.125s$ assumed in the spectral displacement determination.

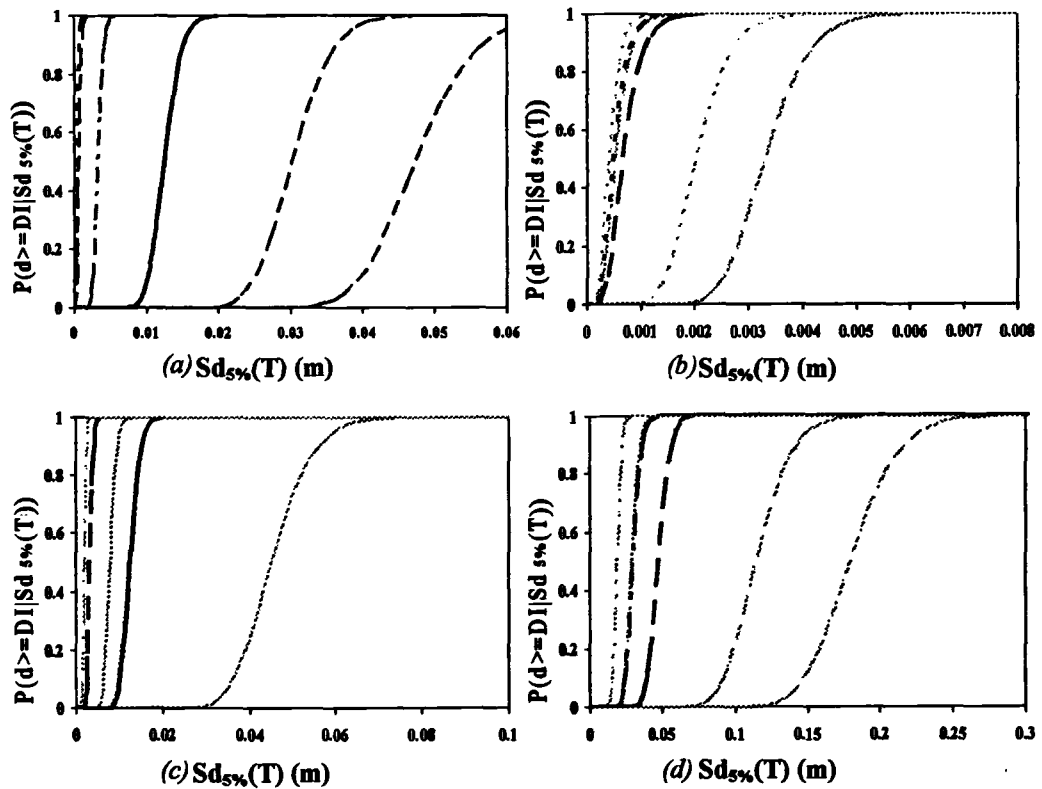


Figure H. 4: The proposed analytical vulnerability curves (dark lines) and 90% confidence bounds (light lines) for the 3-storey infilled frame system using ALLYRP response surface, HRC damage scale and $S_d(T=0.125s)$: (a) Mean curves; (b) Slight and Light damage states; (c) Moderate and Extensive damage states; (d) Partial Collapse and Collapse damage states

Vulnerability Curve		Mean			95% Upper Bound			5% Lower Bound		
RTP ⁽¹⁾	Damage State	$\mu^{(2)}$	$\sigma^{(3)}$	$R^2^{(4)}$	μ	σ	R^2	μ	σ	R^2
Mixed	Slight	-7.60	0.30	0.994	-7.62	0.30	0.993	-6.17	0.30	0.994
	Light	-7.20	0.25	0.996	-6.15	0.25	0.994	-5.60	0.25	0.995
	Mod. to Collapse	Same equations as 2475YRP								

⁽¹⁾ Return period of record suite used in the analyses for the response surface; ⁽²⁾ and ⁽³⁾ Mean and standard deviation of the fitted cumulative lognormal distribution; ⁽⁴⁾ Correlation coefficient of data used in the derivation with the vulnerability curve.

Table H. 2: Summary of the analytical HRC damage state vulnerability curve parameters for the three storey infilled frame system, using performance consistent ground motion intervals, and and $S_d(T=0.125s)$.

Vulnerability Curve		Mean			95% Upper Bound			5% Lower Bound		
RTP ⁽¹⁾	Damage State	$\mu^{(2)}$	$\sigma^{(3)}$	$R^2^{(4)}$	μ	σ	R^2	μ	σ	R^2
95YRP	Serviceability	-7.85	0.60	0.998	-7.81	0.40	1.000	-6.50	0.40	0.998
	Damage Control	-6.17	0.30	0.995	-7.36	0.40	0.999	-5.15	0.20	0.997
	Limited Safety	-4.36	0.16	0.997	-5.36	0.16	0.998	-3.37	0.11	0.997
	Collapse Prevention	-3.89	0.16	0.998	-4.95	0.12	0.999	-2.94	0.10	0.996
475YRP	Serviceability	-7.90	0.28	1.000	-8.27	0.28	1.000	-6.55	0.28	0.999
	Damage Control	-6.25	0.25	0.999	-7.40	0.25	1.000	-5.27	0.25	0.999
	Limited Safety	-4.47	0.22	0.997	-5.49	0.22	0.997	-3.49	0.24	0.997
	Collapse Prevention	-4.00	0.22	0.996	-5.05	0.21	0.998	-3.04	0.23	0.996
2475YRP	Serviceability	-7.40	0.32	1.000	-7.80	0.32	1.000	-6.48	0.32	0.998
	Damage Control	-6.13	0.31	0.996	-7.22	0.33	1.000	-5.16	0.25	0.995
	Limited Safety	-4.35	0.22	0.997	-5.37	0.22	0.998	-3.36	0.22	0.997
	Collapse Prevention	-3.90	0.22	0.998	-4.92	0.20	0.999	-2.95	0.20	0.999

⁽¹⁾ Return period of record suite used in the analyses for the response surface; ⁽²⁾ and ⁽³⁾ Mean and standard deviation of the fitted cumulative lognormal distribution; ⁽⁴⁾ Correlation coefficient of data used in the derivation with the vulnerability curve.

Table H. 3: Summary of the analytical FEMA damage state vulnerability curve parameters for the three storey infilled frame system, derived using $S_d(T=0.125s)$ to characterise the ground motion demand and different scenario earthquake data.

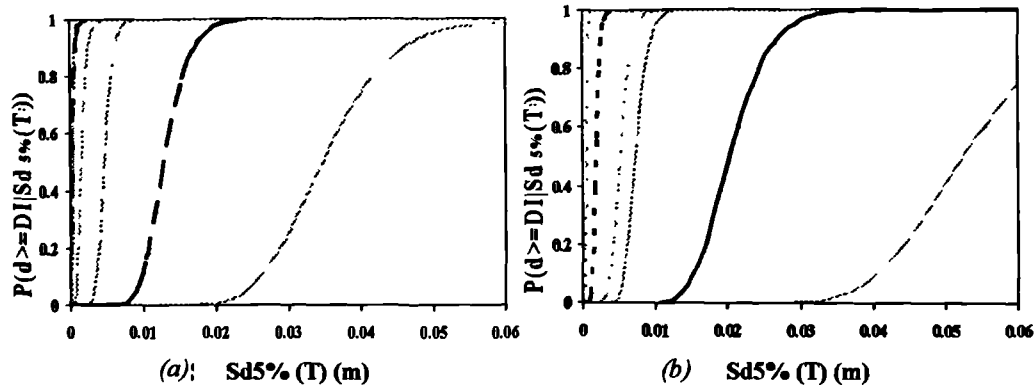


Figure H. 5: The proposed performance consistent analytical vulnerability curves (dark lines) and 90% confidence bounds (light lines) for the 3-storey infilled frame system using the FEMA 273 (ATC 1997) damage scale and $S_d(T=0.125s)$: (a) Serviceability and Limited Safety damage states; (b) Damage Control and Collapse Prevention damage states

Damage State	No. of Bldgs ⁽¹⁾	Non-Weighted Data				Weighted Data			
		Data-sets ⁽²⁾	$\mu(\epsilon)$ ⁽³⁾	COV(ϵ) ₍₄₎	R^2 ⁽⁵⁾	Data-sets	$\mu(\epsilon)$	COV(ϵ)	R^2
		Infilled Frame Data							
Slight	2720	8	0.235	0.922	0.465	2	0.388	0.663	0.450
Light	2720	8	0.261	0.896	0.578	2	0.448	0.557	0.464
Moderate	2720	8	0.391	0.699	0.511	2	0.363	0.991	0.608
Extensive	2460	7	0.008	0.971	0.378	2	0.009	0.474	0.474
		Infilled Frame Data Modified for (T/2)							
Slight	2720	8	0.179	1.097	0.624	2	0.356	0.597	0.461
Light	2720	8	0.217	0.686	0.665	2	0.316	0.203	0.537
Moderate	2720	8	0.180	1.023	0.559	2	0.096	0.478	0.832
Extensive	2460	7	0.009	0.776	0.397	2	0.009	0.506	0.470

⁽¹⁾ Number of buildings; ⁽²⁾ Number of observational datasets; ⁽³⁾ Mean absolute error in the observational exceedence probability prediction ($\epsilon = \text{Abs}(p-o)$); ⁽⁴⁾ Coefficient of variation of the prediction error; ⁽⁵⁾ Correlation coefficient of the analytical predictions with the observational data.

Table H. 4: Summary of the fit of the performance consistent HRC damage state vulnerability curves for the three storey infilled frame system (derived using $S_{d5\%}(T=0.125s)$), with observational data for infilled frames. The comparison is made assuming the spectral displacement demand associated with the observations is determined from structure natural periods that are determined from the Eurocode 8 (CEN 1996) expression (Eq. 4.4), and half this value. (n.b. No data available for High damage state comparisons)

Damage State	No. of Bldgs ⁽¹⁾	Non-Weighted Data				Weighted Data			
		Data-sets ⁽²⁾	$\mu(\varepsilon)$ ⁽³⁾	COV(ε) ₍₄₎	R^2 ⁽⁵⁾	Data-sets	$\mu(\varepsilon)$	COV(ε)	R^2
		All Worldwide Observational Data							
Slight	334976	96	0.548	0.454	0.456	15	0.550	0.411	0.463
Light	334763	90	0.651	0.450	0.459	15	0.657	0.402	0.465
Moderate	327011	67	0.732	0.339	0.481	14	0.734	0.342	0.493
Extensive	325272	54	0.664	0.590	0.510	12	0.686	0.546	0.509
P.C.	288373	36	0.688	0.538	0.503	11	0.512	0.941	0.501
Collapse	73144	22	0.180	1.077	0.499	5	0.247	1.738	0.500
		European Earthquake Data							
Slight	298791	50	0.541	0.473	0.455	13	0.506	0.428	0.461
Light	298578	44	0.670	0.496	0.455	13	0.635	0.435	0.460
Moderate	294784	24	0.633	0.453	0.482	10	0.654	0.394	0.499
Extensive	293844	18	0.444	1.013	0.527	9	0.656	0.569	0.516
P.C.	257624	8	0.608	0.786	0.503	7	0.688	0.658	0.502
Collapse	66172	2	0.594	0.968	0.496	2	0.590	0.983	0.496
		Campania Earthquake (1980, Italy) Data							
Slight	3306	28	0.687	0.203	0.490	8	0.663	0.155	0.495
Light	3093	22	0.904	0.163	0.494	7	0.909	0.109	0.497
Moderate	372	3	0.828	0.300	0.485	2	0.957	0.044	0.500
Extensive	25	1	0.727	0.000	0.500	1	0.727	0.000	0.500
P.C.	25	1	0.819	0.000	0.500	1	0.816	0.000	0.500
Collapse	25	1	0.187	0.000	0.500	1	0.180	0.000	0.500

⁽¹⁾ Number of buildings; ⁽²⁾ Number of observational datasets; ⁽³⁾ Mean absolute error in the observational exceedence probability prediction ($\varepsilon = \text{Abs}(p-o)$); ⁽⁴⁾ Coefficient of variation of the prediction error; ⁽⁵⁾ Correlation coefficient of the analytical predictions with the observational data.

Table H. 5: Summary of the fit of the performance consistent HRC damage state vulnerability curves for the three storey infilled frame system (derived using $S_{d3\%}(T=0.125s)$), with observational data collected in Chapter 5. The comparison is made with all worldwide and European data, regardless of structural typology, survey location and earthquake event, and with data specifically from the Campania (Italy 1980) earthquake, (.population of predominantly low-rise infilled frames designed to old seismic codes).

Damage State	No. of Bldgs ⁽¹⁾	Non-Weighted Data				Weighted Data			
		Data-sets ⁽²⁾	$\mu(\varepsilon)$ ⁽³⁾	$COV(\varepsilon)$ ⁽⁴⁾	R^2 ⁽⁵⁾	Data-sets	$\mu(\varepsilon)$	$COV(\varepsilon)$	R^2
		Low-Rise Building Data							
Slight	10395	50	0.595	0.393	0.468	10	0.598	0.232	0.488
Light	10182	44	0.735	0.402	0.471	10	0.756	0.303	0.480
Moderate	4992	23	0.734	0.386	0.475	7	0.745	0.437	0.491
Extensive	3961	17	0.213	1.466	0.524	6	0.434	0.922	0.530
P.C.	1743	7	0.141	2.155	0.539	4	0.212	1.899	0.546
Collapse	1526	5	0.044	1.840	0.520	4	0.046	1.927	0.537
		Old Seismic Code Building Data							
Slight	308760	32	0.561	0.454	0.455	13	0.520	0.465	0.455
Light	308760	32	0.642	0.417	0.461	13	0.584	0.417	0.463
Moderate	308713	30	0.810	0.225	0.488	13	0.769	0.230	0.488
Extensive	306859	24	0.687	0.588	0.503	11	0.762	0.420	0.504
P.C.	272137	16	0.579	0.782	0.500	11	0.508	0.948	0.501
Collapse	71642	6	0.255	1.469	0.500	4	0.300	1.587	0.500
		Old Seismic Code Infilled Frame Data							
Slight	933	3	0.323	0.955	0.384	2	0.360	1.044	0.393
Light	933	3	0.386	0.870	0.399	2	0.413	0.954	0.407
Moderate	933	3	0.687	0.378	0.477	2	0.690	0.394	0.481
Extensive	933	3	0.131	1.699	0.502	2	0.196	1.372	0.501
		Low-Rise Infilled Frame Data							
Slight	2142	7	0.271	0.813	0.478	3	0.497	0.365	0.479
Light	2142	7	0.300	0.810	0.575	3	0.564	0.292	0.486
Moderate	2142	7	0.502	0.634	0.452	3	0.585	0.700	0.508
Extensive	1882	6	0.072	2.144	0.500	3	0.134	1.618	0.500
		Low-Rise, Old Seismic Code Infilled Frame Data							
Slight	868	2	0.480	0.426	0.478	2	0.480	0.426	0.478
Light	868	2	0.565	0.318	0.488	2	0.565	0.318	0.488
Moderate	868	2	0.835	0.080	0.499	2	0.835	0.080	0.499
Extensive	868	2	0.196	1.379	0.502	2	0.196	1.379	0.502

⁽¹⁾ Number of buildings; ⁽²⁾ Number of observational datasets; ⁽³⁾ Mean absolute error in the observational exceedence probability prediction ($\varepsilon = \text{Abs}(p-o)$); ⁽⁴⁾ Coefficient of variation of the prediction error; ⁽⁵⁾ Correlation coefficient of the analytical predictions with the observational data.

Table H. 6: Summary of the fit of the performance consistent HRC damage state vulnerability curves for the three storey infilled frame system (derived using $S_{aT=0.125s}$), with observational data collected in Chapter 5. The comparison is made with all worldwide and European data, regardless of structural typology, survey location and earthquake event, and with data specifically from the Campania (Italy 1980) earthquake, (population of predominantly low-rise infilled frames designed to old seismic codes).