

Performance-based seismic analysis of reinforced concrete buildings accounting for the angle of seismic incidence

Despoina Skoulidou

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Advisor: Xavier das Neves Romão

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Abstract

Although earthquake engineering has significantly advanced during the past decades, recent examples of seismic events that resulted in disproportionately large losses, including, among others, human and economic losses, prove that the discipline still requires improvements. The weaknesses often met in current practice, usually reflect shortcomings in up-to-date standard provisions and guidelines, thus rendering the revision and the improvement of the latter an imperative task.

In light of this, the present thesis addresses the active area of research dealing with the angle of incidence of the seismic action during structural analysis of buildings. Particularly, the main objective of the thesis is to determine the effect of the angle of seismic incidence and to provide methods to account for it in the context of seismic safety assessment of existing buildings. In accordance with commonly used seismic standards, the input seismic action can be represented either by a suite of ground motion records or by a single response spectrum. As such, two different approaches are applied with respect to the determination of the effect of the angle of seismic incidence, depending on the method of representing the input seismic action.

In the first approach, which occupies the largest part of the thesis, the Performance Based Earthquake Engineering (PBEE) framework proposed by the Pacific Earthquake Engineering Research Centre is applied, explicitly involving uncertainty in the representation of the seismic action. After the numerical modelling of six reinforced concrete (RC) building and the determination of their dynamic characteristics, seismic hazard analysis of the site is performed followed by a state-of-the-art method of ground motion record selection. Groups of ground motions of different sizes are selected to excite the six buildings and the effect of the angle of seismic incidence is defined in all subsequent relevant stages of the procedure. Since the ground motion group size is known to have a significant effect on the seismic response as well, the combined effect of the ground motion group size and that of the angle of seismic incidence is examined.

The study of the effect of the angle of seismic incidence and the ground motion group size starts from the structural analysis stage, and involves post processing of groups of data of selected engineering demand parameters. A probabilistic approach is adopted

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examining both location parameters, commonly used by most current seismic standards, and dispersion parameters, i.e. information relevant to the statistical distribution characterisation of the data. Moreover, the statistical models that best describe the distribution of the aforementioned engineering demand parameters are also analysed. Subsequently, the effect of the angle and the group size on the damage stage is examined, involving the analysis of the collapse fragility functions, as well as of the probability of collapse of the buildings. Finally, the effect of the angle of seismic incidence and the group size on the decision variable stage is studied. The decision variable in the current study corresponds to direct monetary losses, including structural and non-structural components of the RC buildings. The overall outcome of the analysis is the proposal of an optimum combination of number of angles of seismic incidence with a ground motion group size in order to reduce the bias and the variability of the output involved in the different analysis stages down to acceptable levels. The overall results show that different stages are affected differently by the angle of seismic incidence, with smaller effects being found at later stages of the PBEE.

In the second approach, the seismic action is represented by a response spectrum and the Lateral Force method of analysis is employed. The deterministic nature of the seismic action representation in this case requires the determination of the angle of seismic incidence that leads to the most unfavourable response (i.e. the critical angle), instead of an average response obtained considering multiple angles. The linear elastic material properties and the piecewise definition of the response spectrum used in the analysis allow for the derivation of analytical expressions to straightforwardly calculate the critical angle of seismic incidence in this case. Such expressions are provided using a standard-compatible response spectrum and the critical angles of seismic incidence are determined for selected RC building categories.

A summary of the proposed methods to account for the angle of seismic incidence in the seismic assessment of RC building is provided in the last part of the thesis, accompanied by their importance as well as possible difficulties that may arise during their application. Finally, the shortcomings and limitations of the present study are further highlighted, which leads to a series of suggestions for future work on the topic.

Resumo

Apesar os desenvolvimentos importantes alcançados pela engenharia sísmica durante as últimas décadas, as perdas desproporcionalmente grandes que resultaram de eventos sísmicos recentes, incluindo, entre outras, perdas humanas e económicas, mostram que ainda são necessários avanços significativos neste domínio. As limitações frequentemente encontradas na prática atual refletem, em geral, limitações da regulamentação, o que enfatiza a importância de introduzir melhorias e de atualizar essa regulamentação.

Neste contexto, a presente tese aborda questões relacionadas com os efeitos do ângulo de incidência sísmica durante a análise estrutural de edifícios. Em particular, o principal âmbito da tese é quantificar os efeitos do ângulo de incidência sísmica e fornecer métodos para ter em conta esses efeitos no contexto da avaliação de segurança sísmica de edifícios existentes. Neste contexto, são utilizadas duas abordagens diferentes que dependem da representação da ação sísmica, e o efeito do ângulo de incidência sísmica é examinado com base na análise de seis edifícios de betão armado. De acordo com a regulamentação existente, a ação sísmica é representada com base em vários registos sísmicos ou através dum espectro de resposta.

Na abordagem que envolve o primeiro tipo de representação, e que ocupa a maior parte da tese, a análise é feita com base na metodologia de *Performance Based Earthquake Engineering* (PBEE) desenvolvida pelo *Pacific Earthquake Engineering Research Center*, envolvendo a incerteza na representação da ação sísmica. De acordo com esta metodologia, após desenvolver modelos numéricos das seis estruturas e determinar as suas características dinâmicas, a análise de perigosidade sísmica do local é realizada, sendo seguida duma seleção de registos sísmicos. Grupos de registos sísmicos com diferentes tamanhos são selecionados para excitar os seis edifícios e o efeito do ângulo de incidência sísmica é analisado em todas as etapas relevantes do procedimento. Sendo sabido que o tamanho do grupo de registos sísmicos tem um efeito significativo sobre a resposta sísmica, o efeito combinado do tamanho do grupo de registos sísmicos e do ângulo de incidência sísmica é examinado.

O estudo do efeito do ângulo de incidência sísmica começa na etapa de análise estrutural e envolve o pós-processamento de grupos de dados correspondentes a parâmetros de

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resposta selecionados. A análise desses dados é feita com base numa abordagem probabilística onde se determinam medidas de tendência central, usualmente consideradas pela maioria dos regulamentos sísmicos atuais, e medidas de dispersão dos dados, ambos fatores relevantes para definir uma distribuição estatística dos dados. Além disso, são também analisadas as distribuições estatísticas que melhor se ajustam a esses dados. Posteriormente, o efeito do ângulo de incidência sísmica sobre a análise de dano na metodologia PBEE é examinado, envolvendo a análise das curvas de fragilidade de colapso, bem como da probabilidade de colapso dos edifícios. Finalmente, o efeito do ângulo de incidência sísmica nas perdas monetárias diretas, incluindo componentes estruturais e não estruturais dos edifícios, é igualmente estudado. Em termos de resultados gerais, as análises realizadas permitem propor uma combinação ótima do número de ângulos de incidência sísmica e do tamanho do grupo de registos sísmicos que minimize o enviesamento e a variabilidade dos resultados nas diferentes fases da metodologia PBEE. Globalmente, observa-se que os diferentes estágios da metodologia PBEE são afetados de forma diferente pelo ângulo de incidência sísmica, sendo que os efeitos são menores em estágios mais finais da metodologia.

Na abordagem em que a ação sísmica é representada por um espectro de resposta, a análise é feita com base no Método das Forças Laterais. A natureza determinística da ação sísmica, neste caso, requer a determinação do ângulo de incidência sísmica que leva à resposta mais desfavorável (i.e. o ângulo crítico de incidência), em vez de se considerar uma resposta média obtida considerando vários ângulos possíveis. O facto desta abordagem envolver propriedades elásticas dos materiais e do espectro de resposta usado na análise ser definido por troços permitem derivar expressões analíticas para calcular diretamente o ângulo crítico de incidência sísmica. Essas expressões são definidas considerando um espectro de resposta compatível com os espectros regulamentares atuais e os ângulos críticos de incidência sísmica são determinados para vários edifícios.

A última parte da tese apresenta um resumo dos métodos propostos para ter em conta os efeitos do ângulo de incidência sísmica na avaliação sísmica de edifícios de betão armado, destacando a sua importância, bem como possíveis dificuldades que possam surgir na sua aplicação. Por fim, destacam-se algumas limitações do presente estudo, as quais permitem identificar várias sugestões de trabalhos futuros sobre este tópico.

Περίληψη

Παρόλο που ο κλάδος της αντισεισμικής μηχανικής έχει γνωρίσει σημαντική ανάπτυξη τις τελευταίες δεκαετίες, πρόσφατα παραδείγματα σεισμών που είχαν ως αποτέλεσμα δυσανάλογα μεγάλες απώλειες, ζωνών και οικονομικών, μεταξύ άλλων, αποδεικνύουν ότι η βελτίωσή του είναι απαραίτητη. Οι αδυναμίες που συχνά συναντώνται στην πράξη οφείλονται κυρίως σε ελλείψεις και σε ασάφειες των κανονιστικών κειμένων και των ισχυόντων αντισεισμικών κανονισμών, καθιστώντας την αναθεώρησή τους επιτακτική ανάγκη.

Η παρούσα διατριβή εξετάζει τον τρόπο εφαρμογής της γωνίας πρόσπτωσης της σεισμικής δράσης κατά την ανάλυση κτιρίων. Συγκεκριμένα ο κύριος στόχος της διατριβής είναι να προσδιορίσει την επίδραση που έχει η γωνία πρόσπτωσης στην σεισμική απόκριση κτιριακών κατασκευών και να παρέχει μεθόδους ώστε να ληφθεί υπόψη κατά την αποτίμηση της σεισμικής συμπεριφοράς υφιστάμενων κτιρίων. Δύο διαφορετικές προσεγγίσεις εφαρμόζονται για τη μελέτη της γωνίας πρόσπτωσης, ανάλογα με τον τρόπο προσομοίωσης της σεισμικής δράσης. Συγκεκριμένα, σύμφωνα με ισχύοντα κανονιστικά κείμενα, η σεισμική δράση μπορεί να περιέχει αβεβαιότητα, όπως για παράδειγμα με τη χρήση ομάδας επιταχυνσιογραφημάτων, ή να είναι αιτιοκρατικής φύσεως, όπως για παράδειγμα με τη χρήση ενός κανονικοποιημένου φάσματος απόκρισης.

Στην πρώτη προσέγγιση, η οποία καταλαμβάνει το μεγαλύτερο μέρος της διατριβής, εφαρμόζονται μέθοδοι ανάλυσης με βάση την επιτελεστικότητα και συγκεκριμένα ακολουθείται η μέθοδος, που παρέχεται από το Pacific Earthquake Engineering Research Center, χρησιμοποιώντας αβεβαιότητα στην προσομοίωση της σεισμικής δράσης. Ακολουθώντας τη μέθοδο αυτή, μετά την προσομοίωση έξι κτιρίων οπλισμένου σκυροδέματος (Ο/Σ) χρησιμοποιώντας κατάλληλο λογισμικό και την εξαγωγή των ιδιομορφικών χαρακτηριστικών τους, πραγματοποιείται ανάλυση σεισμικής επικινδυνότητας του πεδίου και στη συνέχεια γίνεται επιλογή κατάλληλων επιταχυνσιογραφημάτων. Ομάδες επιταχυνσιογραφημάτων διαφορετικού μεγέθους επιλέγονται για τη διέγερση των έξι κτιρίων και η επιρροή της γωνίας πρόσπτωσης εξετάζεται σε όλα τα ακόλουθα στάδια της μεθόδου. Δεδομένου ότι έχει αποδειχθεί πως

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το μέγεθος της ομάδας των επιταχυνσιογραφημάτων έχει σημαντική επιρροή στα αποτελέσματα της σεισμικής ανάλυσης, η συνδυασμένη επίδραση της γωνίας πρόσπτωσης και του μεγέθους της ομάδας επιταχυνσιογραφημάτων αναλύονται επίσης.

Η επιρροή της γωνίας πρόσπτωσης και του μεγέθους της ομάδας εξετάζεται αρχικά στο δεύτερο στάδιο της μεθόδου, το οποίο περιλαμβάνει την ανάλυση των κτιρίων, μέσω της στατιστικής επεξεργασίας επιλεγμένων παραμέτρων απόκρισης. Οι παράμετροι απόκρισης εξετάζονται πιθανοτικά χρησιμοποιώντας μέτρα θέσης και μέτρα διασποράς των κατανομών τους. Επιπλέον, μελετάται το στατιστικό μοντέλο που μπορεί να χρησιμοποιηθεί ώστε να προσομοιώσει με μεγαλύτερη ακρίβεια και μικρότερο σφάλμα τις κατανομές των παραπάνω παραμέτρων απόκρισης. Στη συνέχεια, η επιρροή της γωνίας πρόσπτωσης και του μεγέθους της ομάδας επιταχυνσιογραφημάτων εξετάζεται στο στάδιο του υπολογισμού των απωλειών και συγκεκριμένα ως προς την καμπύλη τρωτότητας κατάρρευσης και ως προς την πιθανότητα κατάρρευσης των κατασκευών. Τέλος, η επιρροή της γωνίας πρόσπτωσης και του μεγέθους της ομάδας επιταχυνσιογραφημάτων εξετάζεται ως προς το κόστος επισκευής ή/και ανακατασκευής των κτιρίων. Στην παρούσα διατριβή το συνολικό κόστος λαμβάνεται ως το κόστος επισκευής δομικών και μη δομικών στοιχείων της κατασκευής. Το συνολικό αποτέλεσμα της ανάλυσης είναι η πρόταση ενός βέλτιστου συνδυασμού αριθμού γωνιών πρόσπτωσης και μεγέθους ομάδας επιταχυνσιογραφημάτων, ώστε να επιτευχθεί μείωση της επικινδυνότητας, όπως αυτή εκφράζεται στα διάφορα στάδια της ανάλυσης, σε ικανοποιητικά επίπεδα. Σημαντικό εύρημα των αναλύσεων είναι το ότι μεταγενέστερα στάδια της μεθόδου έχουν μικρότερη επιρροή της γωνίας πρόσπτωσης.

Στη δεύτερη προσέγγιση, η σεισμική δράση αντιπροσωπεύεται από ένα κανονικοποιημένο φάσμα απόκρισης και η ανάλυση των κατασκευών πραγματοποιείται χρησιμοποιώντας την Ισοδύναμη Στατική μέθοδο. Η διαφορά αυτής της προσέγγισης με την προηγούμενη έγκειται στο ότι η σεισμική δράση είναι αιτιοκρατικής φύσεως και άρα απαιτείται η γωνία πρόσπτωσης που οδηγεί στην πιο δυσμενή απόκριση, αντί μιας μέσης απόκρισης προκαλούμενης από πλήθος γωνιών. Ο γραμμικός ελαστικός νόμος των υλικών και ο τμηματικός ορισμός του φάσματος απόκρισης επιτρέπουν την εξαγωγή αναλυτικών λύσεων για τον υπολογισμό της κρίσιμης γωνίας πρόσπτωσης. Αναλυτικές λύσεις εξάγονται χρησιμοποιώντας ένα κανονικοποιημένο φάσμα απόκρισης σύμφωνα με κανονιστικά κείμενα και η κρίσιμη γωνία πρόσπτωσης υπολογίζεται για συγκεκριμένες κατηγορίες κτιρίων και παραμέτρους απόκρισης.

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Chapter 1

1 Introduction

1.1 Background and Motivation

The existing building stock has only recently been acknowledged to have a major effect on seismic risk, and generally in economic losses including human casualties (Franchin et al. 2010). In fact, consequences caused by different hazards¹ depend on the specific characteristics of the hazard, on the location where the hazard occurs as well as on the assets which are exposed (Faber 2008). Consequences, also termed losses, should be envisaged as a general concept that includes loss of lives, injuries, damage to the qualities of the environment and economic losses. Losses can be seen to depend on hazard, vulnerability² and exposure³. While hazard is determined by nature (in the particular case of earthquakes), the assessment of which is a matter of research, vulnerability and exposure are human-determined parameters that can be set under control and reduced, respectively.

To shed some more light on those concepts, examples based on data collected from past significant earthquake events are presented in Figure 1.1. It can be observed that earthquake events in different areas led to significantly diversified overall losses, casualties as well as structural damage. For instance, the Christchurch earthquake in New

¹ Hazards are threatening natural or human-induced phenomena that can cause damages and losses when they occur. The severity of a given hazard is usually quantified by their probability of occurrence in a specific area over a given period of time.

² Vulnerability is the susceptibility of a given asset to the damaging effects of a certain hazard.

³ The term exposure represents the risk-relevant information about the assets within a region of interest, where the term asset is used to define something of value.

Zealand (2011, $M_s = 6.3$) caused 181 deaths and led to 15 000 million US\$ in monetary losses, while the L' Aquila earthquake in Italy (2009, $M_s = 6.2$) caused a higher number of deaths, near 300, and led to 6 times less economic losses, namely 2 500 million US\$. In another case, in Reykjavik, Iceland (2008, $M_s = 6.2$), where the population and the structures are much scarcer, the reported economic losses were less than 1 million US\$ and there were neither human casualties nor building collapses. It can be deduced thereafter that since economic losses consist of different sources of losses, e.g. human casualties and injuries, direct economic losses (i.e. structural damage), indirect economic losses (e.g. interruption of activities, downtime), etc., their straightforward comparison impinges difficulties.

Focusing on the building environment, since existing structures worldwide were built in different time periods, it can be inferred that such structures were either not designed according to the same seismic design standards or that many of them were actually designed without considering earthquake-resistant provisions. A consequence of the former situation is the non-uniform distribution of seismic risk, while the latter, i.e. the absence of anti-seismic provisions, may lead to unfavourable structural configurations, both in plan and in elevation, as well as in poor structural detailing, thus increasing earthquake vulnerability. It has also been claimed that even buildings designed with up-to-date seismic provisions fail to meet the target safety performance levels when the plan irregularity is high (Manie et al. 2015). In addition, existing structures may have deteriorated during their lifetime, e.g. through corrosion or fatigue, they might have been physically damaged, repaired or they may have been modified (Melchers 2001). The previously referred conditions lead to the need of assessing the quality of the existing building stock in order to identify the seismic risk levels and to proceed with the implementation of seismic risk mitigation measures, if necessary. Due to the acknowledgement of this situation, standards have begun to be drafted across several countries in order to address and mitigate that risk.

However, the assessment procedure intrinsically possesses a large source of uncertainty, which derives from the actual nature of existing structures, as well as from the techniques to obtain data, and is different from the uncertainty involved in the design of new structures (Franchin et al. 2010; Romão et al. 2012). As a result, it is seen that provisions related to safety assessment procedures need to be more comprehensive, and special attention should also be given to the determination of their uncertainty in order to have

safe and cost-effective evaluation methods, as well as strengthening of structures (Melchers 2001).

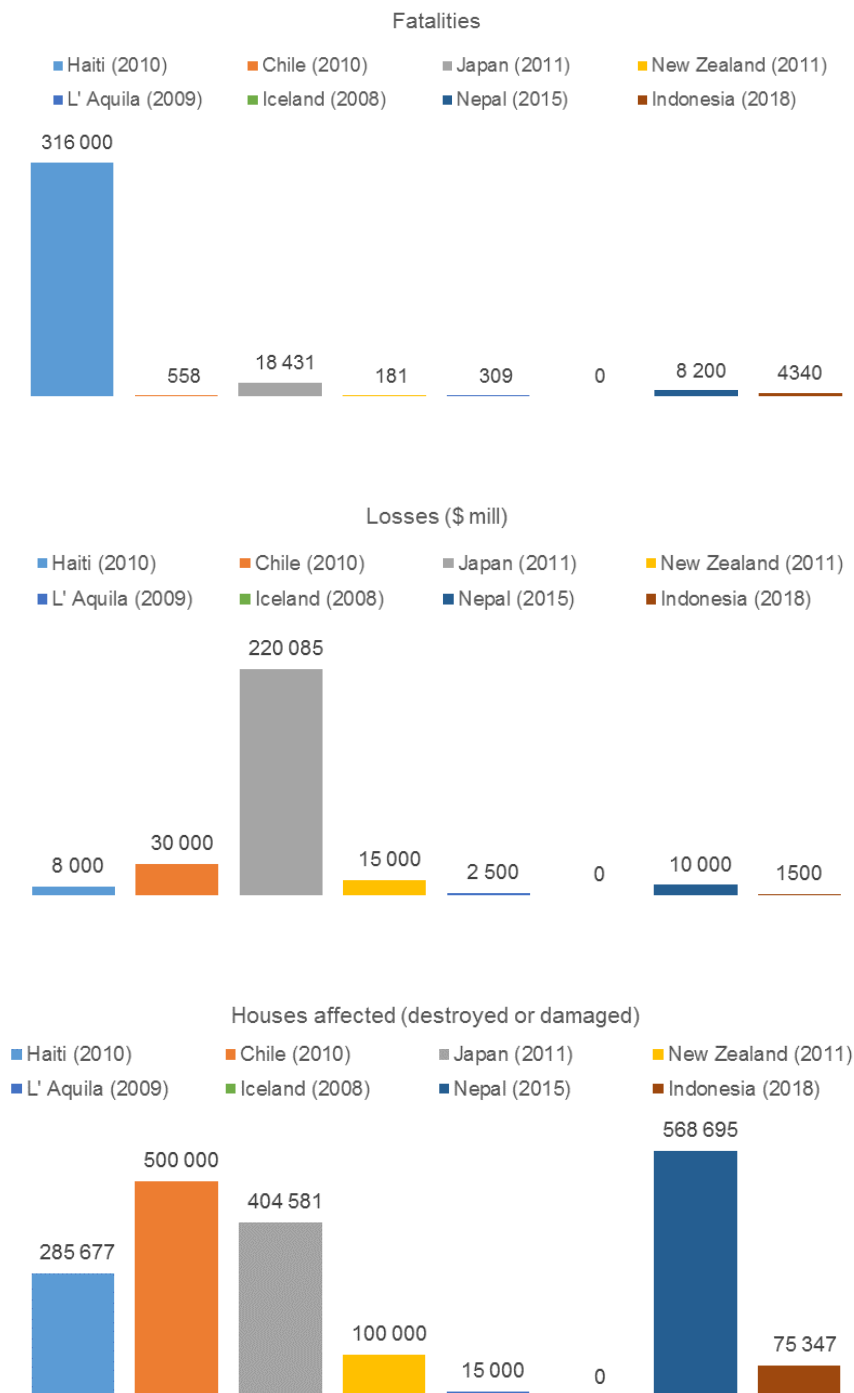


Figure 1.1 Comparison of the number of deaths, economical losses and houses affected from seven recent earthquake events (Source: <http://www.noaa.gov/> National Centers for Environmental Information)

The National Earthquake Hazards Reduction Program (NEHRP) guidelines in 1997 (FEMA 273 1997), which were followed by the prestandard FEMA 356 in 2000 (FEMA-356 2000), can be seen as pioneers of the modern earthquake safety assessment philosophy. Subsequently, several national and international documents were published trying to standardise the assessment and strengthening procedures. The most recent versions of some of those standards are the Swiss standard SIA 269/8 (2017), the New Zealand NZSEE Recommendations (NZSEE 2017), the Unites States IBC (ICC 2018a), IEBC (ICC 2018b) and ASCE 41-17 (ASCE/SEI 41-17 2017), the Italian building code NTC08 (NTC 2018), and the Greek code of interventions KAN.EPE (KAN.EPE. 2013). Regarding the part of the Eurocode suite of documents related to the assessment of existing structures, i.e. Eurocode 8 part 3 (CEN 2005), referred as EC8-3 from now on, its initial drafting started in 2000 and the final version of the standard was published in 2005. Still, its subsequent adoption by each of the European Union member states occurred several years later, e.g. UK adopted EC8-3 in 2010, Greece adopted it in 2011, while Portugal adopted it in 2019.

Even though existing knowledge on seismicity and seismology has increased considerably during the last years (mainly based on information on known or suspected active faults, as well as on the analysis of real ground motion records), the topic of existing structures started being studied not long ago. As previously mentioned, from a regulatory point of view, the inception of dealing with the subject of assessment is chronologically placed 20 years ago. In the particular case of the Eurocodes, this topic started being addressed with the decision to develop EC8-3. It is also recognized that procedures for seismic safety assessment, both in EC8-3 as in other documents, are mostly based on expert judgment rather than on academic research and experimental results, e.g. the definition of the confidence factor in EC8-3 (Franchin et al. 2010).

It has become apparent through years of experience that it is only through practice that the effectiveness of a standard can be validated. As a result, the evolution of a standard can be seen to be directly related to the increase in knowledge on the subject matter, which is largely derived from experience. For instance, the revised version of the Greek seismic design standard EAK2000 (EAK 2000) was drafted and published only after the 1999 earthquake in Athens whose effects brought to light new evidence and revealed several shortcomings of the earlier version of the standard. Another example is the case of ASCE 41-17 that clearly states that data collected after several recent earthquake events have

contributed to the final formulation of its provisions (ASCE/SEI 41-17 2017), e.g. the 1971 Sylmar earthquake (San Fernando, California), the 1985 Michoacán earthquake (Mexico City), the 2010 Chile earthquake, the 2010 and 2011 Christchurch earthquakes (New Zealand), the 2011 Great East Japan earthquake and tsunami, etc. It can thus be seen that current knowledge, especially regarding seismic safety assessment, is still limited by the finite number of events from which data can be obtained and lessons can be learned. Moreover, this knowledge is narrowed even more since, traditionally, the focus of the researchers and standardization committees was mainly on the design of new buildings, rather than on the assessment of older structures designed with not up-to-date or without seismic provisions (Pinto 2005). Thus, the standardization of such an attempt is currently still expected to be bounded and restricted due to the intrinsically limited knowledge on the subject.

Having recognized the source of the shortcomings of current provisions related to the safety assessment of existing structures, EC8-3 will be part of the evolution process of the whole Eurocode system, the goal of which is to lead to the publication of the second generation of EN Eurocodes (source: www.nen.nl/eurocodes2020). The evolution process will consist of 79 discrete tasks in four overlapping phases. Among the 29 tasks involved in the first phase, the evolution of EC8-3 will include the revision, update and extension of that part of the code. In more detail, the goals of the task will include procedures in order to reduce the number of National Choices (the Nationally Determined Parameters), to enhance the ease of use, to update the assessment and retrofitting of buildings and to extend the scope of the standard in order to cover the assessment and retrofitting of bridges. According to Edmund Booth, Chairman of the B/525/8 British subcommittee, the revision and the acceptance of the final draft of the revised EC8 is unlikely to happen before 2020 (source: <http://www.seced.org.uk/>).

1.2 Objectives and Contributions

Technological advances combined with deeper understanding of structural behaviour have resulted in enhanced computational models and improved analysis techniques for the computer-aided seismic analysis of buildings. The use of nonlinear inelastic instead of linear elastic material constitutive laws and the analysis of three-dimensional (3D) in lieu of two-dimensional (2D) simulations comprise relevant examples. However, the introduction of such advanced models is usually accompanied by the simultaneous

involvement of additional parameters beyond the well-known and well-studied parameters present in the earlier format. Such parameters inevitably require additional knowledge for their implementation. As an example, it can be seen that structural analyses based on 2D models and other simplified assumptions are continuously being replaced by more realistic 3D simulations. The recognized limitations of 2D analysis and the continuously advancing engineering software and computational power appear to be the main catalysts towards this change. In this context, a parameter such as the direction of application of the input seismic action with respect to the building's structural axes, also termed the Angle of Seismic Incidence (ASI), is seen to be of particular importance. The effect of the ASI on the seismic demand of buildings has been a research topic for more than four decades and has led to several and important findings. Among others, its dependence on the structural characteristics and the potentially severe underestimation of the structural demand when only one ASI is considered appear to be paramount.

The ASI is involved in the structural analysis stage of the overall performance assessment procedure of a building, both for design and safety verification purposes. The structural analysis stage is an essential stage among all existing assessment methodologies, with several methods of structural analysis being currently available to practitioners for its implementation. The different methods of analysis, prescribed by numerous existing standards and guidelines, involve varying degrees of sophistication and range of applicability. The main factors influencing the selection of the method of analysis involve the purpose of the analysis (in terms of accuracy and precision), the availability of information, the practitioners' expertise, as well as time and cost constraints. However, depending on the selected method of analysis, the different assumptions required regarding the representation of the seismic action have an impact on the interpretation of the effect of the ASI. As such, the seismic input can be represented by a response spectrum or by a time-dependent function. Furthermore, the seismic action can be considered deterministic or can involve various sources of uncertainty, i.e. probabilistic representation of the seismic action. In addition, other factors such as the different assumptions regarding the complexity of the structural model or the simulation of the variability of material properties of the structure may also interfere with the analysis of the effect of the ASI. The material properties are related to the constitutive laws adopted and can range from being linear elastic to nonlinear inelastic, with or without degradation and deterioration mechanisms (the latter is relevant in case of cyclic excursions).

Although the significance of the ASI is well-established among the research community, its quantitative effect on the seismic assessment of buildings, as well as possible ways to account for it, still lack consensus. Even though the concept was introduced to the research community more than four decades ago, the direction of application of the seismic action is still under investigation, and ongoing studies are being carried out to determine its effect, as well as procedures to account for those effects.

In light of this, the main objective of this thesis is to determine the effect of the ASI during the seismic safety assessment of RC buildings and to propose solutions applicable within seismic standard guidelines, such as EC8-3 and FEMA P-58. To this end, different approaches are applied to different methods of analysis, aiming to be consistent with the fundamental assumptions of each method, i.e. accounting for the different representation of the seismic action and the different simulation of the material properties. The analysis will start from the current state-of-the-art method of analysis, using a detailed simulation of the structural model subjected to nonlinear time history analysis (NLTHA) for various levels of seismic intensity. More specifically, the Performance Based Earthquake Engineering (PBEE) framework proposed by the Pacific Earthquake Engineering Research Centre (PEER) will be applied, involving the relevant sources of uncertainty. The PEER PBEE framework comprises 4 distinct stages of analysis: (i) hazard analysis, (ii) structural analysis, (iii) damage analysis, and (iv) loss analysis. Each stage deals with a specific generalised variable: Intensity measure (IM), Engineering Demand Parameter (EDP), Damage Measure (DM), and Decision Variable (DV). The final output of the PEER PBEE framework is to identify and quantify the DVs of primary interest to decision makers accounting for all the relevant uncertainties. The PEER PBEE framework can be summarised into the following Equation:

$$\lambda_{DV}(DV \geq dv) = \int_{IM} \int_{DM} \int_{EDP} G(dv | DM) |dG(DM | EDP)| |dG(EDP | IM)| \left| \frac{d\lambda(IM)}{dIM} \right| dIM \quad (1.1)$$

where G stands for the complementary cumulative distribution function, λ is the mean rate of exceeding the IM and λ_{DV} is the mean rate of exceeding the DV. As previously mentioned, the ASI is involved in the structural analysis stage and thus its effect is expected to propagate to the subsequent stages, i.e. to the damage and decision stages. To analyse this effect, each stage will be studied separately and the effect of the ASI

will be determined. Although unified suggestions will be attempted in the end, focusing on the final product of the PEER PBEE framework, each stage will be also treated independently and partial suggestions will be provided as well.

A different approach will be adopted when involving the Lateral Force Analysis (LFA) method, given the deterministic formulation of the input seismic action. As such, the developed methodology will focus on the estimation of the ASI that leads to the most unfavourable effect on the structure, the critical ASI (ASI_{crit}). Furthermore, due to the linear elastic range of response of the structure, it is possible to develop an analytical methodology that determines straightforwardly the critical ASI.

The conclusions of the research aim to answer the previously referred shortcomings in a rational and practical way. As such, suggestions on how to account for the ASI are provided depending on the purpose and the methods of the analysis. Briefly, in case deterministic seismic action is considered, a methodology is provided for the determination of the ASI that leads to the most unfavourable action effects. In case uncertainty is considered in the simulation of the seismic action, an optimum combination of the ground motion group size and the number of ASIs is provided, taking into account the final outcome of the analysis. The main goal is to reach solutions that account for bias and dispersion introduced by the ASI in the final outcome of a given analysis, while minimising the computational effort.

1.3 Outline of the Thesis

The thesis is composed of eight chapters and three Appendices. Although most of the chapters were designed to be self-contained, the thesis should be read as a continuous sequence of chapters. Due to the chosen format, some repetitions may occur among the different chapters. A brief description of the chapters' contents is given hereafter.

Chapter 1, i.e. the current chapter, starts with a general introduction to the research topic, providing the general background and the motivation of the thesis along with the main scope and the objectives. Furthermore, it briefly describes the contents of each chapter in order to assist the reader to follow the flow of the document.

Chapter 2 begins with a brief definition of the most important terms involved in the research to facilitate the study of the document. It continues with an extensive literature review of existing studies on the ASI, aiming to form the current state-of-the-art on the

topic and to identify the existing lack of knowledge. Next, existing seismic standards and guidelines are reviewed with respect to their provisions on the ASI aiming to highlight the existing gap between available knowledge and current practice. A brief discussion is then presented on conceptual and practical limitations of accounting for the ASI. The chapter ends with the remaining challenges, focusing on those addressed in the current thesis.

Chapter 3 to Chapter 6 involve the analysis of the effect of the ASI from a probabilistic point of view. Particularly, the PBEE framework is applied and the effect of the ASI on its different stages is studied in the four consecutive chapters, as described hereafter. In the application of the probabilistic framework, the seismic action is represented by a group of ground motion records. Since the uncertainty introduced by the size of the group cannot be disregarded when analysing the effect of the ASI, a suitable and efficient methodology presented in **Appendix A** is developed to consider both factors.

In more detail, the methodology presented in **Appendix A** is developed in order to analyse the effect of the ground motion group size on applications where the resampling procedure of the ground motions needs to be statistically consistent. The main advantage of the methodology stems from the consistency with the seismic hazard achieved based on the ground motion group's properties instead of the properties of each individual ground motion. The performance of the procedure is evaluated focusing on the probability of failure considering different limit states and accounting for the possible uncertainty in the limit state definition.

Chapter 3 deals with the probabilistic analysis of the empirical demand of selected engineering demand parameters (EDPs), which are obtained within the demand analysis stage of the PBEE framework. Particularly, EDP distributions at predetermined intensity levels are obtained considering different number of ASIs and different ground motion group sizes. The empirical distributions are compared to a reference case, which corresponds to an empirical distribution obtained using a large number of ASIs and ground motions, and the comparison is performed by analysing three statistics obtained from their empirical distributions at several intensity levels. The examined statistics involve both location and variation parameters and the aim is to determine the effect of the ASI and of the ground motion group size for both deterministic and probabilistic earthquake response analysis.

Chapter 4 continues the investigation on the demand stage of the PBEE framework, but from a distribution fitting point of view. The statistical model that best fits the demand distribution of the previously defined datasets is examined, where the datasets are EDPs obtained considering different combinations of number of ASIs and ground motion group sizes. Both the distribution family and the distribution parameters are studied with respect to the number of ASIs and ground motion group sizes used for each dataset. Potential alterations of the underlying distributions due to the effect of the ASI are highlighted.

Chapter 5 examines the damage stage of the PBEE framework, examining the influence of the number of ASIs and the ground motion group size on the collapse fragility and the corresponding risk of collapse. Initially, the assessment is performed accounting for the previously defined uncertainties, i.e. the ASI and the ground motion group size, and the seismic risk is evaluated with respect to the reference risk, which is obtained considering the same large number of both ASIs and ground motions. Subsequently, additional uncertainty is considered that is derived from the uncertainty of the resistance model related to the occurrence of the collapse limit state. Results show that the ASI and the ground motion group size have a different effect on the collapse risk estimates, with one affecting the bias in the expected value of the estimates and the other their variability.

Chapter 6 focuses on the effect of the ASI on the final stage of the PBEE framework, i.e. the decision variable expressed by the total seismic losses. The seismic losses examined correspond to the direct losses from structural and non-structural components. Even though standards like EC8-3 reach decisions based on structural demand, the final objective of the PBEE methodology is to transform such demand into quantities understandable by stakeholders and by decision makers. As a result, the effect of the ASI and of the ground motion group size on the economic losses is determined, leading to the final outcome, i.e. an optimum combination of ASIs and ground motions that is adequate to predict the expected losses within a given confidence level.

Chapter 7 addresses the ASI effect when LFA, a procedure with deterministic premises, is performed according to EC8-3. Despite the fact that LFA requires several approximations, such as the assumption of linear elastic material behaviour, its simultaneous simplicity renders it very popular in current practice. Given the deterministic nature of the seismic input in this case, an approach different to the one described in the previous chapters is followed and the exact solution of the ASI that leads to the critical structural demand is determined.

Chapter 8 summarizes the research work conducted in this thesis and briefly presents the main findings and conclusions. It highlights the contributions to the research field and, finally, presents remaining challenges and suggestions for future work.

Finally, **Appendices B** and **C** provide information about the selected ground motion records. Tables with data, such as the earthquake name, the magnitude and the recording station, are given in Appendix B, while polar plots with information on the spectral accelerations of the ground motions along different orientations are presented in Appendix C.

Chapter 2

2 The Angle of Seismic Incidence - A review of existing literature and current practice.

Abstract

This chapter reviews research papers that deal with the effect of the angle of seismic incidence on buildings, in order to aggregate the existing knowledge on the topic. Despite the early recognition of the importance of the angle of seismic incidence and the continuous efforts to evaluate its effect on structural demand, research studies are still being produced in this respect. The complexity and difficulty in evaluating such an effect is also evident by the vague guidelines prescribed in relevant standard provisions. A critical evaluation of the current knowledge indicates that the goal of fully determining the effect of the ASI on buildings has not been achieved yet. Furthermore, available procedures found in research documents are not reflected in up-to-date standardised guidelines.

2.1 Introduction

Structural analyses based on two-dimensional (2D) models and other simplified assumptions are continuously being replaced by more realistic three-dimensional (3D) simulations. The recognized limitations of 2D analysis and the continuously advancing engineering software and computational power appear to be the main catalysts towards this change. Enhanced simulations for 3D analysis, however, involve additional parameters that require special treatment, beyond the well-known and well-studied parameters present in 2D analysis. Among others, the direction of application of the input

seismic action with respect to the building's structural axes, also termed the Angle of Seismic Incidence (ASI), has proven to be of particular importance. The effect of the ASI on the seismic demand of buildings has been a research topic for more than four decades (e.g. see Wilson & Button 1982; Smeby & Der Kiureghian 1985) that has led to several and important findings. Among others, its dependence on the structural characteristics and the potentially severe underestimation of the structural demand when only one ASI is considered appear to be paramount.

The ASI is involved in the numerical analysis stage of the overall performance analysis of a building, both for design and assessment purposes. Several methods of structural analysis are currently available to practitioners, which involve varying degrees of sophistication and range of applicability. The implications of using alternative methods of analysis have also been the subject of past studies by several researchers (e.g. see (Mpampatsikos 2008; Masi et al. 2008; Romão et al. 2010)), but no specific answer has been found regarding the selection of the most appropriate one. The purpose of the analysis (in terms of accuracy and precision), the availability of information, the practitioners' expertise, as well as time and cost constraints are factors influencing the selection. The selected representation of the seismic action, defined by a response spectrum or by an accelerogram (or a group of accelerograms), is a parameter that depends on the chosen method of analysis and influences the effect of the ASI. Furthermore, representing the seismic action in deterministic or probabilistic terms also influences the interpretation of the effect of the ASI. Although the significance of the ASI is well-established among the research community, its quantitative effect on the seismic demand of buildings, as well as possible ways to account for its variability, still lack consensus.

In light of this, the present chapter reviews and discusses the current knowledge on the effect of the ASI on the seismic demand of buildings based on the extensive analysis of existing research. A brief presentation of concepts related to the ASI is carried out first, and relevant notations are introduced to provide a solid basis that facilitates understanding of the rest of the document. Next, existing research studies that deal with the ASI are reviewed and discussed to recapitulate the accumulated knowledge and frame the current state-of-the-art. Studies dealing with linear elastic behaviour are discussed separately from studies involving inelastic structural response; the different representation of the seismic action among the several methods of analysis favours this distinction. The

presented studies cover different approaches and different demand parameters addressing the effect of the ASI on seismic demand. Nevertheless, some findings can be generalised and are aggregated in a summary at the end of each section. Following the literature review, provisions from several standards and guidelines referring to the ASI are also reviewed. Critical comments addressing the existing gap between up-to-date standardised guidelines and the available scientific knowledge on this topic are also provided. A final section is also included identifying remaining challenges and possible ways to address them. Finally, it is noted that the current chapter is not expected to contribute new findings to this topic. Instead, it collects existing knowledge and highlights research needs to allow future research to focus on the unresolved aspects and further enhance knowledge.

2.2 Definitions

2.2.1 What is the angle of seismic incidence?

The ASI is defined as the angle θ between the orthogonal *structural axes* of the building and the orthogonal *loading axes* of the seismic action. The *structural axes* of the building are three orthogonal axes, with axes X and Y lying on a horizontal plane (Figure 2.1) and axis Z being orthogonal to that plane (i.e. perpendicular to the plane in Figure 2.1 referred hereon as the vertical axis). *Structural axes* X and Y are apparent in systems with structural and non-structural elements aligned along two orthogonal directions and coincide with the previously mentioned directions, as shown in Figure 2.1 a). In systems without evident orthogonal directions, however, the horizontal *structural coordinate system* becomes more difficult to define and is usually based on engineering judgment. An example of a building without apparent orthogonal directions is presented in Figure 2.1 b). In this case, and unless mentioned otherwise, the term *structural axes* or *structural coordinate system* will refer to an arbitrary set of horizontal and orthogonal axes X and Y.

The majority of the existing buildings have preferred directions of response, possibly with different dynamic properties along different directions. These structures are referred as *azimuth-dependent* (Stewart et al. 2011) and the definition of their *structural coordinate system* is important for further analysis. Unlike *azimuth-dependent* structures, in structures without preferred directions of response, termed *azimuth-independent* (Stewart et al. 2011), such as flag poles, the dynamic properties are the same along all horizontal directions, and defining a particular *structural coordinate system* has no consequences.

Sometimes, the term *principal axes* is also used in the literature or earthquake-related standards to refer to the coordinate system of a building. However, this term usually does not receive the necessary denotation and instead it is used arbitrarily. A clear definition of the term *principal axes* of a building may be found in the studies by Makarios & Anastasiadis (1998a; 1998b) along with a rigorous procedure to determine them.

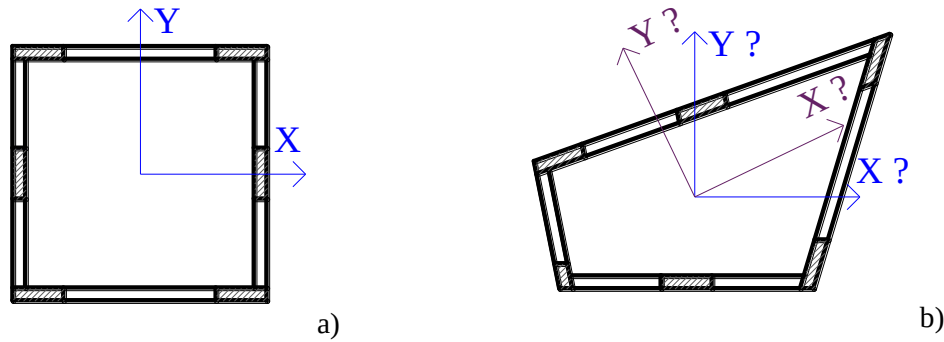


Figure 2.1 Examples of the horizontal structural axes of a building.

The seismic action can be fully represented by a system of three orthogonal *loading axes*: x_0 , y_0 , z_0 . It is commonly accepted that axes x_0 and y_0 lie on a horizontal plane (Figure 2.2) with axis z_0 being orthogonal to that plane (i.e. perpendicular to the plane in Figure 2.2 and defining the vertical component). Conventionally, axes X and x_0 are selected to define the ASI θ , as shown in Figure 2.2. The seismic action involves three translational components along the x_0 , y_0 , z_0 axes and three rotational components about the same set of axes (Newmark 1969). It is common to ignore the rotational components and to take into account the vertical translational component only in specific cases of special site conditions and structural configurations. For the purpose of the current review, the rotational components will be disregarded and only a few comments will be made with respect to the vertical translational component. Moreover, the horizontal seismic action will consist of two orthogonal translational components that are considered to act simultaneously (bidirectional seismic action). As previously referred, the plane defined by x_0 and y_0 is usually horizontal and thus coplanar with the horizontal *structural axes* of the building. In that case, axes Z and z_0 coincide. Studies dealing with seismic components lying on a non-horizontal plane and involving non-vertical components have also been reported in the literature, see for example (Hernández & López 2002) and (Tsourekas & Athanatopoulou 2012).

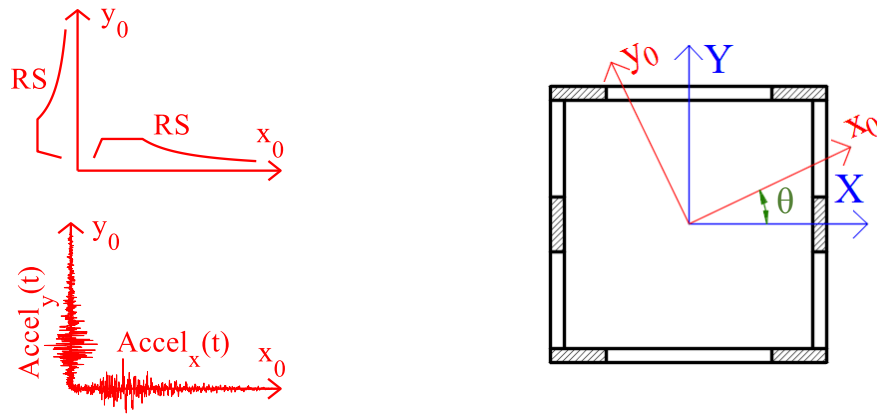


Figure 2.2 Seismic action *loading axes* and the ASI denoted by θ .

In practical terms, the bidirectional seismic action can be represented by two orthogonal response spectra (usually acceleration spectra, but displacement or velocity spectra can also be used) or by a pair of acceleration (or, alternatively, displacement or velocity) time series. When the representation is performed in terms of response spectra, the two spectra (that can have different shapes and intensities) are considered to act along the two horizontal and orthogonal directions x_0 and y_0 (Figure 2.2). When acceleration time series are selected instead, two recorded, simulated or artificial accelerograms can be used depending on the standard provisions under consideration, the availability of real accelerograms and the expertise of the practitioner engineer. Simulated and artificial bidirectional accelerograms require specialised software for their development (e.g. Dagaghi & Der Kiureghian 2018) and will not be discussed herein. Only recorded bidirectional acceleration time series of real events will be further examined and the two orthogonal components will be considered aligned with the x_0 and y_0 directions (Figure 2.2). The term bidirectional acceleration time series, ground motion or record will be used interchangeably throughout the document with the same meaning, unless mentioned otherwise.

2.2.2 What is the ground motion orientation?

The properties of the components of a ground motion, which was generated by a particular event, recorded at a specific site after following a certain source-to-site path, depend on the alignment of the recording instrument with respect to a pre-defined reference system. This reference system is characterised by a set of two horizontal and orthogonal axes x and y that are usually aligned with the North-South and East-West directions, thus with arbitrary directions with respect to the fault (Figure 2.3). These ground motion components a_x and a_y , aligned with the x and y axes as shown in Figure 2.3, are termed

the as-recorded components and are defined by having an as-recorded *orientation*. An *orientation* is defined as a pair of orthogonal directions determined with respect to a given reference system and characterises the properties of the ground motion components. The as-recorded components usually have an arbitrary degree of correlation defined by the correlation coefficient ρ_{xy} given by (Penzien & Watabe 1974):

$$\rho_{xy} = \frac{\sigma_{xy}}{(\sigma_{xx}\sigma_{yy})^{1/2}}, \text{ with } \sigma_{ij} = \frac{1}{s} \int_0^s a_i(t) \cdot a_j(t) dt; i, j = x, y \quad (2.1)$$

where t stands for time, $a_i(t)$ and $a_j(t)$ are the components in their as-recorded *orientation* and s is the ground motion duration.

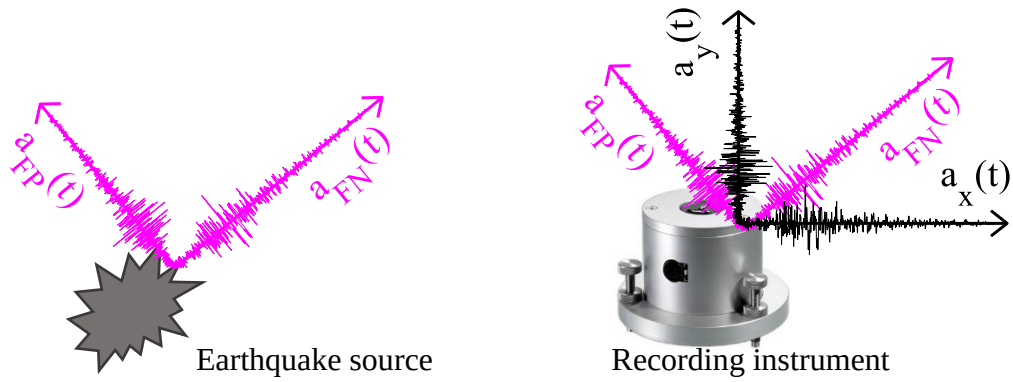


Figure 2.3 Definition of the ground motion orientation, where the terms a_{FP} and a_{FN} stand for the fault-parallel and fault-normal components of the accelerogram, respectively.

Two different components of the same ground motion may be derived by rotating the as-recorded components by an angle α with respect to their as-recorded *orientation* using the following transformation:

$$\begin{bmatrix} a_{x(\alpha)}(t) \\ a_{y(\alpha)}(t) \end{bmatrix} = \begin{bmatrix} \cos(\alpha) & \sin(\alpha) \\ -\sin(\alpha) & \cos(\alpha) \end{bmatrix} \begin{bmatrix} a_x(t) \\ a_y(t) \end{bmatrix} \quad (2.2)$$

where $a_{x(\alpha)}(t)$ and $a_{y(\alpha)}(t)$ are the new components at an orientation α . These new components also possess a degree of correlation that may be different from ρ_{xy} .

Furthermore, the response spectra of the new components may have different frequency contents.

The distance of the recording instrument from the fault affects the properties of the components of a ground motion. Ground motions corresponding to near-field events present directivity and fling effects (Abrahamson 2000; Kalkan & Kunnath 2006), and *orientations* other than the as-recorded are often of particular interest. For example, for the fault-normal (FN) and fault-parallel (FP) *orientations*, specific characteristics of the ground motion will appear, such as the velocity pulse (Kalkan & Kunnath 2007; 2008). Moreover, recent research has also shown that velocity pulses can appear for *orientations* other than the one corresponding to the FN/FP (Reyes & Kalkan 2012a; Reyes & Kalkan 2012b). In contrast to near-field events, the *orientation* of a ground motion corresponding to a far-field event is arbitrary, meaning that no specific characteristic can be associated to a particular *orientation* (Shahi & Baker 2014). In the current document, the effect of the ASI will be mostly discussed with respect to ground motions associated with far-field events. Nevertheless, relevant studies dealing with the ASI for near-field events will also be reported for completeness. It is worth noting here that the distance from the rupture that classifies an event as far-field is not entirely consensual among existing studies. Reference to distances that should be greater than 5 km (Shahi & Baker 2014), 15 km (Campbell & Bozorgnia 2007) or 30 km (Bradley & Baker 2015) can be found, depending on the study.

The *orientation* is a property of the ground motion that is also reflected in its derivative response spectra. The response spectra of two components, obtained from a particular *orientation* of a ground motion, reflect the properties (i.e. the frequency content) of that specific *orientation* and hence, as previously mentioned, alternative *orientations* of the same ground motion may correspond to response spectra with different properties. This means that if a different *orientation* of a pair of response spectra is requested, this orientation is first determined in the original/source/generator ground motion using Eq. (2.2) and then the new pair of response spectra is derived. In seismic codes, the provided response spectrum usually comprises a normalised spectrum derived from an ensemble of records (Riddel 1996) and thus cannot be linked to any particular *orientation*.

2.2.3 Ground motion orientation versus the angle of seismic incidence

During the analysis of a building performed either for seismic design purposes or to evaluate its seismic performance, both the ground motion orientation and the ASI are

involved. However, these two parameters are involved in different stages of the analysis procedure. The ground motion orientation is involved solely in the selection and scaling process of the ground motions, whereas the ASI is only associated with the structural analysis.

During the ground motion selection process, the candidate ground motions usually come from different sources and locations that may not be connected to the building site. Additionally, their orientation is the one provided by a certain ground motion database and usually coincides with the orientation of the recording instrument (as-recorded components), which is arbitrary with respect to the fault. As-recorded components of ground motions can be found, for example, in the European Strong-Motion Database (Twidwell et al. 2014) and the Italian Accelerometric Archive (Luzi et al. 2008). Alternatively, the FN and FP components may be provided after transforming the as-recorded components, such as the case of ground motions provided by the NGAwest2 database (Ancheta et al. 2014). Additional relevant *orientations* involve those in which the two components become uncorrelated (Penzien & Watabe 1974) or fully correlated. The uncorrelated components, also termed principal components, correspond to the *orientation* where the correlation coefficient given by Eq. (2.1) becomes zero. The orientation of the uncorrelated components with respect to the as-recorded components (α_{uncor}) is given by (Penzien & Watabe 1974) and (Clough & Penzien 1993):

$$\tan 2\alpha_{\text{uncor}} = \frac{2 \cdot \sigma_{xy}}{\sigma_{xx} - \sigma_{yy}}, \quad (2.3)$$

where σ_{xx} , σ_{xy} , σ_{yy} are derived in the same way as in Eq. (2.2). The angle α_{uncor} determined from Eq. (2.3) plus 45° defines the fully correlated components.

Once the desired ground motion orientation is established, the ground motion selection is performed. Different selection procedures can be used depending on the purpose of the analysis. A recent extensive review of ground motion selection methods can be found in Katsanos et al. (2010). The typical approach involves matching of spectral ordinates of a ground motion with the ordinates of a target response spectrum within a predefined period range. The spectral ordinates S_{ax} and S_{ay} of the two orthogonal components of the ground motion oriented along axes x and y , respectively, are then combined to form one *representative component* (Beyer & Bommer 2006). Axes x and y have been used so far to refer to the as-recorded orientation but they are used here to refer to any selected

orientation. Currently, the preferred *representative component* is the geometric mean (Stewart et al. 2011), determined by:

$$Sa_{gm} = \sqrt{Sa_x \times Sa_y} \quad (2.4)$$

Alternative definitions involve the Square Root of the Sum of the Squares (SRSS) combination of Sa_x and Sa_y . Both previously mentioned definitions of the *representative component* depend on the *orientation* of the two orthogonal components, thus indicating that the chosen *orientation* of the ground motion has an impact on the ground motion selection (Beyer & Bommer 2007). This fact urged researchers to develop orientation-independent measures. An example of such measures is the *orientation* in which the geometric mean of the two horizontal components reaches its median value, termed GMRotI50 and that is discussed in Boore et al. (2006). This measure has been developed in order to be period-independent and has been adopted in the ground motion prediction equations (GMPEs) developed within the Next Generation Attenuation project (Power et al. 2008). Other definitions of the *representative component* involve the maximum component, the arbitrary components, etc. (Beyer & Bommer 2006). Additionally, it should be noted that the selected representation of the ground motions needs to be compatible with the GMPEs used for the hazard definition. The latter is particularly important in case the target response spectrum is not provided by a standard, but obtained from probabilistic seismic hazard analysis using specific GMPEs, such as in the derivation of a conditional spectrum (Baker 2010). Once the ground motion selection is performed considering a particular *orientation* of the ground motion, such *orientation* must be kept constant during the analysis in order to maintain the earthquake scenario.

During the structural analysis, which follows the ground motion selection stage, a numerical model of the building is analysed using an appropriate software by applying the previously selected ground motions along a set of orthogonal *loading axes* x_0 and y_0 , as presented in Figure 2.2. The records are applied unchanged along the *loading axes* which may be rotated with respect to the building's *structural axes* X and Y, thus defining an ASI (θ as shown in Figure 2.2) different than zero. Hence, during the structural analysis, the *orientation* of the ground motions is constant (i.e. the ground motion properties remain unchanged) and only the ASI can change. Taking into account the definition of the ground motion *orientation* and the definition of the ASI, they can be seen

to correspond to two different concepts involved in two separate stages of the analysis procedure. The ground motion *orientation* is accounted for when defining the seismic hazard, it is a characteristic of the ground motion and remains unchanged throughout the structural analysis stage. On the other hand, the ASI is introduced in the structural analysis stage, it can change and depends on the characteristics of the structure. Even the previously referred orientation-independent *representative components* can induce different demands when applied to *azimuth-dependent* structures along different ASIs (Stewart et al. 2011).

2.2.4 The angle of seismic incidence of a group of ground motions

Special attention should be given to the case where the bidirectional seismic action is defined by a group of ground motions (instead of a single ground motion) in which the average spectral ordinates of their *representative components* matches a certain target response spectrum. In this situation, the average of the *representative components* of all ground motions represents the earthquake scenario. Therefore, that average should remain unchanged throughout the subsequent stages of the analysis. As a consequence, all ground motions of the group should be applied along the same angle that corresponds to the ASI of the group. Exceptional cases where different ASIs are assigned to individual ground motions within a group should be followed by proper justification.

2.2.5 Critical angle of seismic incidence

The application of the bidirectional seismic action along an arbitrary ASI yields a value of a given response parameter, an Engineering Demand Parameter (EDP), that is termed *final demand*. In case the seismic action is represented by two orthogonal response spectra, the *final demand* is evaluated using well known directional combination rules (which combine the timewise non-simultaneous peak responses of each orthogonal axis). Alternatively, in case a group of ground motions is used, the *final demand* corresponds to the average response among all ground motions applied along the same ASI. In both cases, the maximum demand considering all possible ASIs, i.e. the maximum of all possible *final demands*, is termed the *critical demand*, and the ASI that leads to the *critical demand* is termed the *critical ASI* or ASI_{crit} .

2.3 Previous research addressing the effect of the angle of seismic incidence on structural response: Linear elastic methods of analysis

Three methods are currently available for the analysis of buildings with linear elastic behaviour:

- Response Spectrum Analysis (RSA), or pseudo-dynamic analysis (Wilson & Button 1982), which is considered as the reference method of analysis in the majority of existing seismic standards.
- Lateral Force Analysis (LFA), which is a simplified version of the RSA that only considers the fundamental mode of vibration of the structure. The simplified nature of this method makes it the easiest to use and, therefore, the most popular (Toranzo-Dianderas 2010), though it has a limited range of applicability.
- Linear Time History Analysis (LTHA), according to which the structure is analysed with bidirectional acceleration time series by solving the dynamic equations of motion at each time step.

2.3.1 Research involving Response Spectrum Analysis

Bidirectional RSA is performed by applying the two horizontal components of the seismic action, which are represented by two orthogonal response spectra, separately along two orthogonal directions (the *loading axes* as defined in Section 2.2.1) and by carrying out unidirectional modal analysis. The modal combination rules are used to obtain the maximum demand for each direction and these maximum demands are then combined using directional combination rules to obtain the *final demand*. The response spectra should be uncorrelated in order to avoid the need to account for interdependences (Anastassiadis 1993). For the case of isotropic excitation, which is the case prescribed by existing standards, this uncorrelatedness is known to be true (Rosenblueth & Contreras 1977). In isotropic excitation, the same response spectrum (intensity and shape) is applied in both horizontal and orthogonal directions. Furthermore, combining the uncorrelated components is expected to lead to the *critical demand*. The most commonly used procedures to combine the maximum effects obtained from two unidirectional excitations along two orthogonal directions are the percentage combination rules, such as the 30% rule (Rosenblueth & Contreras 1977) and the 40% rule (Newmark 1975), or the SRSS. Additional combination rules, such as the 60% combination rule or the sum of the absolute values, can be found in the literature (e.g. see (MacRae & Mattheis 2000)).

The SRSS is valid when the unidirectional responses are uncorrelated and leads to *final demands* that do not depend on the ASI when the two orthogonal spectra have the same shape and intensity (Wilson et al. 1995; Smeby & Der Kiureghian 1985; López & Torres 1997). On the contrary, when the two spectra have different intensities, the *final demands* obtained by the SRSS rule vary with the ASI (López et al. 2001; Anastassiadis et al. 2002) and the *critical demand* is not necessarily obtained for the case where the analysis is performed along the *structural axes* (López & Torres 1997). Nevertheless, the ratio of the intensities of the two orthogonal spectra does not affect the value of the ASI_{crit} . Furthermore, the vertical component of the seismic action does not affect the ASI_{crit} (López & Torres 1997). As far as the percentage combination rules are concerned, the *final demand* is seen to depend on the ASI (Wilson et al. 1995; López et al. 2001) and, generally, the ASI_{crit} does not coincide with the building's *structural axes*.

Since the *final demand* depends on the ASI and the level of conservativeness of the demand is uncertain when only one ASI is considered, a rigorous procedure that determines the ASI_{crit} and the *critical demand* was developed (Smeby & Der Kiureghian 1985; Menun & Der Kiureghian 1998; López et al. 2000a). The procedure, which involves the CQC3 combination rule, takes into account the correlation and the relative intensity of three orthogonal components, where the minor component is the vertical one. A modified expression, termed CQCC3, was also derived to account for the orthogonal components lying on a non-horizontal plane (Hernández & López 2002). The CQC3 and CQCC3 combination rules are simple to apply and do not increase the computational effort (López et al. 2001). Moreover, they lead to the *critical demand* providing a conservative basis for design and assessment procedures. Nevertheless, the careful use of these rules was advised by Anagnostopoulos (2002) in a response-paper to the contribution by López et al. (2001). The author's concerns were related to the fact that not all demand parameters reach their critical values simultaneously, i.e. for the same ASI, and that considering such concurrent maximum demands does not necessarily lead to a safer structure. The author suggests to always perform a capacity design procedure, even when the *critical demands* are used.

Based on the fact that different EDPs reach their maximum values under different ASIs, López et al. (2000a) did not focus on determining the ASI_{crit} , but on calculating the *critical demand* as a function of the SRSS response when only one ASI was considered. The developed procedure is based on the independent unidirectional application of the

orthogonal response spectra along the *structural axes* and the SRSS combination of the unidirectional responses. The maximum response was found to be bounded by $\sqrt{2}$ times the SRSS response when the two horizontal spectra had the same shape and was equal to the SRSS response when the spectra also had the same intensity, in agreement with (Wilson et al. 1995; Smeby & Der Kiureghian 1985; López & Torres 1997). Additional research by the same authors found that maximum response depends on the intensity ratio of the two spectra, on the correlation coefficient of the orthogonal responses and on the ratio of the orthogonal responses (López et al. 2000b).

The conservativeness of the percentage combination rules when only one ASI is used was evaluated by Fernandez-Davila & Cruz (2008) using parametric analyses and nonlinear structural models. Based on structure-specific results, the authors considered the results obtained from a 60% combination rule to be acceptable when compared to the maximum of the average response due to twenty records applied along several ASIs. The SRSS and the 40% rules were also studied, showing inferior behaviour, but these results were not discussed by the authors.

An alternative way to account for the *critical demand* was proposed by Potnis et al. (2012). These authors used an envelope spectrum suggested by Boore (2010) and concluded that analysing the building using only one ASI, separately along the two *structural axes* and without combining the directional responses, is adequate to predict the *critical demand* obtained by parametric LTHA. The results were closer to the corresponding *critical demand* than those obtained using existing combination rules, but only symmetric buildings were analysed. This latter aspect introduces an important limitation to the proposed procedure since modal coupling, a common characteristic of unsymmetrical buildings, might change the conclusions.

2.3.2 Research involving Lateral Force Analysis

In LFA, only the fundamental mode of vibration is considered in the analysis, making it appropriate only for systems with a large participation of the first translational mode. Similar to the RSA, the seismic action is applied along two orthogonal directions and the *final demand* is obtained by combining the demands from the two unidirectional analyses. All directional combination rules discussed in Section 2.3.1 can be used with LFA.

Morfidis et al. (2008) studied the effect of the ASI when LFA was applied for design purposes. These authors reported that the seismic demand (measured in terms of the longitudinal reinforcement in columns and by using the SRSS combination of the

orthogonal effects) depends on the ASI and that the design base shear could increase up to 29% for ASIs other than the one aligned with the *structural axes*. Additionally, it was found that when the fundamental uncoupled periods of the building fell within the horizontal branch of the spectrum, the design base shear was not affected by the ASI. In any other case though, the response changed as a function of the ASI, but no solution was provided for the definition of the ASI_{crit} or the *critical demand*.

2.3.3 Research involving Linear Time History Analysis

In LTHA the structural model is analysed in the time domain using bidirectional ground motions where the two components are applied either simultaneously or separately along the *loading axes* considering any ASI. In the former case, the *final demand* is obtained directly from the analysis results, whereas in the latter case it is determined by combining the maximum demands obtained from the two unidirectional analyses using a directional combination rule.

In bidirectional LTHA (i.e. considering the simultaneous application of the two orthogonal components), the *critical demand* is not generally achieved when a ground motion is applied along the *structural axes*. Instead, it can be found for ground motions applied along any other ASI (Fernandez-Davila et al. 2000; Athanatopoulou 2005). Furthermore, the application of ground motions having alternative *orientations*, such as the principal *orientation* as defined by Eq. (2.3), along the *structural axes* was also studied (Tsourekas et al. 2009) leading to similar conclusions. In particular, Tsourekas et al. (2009) showed that the demand obtained from the uncorrelated components may be lower than the demand obtained from the as-recorded components applied only along the *structural axes*. The ASI_{crit} and the corresponding *critical demand* were determined by Athanatopoulou (2005) using an analytical procedure that manipulates preliminary analysis results of the ground motion components applied separately⁴ along a set of *loading axes*, i.e. considering only one ASI. The proposed analytical formulae were also extended to account for the vertical component of the seismic action. The latter is of particular importance in LTHA, since considering the vertical component changes the ASI_{crit} , a fact that did not hold true for RSA (López & Torres 1997). The proposed procedure can be applied for ground motions having an arbitrary degree of correlation,

⁴ The total time series of the requested response parameters obtained from the unidirectional analyses are required to implement this analytical procedure. It should not be confused with the combination of the maximum responses using directional combination rules.

i.e. any *orientation*, and practical applications have been explored by Kostinakis et al. (2012). The authors showed that using this analytical procedure to select the internal forces for sectional design led to reinforcement ratios that were independent from the selected *orientation* of the ground motion.

Despite being able to determine the ASI_{crit} of an EDP under a given record, no specific trends could be deduced for the ASI_{crit} or for the *critical demand* for most of the cases. Instead, the ASI_{crit} for a given EDP appeared to change for different records and the same record could lead to different $ASIs_{crit}$ for different EDPs (Athanatopoulou 2005). Some specific trends were only observed for a few building configurations and ground motion characteristics. More specifically, in the special case of buildings that are symmetric with respect to the two *structural axes* and, in addition, have the same stiffness along those axes, the *critical demand* is not a function of the ASI for either local EDPs, e.g. moments, shear forces, or global EDPs, e.g. displacements, (Kostinakis et al. 2008). Furthermore, in case of isotropic excitation (i.e. the same component is applied in both *loading axes*), specific ASI_{crit} values were determined for buildings having one or two axes of symmetry and for various EDPs (Kostinakis et al. 2008). The corresponding *critical demand* at those $ASIs_{crit}$ values was equal to $\sqrt{2}$ times the demand obtained for θ equal to 0° . For buildings and ground motions without these characteristics, only structure- and EDP-specific results could be reported. For example, the critical displacement was found to be 80% larger than the one obtained for the records applied along the structural axes in a single-storey building (Athanatopoulou 2005). Moreover, the critical moment was 190% larger in the case of a different single-storey building (Tsourekas & Athanatopoulou 2012). An alternative proposal by Marinilli & López (2008) suggested performing LTHA along a reduced number of ASIs in a range of ASIs predicted by RSA results, in order to decrease the computational effort.

As previously mentioned, LTHA results can also be associated with the directional combination rules. In this case, LTHA is performed by applying the two components separately along the *loading axes* considering any ASI and the directional combination rules are then used to obtain *final demands*. Similar to RSA, the *critical demand* depends on the selected ASI and studies have been carried out to evaluate the conservativeness of the combination rules in case only one ASI is used (usually selected to be 0°). The evaluation procedure involves determining the *critical demand* using analytical formulae, i.e. the one proposed by Athanatopoulou (2005), or a parametric analysis with the

simultaneous application of the two components along several ASIs, where the angle step is defined according to the authors' sensitivity. Several conclusions have been reached on this matter, depending on the structure as well as on the selected EDP. The research of Kostinakis et al. (2013) and Tsourekas & Athanatopoulou (2013) focused on regular buildings, torsionally-balanced and -unbalanced, and reported that combination rules (30%, 40% and Regulatory Guide rules (Regulatory Guide 2006)) significantly underestimated the *critical demand*. On the other hand, Anagnostopoulos (1981) and Aksoylu et al. (2016) concluded that, in structures with clearly defined *structural axes*, combination rules (SRSS and the 40% or 60% rules), led to acceptable results since the underestimation was not significant. It is worth noting that Aksoylu et al. (2016) also considered buildings with irregularities in elevation in their study. An alternative combination rule, corresponding to an amplification by 20% of the maximum response due to unidirectional excitation, was found more suitable to represent the *critical demand*, based on results of five-storey RC buildings (Fernandez-Davila et al. 2000). In the same study, the authors showed that the 30% and the SRSS combination rules could underestimate the *critical demand*.

Different *orientations* of the ground motion (principal and FN) were examined by Reyes-Salazar et al. (2008) to validate the directional combination rules for alternative *orientation* schemes. These authors analysed local and global EDPs of several structures and observed that both the 30% and the SRSS combination rules had the tendency to produce unconservative demands for both the principal and the FN components. Still, slightly less unconservative results were observed for the principal components and for the 30% rule. However, due to the disproportionate effort required by parametric analysis to obtain the *critical demand*, the authors considered the results obtained using only the principal components as acceptable.

Still with respect to LTHA, but from a probabilistic point of view, a methodology was recently proposed by Nievas & Sullivan (2017) to account for both the ground motion *orientation* and the ASI in the context of the Performance Based Earthquake Engineering (PBEE) framework. The methodology was developed for elastic single-mass oscillator systems with the same dynamic characteristics along two perpendicular axes (*azimuth-dependent*) or along all directions (*azimuth-independent*), and determined the probability of exceeding a predefined limit state in terms of spectral displacements. To do so, the authors assigned a theoretical distribution to the ratio between the spectral displacement

at any *orientation* and the maximum spectral displacement along all *orientations* for a given period, based on findings by Hong & Goda (2007), and integrated that probability distribution with the corresponding hazard curve. The methodology led to the exact solution for the case of *azimuth-independent* structures and to approximate results in the alternative case.

2.3.4 Summary of research involving linear elastic methods of analysis

The effect of the ASI has been extensively but not exhaustively studied for building structures modelled by systems with linear elastic response. Since the main focus of existing research was on determining the ASI_{crit} and the corresponding *critical demand*, analytical solutions have been proposed for their explicit derivation for RSA and LTHA. As far as LFA is concerned, similar solutions are still unavailable. Nonetheless, the potential underestimation of the demand when only one ASI is considered has been highlighted for all methods of analysis. However, no universal quantitative evaluation of the degree of underestimation was able to be defined. Instead, most of the reported results were structure- and EDP-dependent. The lack of unified conclusions is mainly attributed to the fact that, except in certain cases where buildings have specific configurations, little connection was found between the properties of the building, the seismic action characteristics and the ASI.

2.4 Previous research addressing the effect of the angle of seismic incidence on structural response: Nonlinear inelastic methods of analysis

Elastic behaviour is seldom observed in real structures subjected to strong earthquake excitations. Instead, structures undergo deformations beyond yielding and, in many occasions, enter deeply into the inelastic range of response. In order to achieve a more realistic representation of real-life responses on structures, the simulation of yielding, degradation and deterioration of material behaviour is often integrated into the structural models. However, once inelastic analysis is performed using nonlinear material properties, the superposition of the results obtained from separate unidirectional analyses ceases to be valid and thus expressions proposed for linear analysis cannot be applied. Moreover, additional factors, e.g. the seismic input intensity, have been found to affect the ASI_{crit} and the *critical demand* in the nonlinear-inelastic domain of response. Both static (Pushover) and dynamic (NLTHA) have been studied in literature, however, no

universal conclusions could be drawn. Instead, most observations were structure- and record-specific and only some general behavioural trends were reported.

In nonlinear approaches, the effect of the ASI was assessed by performing parametric analysis with the selected ground motions applied along various angles, aiming to cover the whole range of ASIs. As in the case of linear analysis, the angle step was a pure choice of the analyst. The evolution of the *final demand* for various ASIs as well as the *critical demand* were then evaluated with respect to the *final demand* obtained from the application of the seismic action along the *structural axes*, which corresponds to current practice. Alternatively, the comparison was performed with the *final demand* defined as the combination of the maximum demands resulting from unidirectional excitation of the structure, despite the fact that the implementation of combinational rules in inelastic analysis lacks scientific basis.

2.4.1 Research involving Nonlinear Time History Analysis

The demand variations with the ASI observed in LTHA were found to be much more pronounced when nonlinear material properties were considered and the variability of the structural response appeared to become larger with increasing the level of structural inelasticity (Magliulo et al. 2014; Fontara et al. 2015). This variability, was also found to depend on the existing reinforcement and on the design procedure (Kostinakis et al. 2013). More particularly, Kostinakis et al. (2012) and Kostinakis et al. (2013) reported that in case the sectional forces were selected from a procedure that determined the ASI_{crit} in LTHA, the variability of inelastic response, measured in terms of a global damage index, was only slightly influenced. The aforementioned results, however, were obtained based on a single-storey regular building and no similar results exist for different structural configurations.

Along the same lines, a good correlation was found between the maximum interstorey drift of realistic 3D models with nonlinear material properties and equivalent two degree-of-freedom linear models when multiple ASIs were employed (Sebastiani et al. 2014). Based on that, the authors proposed a methodology involving parametric analysis with the linear model in order to determine the ASI_{crit} and to perform NLTHA with the realistic model. This procedure, however, was only tested in single-storey buildings and individual ground motions, while, in general, the ASI_{crit} of an EDP differed for linear and nonlinear response for both single- and multi-storey buildings (Kalkan & Reyes 2015; Reyes & Kalkan 2015; Khoshnoudian & Poursha 2004).

The fundamental period of vibration, the torsional irregularity and the level of inelastic behaviour of the building were also found to significantly affect the ASI_{crit} (Rigato & Medina 2007). Based on the fact that the ASI_{crit} could not be easily identified, the authors tried to characterize the ratio of the *critical demand* to the *final demand* obtained from the application of the seismic action only along the *structural axes*. The proposed values of the previously defined ratio, determined in terms of drift, ductility and slab rotation, would vary between 1.1 and 1.6 with the fundamental period irrespective of the amount of torsional irregularity. Furthermore, the effect of the ASI was found to be much more pronounced in buildings with structural irregularities compared to buildings with regular configurations (Cantagallo et al. 2012; Cantagallo et al. 2015).

In agreement with linear methods of analysis, the effect of the variability of the demand with the ASI appeared to be dependent on the EDP under consideration. Different EDPs were examined in different studies and the selection was mainly dependent on the purpose of the analysis. Particularly, local EDPs, such as axial forces in columns or plastic rotations were found to be much more variable when compared to global EDPs, such as the base shear or top displacements (Hosseini & Salemi 2008; Magliulo et al. 2014; Reyes-Salazar et al. 2008). Hosseini & Salemi (2008) investigated a regular building and focused on results of individual ground motions, while Magliulo et al. (2014) focused on an irregular building and on the average demand among all ground motions of the group. With respect to that, it is worth noting that the demand of an EDP obtained from only one record was much more variable with the ASI when compared to the average demand of the same EDP over a group of records (e.g. see Tsourekas et al. 2009). It is also worth noting that reported variations of the average demand could reach as much as 170% even for symmetric buildings (Kostinakis et al. 2017). An exception to these results comprises cases involving symmetric buildings with equal stiffness along the *structural axes*, in which the ASI did not affect significantly particular classes of EDPs, such as vectorial parameters. Different damage indexes, global and local, were also examined with respect to their sensitivity to the ASI, concluding that the one proposed by Park & Ang (1985) was the least affected (Morfidis et al. 2013).

The selected *orientation* of the ground motions corresponding to far field events was also examined. Despite showing different responses for the same ASI, a given record would lead to approximately the same extent of structural damage, expressed by a modified Park and Ang damage index, for the ASI_{crit} (Fontara et al. 2015). In the previous case, the

records corresponding to different *orientations* were amplified by the same scale factor. In case different *orientations* result in different scale factors, the responses would exhibit more variability (Fontara et al. 2015). The effect of the ground motion *orientation*, though, was found to be much more pronounced in case of near-field events. Reyes & Kalkan (2012a) and Archila et al. (2017) observed that velocity pulses could appear in orientations other than the FN/FP and that the *critical demand* was not necessarily obtained when the FN/FP components were applied along the principal structural axes (Kalkan & Kwong 2014). Maximum drift appeared to be polarized in the direction in which an apparent velocity pulse with a period close to the fundamental period of the structure was observed when the structure performed linearly. Period elongation due to inelasticity, though, weakened this polarization effect. Even for near field effects, the FN or the principal components did not necessarily lead to the peak median demand in terms of drifts, and thus could underestimate the demand, both in single-storey and multi-storey buildings (Kalkan & Reyes 2015; Reyes & Kalkan 2015). Based on these observations, Kalkan & Reyes (2015) and Reyes & Kalkan (2015) proposed to consider two ground motion *orientations*, the maximum direction and the FN/FP, and to use the maximum response among them for design purposes.

Reyes & Kalkan (2012b) also highlighted the importance of using a large number of ground motions when only one ASI is considered. The authors reported that when a small number of records was used, the underestimation of the median response could be larger than 50%. Instead, for a large number of ground motions, the underestimation was less than 20%. Similar findings were also previously mentioned by Rigato & Medina (2007). More particularly, these authors came up with the idea of defining a required number of GMs and ASIs in order to estimate the peak inelastic deformation with a specified precision and level of confidence. In line with this, Beyer & Bommer (2007) and Stewart et al. (2011) argued that when structural performance is assessed in probabilistic terms, using the ASI_{crit} is seen to lead to an overly conservative demand assessment. In light of this, some researchers have been proposing alternative approaches to obtain adequate probabilistic demand distributions while accounting for the randomness of the ASI. In this context, Lagaros (2010a) investigated the effect of the ASI in a loss estimation framework of RC buildings and concluded that 20 ground motions applied along 5 random ASIs were adequate to take into account the randomness of both the ASI and of the ground motion. Lagaros (2010b) also incorporated the ASI in a PBEE framework

involving Multidirectional Incremental Dynamic Analysis (MIDA). The reasoning behind the analysis was that, in MIDA, the direction that the two components should be applied in to induce *critical demand* is difficult to define. Hence, the author suggested using 30 records matched with an equal number of ASIs, sampled using the Latin Hypercube sampling method, in order to achieve a good approximation of the fragility functions considered as target (fragility functions based on 100 pairs). More recently, Giannopoulos & Vamvatsikos (2018) analysed structural demand distributions obtained from Incremental Dynamic Analysis (IDA) and showed that, in a scenario where only a fixed number of analyses is performed, the number of ground motions has a larger influence on obtaining an accurate structural demand distribution, when compared to that of the number of ASIs being considered.

The representation of the seismic action in probabilistic terms might lead to the need to describe the resulting demand using a statistical model. The most common model currently used in earthquake engineering is the lognormal model (e.g. see Ibarra & Krawinkler 2005) due to the various advantages it possesses over alternative statistical models. Among others, it can be fully described with two parameters and it takes only positive values. Moreover, several widely used goodness-of-fit tests developed for the normal distribution model can be applied to test the adequacy of the model since the logarithms of the values follow a normal distribution. Although, several studies can be found that test the adequacy of the lognormal model to fit various data, e.g. EDPs (Romão et al. 2011) and fragility curves, in 2D and in 3D analysis when only one ASI is involved, no similar studies exist for the specification of the probabilistic model used for the same parameters when multiple ASIs are employed. Nevertheless, the lognormal model has been used so far in these cases as well, e.g. see (Lagaros 2010a), following common practice.

Another issue that has been extensively studied in 2D analysis, but has been overlooked when the ASI is involved in 3D analysis, is the efficiency and the sufficiency of the Intensity Measure (IM). Relevant research was performed by Kostinakis & Athanatopoulou (2016) in the context of MIDA, in which the efficiency of various IMs was studied considering the effect of the ASI. The authors found that the efficiency of the examined IMs was dependent on the structural characteristics and on the level of inelasticity. The commonly used spectral acceleration at the first period of vibration was evaluated as relatively efficient.

The soundness of the combination rules in nonlinear analysis was studied in one of the earliest studies addressing the ASI in the nonlinear range of response (MacRae & Mattheis 2000). The authors showed that the determination of the response, represented by interstorey drifts, based on several combination rules (e.g. 30%, 40%, SRSS) depended strongly not only on the ASI, but also on the selection of the reference axes used to measure the response. Such a demand could either underestimate or overestimate the *critical demand*. Similar conclusions in terms of underestimation were also reached by Reyes-Salazar et al. (2008) regarding the 30% and the SRSS demands. Special attention was given to the large variability of the results especially for the local EDPs. Recognising, though, the disproportional effort required to achieve critical demands based on parametric NLTHA using multiple ASIs, the authors considered the combination rules as acceptable. Similar conclusions were found by Emami & Halabian (2015) and Amarloo & Emami (2019) for regular and irregular in-plan buildings, respectively. The authors of the latter approach additionally proposed a methodology to determine the percentile value of the percentage combination rules considering particular characteristics of the seismic action and of the building.

2.4.2 Research involving Pushover Analysis

Although Pushover Analysis was initially developed for applications in planar (2D) structural models, its concept broadened fast to accommodate the analysis of 3D structural models with and without structural irregularities. The effect of the ASI on Pushover Analysis was examined by performing multi direction Pushover Analysis, i.e. by applying a set of orthogonal vertically distributed lateral forces along different ASIs. The assessment of the building behaviour was performed based on multi-directional capacity curves and on limit (or capacity) domains (Marino & Petti 2011). Limit (or capacity) domains represent the polar plots presenting the limit states of an EDP, e.g. top displacement or base shear, obtained for Pushover Analysis along various ASIs.

Marino & Petti (2011) developed limit domains for the collapse limit state for plan-asymmetric buildings using both triangular and uniform vertical distributions of lateral forces and showed that performing Pushover analysis along the structural axes may overestimate the building capacity. The least seismic resistant direction, i.e. the direction with the lower base shear, of different structures was further identified, highlighting its dependence on the ASI. Since the referred study involved asymmetric buildings whose response was not significantly influenced by higher modes, the standard vertical

distributions of lateral loads were deemed adequate. The use of alternative Pushover analysis, such as adaptive Pushover, was suggested in the opposite case. Mazza (2014) applied the procedure described in Marino & Petti (2011) in a different asymmetric building and derived capacity domains for various limit states. The results were in accordance with those of Marino & Petti (2011) and showed that the least seismic resistant direction can be different from the one corresponding to the structural axes. Furthermore, the author showed that the least seismic resistant direction may also differ for the different limit states, as well as for the different EDPs, i.e. base shear or displacements.

Emami & Halabian (2015) studied the effect of the ASI in symmetric buildings by using adaptive multi-directional Pushover analysis, while an extension of the former study to include asymmetric buildings was performed by Amarloo & Emami (2019). Although in both studies the Pushover results served mainly to define the limit states along directions different than the structural directions to be used for NLTHA, relevant conclusions were also reached for Pushover analysis. As such, it was found that the yield mechanism and the ductility demand depended on the ASI. Furthermore, ductility capacity reduction was observed when changing the ASI, based on the multi directional capacity curves. Although all of the presented studies highlighted the importance of considering different ASIs when performing Pushover analysis, no methodology to determine the critical ASI or the critical demand was proposed.

2.4.3 Summary of research involving nonlinear inelastic methods of analysis

The effect of the ASI was found to be extensively studied using nonlinear inelastic methods of analysis. However, despite the numerous studies, no uniform conclusions were deduced regarding the effect of the ASI or ways to account for it. Few exceptions to the latter were found with suggestions on using amplification factors in order to obtain the *critical demand* instead of performing parametric analyses. Nonetheless, their validation and verification is limited in extent. In addition, important information obtained from the literature is the introduction of the ASI in probabilistic analyses. In this case, the objectives shift from trying to estimate the ASI_{crit} and the *critical demand* to the definition of a sufficient number of ASIs that is able to provide an estimate of a *demand statistic*, such as the median demand, with an acceptable degree of confidence that is in agreement with the premises of the probabilistic analysis under consideration.

2.5 Treatment of the Angle of Seismic Incidence in standard provisions

Based on the extensive literature review presented in Sections 2.3 and 2.4, a general lack of consensus can be observed with respect to the implementation of measures to account for the effect of the ASI. More specifically, no agreement was reached on the effects of the ASI on the demand parameters studied and no uniform solution was provided for accounting for these effects. This conflict can be further seen to affect existing standard provisions. The present Section contains an overview of existing provisions related to the ASI, as prescribed by different standards and guidelines. A separate revision is dedicated to European standards, while various documents from different areas are also discussed.

2.5.1 European standards

The Eurocode suite of standards, of which Eurocode 8 (EC8) is a part of, is currently under revision and a new version is expected to be available to the public soon. At the time of this study, a second revision of the EC8 provisions has been concluded and the final draft is already circulating among the research community. Hence, the discussion on the EC8 provisions will also include the latest EC8 version (although it is still in draft format). In both versions, Part 3 of EC8 (EC8-3) (CEN 2005) makes reference to Part 1 of EC8 (EC8-1) (CEN 2004) for the direction of application of the seismic action on a building and the respective section of EC8-1 was subjected to modifications. The somehow ambiguous provisions of the earlier version, which required the application of the seismic action along all *relevant* horizontal directions, has been replaced by the requirement of only one ASI, arbitrarily selected with respect to the principal directions of the building. The referred ambiguity attributed to the former EC8 version refers to the term *relevant*, since a definition or an explanation of the term was not provided in any clause of the standard. An exception to this ambiguity is the case of buildings with resisting elements along two perpendicular axes, in which the *relevant* directions were selected to coincide with the previously-mentioned axes. Consequently, in EC8 the determination of the ASI or the ASIs was left to the judgement of the practitioner engineer. In the draft version of the future EC8, this ambiguity disappears, as well as the notion of multiple ASIs, since defining only one direction of application, arbitrarily selected, is considered to be adequate. However, no reference is provided to justify this alteration.

Regarding other national European standards, the Greek seismic codes for design EAK (2003) and for assessment KAN.EPE (2017) do not require the use of multiple ASIs nor

the determination of the ASI_{crit} , despite recognising that the seismic action may act on the structure from any direction. Instead, both standards prescribe that the ASI should coincide with the principal axes of the structure for all methods of analysis. It is explained, though, in the commentary section of EAK (2003) that for the case of RSA, which is considered the reference method of analysis, performed with the same response spectra along the two orthogonal directions and with the SRSS combination of the directional effects, the demand does not depend on the ASI (Anastasiadis 1993). Furthermore, EAK (2003) provides a clear definition of the term principal axes of a building accompanied by a rigorous procedure for their determination (Makarios & Anastasiadis 1998; Makarios & Anastasiadis 1998). Similar to the Greek code, the Italian Seismic Standards NTC (2018) and the Swiss Standards SIA 269/8 (2017), prescribe the application of the seismic action only along the structural/principal axes of the buildings, while they do not make any reference to the effect of the ASI.

2.5.2 Other national and international standards and guidelines

As far as other international standards are concerned, ASCE/SEI 41-17 (2017) requires that when NLTHA is carried out, the ASI should be randomised among the different records of the same group, even if the group was defined using spectral matching techniques. This provision, which contradicts the idea discussed in Section 2.2.4, aims to achieve approximately equal input along all orthogonal axes of the building. Additional guidelines are provided for the implementation of the records along the randomised ASIs. In any case, though, the ASCE/SEI 41-17 (2017) provisions prescribe that orthogonal directions X and Y should be employed for the application of the bidirectional action and the most critical one should be used, i.e. the direction that leads to the most unfavourable effect. The International Building Code (ICC 2018a) and the International Existing Building Code (ICC 2018b) refer to ASCE/SEI 7 for the application of the seismic action. The provisions related to the direction of loading remained unchanged between the consecutive versions of ASCE/SEI 7, thus the ASCE/SEI 7-16 (2016) provisions are similar to those of ASCE/SEI 7-10 (2010) and define the ASI as a function of the building classification/category. The lowest categories require no consideration of additional ASIs other than the one aligned with the main *structural axes*. Moreover, bidirectional effects are allowed to be neglected in that case as well. Higher building categories require the consideration of the ASI that leads to the most critical load effects for all methods of

analysis. Hence the buildings should be analysed with the seismic action applied along multiple directions, in order to determine the critical one.

The FEMA P-1050-1/2015 (2015) seismic provisions recommended by the National Earthquake Hazards Reduction Program also require the definition of the most critical direction of loading. There is nonetheless a special case which involves buildings with orthogonal seismic force resisting systems where the critical direction may be considered to coincide with the main structural axes. Similarly, the New Zealand standard NZSEE (2017) prescribes that the analysis should consider multiple ASIs when the application along the structural axes does not produce the most adverse effects (critical direction) or when the structural axes are not clearly defined.

2.6 Conceptual issues and remaining challenges

Sections 2.3 and 2.4 contain an extensive literature review of past studies dealing with the effect of the ASI on building structures. Even though the methods and the tools used by the different researchers to estimate the effect of the ASI generally vary, two main approaches can be identified. These approaches can be briefly described as: estimation of the critical response/demand and as a consequence of the ASI_{crit} , and estimation of an average response considering multiple ASIs. The consideration of either of the two approaches has been seen to depend on the representation of the seismic action and particularly on whether the latter is expressed in probabilistic or deterministic terms. As such, in case the seismic action is represented by a group of ground motion records, selected in order to account for relevant sources of uncertainty, the pursuit of an average effect has been proposed (Beyer & Bommer 2007; Steward et al. 2011). On the contrary, in case a response spectrum is used or a unique ground motion, i.e. a deterministic simulation of the seismic action, the maximum possible effect of the ASI is sought. Nevertheless, since the interpretation of the ASI relies on the judgement of each individual researcher, probabilistic studies can also be found aiming for the critical demand, e.g. Nievas & Sullivan (2017).

Additional constraints complementing the conceptual issues of dealing with the ASI can be seen to be related with computational costs, as stressed by several authors. Unless there is a methodology to determine the ASI_{crit} , the calculation of the critical demand requires a full parametric analysis considering all possible ASIs. Such a limitation has been condemned by several authors, who further criticised existing standard provisions

requiring the computation of the critical response without providing further guidance. Similar limitations apply not only for the determination of the critical demand, but also for the average demand which requires an adequate number of ASIs to obtain unbiased demand percentiles or statistics of central tendencies, e.g. see Lagaros (2010b). Additional to the time and power consuming task of analysing the building using multiple ASIs, the treatment of the results obtained from such a parametric analysis is not a negligible task either. The consideration of multiple ASIs results in the generation of multiple results of the same parameter that require post processing. Furthermore, an additional parameter that affects the analysis time as well as the quality of the analysis results is the maximum number of ASIs or the precision of the angle step to be used in case multiple ASIs are implemented. The angle step, however, is not provided in any of the previously discussed standards and thus the decision is left to the experience of the practitioner engineer. Angle steps of 1 to 30 degrees are, for example, common values found in literature, undoubtedly affecting the accuracy of the results.

Based on the proven importance of the ASI in the analysis of structures, as well as on the acknowledged burdens for accounting for its effects, the focus of the proposed research was on developing methodologies to either calculate the ASI_{crit} or to determine the minimum number of ASIs required to adequately represent probabilistic demand. Even though relevant procedures/suggestions can be found in the literature, they are still limited in extent or restricted in their applicability by specific assumptions, thus leaving room for improvement. The main challenges that still remain and that were pinpointed throughout the review can be summarised as follows.

Regarding the consideration of the ASI in probabilistic analysis, none of the available studies examines the effect of the ASI in all stages of the seismic assessment procedure and provides conclusions regarding each phase of the procedure as well as in the overall procedure. Different researchers have reached different conclusions regarding the required number of ASIs, or regarding the combination of number of ASIs and number of ground motions, depending on the stage of the procedure considered. As a result, different suggestions can be found for the demand, damage and loss stages that were unarguably further affected by the distinct underlying assumptions adopted by each researcher. As far as the determination of the ASI_{crit} is concerned, methodologies for its straightforward determination have been developed for the RSA and the LTHA. However, similar solutions regarding LFA do not exist currently.

In light of the previously referred shortcomings, the main purpose of the present thesis is to examine the effect of the ASI from a probabilistic point of view, studying all different stages of the PBEE framework that the ASI is involved in, providing suggestions for each of them having a common basis, e.g. the analysed structures and the modelling techniques. As such, the thesis provides suggestions regarding an optimum combination of number of ASIs and number of ground motions to be used for the adequate determination of structural demand, the failure risk and ultimately the expected losses using the PBEE framework. Finally, the last purpose of the thesis is to provide a straightforward solution for the determination of the ASI_{crit} during the performance assessment using LFA.

Chapter 3

3 The significance of considering multiple angles of seismic incidence for estimating engineering demand parameters

Based on:

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Abstract

The accurate determination of demand during the seismic analysis of buildings requires that all factors having a significant contribution must be accounted for. The present chapter examines the effect of the angle of seismic incidence of earthquake ground motions on the inelastic demand of reinforced concrete buildings and defines its significance for different engineering applications, including both deterministic and probabilistic analyses. Three commonly used engineering demand parameters are examined by analysing three statistics obtained from their empirical distributions at several intensity levels. The results show that the influence of the angle of seismic incidence is different for the three statistics, and that this effect depends also on the engineering demand parameter under consideration. Central tendencies of the demand appear to be less affected by the angle of seismic incidence when compared to its variability, thus showing the potentially larger influence of the angle of seismic incidence for probabilistic analyses rather than for standard-based (i.e. deterministic) applications.

Finally, recommendations on the minimum number of angles of seismic incidence that should be considered are given as a function of the number of ground motions used in the analysis.

3.1 Introduction

When performing the seismic analysis of a structure, either for design or assessment purposes, a decision needs to be made regarding the angle of seismic incidence (ASI) that is selected. The ASI represents the direction of application of the seismic action on the structural model and its adequate selection is relevant when analysing three-dimensional models. Over the past decades, this issue has received significant attention from the research community. However, despite the research that was carried out and the particular improvements that were achieved, there is still a lack of consensus about the more adequate way of dealing with the ASI. This issue is apparent when analysing scientific research publications and recent earthquake-related standards.

Existing research focuses mainly on determining the effect of the ASI on structural response and on ways to account for this effect. Regarding the former, the various opinions that can be found in the literature vary according to the type of structure being analysed, the method of analysis, the purpose of the analysis and the engineering demand parameters (EDPs) that are examined. For example, when assessing the impact of considering different ASIs in structural analyses, non-negligible effects were observed by Fontara et al. (2015), who examined member-level EDPs obtained from nonlinear dynamic analyses of single-storey reinforced concrete (RC) buildings, and by Kostinakis et al. (2015), who performed similar analyses in multi-storey RC buildings and examined global structural damage indices. Similar results were also found by Kalkan and Reyes (2015), who analysed multi-storey steel buildings and examined global EDPs. Moreover, Lucchini et al. (2011) and Nguyen and Kim (2013) determined the nonlinear response of asymmetric single-storey buildings and also showed that maximum structural response depends on the ASI. On the contrary, negligible effects were found by Kostinakis et al. (2018) when analysing specific building types and EDPs. Furthermore, Wilson et al. (1995) showed that the response in terms of any EDP is independent from the ASI when response spectrum analysis is performed using the same spectrum along two orthogonal directions and directional effects are combined using the square-root-sum-of-squares rule.

Regarding existing proposals to account for the effect of the ASI, existing approaches depend on the way the seismic action is represented. When the seismic action is considered to be deterministic, one of the most common approaches involves determining the critical ASI, which represents the ASI that leads to the most unfavourable demand. In this context, Smeby and Der Kiureghian (1985) and Menun and Der Kiureghian (1998) provided a methodology to derive the critical ASI when using response spectrum analysis. When linear time history analysis is selected, the critical ASI can be determined using the analytical formulae derived by Athanatopoulou (2005). However, when the seismic action is considered in probabilistic terms, e.g. by a group of ground motions accounting for record-to-record variability, the approach based on determining the critical ASI has been claimed to be overly conservative (Beyer and Bommer 2007; Steward et al. 2011) and the use of multiple ASIs has been proposed as an alternative. For example, Rigato and Medina (2007) proposed the idea of defining an adequate number of ground motions and number of ASIs to estimate inelastic demand with a predefined confidence level. In a loss assessment framework, Lagaros (2010a) sampled the ASI from a uniform distribution and concluded that 20 ground motions applied along 5 random ASIs are adequate to account for the randomness of both the ground motion and the ASI on the EDP estimates. Lagaros (2010b) also incorporated the effect of the ASI in a probabilistic framework involving Multidirectional Incremental Dynamic Analysis and suggested combining 30 records with an equal number of ASIs. More recently, Giannopoulos and Vamvatsikos (2018) concluded that the ground motion group size has a much larger effect than that of the ASI and thus suggested that the contribution of the ASI could be ignored. This summary of existing research shows that a variety of conclusions were reached and different suggestions were made regarding the effect of the ASI and on ways to account for this effect. However, no general agreement was found. This lack of consensus is also reflected in the provisions found in recent earthquake-related standards, which can either disregard the effect of the ASI, e.g. EAK (2003), NTC (2018), or prescribe the use of the critical ASI, e.g. ASCE/SEI (2017), NZSEE (2017) or EN1998-1 (2004).

In this context, further insights are presented herein on the effects of the ASI for applications where the seismic action is represented by ground motion records. In particular, the present research provides a detailed quantification of the effect of the ASI and of the ground motion group size on the seismic demand of six multi-storey RC buildings, focusing on specific EDPs regularly used to assess global behaviour. Three

descriptive statistics of the empirical EDP distributions representing their central tendency and dispersion are examined to obtain results relevant for both deterministic and probabilistic earthquake response analysis. The distinct behaviour of each statistic is examined using a probabilistic framework involving reference values of those statistics for each EDP. The results lead to recommendations about the adequate number of ASIs required to obtain robust estimates of the EDP statistics, within acceptable margins of error, for different ground motion group sizes.

3.2 Methodology

The present research analyses seismic demand distributions of three EDPs, obtained from six RC buildings with infilled frames, considering the effect of the number of ASIs and the ground motion group size. The selected EDPs are the maximum interstorey drift ratio (ISD), the peak floor acceleration (PFA) and the maximum roof drift (RD) due to their explicit involvement in performance-based assessment procedures. The different EDP distributions are obtained from the nonlinear dynamic analysis of the buildings, considering bidirectional ground motions in groups of size n ranging from 10 to 40, applied along 1 to 12 ASIs and for 4 intensity levels. Ground motion groups of size 40 are initially selected for each building in order to be compatible with a response spectrum that represents the seismic scenario. Subsequently, the groups of 40 records are regrouped to create ground motion groups of smaller sizes equal to 10, 15, 20, 25, 30 and 35, according to a regrouping procedure. The regrouping procedure, takes into account the requirements needed to maintain compatibility with the seismic scenario for all new groups and yields a total number of 100 admissible groups for each new size n (for a detailed description of the procedure see Appendix A).

All ground motions of each group are applied to the buildings along different numbers of ASIs. The ASI of a bidirectional ground motion is defined by the angle between one preselected component of the ground motion and a structural axis. Since only far-field records are considered in the current study, the reference ground motion component is defined arbitrarily and the angle it forms with structural axis X of the building defines the ASI. Given that the structural axis X of a building can have a random orientation with respect to the reference ground motion component, all ASIs are considered to be equally likely within the range $[0^\circ \ 180^\circ)$, assuming an angle step of 15° . In order to be consistent with current practice, when a single ASI is used, the direction of application of the seismic

action coincides with the structural axes of the building, i.e. all ground motions of a group are applied along 0° . When multiple ASIs are used, the ASIs equidistantly cover the total angle range, with the first ASI always being 0° . For instance, using two ASIs corresponds to 0° and 90° , while three ASIs corresponds to 0° , 60° and 120° . The overall procedure leads to structural demand results of 100 groups of ground motions for each combination of group size n and number of ASIs, which are adequate for statistical post-processing. The notation n_ASI will be used to refer to results obtained from a given combination of a ground motion group of size n applied along ASI number of angles.

The 100 demand distributions of each EDP and n_ASI combination are analysed using three descriptive statistics, i.e. the mean, the median and the standard deviation (std), in order to provide information about their central tendencies and dispersions. It is noted that the values of these statistics are estimates of the true, yet unknown, values of those parameters due to the finite number of ASIs and ground motions that are considered to derive them. For each particular combination of n and ASI, a kernel distribution is then fitted to the 100 estimates of each statistic. The kernel distribution was selected in order to obtain a smooth nonparametric representation of the distribution of the sample data, while avoiding the need to consider specific parametric distributions that would require further assumptions. Figure 3.1 shows the kernel distribution (in a violin plot format) of the 100 estimates of a given statistic normalized by the reference value of the statistic, which is defined by the value of the statistic obtained from the group of 40 ground motions applied along all 12 ASIs.

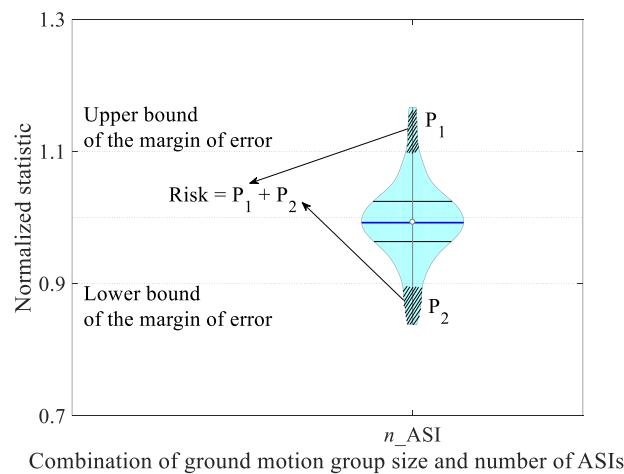


Figure 3.1. Violin plot of the 100 statistic estimates and definition of risk (the black and blue horizontal lines of the plot correspond to the interquartile range and the median of the data, respectively, while the white dot represents the mean of the data).

It is noted that the maximum and the minimum values of the violin plot correspond to the maximum and minimum values of the 100 estimates, respectively.

Each distribution is then compared to the reference value of the corresponding statistic using a procedure that requires the definition of a margin of error around the reference value. This margin of error establishes that an estimate $\hat{X}$ of a certain statistic is within an interval which is a function of the reference value of the statistic X_{ref} such as:

$$X_{ref} \times (1 - ME) \leq \hat{X} \leq X_{ref} \times (1 + ME) \quad (3.1)$$

where ME is the width of the margin of error in percentage (see Figure 3.1). The margin of error is then used to determine the risk, i.e. the probability of being outside the margin of error, which is also illustrated in Figure 3.1 by the hatched areas P_1 and P_2 . Admissible risk levels are subsequently selected and the bias of the statistic estimates is then determined by analysing the number of cases exceeding the admissible risk levels for each combination of n and ASI. The previously referred number of cases non-conforming with a predefined risk level is presented as a ratio over the total number of cases (i.e. 24 cases in total, corresponding to the 6 buildings and the 4 seismic intensity levels) and termed *risk exceedance ratio* hereafter. A high value of the *risk exceedance ratio* indicates there is a larger number of biased estimates of a given EDP statistic for the particular combination of n and ASI under analysis.

3.3 Description of the selected case studies

3.3.1 General characteristics of the buildings

Six RC buildings with infilled frame systems are analysed. The selected buildings have configurations ranging from low- to mid-rise buildings with and without in-plan irregularities. The plan views of a typical storey of the 3-storey irregular (3-Ir), the 4-storey irregular (4-Ir) and the 5-storey irregular (5-Ir) are presented in Figure 3.2, along with the design details. The height of the first storey is 3.5 m and the height of all upper storeys is 3.0 m. Similarly, Figure 3.3 shows the plan views of a typical storey of the 3-storey regular (3-R), the 4-storey regular (4-R) and the 5-storey regular (5-R) building and the design details of the end section of the elements. All regular buildings have a unique storey height equal to 3.0 m. The concrete strength f_c and the yield strength of the reinforcing steel f_y are equal to 25 MPa and 500 MPa, respectively, for all buildings.

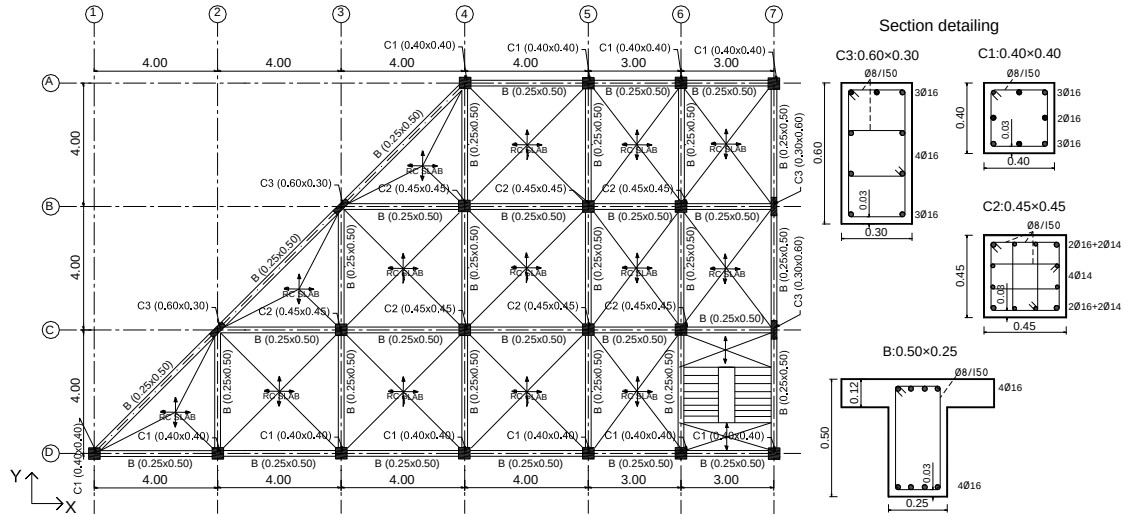


Figure 3.2. Plan view of a typical storey of the 3-Ir, 4-Ir and 5-Ir buildings, the considered structural axes and the design details (all dimensions are in m).

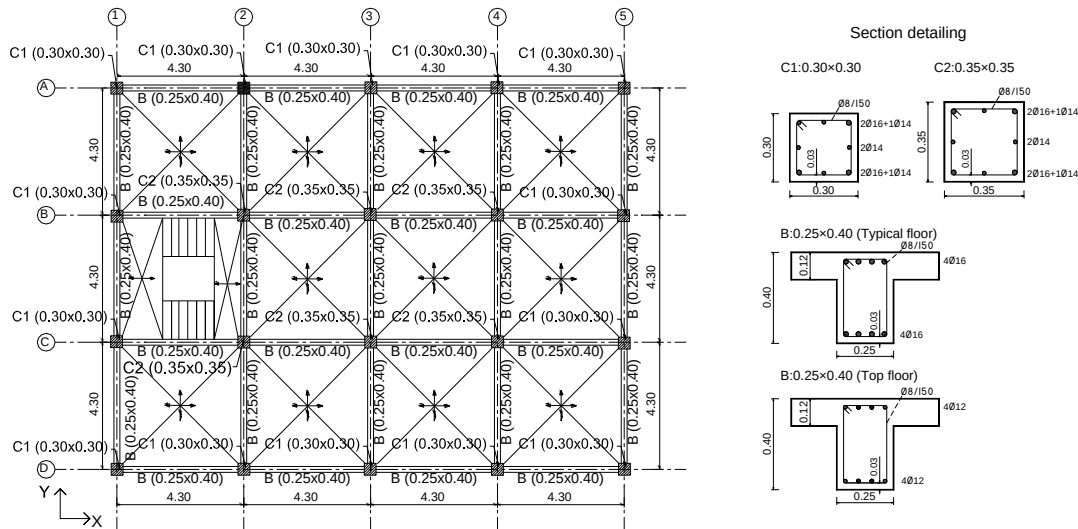


Figure 3.3. Plan view of a typical storey of the 3-R, 4-R and 5-R buildings, the considered structural axes and the design details (all dimensions are in m).

All buildings are defined as existing buildings located in Lisbon, Portugal, designed for gravity loads and with a minimum lateral strength to account for wind loads. Specifically, a non-structural permanent load of 4 kN/m^2 is considered to be uniformly distributed on all floors of both building categories, additional to the slab self-weight (all two-way solid slabs have a thickness of 0.12 m). Additionally, the quasi-permanent component of a uniform live load of 3 kN/m^2 is also assigned to all floors, except to the top storey floor where this load is 1 kN/m^2 . Staircases are modelled only as permanent and quasi-permanent loads, which are transferred to the supporting beams and applied uniformly.

The corresponding loads are 7.75 kN/m and 8.60 kN/m for the permanent and the quasi-permanent loads, respectively. Masonry infills are present in all peripheral frames and represent a permanent uniform load of 7 kN/m. For modelling simplification purposes, a unique beam section with symmetric reinforcement configuration is used for each floor

3.3.2 Structural modelling

Three-dimensional models of the buildings are simulated in the OpenSees (OS) computer software (McKenna 2011) considering mean values of all material and geometrical properties.

3.3.2.1 Modelling of the structural elements

A lumped plasticity approach is adopted to simulate the inelastic behaviour of both beams and columns. Phenomenological hysteresis laws are assigned in rotational springs located on both ends of all structural elements to simulate inelastic flexural behaviour. Two independent springs are assigned to each end of the columns, one for each orthogonal direction, while one spring is assigned to each end of the beams modelling the in-plane flexural behaviour. Due to the nature of the selected inelastic modelling approach, biaxial moment interaction or axial force moment interaction are not considered when modelling the behaviour of columns (Ricci et al. 2018). The hysteretic flexural behaviour is defined by a moment-rotation law that is simulated using the *hysteretic* material provided by OS. The properties of the backbone curve of all columns are determined considering the axial force due to vertical loads of the seismic combination ($G+0.3Q$), where G and $0.3Q$ stand for all permanent and quasi-permanent loads, respectively. The yielding moment (M_y) and the yielding rotation capacity (θ_y) are determined according to Panagiotakos and Fardis (2001). The capping (θ_{cap}) and the ultimate (θ_u) rotation capacities are computed according to Haselton et al. (2008) along with the corresponding capping (M_{cap}) and residual (M_r) moments, where the latter is considered to be 20% of M_{cap} (see the backbone curve in Figure 3.4(a)). Stiffness, strength and unloading stiffness degradation are considered in the hysteresis curves as shown in Figure 3.4(a). The unloading stiffness degradation is implemented in OS via the *beta* factor of the *hysteretic* material. A *beta* factor of 0.75 is used for the columns and of 0.85 for the beams in order to introduce higher degradation for the latter (Dolšek and Fajfar 2005). The *hysteretic* material parameters related to pinching and to damage due to ductility and dissipated energy are all set equal to zero. The springs are connected in-series with a linear elastic interior element. A stiffness modification factor equal to 10 is applied according to Zareian and

Medina (2010) to account for the effect of the series connection on the total stiffness of the element. Beam-column node failure is not considered and rigid nodes are modelled in all element cross-sections.

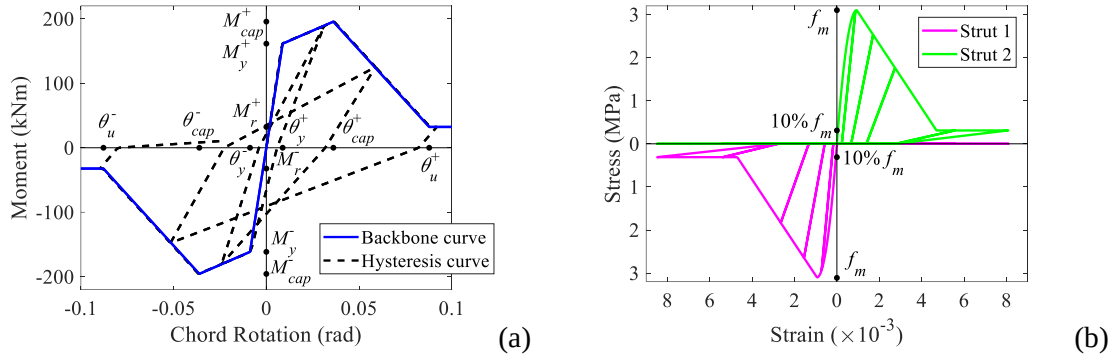


Figure 3.4. Moment-rotation backbone curve and hysteresis loop simulation for a column (a) and stress – strain relationship of the diagonal struts (in compression) representing the infills, measured in the diagonal direction (b)

3.3.2.2 Modelling of the infills

Masonry infills are considered in all peripheral frames and are modelled with a single strut active only in compression. Two diagonal struts (strut 1 and strut 2 as shown in Figure 3.4(b)) are used to simulate one infill and are connected to the beam-column joints. The equivalent area of each strut is established based on the maximum lateral force of the infill (Žarnić and Gostič 1997), transformed to the direction of the diagonal, and on the masonry compressive stress f_m . Maximum strength is assumed to be reached at an interstorey drift of 0.2% (Dolšek and Fajfar 2008). The lateral displacement of each infill is transformed into the diagonal displacement for the subsequent definition of the strain of the strut. The parameters obtained, i.e. the maximum stress and strain, are used to define the masonry material with zero tensile strength simulated by the *Concrete01* constitutive model (Figure 3.4(b)), (Noh et al. 2017). The stress f_m is equal to 3.10 MPa and all infills have a thickness of 0.15m. Additionally, a residual stress equal to 10% of the maximum stress is considered for numerical stability, which is reached at an interstorey drift five times the interstorey drift at maximum strength.

3.3.2.3 Additional considerations

Geometric nonlinearities are incorporated in the form of P-Δ effects. Furthermore, soil structure interaction phenomena are not modelled and all structures are considered fixed to the ground. The first and second fundamental periods of vibration of the six buildings,

with and without the masonry infills, are presented in Table 3.1. The terms infilled and bare structure refer to the structure with and without the infills, respectively. Finally, the average of the first two periods of the infilled structure and the first two periods of the bare structure T^* is determined (also shown in Table 3.1) to be used for the ground motion selection. Involving periods of vibration of the bare structure in the definition of T^* , is conceptually similar to accounting for the apparent period elongation of the structure after yielding and failure of the infills.

Table 3.1. First and second mode periods of vibration of the buildings

Period\Building	3-R (s)	4-R (s)	5-R (s)	3-Ir (s)	4-Ir (s)	5-Ir (s)
T_1, T_2 (infilled)	0.31,0.25	0.41,0.31	0.52,0.39	0.21,0.15	0.29,0.20	0.37,0.26
T_1, T_2 (bare)	0.73,0.72	0.96,0.93	1.18,1.15	0.39,0.35	0.55,0.47	0.70,0.60
T^*	0.50	0.66	0.82	0.27	0.38	0.48

3.4 Seismic action and additional modelling techniques

3.4.1 Seismic hazard, ground motion selection and regrouping procedure

The ground motion record selection is performed using the recently developed SeleEQ software (Macedo and Castro 2017) using a Conditional Spectrum (CS) in terms of the 5% damped spectral acceleration (Baker 2010; Lin et al. 2013a) as the target spectrum. First, the probabilistic seismic hazard analysis of the site (Lisbon) was performed within the SeleEQ software using the open source software OpenQuake (Pagani et al. 2014) and the seismic hazard model developed by the SHARE project (Woessner et al. 2015). The GM prediction equations proposed by Atkinson and Boore (2006) and Akkar and Bommer (2010) were used with a weight of 70% and 30%, respectively. The geometric mean spectral acceleration is employed for the definition of the seismic hazard in order to ensure compatibility with the spectral acceleration used in the ground motion selection (Baker and Cornell 2006), as described next. The annual seismic hazard curve H_{IM} was determined at T^* for each building of the benchmark site and is shown in Figure 3.5(a). Note that since T^* of the 5-Ir building is equal to 0.48s, the seismic hazard curve considered for this case is also the one determined for the 3-R building that has $T^*=0.50$ s.

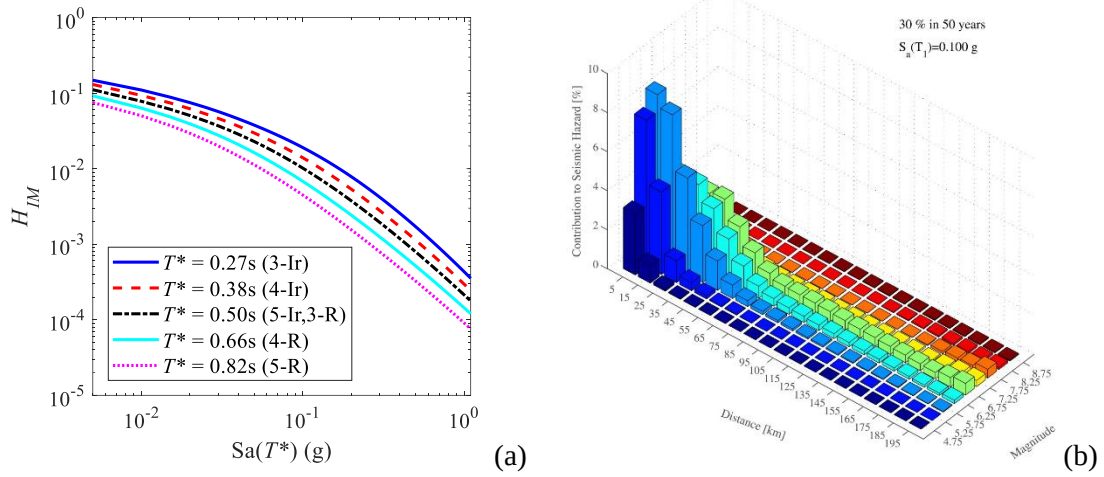


Figure 3.5. Seismic hazard curves for Lisbon and for the periods of interest (a) and seismic hazard disaggregation for $T^* = 0.66s$ and 30% in 50 years (b).

Disaggregation of the hazard was then performed for three probabilities of exceedance, i.e. 30%, 10% and 5% in 50 years, at T^* for each building. An illustrative example of the hazard disaggregation is shown in Figure 3.5 (b) for the 4-R building and for the 30% probability of exceedance in 50 years. The results are next used to build four CSs using the methodology proposed by Lin et al. (2013a; 2013b; 2013c). A preliminary ground motion record selection was performed using the NGA-WEST2 database (Ancheta et al. 2014) based on seismological and strong motion parameters, such as magnitude, epicentral distance and average shear wave velocity. More specifically, ground motions with magnitude larger than 5.5, closest distance from source to site larger than 10 km and soil shear wave velocity, in upper 30m of soil, greater than 200 m/s were used. Subsequently, the final selection and scaling of a group of 40 bi-directional records (for each probability of exceedance) was carried out by ensuring compatibility between the target spectrum, i.e. the CS, and the average of the geometric means of the horizontal components of all pairs. This compatibility was ensured by minimizing the difference between statistics of the target spectrum (i.e. the mean and standard deviation of the logarithms of the spectral accelerations) and the same statistics of the group, within a period range between $\min(0.1 \text{ s}, 0.2T^*)$ and $1.5T^*$, as described in Macedo and Castro (2017). An additional criterion was included in the process which involves minimizing also the skewness of the logarithms of the spectral accelerations to a value close to zero for the same period range. This criterion is based on the fact that the spectral accelerations are expected to follow a multivariate lognormal distribution (Jayaram and Baker 2008) and, therefore, the logarithms of the spectral accelerations are expected to follow a

multivariate normal distribution. Overall, 3 groups of 40 records are selected for each building, each corresponding to a predefined probability of exceedance (see Appendix B for a complete list of the GMs). To illustrate the results of the ground motion selection process, Figure 3.6 shows the geometric mean spectra of the selected 40 records along with the CS for the three probabilities of exceedance for the 3-R building. Furthermore, in order to highlight the directionality characteristics of the selected groups of GMs, and particularly the fact that they are representative of far-field records, polar plots of the $S_a(T^*)$ of all groups are presented in Appendix C.

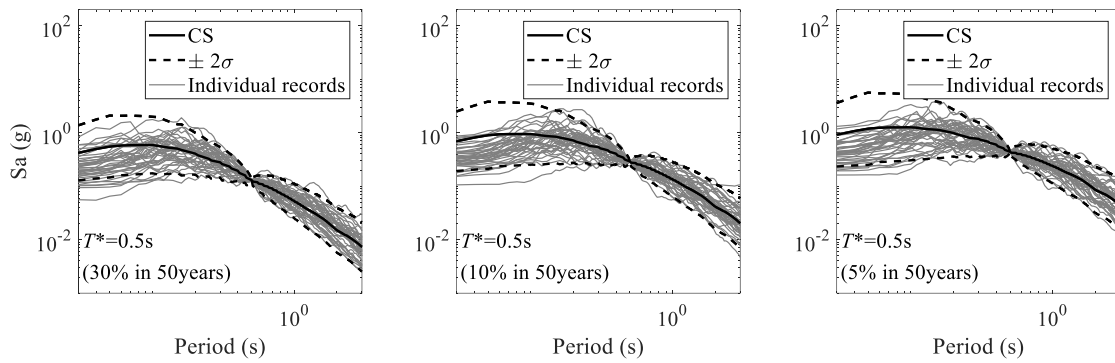


Figure 3.6. The CS with the geometric mean spectra of the 40 records for each probability of exceedance, $T^* = 0.50s$

Each initial group of size $n = 40$ is then regrouped to create ground motion groups of smaller size $n = 10, 15, 20, 25, 30$ and 35 by implementing a regrouping procedure. The regrouping procedure creates groups of ground motions that represent a seismic scenario consistent with that of the reference group of size 40 and a detailed description of the procedure can be found in Appendix A. A total of 100 groups are created for each new group size n using this procedure and illustrative examples of the geometric mean spectra of two groups of size $n = 35$ and $n = 15$ are presented in Figure 3.7(a) and (b), respectively. The groups presented are representative of large and small size n , respectively, and were obtained from the group with $T^* = 0.50s$, i.e. for the 3-R building, and for a 5% in 50 years probability of exceedance.

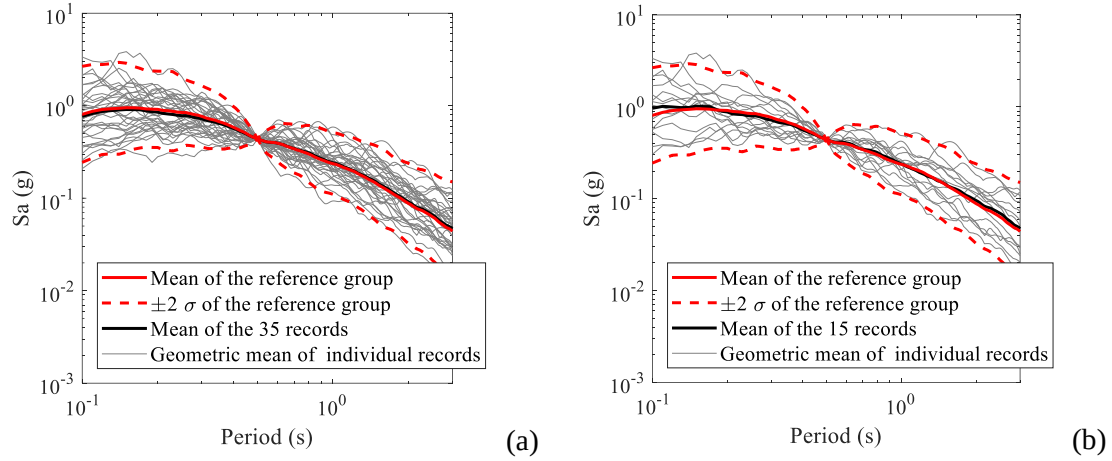


Figure 3.7. The individual geometric mean spectra of all ground motions of a group of size $n = 35$ (a) and $n = 15$ (b), along with their average for the 3-R building. The average of the reference group of 40 records and the standard deviation are also shown for comparison.

3.4.2 Probabilistic demand model

The response of the six 3D RC building models presented in Section 3.3 is determined by performing nonlinear dynamic analysis using the ground motion groups defined in Section 3.4.1. Each building is analysed using the three groups of 40 ground motions, each corresponding to a predefined probability of exceedance, all applied along 12 ASIs, as explained in Section 3.2. Furthermore, an additional higher intensity is also considered for all buildings in order to generate EDP sets involving larger levels of inelastic response. This fourth intensity level is generated by multiplying the group of ground motions obtained for the 5% in 50 years probability of exceedance with a scale factor (SF). The SFs were derived for each building separately in order to ensure a) the highest possible intensity where none of the scaled ground motions lead to the numerical collapse of the building and b) an intensity level with the same probability of exceedance across all buildings. The SFs that were derived according to these conditions correspond to a return period of about 1300 years for all buildings. The spectral accelerations $Sa(T^*)$ and the corresponding SFs for all groups of records are summarized in Table 3.2. Numbers 1 to 4 are assigned to each intensity for reference in the following sections and are presented in the second column of Table 3.2.

Table 3.2. Spectral accelerations used and the corresponding scale factors

Building		3-Ir		4-Ir		5-Ir, 3-R		4-R		5-R	
Probability of exceedance	Int.	Scale factor	Sa($T^*=0.27$ s) g	Scale factor	Sa($T^*=0.38$ s) g	Scale factor	Sa($T^*=0.50$ s) g	Scale factor	Sa($T^*=0.66$ s) g	Scale factor	Sa($T^*=0.82$ s) g
30% in 50 years	(1)	1.00	0.21	1.00	0.17	1.00	0.13	1.00	0.10	1.00	0.07
10% in 50 years	(2)	1.00	0.45	1.00	0.36	1.00	0.29	1.00	0.22	1.00	0.17
5% in 50 years	(3)	1.00	0.65	1.00	0.54	1.00	0.44	1.00	0.34	1.00	0.27
	(4)	1.15	0.75	1.17	0.63	1.20	0.53	1.22	0.41	1.21	0.32

Only sources of uncertainty related to the seismic input are taken into account, while the rest of the parameters involved, e.g. material properties, dimensions and member capacities, are considered with their mean values. More specifically, uncertainty stems from record-to-record variability and ASI-to-ASI variability, i.e. variability of the obtained parameters related to the finite number of records and of ASIs used to determine the demand distribution. It can be thus said that the obtained descriptive statistics are estimates of the true, yet unknown, statistics due to the finite number of records (Baker and Cornell 2008) and the finite number of ASIs. The uncertainty of the estimates will be examined using the proposed *regrouping* procedure described in Section 3.4.1. The proposed procedure examines both the effect of using a finite ground motion group and a finite number of ASIs, determining the variability of the statistic estimates as well as the bias of the expected value with respect to a reference value. Although the true statistics are inherently unknown, the reference statistics considered in the current study were obtained using 40 records and 12 ASIs. As referred before, the statistics examined are the mean, the median and the std of the three EDPs.

3.5 Results obtained for central tendencies

3.5.1 Setting the margin of error for the mean and the median

A set of tentative margins of error defined around the reference value of each statistic are first analysed before establishing the final margin of error that is going to be used in the subsequent analyses. The final margin of error is defined based on a preliminary sensitivity analysis that uses all the ground motion group sizes n employed in this research and considers that all ground motions are applied along the 12 ASIs. By using all 12 ASIs,

the effect of the ASIs is fully accounted for and the results that are obtained only reflect the influence of having different ground motion group sizes n . The margins of error that are tentatively analysed are $\pm 5\%$, $\pm 10\%$ and $\pm 15\%$ around the reference value of the statistic (see Section 3.2). For all three margins of error, the risk (see Section 3.2) is determined for the EDPs of all the buildings and all the intensity levels. Since no specific trends were observed among the risk values obtained for the different buildings and for the various intensity levels, the risk values of all six buildings and all four intensity levels are aggregated and treated together for each EDP statistic in this sensitivity analysis. Subsequently, four admissible risk levels are considered, i.e. 5%, 10%, 15% and 20%, and the number of cases exceeding these admissible risk levels defined as a percentage of the total number of cases, i.e. the *risk exceedance ratio* (see Section 3.2), is analysed to select the most adequate margin of error. To illustrate the results that were obtained, Figure 3.8 presents the *risk exceedance ratios* obtained for the mean value of the logarithms of ISD. Similar analyses were performed for both central tendencies (the mean and the median) and for all EDPs, i.e. ISD, PFA and RD.

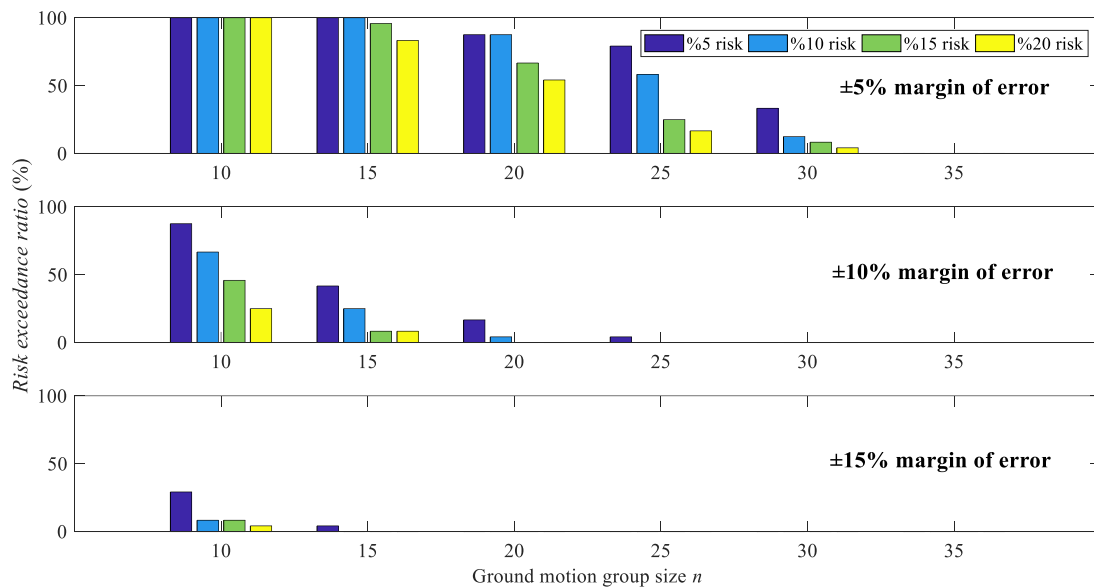


Figure 3.8. *Risk exceedance ratios* for the mean value of the logarithm of ISD, for four admissible risk levels and three margins of error, considering different ground motion groups sizes, and considering all the 12 ASIs, buildings and seismic intensities.

As referred in Section 3.2, the narrower the margin of error, the higher the confidence that the results are better estimates of the reference value. However, as observed in Figure 3.8, the narrower the margin of error, the higher the *risk exceedance ratio* for a given risk

level. Since the objective is to have reasonably low *risk exceedance ratios* for the different risk levels, a compromise needs to be made regarding the margin of error. With that in mind, it is observed that, for the $\pm 5\%$ margin of error, the *risk exceedance ratios* are high for all risk levels. When considering, as a guidance, the group of 20 ground motions, since this number of ground motions is prescribed by seismic guidelines (e.g. see (CNR 2013)) and was also suggested for more advanced probabilistic analyses (e.g. (Giannopoulos and Vamvatsikos 2018)), it is observed that *risk exceedance ratios* are above 50% for all risk levels. Therefore, it can be inferred that the $\pm 5\%$ margin is too strict to obtain the desired trends. On the contrary, when a large range is used, i.e. the $\pm 15\%$ margin of error, almost all *risk exceedance ratios* are zero, thus indicating that this margin is too lenient. When considering the $\pm 10\%$ margin of error and the group of 20 ground motions, the *risk exceedance ratio* is acceptably low for the several risk levels (i.e. it is 0% for the higher risk levels and less than 20% for the lower risk levels). Even though only the results obtained for the mean of the ISDs (in logarithmic scale) are presented, similar observations were obtained for the rest of the EDPs and for both the mean and the median. As a result, the $\pm 10\%$ margin of error was seen to be adequate for all the cases and will be the margin of error considered in the subsequent analyses presented in the following.

3.5.2 Analysing the effect of the ASI on the estimates of the mean and the median

After defining the $\pm 10\%$ margin of error with respect to the reference value for a given EDP statistic, intensity level and building, the risk is calculated for all n_{ASI} combinations of the corresponding EDP statistic, as described in Section 3.2. As referred in Section 3.5.1, the risk values of all six buildings and all four intensity levels are aggregated and treated together for each EDP statistic. Figure 3.9 shows the risk values determined for the median of the logarithm of ISD and for all n_{ASI} combinations. The vertical dashed lines in Figure 3.9 separate the results obtained from different ground motion group sizes n . As presented in Section 3.2, four admissible risk levels are defined and are shown by the horizontal dashed lines in Figure 3.9. The corresponding *risk exceedance ratios* are then derived and plotted in Figure 3.10 for each n_{ASI} combination. Similar plots are created for the rest of the EDPs and central tendency statistics that are examined and discussed hereafter.

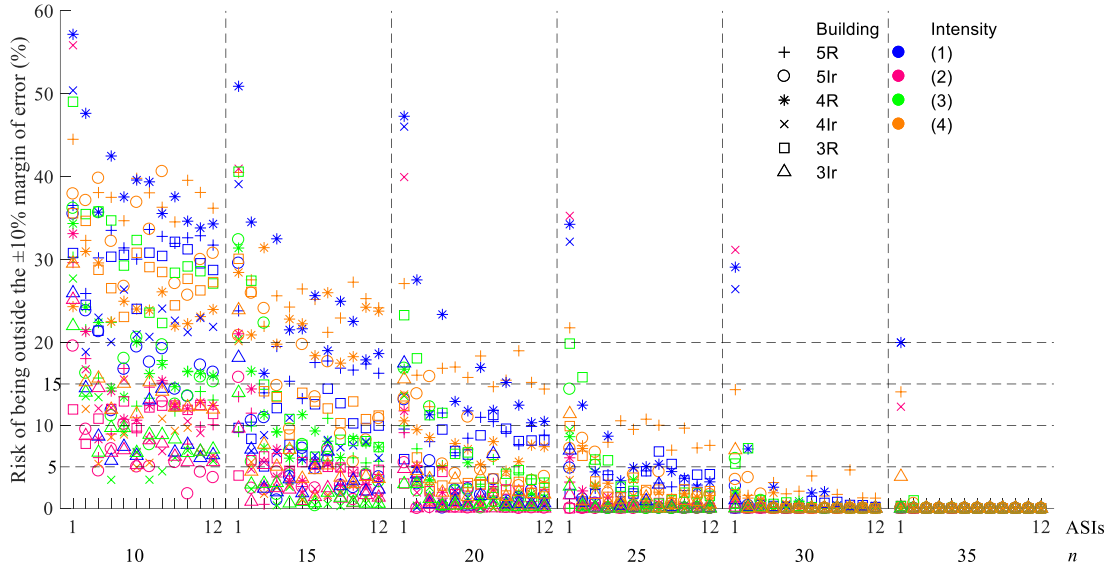


Figure 3.9. Risk values of the median of the logarithm of ISD.

The main effect of the ground motion group size was previously discussed in Section 3.5.1, where the increase of the size n was seen to lead to a reduction of the *risk exceedance ratio*. As referred, the individual effect of the ground motion group size can be determined when the ASI is fully accounted for, i.e. when all 12 ASIs are considered. The effect of the ASI for a given ground motion group size can be thus determined by comparing the previously referred case (n_{12}) to cases where a lower number of ASIs is used. To illustrate these results, Figure 3.10 shows the *risk exceedance ratio* as a function of the selected number of ASIs for the median of the logarithm of ISD. In particular, it can be seen that using only 1 ASI leads to a significant increase of the *risk exceedance ratio* for the overwhelming majority of the cases. This increase depends on the admissible risk level and on the ground motion group size that are considered, and can be up to more than 6 times the *risk exceedance ratio* obtained when considering all 12 ASIs, e.g. see the case where $n = 20$ with an admissible risk level of 10%. However, the two extreme cases, i.e. the largest values of n and admissible risk level and the smallest values of n and admissible risk level are exceptions to these general observations, for which using 1 or more ASIs has little or no influence on the *risk exceedance ratio*.

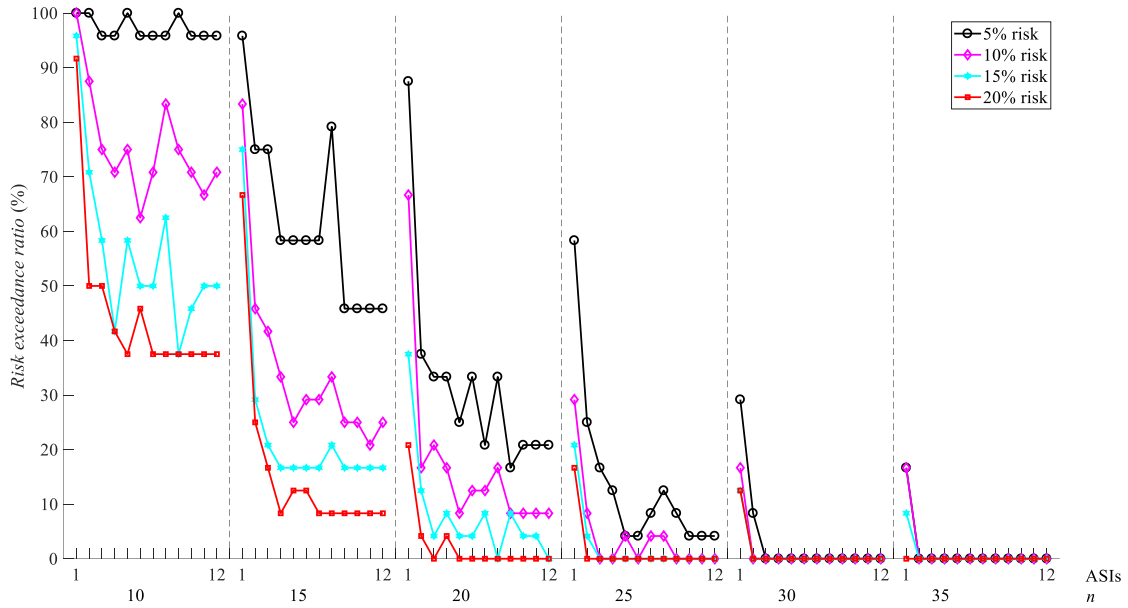


Figure 3.10. Risk exceedance ratios corresponding to the median of the logarithm of ISD.

Figure 3.10 shows that, for the larger ground motion group sizes, i.e. 30 and 35, using only 1 ASI can lead to a *risk exceedance ratio* up to 30%. On the other hand, using 2 or more ASIs leads to *risk exceedance ratios* that are zero for almost all the considered risk levels. In order to shed more light on the reason behind the increase of the *risk exceedance ratio* for cases involving 1 ASI and a larger ground motion group, Figure 3.11 shows the violin plots (see Section 3.2) of the median of the logarithm of ISD obtained for the 4-R building including a margin of error of $\pm 10\%$. These results are obtained for the seismic intensity corresponding to a 30% probability of exceedance in 50 years, when n is 30 and for all the ASI cases (similar trends are observed for the remaining buildings and intensity levels, and the results of this building are only presented to support the following discussion). The results show that, when 12 ASIs are considered, the whole distribution of the medians is within the margin of error thus leading to zero risk. This is mostly due to the small dispersion of the median estimates and the fact that their distribution is relatively well centred with respect to the reference value. Similar trends are also found for cases involving less than 12 ASIs, except for the case using only 1 ASI. In this case, the distribution of the medians is biased and tends to overestimate the reference value. This bias generates a situation where a large part of the distribution of the medians is outside the margin of error. This aspect combined with an increase in the dispersion of the distribution (when compared to cases involving a larger number of ASIs), leads to a risk value of about 30% (the value can be seen in Figure 3.9). Although the referred bias

of the presented case tends to overestimate the reference value, there are other cases where the existing bias tends to underestimate the reference value. Furthermore, no features were identified that would indicate a specific predisposition towards overestimation or underestimation.

Going back to Figure 3.10, focusing on the *risk exceedance ratios* of the intermediate group sizes, i.e. 20 and 25, reveals slightly different trends, especially for the lower levels of admissible risk (5% and 10%). In these cases, using only 1 ASI leads to large *risk exceedance ratios*, which can go up to 85%, and increasing the number of ASIs leads to their gradual reduction. Generally, more than 2 ASIs are required to reach a *risk exceedance ratio* close to that of the n_{12} case (i.e. within $\pm 10\%$). For the higher levels of admissible risk (15% and 20%), the *risk exceedance ratios* are about 30%, and can decrease either to zero or to a value close to that of the n_{12} case (i.e. within $\pm 10\%$) by using only 2 ASIs. Hence, the use of more than 2 ASIs in these cases has no practical benefits to reduce the *risk exceedance ratio*.

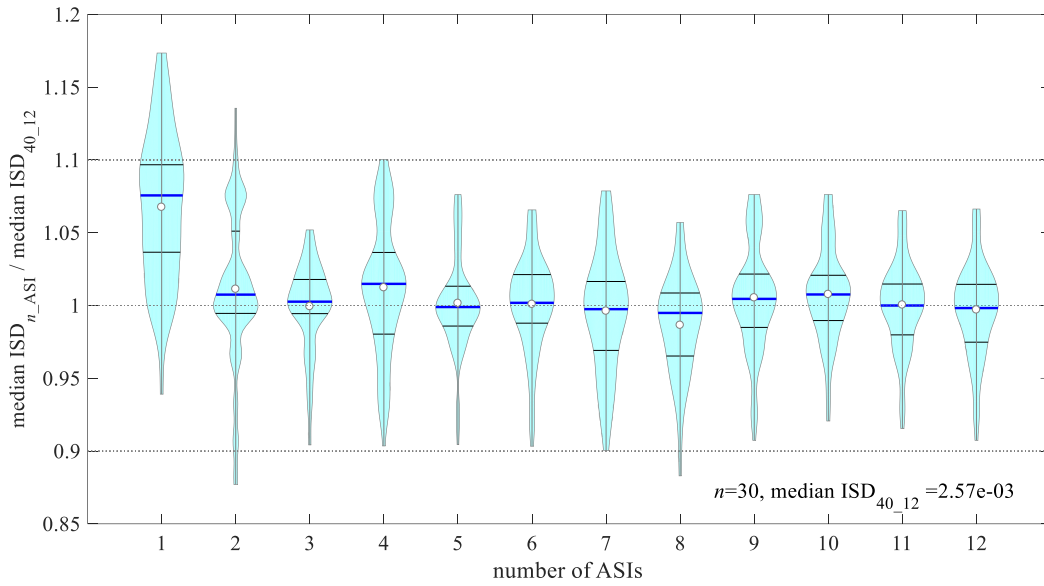


Figure 3.11. Violin plots of the median of the logarithm of ISD for $n=30$ and all the cases of ASIs (results obtained for the 4-R building and the 30% in 50 years probability of exceedance)

Regarding the smaller ground motion group sizes, i.e. 10 and 15, the effect of the ASI appears to be similar to that discussed for the intermediate ground motion group sizes, except for the fact that *risk exceedance ratios* are much larger given the larger variability of the distribution when these smaller ground motion group sizes are involved. As such,

using only 1 ASI and 10 ground motions always leads to *risk exceedance ratios* larger than 85%, while the lowest *risk exceedance ratios* that can be reached by considering the more lenient 20% admissible risk level can still be as high as 40%. A representative example of the violin plots obtained when using a small ground motion group size is presented in Figure 3.12 that shows the results of the 3-R building obtained for the case involving a group of 10 ground motions, and that the high risk values originate from the large variability. Although Figure 3.12 shows that the variability can be larger when only 1 ASI is used, the risk obtained using all 12 ASIs is already too high and no practical improvements are achieved by increasing the number of ASIs in this case. Much lower *risk exceedance ratios* are reported when using 15 ground motions, which, for the case of a 20% admissible risk level, can be as low as 10% and can be reached by considering at least 3 ASIs.

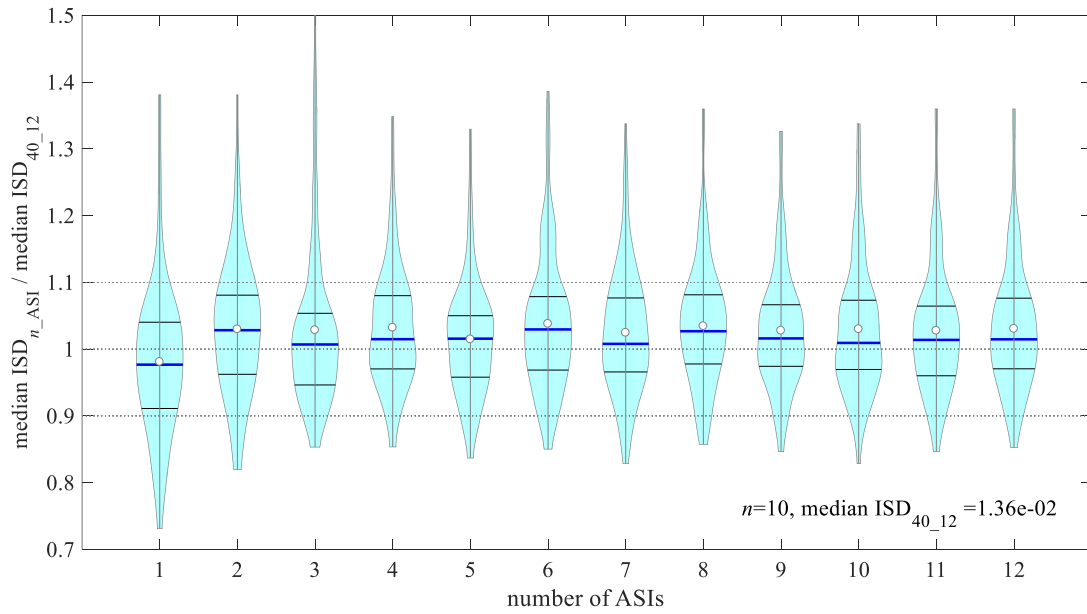


Figure 3.12. Violin plots of the median of the logarithm of ISD for $n=10$ and all the cases of ASIs (results obtained for the 3-R building and a seismic intensity of 1.2 times the seismic intensity corresponding to a 5% in 50 years probability of exceedance).

Following the same rationale of Figure 3.10, Figure 3.13 shows the *risk exceedance ratios* obtained for the median of the logarithms of RD, while Figures 3.14 and 3.15 show the *risk exceedance ratios* obtained for the mean and the median of the logarithms of PFA, respectively. For the ISD and the RD, since the results obtained for the statistics of the mean are similar to those obtained for the statistics of the median, only the latter is

discussed for the sake of brevity. As such, Figure 3.13 shows that the effect of the ASI on the median of the logarithms of RD can be accounted for by using 2 ASIs for most of the cases examined (i.e. the risk exceedance ratios are in the range of the value obtained for the n_{12} case $\pm 10\%$, for the majority of combinations of ground motion group size and admissible risk level). On the other hand, for the statistics of PFA the situation is different. The ASI seems to have a much lower influence on the mean value of the logarithms of PFA, since it shows to have a negligible effect when larger ground motion group sizes are involved, and a minor effect in the rest of the cases. In most of the situations, the effect of the ASI can be accounted for as proposed for the median of the logarithms of RD, i.e. by using 2 ASIs. On the contrary, larger *risk exceedance ratios* are observed when examining the results obtained for the median of the logarithms of PFA. In these cases, more ASIs are required to reach the *risk exceedance ratio* obtained with 12 ASIs (Figure 3.15) and, as observed for the median ISD, a gradual decrease of the *risk exceedance ratio* occurs with the increase of the number of ASIs for most ground motions group sizes and admissible risk levels.

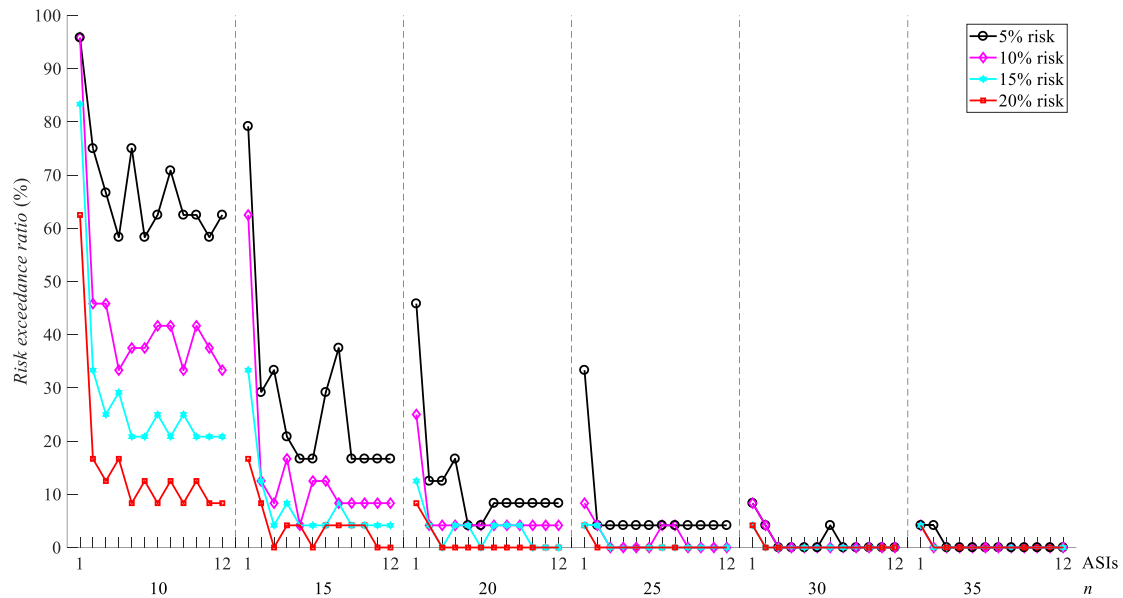


Figure 3.13. *Risk exceedance ratios* obtained for the median of the logarithm of RD.

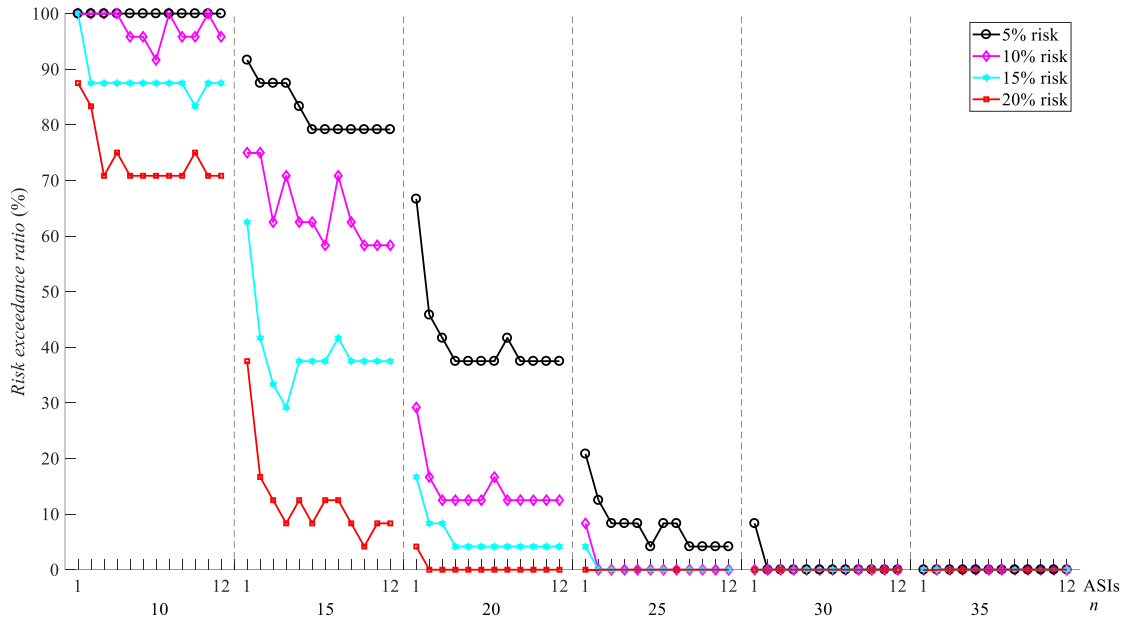


Figure 3.14. *Risk exceedance ratios* obtained for the mean of the logarithm of PFA.

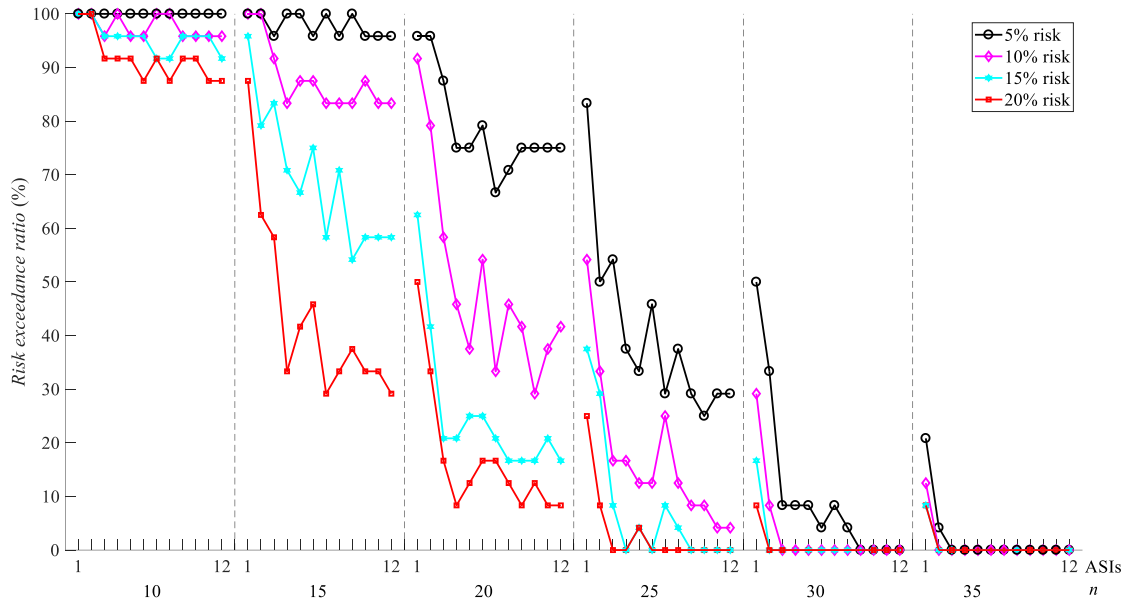


Figure 3.15. *Risk exceedance ratios* obtained for the median of the logarithm of PFA.

3.6 Results obtained for the variability measure

3.6.1 Setting the margin of error for the standard deviation

Following the rationale applied in Section 3.5.1 to analyse the central tendencies, tentative margins of error are also defined for the std of the logarithms of the EDPs, considering only the effect of the ground motion group size n , i.e. using all 12 ASIs. However, in this case, the final margin of error is expected to be larger than those of the central tendencies when considering the same sample size. A simple proof for this

statement is provided based on known statistical results. When a sample of size n is taken from a normal distribution with a mean μ and a standard deviation σ , a margin of error ME_m for the estimate of the mean can be established by combining Eq. (3.1) with the expression of the confidence interval for the mean of a normal distribution with known variance (Montgomery and Runger 2003):

$$ME_m = \pm z_{1-\alpha/2} \cdot CoV / \sqrt{n} \quad (3.2)$$

where CoV is σ/μ and $z_{1-\alpha/2}$ is the $(1-\alpha/2)$ percentage point of the standard normal distribution associated to a certain level of confidence α . Similarly, as proposed by Thompson and Endriss (1961), an approximate margin of error ME_{std} for the estimate of the standard deviation can be defined by:

$$ME_{std} = \pm z_{1-\alpha/2} / \sqrt{2 \cdot (n-1)} \quad (3.3)$$

The width of these margins of error can be compared by enforcing the same level of confidence. For example, Figure 3.16 presents the evolution of the ratio ME_{std}/ME_m for a confidence level α of 5% and three values of the CoV as a function of the sample size n . As can be seen, the ratio is always higher than 1, which indicates that, for a given sample size n , larger margins of error are required to ensure that the standard deviation is estimated with the same confidence level of the mean.

Given these results, three margins of errors are tentatively examined for the std, i.e. $\pm 10\%$, $\pm 15\%$ and $\pm 20\%$. Following the rationale behind the results presented in Figure 3.8 for the estimates of the mean value of the logarithms of ISD, Figure 3.17 presents the *risk exceedance ratios* obtained for the estimates of the std of the logarithms of ISD. Results are presented for the same admissible risk levels, i.e. 5%, 10%, 15% and 20%, and considering an aggregation of the results obtained for all six buildings and all four seismic intensity levels. As for the estimates of the mean value, the same analyses were performed for all EDPs, i.e. ISD, PFA and RD, and results with similar trends were obtained. By following principles like those adopted in Section 3.5.1 to establish the margin of error for the subsequent analyses, a $\pm 20\%$ margin of error was seen to be adequate for all the cases and was selected to perform the analyses for examining the effect of the ASI on the estimates of std.

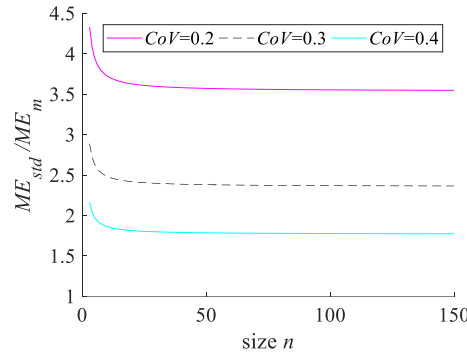


Figure 3.16. Evolution of ME_{std}/ME_m with the size n for different CoV values.

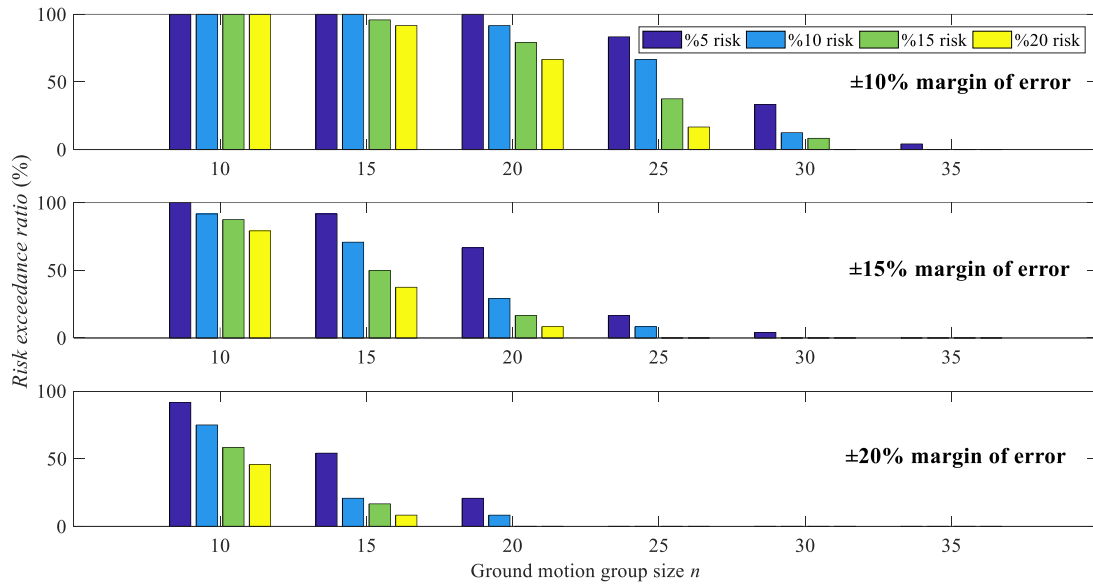


Figure 3.17 Risk exceedance ratios for the std of the logarithm of ISD, for four admissible risk levels and three margins of error, considering different ground motion groups sizes, and considering all the 12 ASIs, buildings and seismic intensities.

3.6.2 Analysing the effect of the ASI on the estimates of the standard deviation

The results of the effects of the ASI on the estimates of the standard deviation are presented following the format used for the central tendencies in Section 3.5.2. In particular, results focus mostly on the values obtained for the *risk exceedance ratios* as a function of the several combinations of n and ASI for the four levels of admissible risk. Figures 3.18, 3.19 and 3.20 show the results obtained for the std of the logarithms of ISD, PFA and RD, respectively. Results indicate that using only 1 ASI leads to *risk exceedance ratios* that are much larger than those obtained when all 12 ASIs are used, for all EDPs and for most of the ground motion group sizes and risk levels. Furthermore, the high values of the *risk exceedance ratios* that are observed (up to 100% in some cases) are only seen to reduce to values close to the n_{12} *risk exceedance ratios* when considering

2 to 4 ASIs. The required number of ASIs to achieve this reduction depends on the EDP, on the ground motion group size and on the admissible risk level. Involving larger ground motion group sizes and higher admissible risk levels generally requires the use of a lower number of ASIs. In addition, results also indicate that estimates of the std of the logarithms of PFA appear to be less sensitive to the number of ASIs. On the contrary, the estimates of the std of the logarithms of ISD and RD show larger variations.

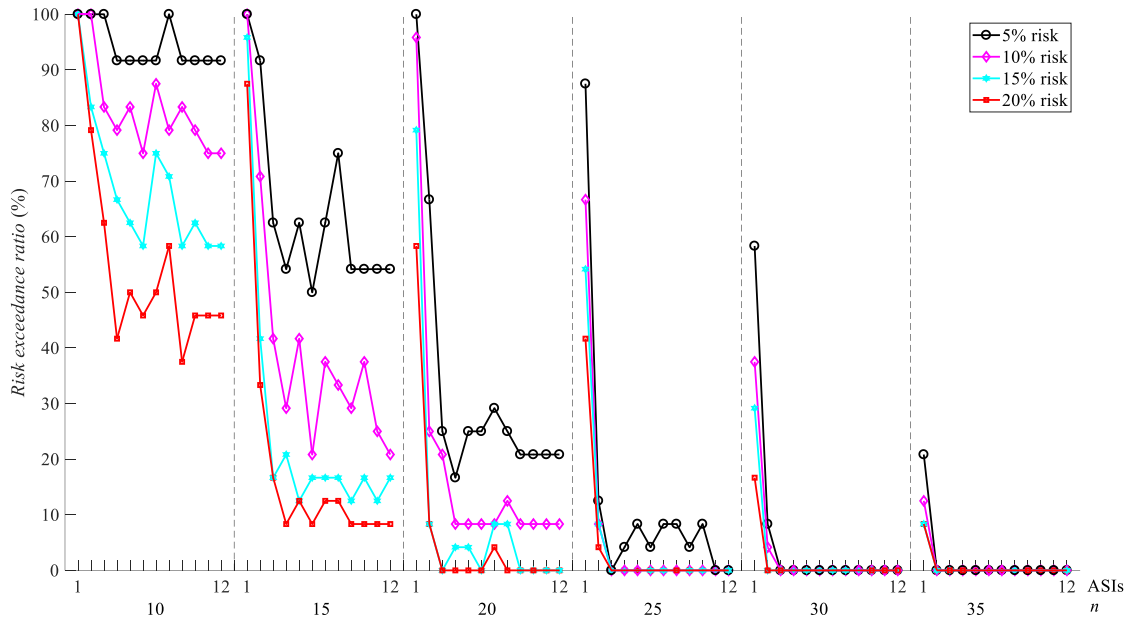


Figure 3.18. Risk exceedance ratios obtained for the std of the logarithm of ISD.

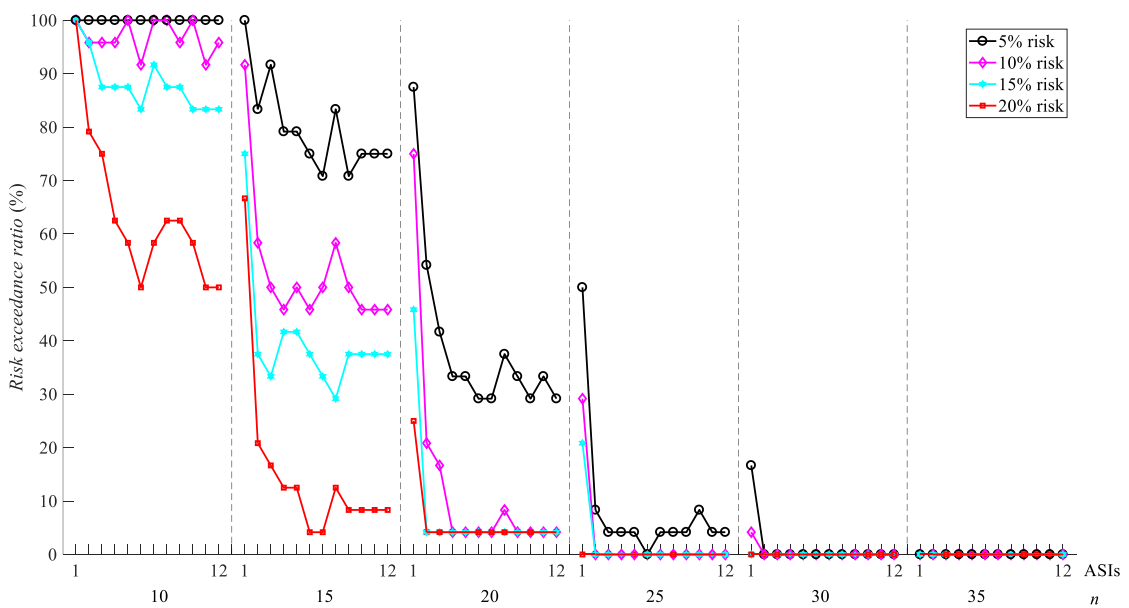


Figure 3.19. Risk exceedance ratios obtained for the std of the logarithm of PFA.

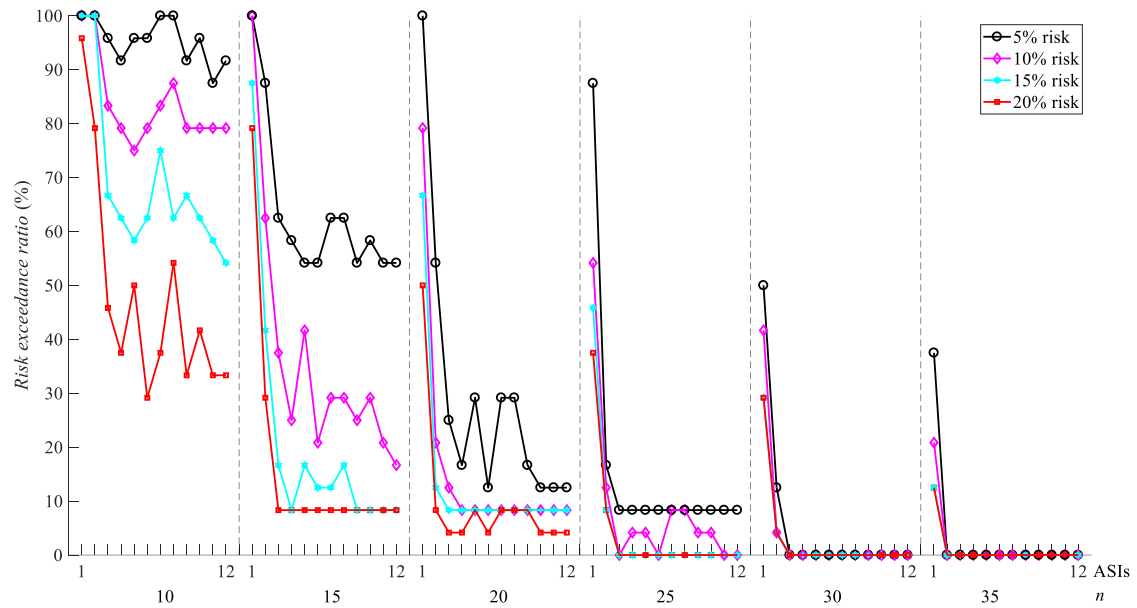


Figure 3.20. *Risk exceedance ratios* obtained for the std of the logarithm of RD.

To complement these results, violin plots of the std of an example case are also provided, highlighting the more common trends of the effect of the ASI on this statistic. The violin plots presented in Figure 3.21 correspond to those of the std of the logarithm of ISD for the 5-R building considering the $\pm 20\%$ margin of error. These results are those obtained for the seismic intensity corresponding to a 10% probability of exceedance in 50 years, when n is 20 and for all the ASI cases. Given the larger margin of error that is adopted, Figure 3.21 shows that small risk values are obtained for a number of ASIs equal or larger than 3. On the other hand, if only 1 ASI is used, the portion of the violin plots that is outside the margin of error is significant, due to the combined effect of an increase in the variability and in the bias of the estimates, and leads to a larger risk value ($\sim 25\%$). The bias and the variability are slightly reduced when 2 ASIs are employed, but this scenario still leads to a risk value that is significantly larger than the value obtained when using all 12 ASIs. For the presented case, it can be seen that using more than 3 ASIs has no relevant effect on the value of the risk.

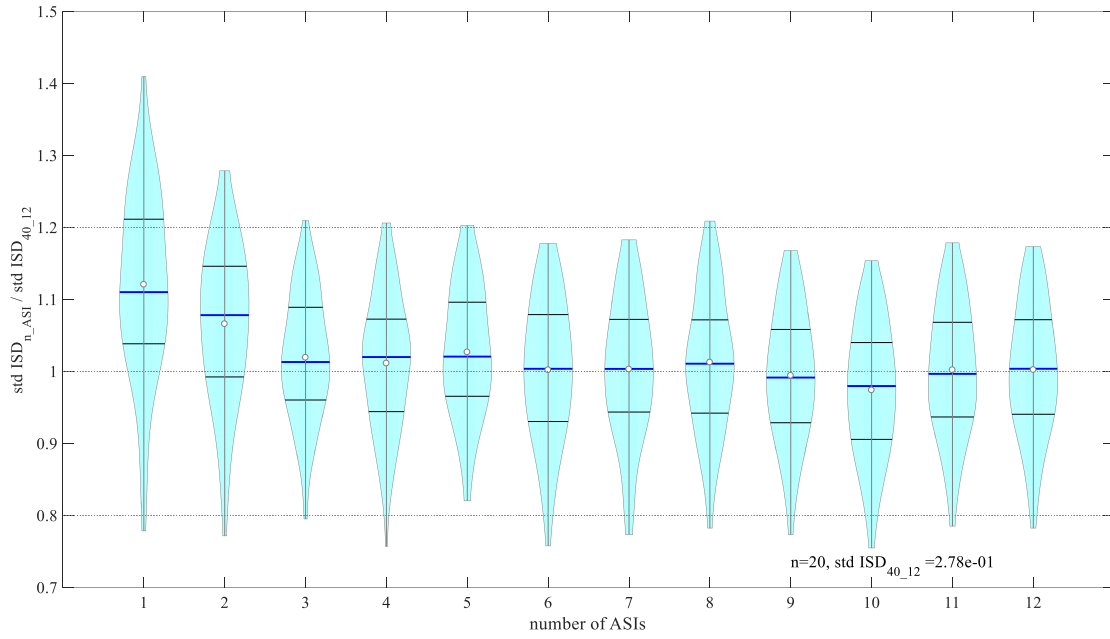


Figure 3.21. Violin plots of the std of the logarithm of ISD for $n=20$ and all cases of ASIs (results obtained for the 5-R building and a seismic intensity corresponding to a 10% in 50 years probability of exceedance).

3.7 Discussion

One of the most apparent features observed in the results presented in Sections 3.5 and 3.6 is that the effect of the ground motion group size has a larger influence on the estimates of seismic demand when compared to that of the ASI, a fact which is in agreement with the conclusions of Giannopoulos and Vamvatsikos (2018). In particular, the ground motion group size governs the minimum values that can be expected for the *risk exceedance ratios*, irrespective of the number of ASIs that are considered. As a reminder, it is noted that the effect of the ground motion group size is obtained herein when all 12 ASIs are used, i.e. when the ASI is fully accounted for. As such, it was observed that, by increasing the ground motion group size, the *risk exceedance ratios* decrease (or remain equal to zero) for all the cases that were studied. As far as the ASI is concerned, its effect on the *risk exceedance ratios* appears to depend on the EDP and the statistic that is examined. Furthermore, for most of the cases that were analysed, and again depending on the EDP and the statistic, using an insufficient number of ASIs could lead to large *risk exceedance ratios* when compared to the corresponding value obtained for the n_{12} case. On the other hand, for each EDP and statistic, there is a certain number of ASIs beyond which the reduction of the *risk exceedance ratios* can be considered negligible. Before addressing a more detailed discussion of the effect of the ASI on the

different statistics, it is worth noting that using ground motion groups with a size of 10 and sometimes of 15 led to high *risk exceedance ratios*, even when considering large admissible risk levels. This trend was found to be valid for all the EDPs and statistics that were analysed and is attributed to the large variability of the statistics associated to small ground motion group sizes, as shown for example in Figure 3.12.

Focusing on the effect of the ASI when estimating central tendencies, the mean of the logarithm of PFA and the mean and the median of the logarithm of RD were seen to be the EDPs that are least affected. For these EDPs, it was seen that using only 1 ASI could lead to larger *risk exceedance ratios* when compared to the n_{12} case, but for n larger than 15 and for an admissible risk higher than 5% this increase was found to be negligible. When using 2 ASIs, the *risk exceedance ratios* decreased to values close to that of the n_{12} case and, in most of the cases that were analysed, using more than 2 ASIs had only a negligible effect on further reducing the *risk exceedance ratio*. For the larger ground motion group sizes and for the larger admissible risk levels, it was also observed that 1 ASI was enough to capture the overall effect of the ASI, leading to *risk exceedance ratios* similar to those obtained when using all 12 ASIs.

Unlike the previous EDPs, the median of the logarithm of PFA and the mean and the median of the logarithm of ISD were found to be much more prone to variations depending on the number of ASIs that is used. As such, using 1 ASI was found to lead to much larger *risk exceedance ratios* when compared to the corresponding n_{12} case for all ground motion group sizes and when considering low admissible risk levels. For these cases also, only a gradual decrease of the *risk exceedance ratio* was observed when increasing the number of ASIs, and the number of ASIs required to reach the n_{12} *risk exceedance ratios* varied. This variation depended both on the ground motion group size and the examined EDP. For larger groups sizes (30 and 35), usually 2 ASIs provided results close to that of the n_{12} case, while for smaller group sizes (10 and 15) the variation of the *risk exceedance ratios* with the ASI was either too high or the *risk exceedance ratios* due to the ground motion group size only (i.e. the n_{12} case) were so large that the ASI had no practical influence in the results. For the intermediate ground motion group sizes (20 and 25), 2 to 4 ASIs were usually sufficient to obtain *risk exceedance ratios* with a value close to that of the n_{12} case.

Regarding the effect of the ASI on the std of the demand distributions, the ISD and the RD appeared to be similarly influenced and the effect of the ASI was found to be much

more pronounced when compared to its effect on estimates of the central tendencies. Even though a larger margin of error had to be adopted when analysing the std of the demand distributions, using only 1 ASI was found to lead to *risk exceedance ratios* that are excessively high in most of the cases. For example, the *risk exceedance ratio* obtained for the case of RD is 100% when only 1 ASI is used, while it is less than 20% when all 12 ASIs are employed. In order to reduce the *risk exceedance ratios* to a value close to that of the n_{12} case, 2 to 4 ASIs were required, depending on the ground motion group size and the admissible risk level. When analysing the effect of the ASI on the std of the PFA distributions, it can be seen to be smaller than for the previous EDPs, as long as ground motion groups of size 25 or more are considered and the admissible risk level is higher than 5%. In most cases, using 2 ASIs was sufficient to obtain *risk exceedance ratios* with a value close to zero. Nevertheless, when ground motion group sizes larger than 30 are involved, the ASI appeared to have no effect on this EDP.

3.8 Conclusions

The proposed simulation study examines the effects of the ground motion group size and the number of ASIs on the empirical demand of selected EDPs. These effects were analysed using specific statistics of the empirical distributions of the EDPs that characterise their central tendency and their dispersion. The relevance of these effects on each EDP statistic is analysed by quantifying a *risk exceedance ratio* that measures how far a given estimate of the statistic is from its reference value, within a certain margin of error and considering a certain admissible risk of not complying with that margin of error.

In the overall, it can be concluded that, depending on the purpose of the analysis and on the EDP under consideration, the ASI can have either a negligible or a determinant effect on the closeness between the estimate of the EDP statistic and its reference value. Based on the results that were obtained, it is found that, if only mean/median values of the EDPs are required (e.g. in the context of standard-based analysis procedures), using only 1 ASI can be sufficient as long as a large enough ground motion group size is also considered (at least 30 ground motions depending on the level of admissible risk). In most of the cases, though, using 2 or 3 ASIs appears to be a safer choice. On the other hand, using more than 4 ASIs does not seem to provide relevant benefits. The appropriate number of ASIs that leads to the required closeness between the estimate of the EDP statistic and its reference value was found to depend on the ground motion group size and on the level of

admissible risk: considering larger group sizes and higher admissible risk levels was seen to require a lower number of ASIs. When performing probabilistic analyses that may also require information about a measure of the variability of the EDP distribution such as the std, using only 1 ASI was seen to be inadequate. In this case, using 2 to 4 ASIs is highly suggested, depending again on the admissible risk level and on the ground motion group size. Unlike for estimating central tendencies, even with large ground motion group sizes (i.e. 30 and 35), 2 ASIs seem to be necessary, while for smaller ground motion group sizes (i.e. 20 and 25) up to 4 ASIs can be required. Estimating the variability of the PFA is, however, an exception to this rule since it appears to be insensitive to the effect of the ASI when 30 or more ground motions are used. Furthermore, in most cases where ground motion group sizes of 20 or 25 are used, the variability of the PFA can also be captured by considering only 2 ASIs, as long as the admissible risk level is higher than 5%. Finally, it is worth restating that, for all the statistics considered, the ground motion group size has a determinant contribution to the *risk exceedance ratios* and the use of at least 20 ground motions is suggested in all situations.

Chapter 4

4 The effect of the Angle of Seismic Incidence when defining a statistical model for structural demand

Based on:

Skoulidou, D., Pereira, N., & Romão, X. (2018). The effect of the angle of seismic incidence when defining a statistical model for structural demand. In the *16th European Conference on Earthquake Engineering*, Thessaloniki, Greece.

Abstract

This chapter continues analysing the probabilistic seismic demand of selected EDPs at predefined intensity levels. In this context, the statistical distribution of three demand parameters is examined, obtained from three-dimensional analysis of RC buildings considering multiple angles of seismic incidence and various ground motion group sizes. Six statistical models are fitted to the EDP datasets and their performance is examined based on quantile-quantile plots as well as on the corresponding coefficient of determination and the root mean square error. The results show that using more than one angle of seismic incidence does not affect the type of demand distribution that best fits the datasets in the majority of the cases studies. The lognormal distribution exhibits higher performance when compared to the rest of the distributions, while in many cases its superior behaviour is marginally better than that of the Gamma and the Weibull distributions. Goodness-of-fit tests with power against specific distribution models are further applied to the datasets to examine the suitability of the aforementioned models in

a more quantitative manner. The superiority of the performance of the lognormal and the Gamma distributions is shown again, in accordance with the preliminary results. Finally, the particular effect of the number of ASIs on the empirical data is highlighted along with suggestions regarding the distribution selection in case a higher number of ASIs is used.

4.1 Introduction

Probabilistic analyses often require making assumptions regarding the statistical distribution of the variables involved. The lognormal distribution is frequently used to describe the statistical distribution of different engineering demand parameters (EDPs) at discrete intensity levels or of the fragility function for distinct limit states (e.g. see Ibarra and Krawinkler 2005). The assumption of the lognormality of the data has been analysed by several researchers, both for two-dimensional and for three-dimensional (3D) structural analyses (e.g. see Romão et al. 2011). In the latter case though, relevant results exist only for structural analyses performed with one angle of seismic incidence (ASI), while limited work has been done in probabilistic seismic analysis involving multiple ASIs, e.g. see the research by Lagaros (2010a, 2010b), in which the lognormal data assumption was adopted in both studies.

In this context, the objective of the current chapter is twofold. Based on the results of 3D structural analyses performed considering multiple ASIs, the soundness of the lognormal statistical model is first examined when used to characterize the probability distribution of structural demand. Secondly, the performance of alternative statistical models to characterise the distribution of the structural demand is also evaluated. As such, the goodness-of-fit of six statistical models, including the lognormal distribution, is analysed when fitting them to EDP samples. The EDP samples are obtained considering the number of ground motions (GMs) as well as the number of ASIs, as described in Chapter 3.

The performance of all six statistical models is initially analysed using quantile-quantile (Q-Q) plots whose assessment is performed visually and by using the coefficient of determination (R^2) and the root mean square error (RMSE). The R^2 and the RMSE are used to evaluate the adequacy of the linear regression between the theoretical and the empirical quantiles. The statistical models demonstrating the best performance based on the preliminary evaluation using the Q-Q plots are further explored and their performance is evaluated using statistical goodness-of-fit tests. The results aim to highlight the

influence of the ASI in probabilistic analysis and to point out possible implications in the development of a statistical model of the demand to be used for further risk and loss structural analyses.

4.2 Methodology and statistical models analysed

The current study evaluates the performance of different statistical models to characterise the demand distribution obtained from 3D analyses when multiple ASIs are considered. Additionally, further insights are provided regarding the unconditional use of the lognormal model to represent seismic demand data. The results are obtained for the six buildings presented in Section 3.3, which were subjected to nonlinear dynamic analysis for bi-directional GM groups of size n equal to 10, 15, 20, 25, 30, 35 and 40, applied along 1 to 12 ASIs, as per Section 3.4. Three EDPs are considered to represent seismic demand: the maximum inter-storey drift ratio (ISD), the maximum roof drift ratio (RD) and the peak floor acceleration (PFA). The ISD and the PFA are defined as the maximum ISD and PFA of the two structural directions across the height of the buildings, respectively, while the RD corresponds to the square root of the sum of squares (SRSS) combination of the displacements of two directions measured at the centre of mass of the top storey. The probabilistic analysis is then performed for EDP samples obtained from 4 discrete intensity levels (see Section 3.4) analysed using GM groups of size n and a number of ASIs. In the following, the notation n_ASI will be used to refer to the results of a particular size n and number of ASIs.

Five statistical models are considered in addition to the lognormal distribution and fitted to the EDP data of each sample. The distributions considered are the normal, the lognormal, the Gamma and the three sub-families of the generalized extreme value (GEV) family of distributions, i.e. the Gumbel (Type I extreme value distribution), the Fréchet (Type II extreme value distribution) and the Weibull (Type III extreme value distribution). The GEV is a flexible three parameter model (location, scale, shape) that, depending on the shape parameter, can represent the three different distribution families. The lognormal and GEV models are selected due to their suitability to characterise the distribution of maximum (extreme) values. The normal model is considered due to its widespread use in statistical analysis and also due to its performance for some types of demand data (e.g. see Romão et al. 2011). Finally, the Gamma distribution is considered given its flexibility to model different types of data.

The suitability of a probabilistic model to fit the sample data is analysed in two steps. Initially, all 6 statistical models are analysed using Q-Q plots and two measures of linear fit between the empirical and the theoretical quantiles. Subsequently, the distribution models that presented the best fittings are further explored for their adequacy employing suitable goodness-of-fit tests. Further details on the formulation of the measures of linear regression as well as on the statistical tests are given in the following sections.

4.3 Examining the distribution family using Q-Q plots

4.3.1 Formulation of the R^2 and the RMSE measures

The suitability of a probabilistic model to fit the sample data is initially analysed using Q-Q plots whose assessment is carried out qualitatively by visual inspection and by determining the linear fit between the empirical and the theoretical quantile data using the coefficient of determination R^2 and the root mean square error RMSE. These measures are formulated using the sum of square errors (SSE), i.e. the sum of the squares of the differences between the theoretical (q_{theor}) and the empirical quantiles (q_{emp}), and the total sum of squares (TSS), i.e. the sum of the squares of the differences between the empirical quantiles and the average of the empirical quantiles $\bar{q}_{emp}$, and are obtained from the following expressions:

$$RMSE = \sqrt{1/m \sum_{i=1}^m (q_{theor,i} - q_{emp,i})^2} \quad (4.1)$$

$$R^2 = 1 - \frac{SSE}{TSS} \quad (4.2)$$

where m is the size of the sample of data. Although both measures are indicators of the accuracy of the regression models, their different formulations provide different information about how well the regression model compares to the data. The RMSE is an absolute measure that indicates the absolute fit of the model to the data and it has the useful property of being in the same units as the sample data. Lower values of the RMSE indicate better fit. The R^2 , on the other hand, is a relative measure of the improvement in prediction from the regression model compared to the mean model. It usually ranges from 0 to 1, with 1 indicating perfect prediction. An illustrative example of the procedure is presented in Figure 4.1 for an ISD sample and for two statistical models, the normal and the lognormal.

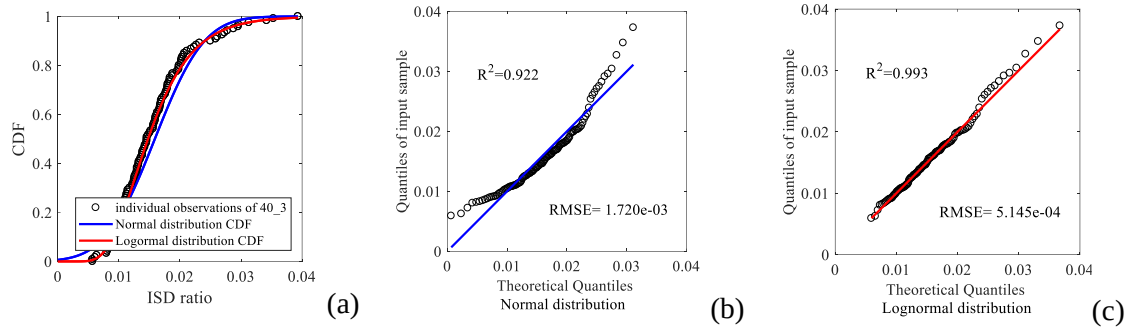


Figure 4.1 Cumulative distribution functions (CDF) of the normal and lognormal distributions fitted to a 40_3 ISD sample obtained for an intensity of 0.32g (a); Q-Q plots for the normal (b) and the lognormal (c) model.

The ISD sample corresponds to a 40_3 example obtained from the 5-storey regular building (presented in Section 3.3) and for a seismic intensity equal to 0.32g (intensity (4) as defined in Section 3.4.2 and Table 3.2). The cumulative distribution functions (CDFs) of the two statistical models fitted to the ISD sample are shown in Figure 4.1 (a), along with the empirical data. Figure 4.1 (b) and (c) represent the Q-Q plots for the normal and the lognormal model, respectively. The poor fitting of the normal model is evident from the visual analysis of the Q-Q plot (i.e. the points do not fall on a straight line) and its shape further suggests that the sample data is skewed. On the other hand, the visual analysis of the Q-Q plot for the lognormal distribution shows this distribution provides a better fit to the data, as seen by the straight line formed by the points in Figure 4.1 (c) that only deviate slightly from the $y = x$ line. Even though a process based on the visual analysis of Q-Q plots is likely to be able to support the identification of distributions that are best to fit the data, its implementation for a significant number of cases is not practical. Therefore, the use of quantitative measures given by R^2 and RMSE is selected instead to automate the identification process. For the case of Figure 4.1, it can be seen that the R^2 is larger and, at the same time, the RMSE is smaller for the lognormal distribution, which corroborates the assessment based on the visual analysis. Nevertheless, although comparing these two measures for different distribution alternatives is able to identify the best candidate distributions to fit the data, these distributions may not be adequate enough and their suitability is further analysed in a second step using goodness-of-fit testing, as referred before.

To further illustrate the first step in the identification of candidate distributions, an additional example is presented in Figure 4.2 that shows the Q-Q plots and the

corresponding R^2 and RMSE values of an ISD sample fitted with all 6 statistical models. The sample is obtained from the analysis of the 5-R building, for one group of 35_4 and for an intermediate ground motion intensity level with $S_a(T^*)=0.27g$, as shown in Table 3.2. The figure shows that the lognormal and the Gamma distributions appear to be the more adequate for this particular ISD sample, followed by the Weibull distribution. The normal distribution demonstrates a weaker fit when compared to the previously referred distributions, while the Gumbel and the Fréchet distribution fits exhibit very low performance.

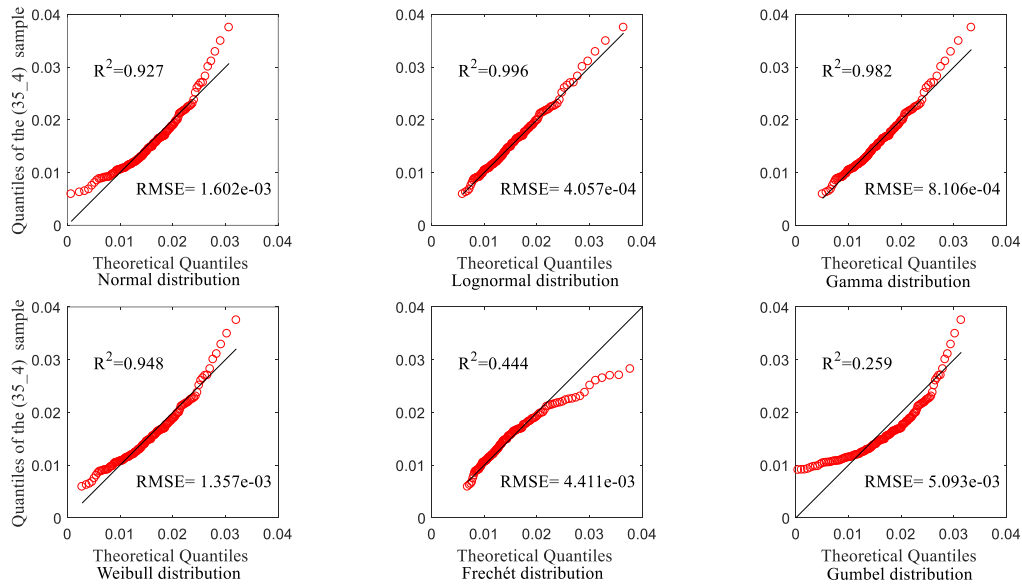


Figure 4.2 Q-Q plots and the corresponding R^2 and RMSE values of all considered distributions for a 35_4 ISD sample that corresponds to inelastic response of the 5-R building.

4.3.2 Q-Q plot results: R^2

The mean of the R^2 values of the 100 groups of selected combinations of record sizes n and number of ASIs are presented in Figure 4.3 to Figure 4.8 for all buildings. Specifically, the 35_1 and 35_12 results of the ISD, RD and PFA are presented in Figure 4.3, Figure 4.4 and Figure 4.5, respectively. Likewise, the 15_1 and 15_12 results of the ISD, RD and PFA are presented in Figure 4.6, Figure 4.7 and Figure 4.8, respectively. Intensity (1)-(4) in all figures refers to those identified in Table 3.2. The results obtained from the groups of size $n = 35$ are representative of larger group sizes ($n \geq 25$), while the results obtained from the groups of size $n = 15$ are representative of smaller group sizes ($n \leq 20$). Given the expected dependence of R^2 and RMSE on the sample size of the fitted

data, the results should be interpreted separately for each n_{ASI} combination and the focus should be given to the relative variations of R^2 among the different distributions for each n_{ASI} .

It is observed that the lognormal model presents the best fitting for most of the cases studied, involving the different buildings, number of ASIs and intensity levels, followed closely by the Gamma distribution. This trend holds true for all EDPs examined herein and for both ground motion group sizes. The Weibull distribution also presents good fitting performance, sometimes even superior to the two previously referred distributions, see for example the 35_1 and 35_12 fitting of the PFA obtained from the 5-R building for the mixed elastic-inelastic response (Intensity 2) shown in Figure 4.5. Still, these differences can be characterized as negligible. Regarding the adequacy of the normal distribution, it can be seen to vary significantly mainly as a function of the level of inelasticity, where cases involving higher inelastic demand, i.e. for a higher intensity, are seen to exhibit a lower fitting performance. As for the rest of the two sub-families of the GEV distribution, i.e. the Fréchet and the Gumbel, their performance is, in the majority of the cases, inferior to the rest of the distribution models examined. Regarding the Gumbel model, very low R^2 values are observed for all intensity levels, all EDPs and buildings and all n_{ASI} combinations presented in Figure 4.3 to Figure 4.8. As far as the Fréchet model is concerned, it exhibits an extremely low performance, as indicated by the R^2 values found for most of the cases. However, for specific cases, such as the mainly inelastic response of the 3-R building shown in Figure 4.3 to Figure 4.8, the Fréchet distribution appears to fit better than the rest of the models.

It can be further observed in the same figures that using 1 or 12 ASIs does not seem to significantly affect the type of statistical model that best fits the demand distribution. In particular, the range of R^2 values found for the normal, lognormal, Gamma and Weibull models is uninfluenced in most of the cases, with only a few exceptions (e.g. see the mainly inelastic response (Intensity 4) of the 5-Ir building shown in Figure 4.3). On the contrary, larger discrepancies are observed for the Fréchet and the Gumbel distributions. Nevertheless, these differences involve very low R^2 values in most of the cases. The few cases already mentioned earlier, in which the Fréchet model fits data that none of the remaining models is able to, are important exceptions to this general statement. Nevertheless, it needs to be reminded at this point that the relative difference between the R^2 values of samples involving the same n , but different number of ASIs, should be

examined due to the dependence of R^2 on the sample size. As an overall comment on this respect, it can be said that the smallest variations can be seen for the RD and the elastic response of the buildings analysed for large n values, while the largest variations are found for the ISD and for the mainly inelastic response of the buildings analysed for smaller n values.

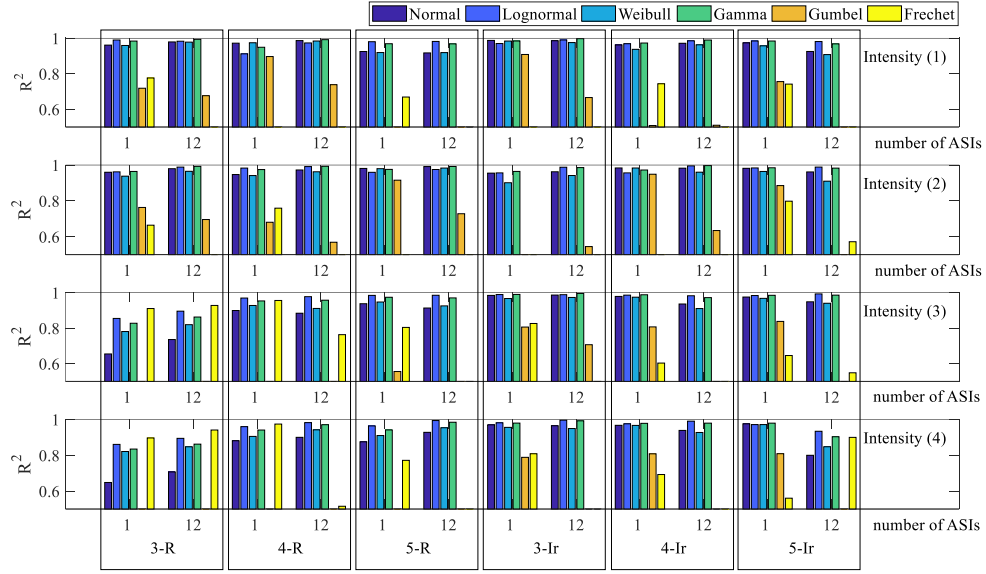


Figure 4.3 Mean R^2 obtained from ISD Q-Q plots for 35_1 and 35_12, for all buildings.

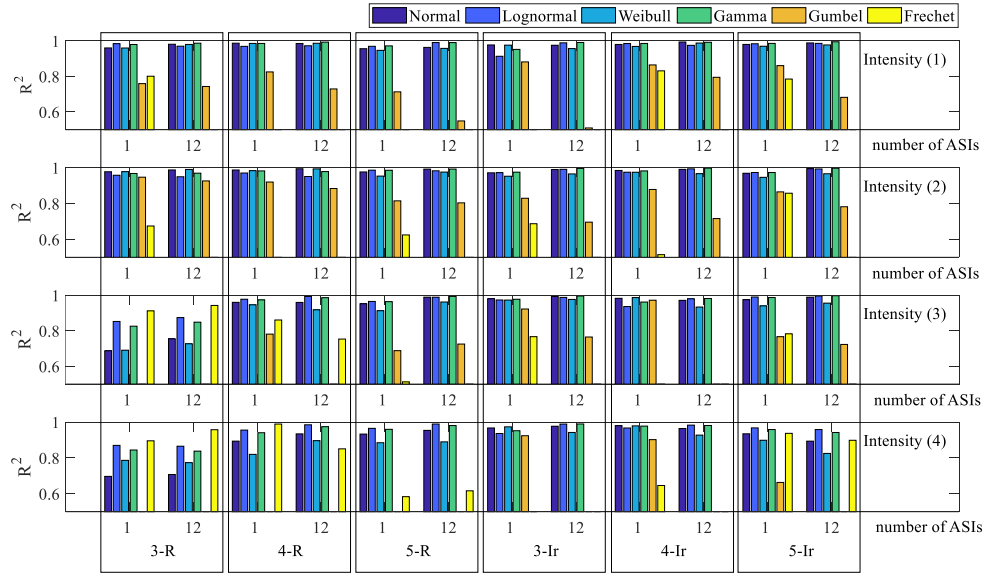


Figure 4.4 Mean R^2 obtained from RD Q-Q plots for 35_1 and 35_12, for all buildings.

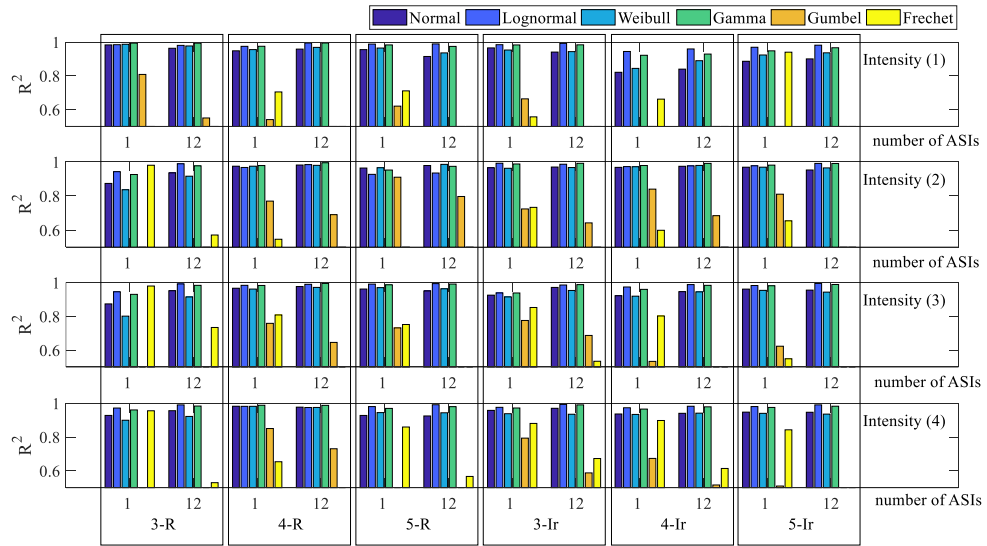


Figure 4.5 Mean R^2 obtained from PFA Q-Q plots for 35_1 and 35_12, for all buildings.

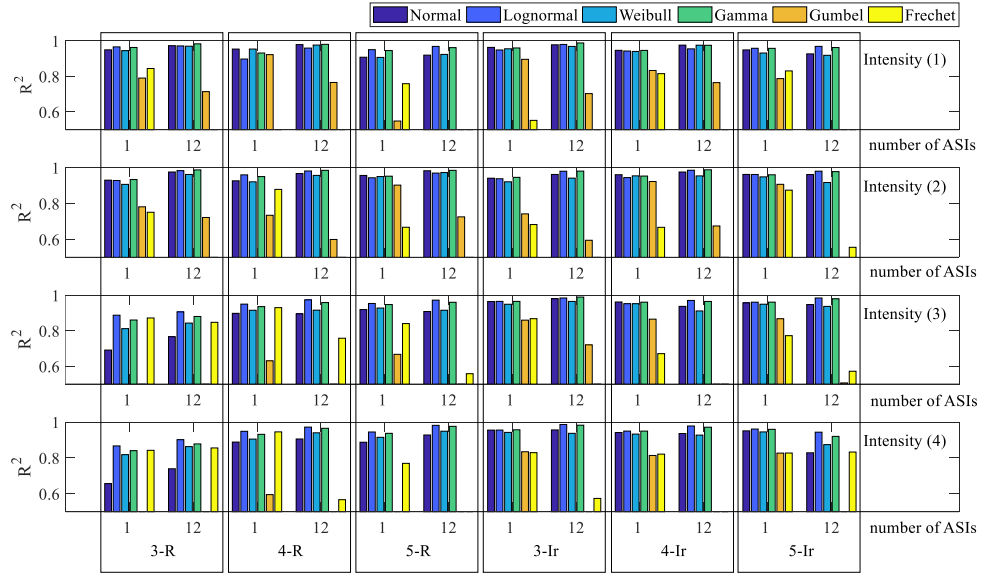


Figure 4.6 Mean R^2 obtained from ISD Q-Q plots for 15_1 and 15_12, for all buildings.

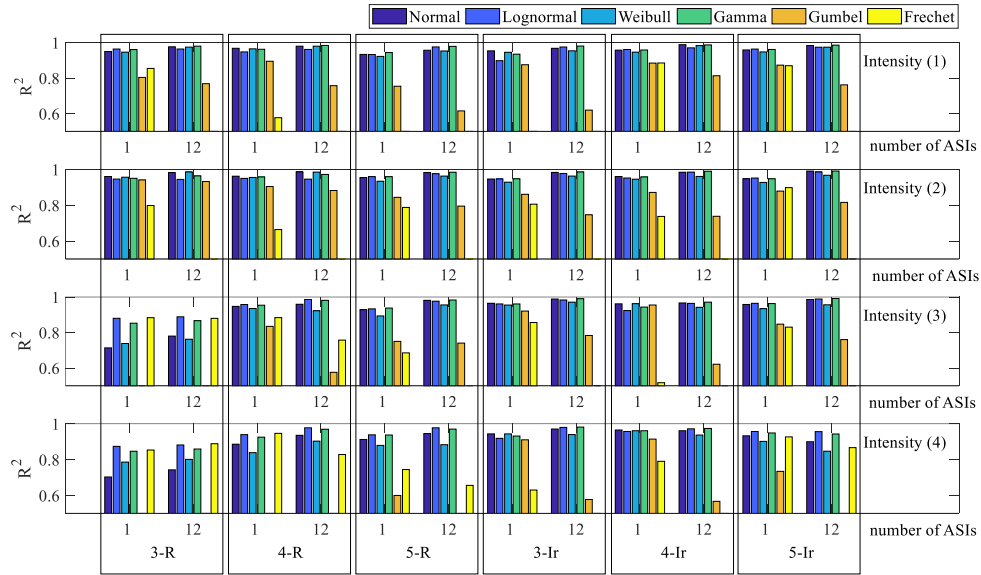


Figure 4.7 Mean R^2 obtained from RD Q-Q plots for 15_1 and 15_12, for all buildings.

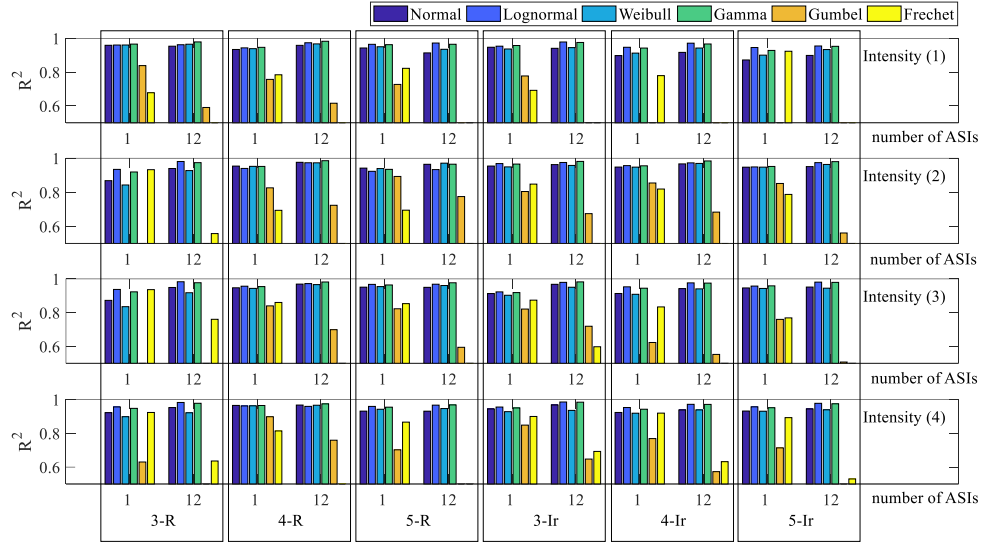


Figure 4.8 Mean R^2 obtained from PFA Q-Q plots for 15_1 and 15_12, for all buildings.

In order to have a more complete picture of the fitting performance of the different models, the coefficient of variation (cov) of the R^2 values for the 100 groups is also examined. Since similar trends were observed for all three EDPs, only the cov values obtained for the R^2 of the ISD are presented and discussed. Following the same presentation format as the one used for the R^2 mean values, Figure 4.9 and Figure 4.10 show the cov values of R^2 for the ISD samples of sizes 35 and 15, respectively. The cov of the lognormal and the Gamma models are always below 10% and, in the vast majority of cases, below 5%. The latter observation in combination with the high mean R^2 values suggest that the lognormal and the Gamma model are the most adequate to fit the sample data. The cov of the R^2 values for the normal and the Weibull models is also low, lower than 10%, for the larger GM group sizes, while it is higher in case smaller GM group sizes are used, reaching up to 30%. In contrast to the relatively low variability observed for the previously discussed four models, the Gumbel and the Fréchet models exhibit much higher cov values. These cov values may be higher than 50%, indicating a large variability. It is noted that 0 cov values indicate the inability to fit instead of zero variability and these appear in cases in which very low mean R^2 values were simultaneously observed. The initial fitting is performed using the maximum likelihood method with 400 iterations and a tolerance of $10e-5$. However, no better performance could be achieved by changing the tolerance or by increasing the number of iterations. Finally, as expected, larger cov values are associated with smaller GM group sizes,

however, no constant trends can be observed regarding the variability of the models when 1 or 12 ASIs are used.

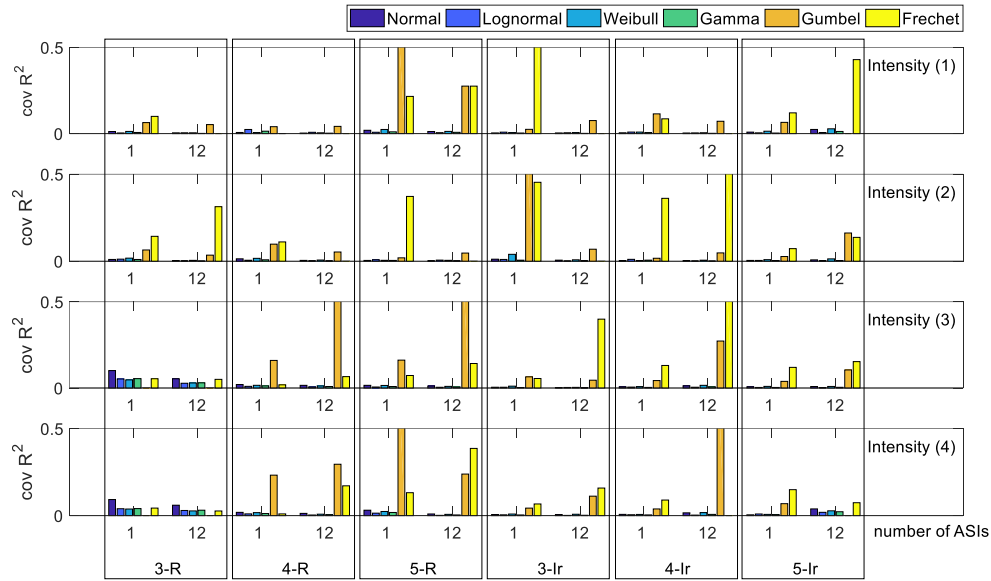


Figure 4.9 cov of R^2 obtained from ISD Q-Q plots for 35_1 and 35_12, for all buildings.

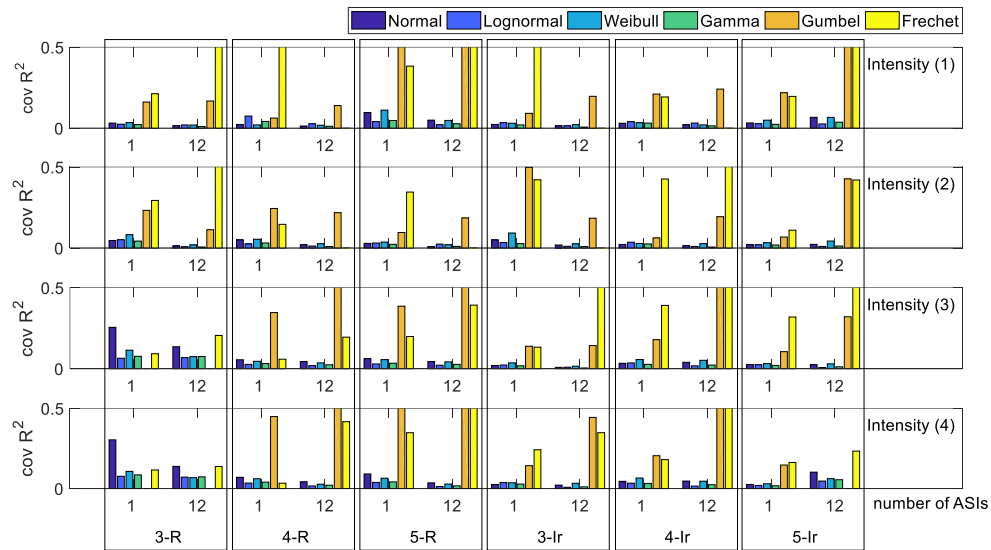


Figure 4.10 cov of R^2 obtained from ISD Q-Q plots for 15_1 and 15_12, for all buildings.

4.3.3 Q-Q plot results: RMSE

In order to demonstrate the evolution of the fitting performance between the alternative distribution models when using different numbers of ASIs, the RMSE is also employed, following a slightly different presentation format than the one used for the R^2 results. The RMSE values are aggregated over all buildings, and an average RMSE value is obtained

(average over the 100 cases of all six buildings for a given n_{ASI} combination) for each statistical model. The results are plotted in Figure 4.11 to Figure 4.16, with each figure presenting a particular EDP. Furthermore, two group sizes n are considered which correspond to 30, shown in Figure 4.11, Figure 4.12 and Figure 4.13, and 20, shown in Figure 4.14, Figure 4.15 and Figure 4.16. Unlike the results presented for R^2 , the RMSE results are presented for different GM group sizes in order to show a wider variety of situations.

As referred before, since the RMSE values depend on the sample size, only a comparative assessment of the different models can be performed when the different n_{ASI} combinations lead to different sample sizes. As a general trend, it can be seen that the majority of the RMSE values increase with the intensity, which indicates a poorer fitting (it is reminded that lower RMSE values indicate better fitting).

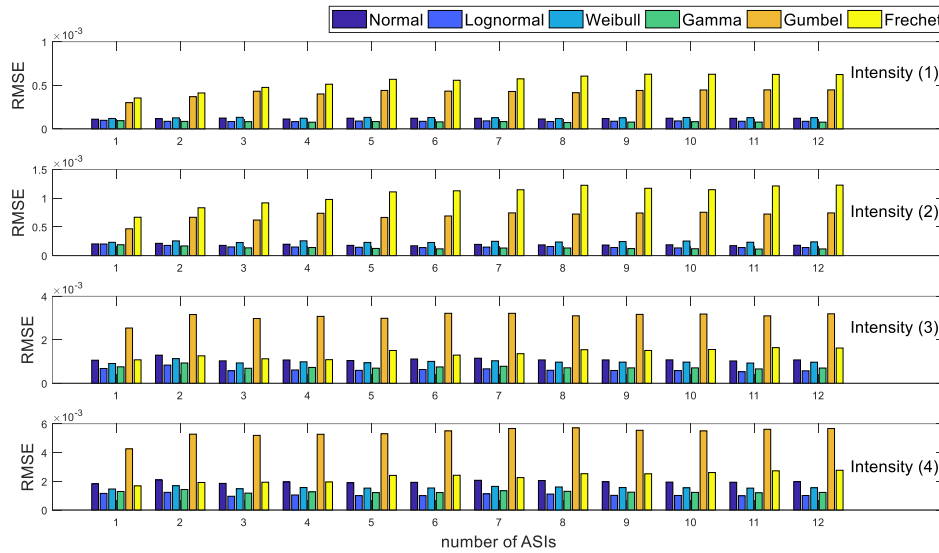


Figure 4.11 Mean RMSE of all buildings obtained from ISD Q-Q plots for ASIs 1 to 12 and for $n = 30$.

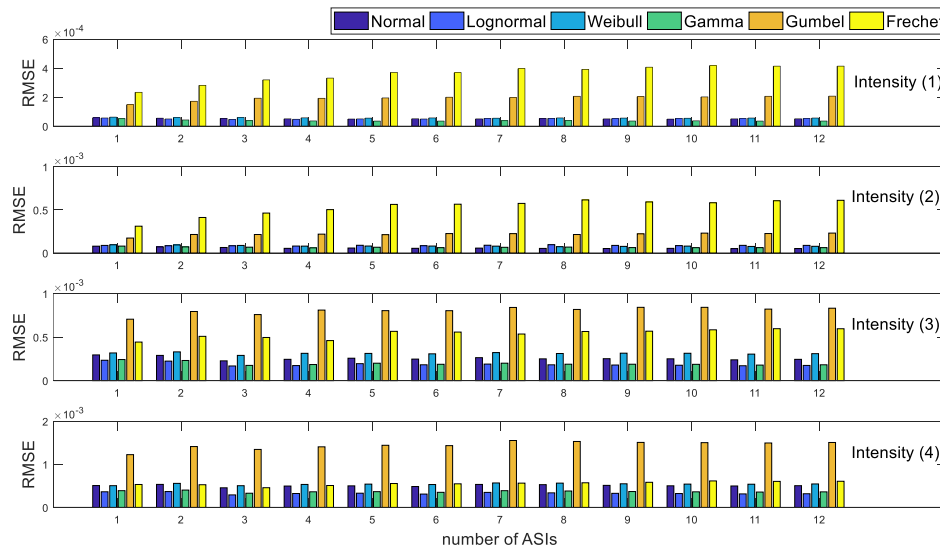


Figure 4.12 Mean RMSE of all buildings obtained from RD Q-Q plots for ASIs 1 to 12 and for $n = 30$.

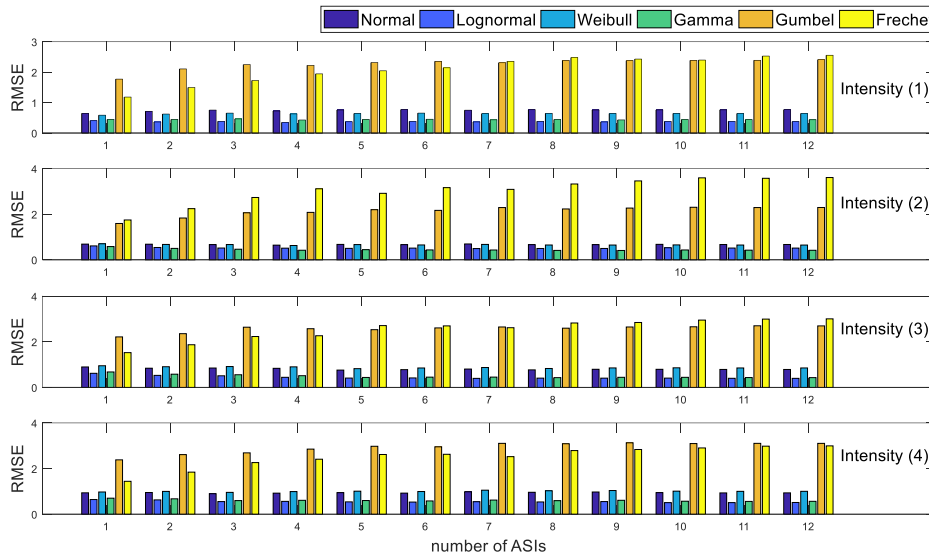


Figure 4.13 Mean RMSE of all buildings obtained from PFA Q-Q plots for ASIs 1 to 12 and for $n = 30$.

The most noticeable trend that can be observed in the majority of the figures, which is in agreement with the results found for R^2 , is that the performance of the Gumbel and the Fréchet models is always worse than that of the rest of the models. More specifically, it can be seen that for low intensities, i.e. when the structures exhibit mostly elastic structural behaviour, and for all EDPs, the Fréchet model exhibits the largest RMSE

values for both GM group sizes. On the contrary, for higher intensities, the Gumbel model is the one presenting the largest RMSE values. These observations are generally true for the three EDPs that are examined, as well as for both GM group sizes presented. A few exceptions are nonetheless found, such as for the PFA and for specific n_ASI combinations, shown in Figure 4.13 and Figure 4.16, where the order of the two models is reversed.

The rest of the distributions, i.e. the normal, the lognormal, the Weibull and the Gamma, exhibit much lower mean RMSE values, hence a better performance. Among these, the lognormal model appears to have the best performance, followed closely by the Gamma model. The Weibull model presents adequate behaviour as well, while the last is the normal model, which has the largest RMSE values (among the four) for most of the n_ASI combinations, as well as for most intensities. Exceptions to this last observation comprise the RD RMSE values for intensity (2) (see Figure 4.12 and Figure 4.15), where all four models show similar behaviour and the normal distribution exhibits the lowest RMSE values.

Overall, the trends observed for the RMSE values are consistent with the conclusions established from the analysis of the R^2 results. Both measures indicate that the lognormal model appears to be the most appropriate to fit the data. Moreover, the Gamma model appears to have a similarly adequate performance, while that of the Weibull model seems to be slightly worse. The analysis of demand samples involving more than 1 ASI does not seem to affect significantly the performance of a model selected to represent the demand distribution of the EDPs for any intensity level, with the exception of the Fréchet model. More particularly, the Fréchet model exhibits low RMSE values, comparable to those of the Weibull and normal models, for the RD and ISD distributions, for the largest intensity and when a small number of ASIs are used (e.g. see Figure 4.11, Figure 4.12, Figure 4.14 and Figure 4.15). For higher number of ASIs, however, the performance of the model deteriorates when compared to that of the rest of the models.

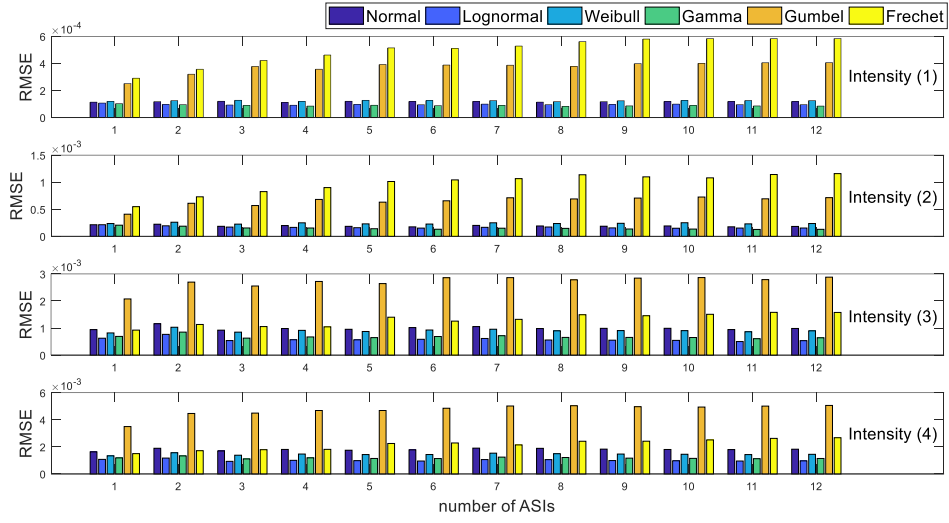


Figure 4.14 Mean RMSE of all buildings obtained from ISD Q-Q plots for ASIs 1 to 12 and for $n = 20$.

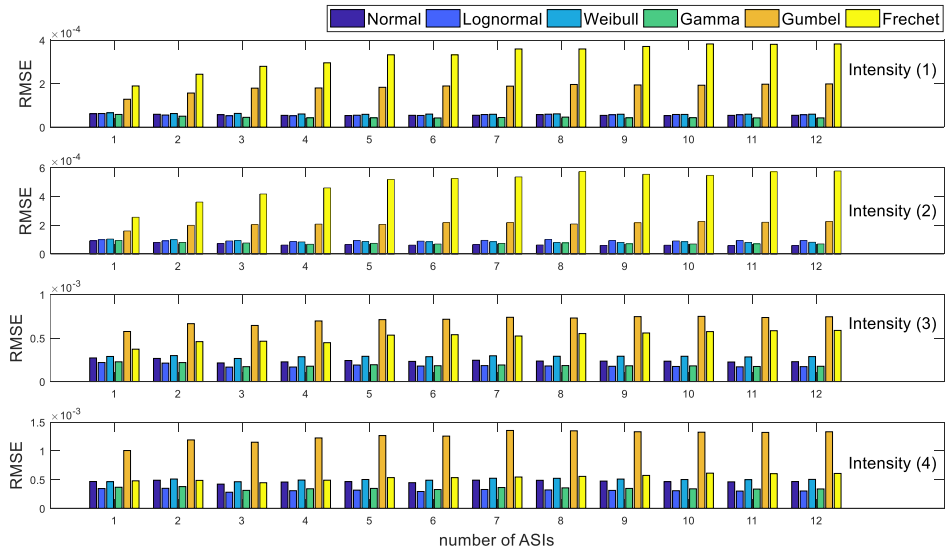


Figure 4.15 Mean RMSE of all buildings obtained from RD Q-Q plots for ASIs 1 to 12 and for $n = 20$.

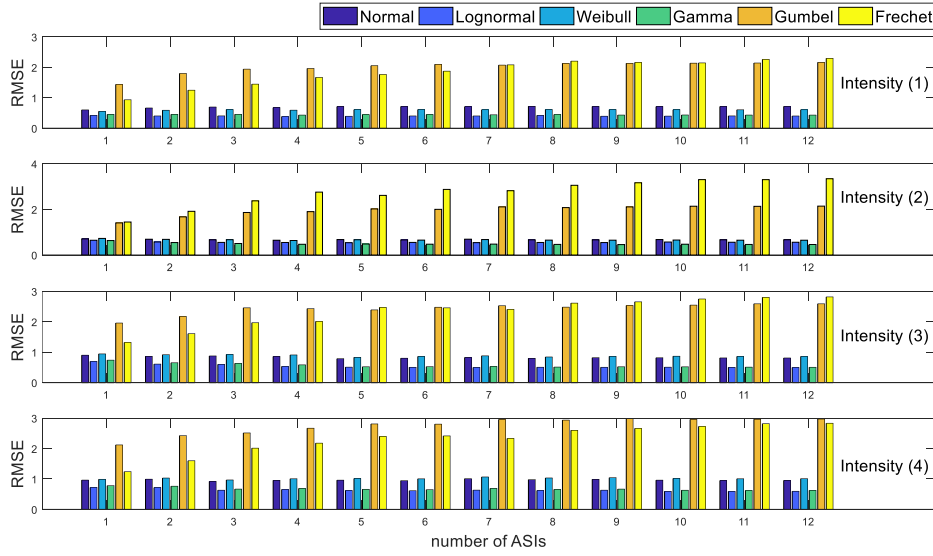


Figure 4.16 Mean RMSE of all buildings obtained from PFA Q-Q plots for ASIs 1 to 12 and for $n = 20$.

As far as the variability of the RMSE is concerned, Figure 4.17 and Figure 4.18 show the cov of the RMSE values for the ISD obtained from all buildings, for two GM group sizes and for 1 to 12 ASIs, following the presentation format adopted for the mean RMSE values. In particular, Figure 4.17 shows the results for $n = 30$ and for the 4 intensities, while Figure 4.18 shows the results for $n = 20$ and for the 4 intensities.

The cov values obtained for the RMSE results are generally larger than those obtained for the R^2 results, due to the different formulation of the two measures. Nonetheless, as expected and in line with the results obtained for the R^2 variability, larger cov values are found for samples with lower GM group sizes. Although the relative behaviour of the mean values did not seem to vary when increasing the ASI, the cov shows slightly different trends. In this case, a more spiky format is observed with the increase of the number of ASIs, indicating the differences in the relative behaviour of the alternative models. While all models present similar covs when only 1 ASI is used, the lognormal and the Gamma model exhibit higher deviations when increasing the ASI, especially for the higher intensities. Covs in the range of 25%-30% are found for the lognormal and Gamma models for the larger GM group size (Figure 4.17), while for the smaller GM group size the covs may reach 40% (Figure 4.18). Although lower cov values are observed for the Gumbel and Fréchet models, these are usually associated with larger mean RMSE values. Finally, the Weibull model presents relatively low mean RMSE

values for the majority of the cases studied (EDPs and n_{ASI} combinations) along with moderate cov values.

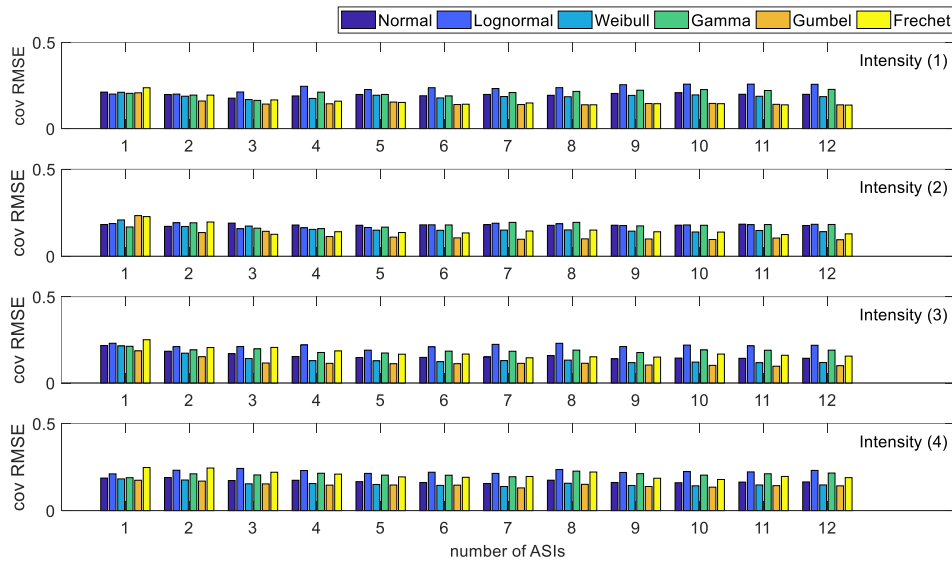


Figure 4.17 cov RMSE of all buildings obtained from ISD Q-Q plots for ASIs 1 to 12 and for $n = 30$.

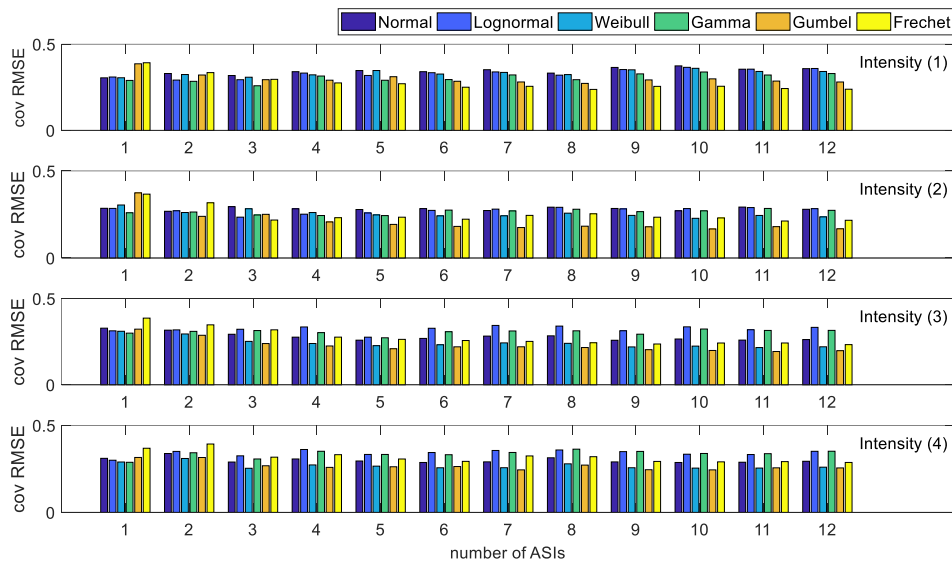


Figure 4.18 cov RMSE of all buildings obtained from ISD Q-Q plots for ASIs 1 to 12 and for $n = 20$.

4.3.4 Additional remarks

Taking into account the results of both measures of linear fit, i.e. the mean of the R^2 and RMSE values, for all three EDPs examined, as well as their variability expressed by the cov, the following can be established:

- In the majority of the cases, the distribution model that best fits the data does not change substantially by increasing the number of ASIs;
- The performance of a given distribution model seems to deteriorate with the increase of the seismic intensity, i.e. with the increase of the inelastic behaviour of the structure;
- The GM group size does not affect the mean of the R^2 and RMSE values, but it affects their variability, with lower cov values usually found when larger sizes are used;
- The lognormal distribution appears to be the model whose performance is better and more consistent for most of the cases examined, namely for all three EDPs, their various intensities and the several n_{ASI} combinations;
- The Gamma distribution is ranked as the second most adequate model, which exhibits marginally lower performance when compared to that of the lognormal distribution;
- The performance of the Weibull model seems to be inferior to both the lognormal and Gamma distributions. Nonetheless, it has quite stable mean R^2 and RMSE values and relatively low cov values, even when a low number of ground motions is used;
- The Normal distribution presented significantly inferior performance when compared to that of the three aforementioned models, with respect to both the mean and the variability of R^2 and RMSE;
- The Gumbel and the Fréchet models presented the worst performance for most of the examined cases. In addition, their suitability seems to interchange from case to case. Nevertheless, the Fréchet model exhibits a good performance in some cases and that is even superior to that of the lognormal model.

4.4 Examining the distribution type using goodness-of-fit tests

4.4.1 Statistical tests

The adequacy of the distribution models that presented the best fittings based on the results shown in Section 4.3 is further explored using goodness-of-fit tests. Goodness-of-fit tests are statistical tests that are used to show how well a probabilistic model fits a set of observations. These tests are based on a null hypothesis, i.e. that the data are extracted from a certain distribution type, and provide a measure to prove if the null hypothesis is rejected in favour of an alternative hypothesis or if the null hypothesis cannot be rejected, at a preselected significance level. Most tests are developed to have power against specific distributions (D'Agostino and Stephens 1986) and thus different tests are required to test different types of distributions.

The distribution models examined herein are the lognormal, the Gamma and the Weibull distributions, given their better performance indicated by the Q-Q plots and the measures of their linear regression. Additionally, the Fréchet distribution is also examined since it presented the interesting property of showing good results in specific cases where none of the alternative distributions did. In terms of tests, the Shapiro Wilk test (Royston 1995) (S-W) is used to test for the lognormal distribution, by checking for the normality of the logarithm of the data. Existing literature (e.g. see Mendes and Pala 2003; Keskin 2006; Romão et al. 2010; Razali and Wah 2011) has ranked the S-W among the most powerful tests for normality for different types of alternative distributions as well as for various sample sizes. For the distributions belonging to the extreme value family, i.e. the Weibull and the Fréchet, the test created by Tiku and Singh (1981) (T-S) is used, according to which a two sided goodness-of-fit test is proposed to test for the two parameter Weibull distribution. This test has been ranked among the most powerful for detecting departures from the Weibull distribution according to a recent study performed by Krit et al. (2019). To test for the Fréchet distribution, the inverse of the data is used and the Tiku and Singh (1981) test is then applied (the following notation will be used to refer to the T-S test using the inverse of the data: $T-S(x^{-1})$). The statistical test developed by Betsch and Ebner (2019) (B-E) is employed to test for the Gamma distribution. Despite the proven strengths and weaknesses of the selected tests presented in the referred studies, their power may also depend on the range of the distribution parameters. For this reason, a limited preliminary simulation study is performed using a Monte Carlo procedure in order to

evaluate the power of the three tests against the four selected distribution models using parameters close to those expected to be found for the samples of EDPs studied herein.

4.4.2 Power of the tests

The power of each test is examined for the four alternative distributions, i.e. the lognormal (LN), the Gamma (G), the Weibull (W) and the Fréchet (Fr), with parameters similar to those expected for the ISD and the PFA samples of the present study. The probability density function plots (PDF), as well as the cumulative distribution function plots (CDF) of the examined cases are shown in Figure 4.19 and Figure 4.20, respectively. The power analysis was performed by simulating 100000 samples of each distribution with three sample sizes (25, 50 and 100). Each test was applied to all the samples of each distribution to determine the corresponding rejection rates for each type of data. All the tests were applied considering a significance level α of 5%, which means that when the test developed for the X distribution is applied to multiple samples coming from the X distribution, the rejection rate corresponds to the Type I error and should be 5%. On the contrary, when the test developed for the X distribution is applied to multiple samples coming from any other type of distribution, the ideal rejection rate would be 100%. The rejection rates obtained for each test and for the three sample sizes are presented in Table 4.1 for the distributions simulating ISD data and in Table 4.2 for the distributions simulating PFA data. The parameters of each distribution are shown in parenthesis and correspond to the mean and the standard deviation of the log of the data for the lognormal distribution, and to the shape and scale for the rest of the distributions.

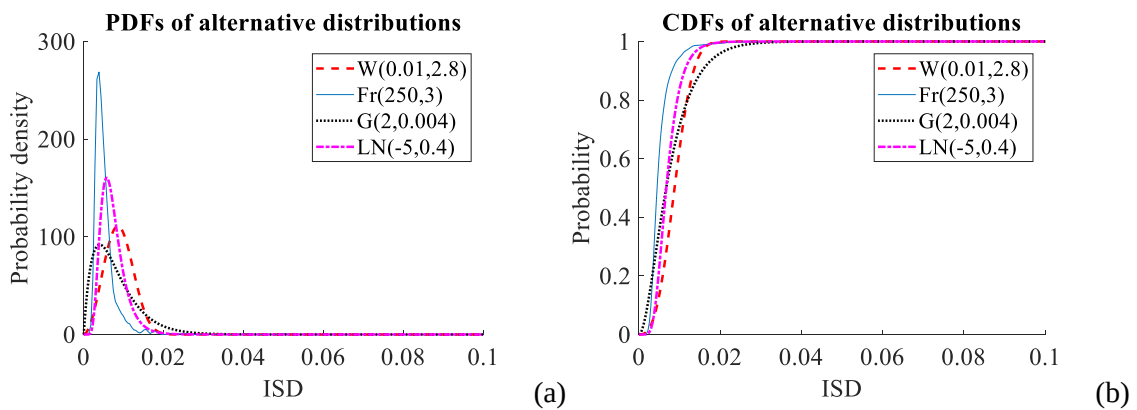


Figure 4.19 PDF (a) and CDF (b) plots for the alternative distributions. ISD case.

The results shown in both Table 4.1 and Table 4.2 indicate that all three tests have similar Type I errors, which are very close to the selected significance level (5%), regardless of the sample size and the parameter range. On the contrary, when tests are applied to

alternative distributions, the rejection rates vary, depending on the test, the distribution and its parameters, as well as on the sample size. As expected, in all cases the rejection rates increase with the sample size. Large sample sizes increase the tests' sensitivity, thus leading to inherently higher rejection rates. On the other hand, when using small sample sizes the test's capacity to identify a lack of departure from the null hypothesis decreases, hence leading to lower rejection rates.

As far as the ISD is concerned (Table 4.1), the S-W test shows low power ($\sim 23\%$) against the Gamma model for the smallest sample size, while its power increases to 43% for a sample size of 50 and to more than 70% for sample size of 100. The B-E test on the other hand, presents very low power against the lognormal model for all samples sizes (lower than 30%). It is worth pointing out that the rejection rate of the B-E against the lognormal for a sample size of 25 is very close to α , which indicates that the B-E test is not powerful against the lognormal distribution for the parameters under consideration. Regarding the rejection rates of the S-W against the Weibull model, they range from low ($< 40\%$) for small sample sizes to high ($> 90\%$) for large sample sizes. On the contrary, the B-E shows much lower power against the Weibull, lower than 60% even for the largest sample size examined. Both the S-W and the B-E show moderate-to-high power against the Fréchet model, with the B-E being more powerful. Regarding the T-S test, it shows low power against the Gamma model, especially for small sample sizes, while its power against the lognormal model can be characterised as moderate-to-high, depending on the sample size.

Table 4.1 Empirical rejection rates for $\alpha = 0.05$. ISD case.

Alternative	size	S-W	B-E	T-S	T-S(x^{-1})
LN(-5,0.4)	25	5.08	6.59	42.48	42.43
	50	5.03	12.14	74.33	74.27
	100	4.93	24.76	96.56	96.53
G(2,0.004)	25	22.84	5.75	9.11	63.18
	50	42.52	5.24	12.95	92.06
	100	71.99	5.09	21.13	99.81
W(0.01,2.8)	25	38.89	18.17	6.23	90.02
	50	68.98	32.70	5.99	99.69
	100	94.20	56.72	5.77	100.00
Fr(250,3)	25	39.03	55.28	90.08	6.28
	50	68.86	88.42	99.70	5.94
	100	94.18	99.65	100.00	5.83

Regarding the power of the tests against distributions with parameters compatible with the PFA data, the results are presented in Table 4.2, while the PDFs and CDFs of the corresponding distributions are shown in Figure 4.20. The rejection rates of S-W against the Gamma distribution and of B-E against the lognormal distribution are much lower in this case, suggesting a lower performance of these tests for the considered parameters. The power of S-W against the Weibull is moderate to high in this case as well, while rates similar to those found for the ISD results are obtained for the B-E test. On the other hand, T-S is seen to exhibit similar or higher rejection rates against the alternative distributions when compared to those obtained for the ISD results, thus indicating higher performance.

In the overall, these results allow concluding two aspects about the performance of the tests. The first is that, on average, the target value of Type I error is met without significant discrepancies. The second is that the ability of the tests to detect departures from the null hypothesis is not uniform across the tests, and is strongly dependent on the shape of the empirical data (irrespective from the distribution type from which it was generated) and the sample size. As such, when a given test fails to reject the null hypothesis for a given sample, it might be due to the small sample size that is involved (e.g. lower than 50) but it might also be due to the fact that the data does not present a strong departure from the distribution underlying the null hypothesis and could in fact be modelled using that distribution. Further results illustrating this issue are presented in the following using real EDP samples.

Table 4.2 Empirical rejection rates for $\alpha = 0.05$. PFA case

Alternative	size	S-W	B-E	T-S	T-S(x^{-1})
LN(1.9,0.35)	25	5.06	3.83	42.50	42.47
	50	4.99	7.99	74.20	74.25
	100	4.97	17.29	96.53	96.49
G(9,0.7)	25	8.74	2.74	22.91	81.51
	50	13.19	3.49	43.07	98.60
	100	22.00	4.10	73.40	100.00
W(10,3)	25	39.01	17.74	6.19	90.06
	50	68.81	34.12	6.03	99.69
	100	94.23	59.57	5.87	100.00
Fr(0.15,3.4)	25	38.90	47.85	90.02	6.27
	50	68.87	84.37	99.70	6.01
	100	94.18	99.46	100.00	5.86

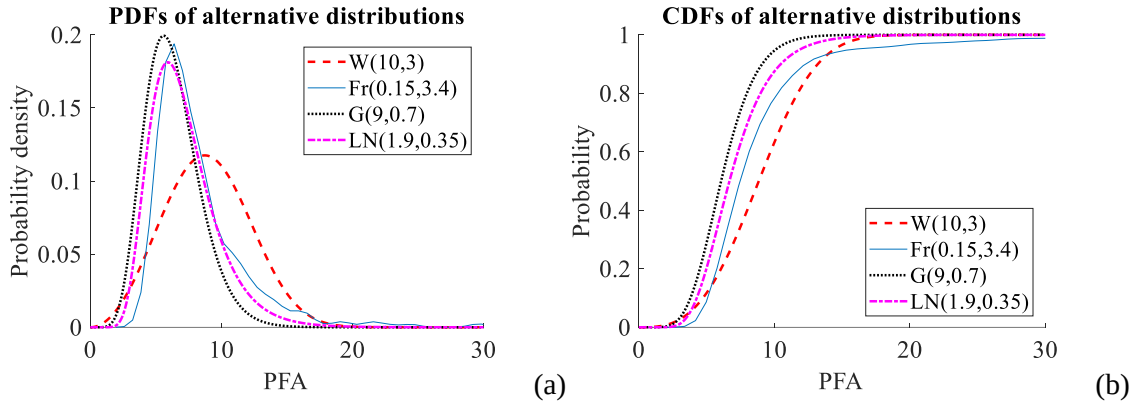


Figure 4.20 PDF (a) and CDF (b) plots for the alternative distributions. PFA case.

4.4.3 Rejection rates obtained for real EDP data

The selected tests were applied to the 100 EDP samples (ISD, PFA, RD) of all different combinations of number of ASIs and GM group size (n_{ASI} combinations) of the six buildings considering a 5% significance level. For each EDP sample, each test returned a value of 1 if the distribution hypothesis that it was tested against was rejected. Otherwise, the test returned a value of 0, signifying that the distribution hypothesis could not be rejected. The outcomes of all 100 groups of all six buildings for a given n_{ASI} combination were next added to obtain a global rejection rate. The lower the rejection rate, the better the performance of a given distribution model.

Due to the large number of results, only selected cases are presented herein for discussion. Additionally, given the inherent dependence of the results on the sample size, the comparison is performed between n_{ASI} combinations that result in the same sample size m . In light of this, the following combinations will be compared hereafter:

- $m = 20$: {10_2, 20_1},
- $m = 50$: {10_5, 25_2}
- $m = 70$: {10_7, 35_2}

Furthermore, results corresponding to different intensities are presented and discussed separately, since some of the trends appear to change with the level of structural inelasticity. Finally, results are presented for the three different EDPs, i.e. ISD, PFA and RD. The following figures (Figure 4.21, Figure 4.23 and Figure 4.24) show the rejection rates obtained for the four alternative distribution models for the ISD, RD and PFA samples, respectively, for the preselected n_{ASI} combinations. In particular, rows 1, 2 and 3 of each figure show the results for the sample sizes of 20, 50 and 70, respectively,

while columns 1 and 2 show the results for intensities 2 and 4, respectively. It is reminded that the intensity levels are those presented in Table 3.2.

The most noticeable feature in Figure 4.21 is that, as expected, the rejection rates increase with the sample size. Regarding the results for the lower intensity (left column), the Gamma and the lognormal models exhibit lower rejection rates for all sizes m and n_ASI combinations. It is interesting to point out that these rejection rates are below 10% when using up to 2 ASIs for all GM group sizes (i.e. for low number of ASIs) and rise up to ~30% when increasing the number of ASIs.

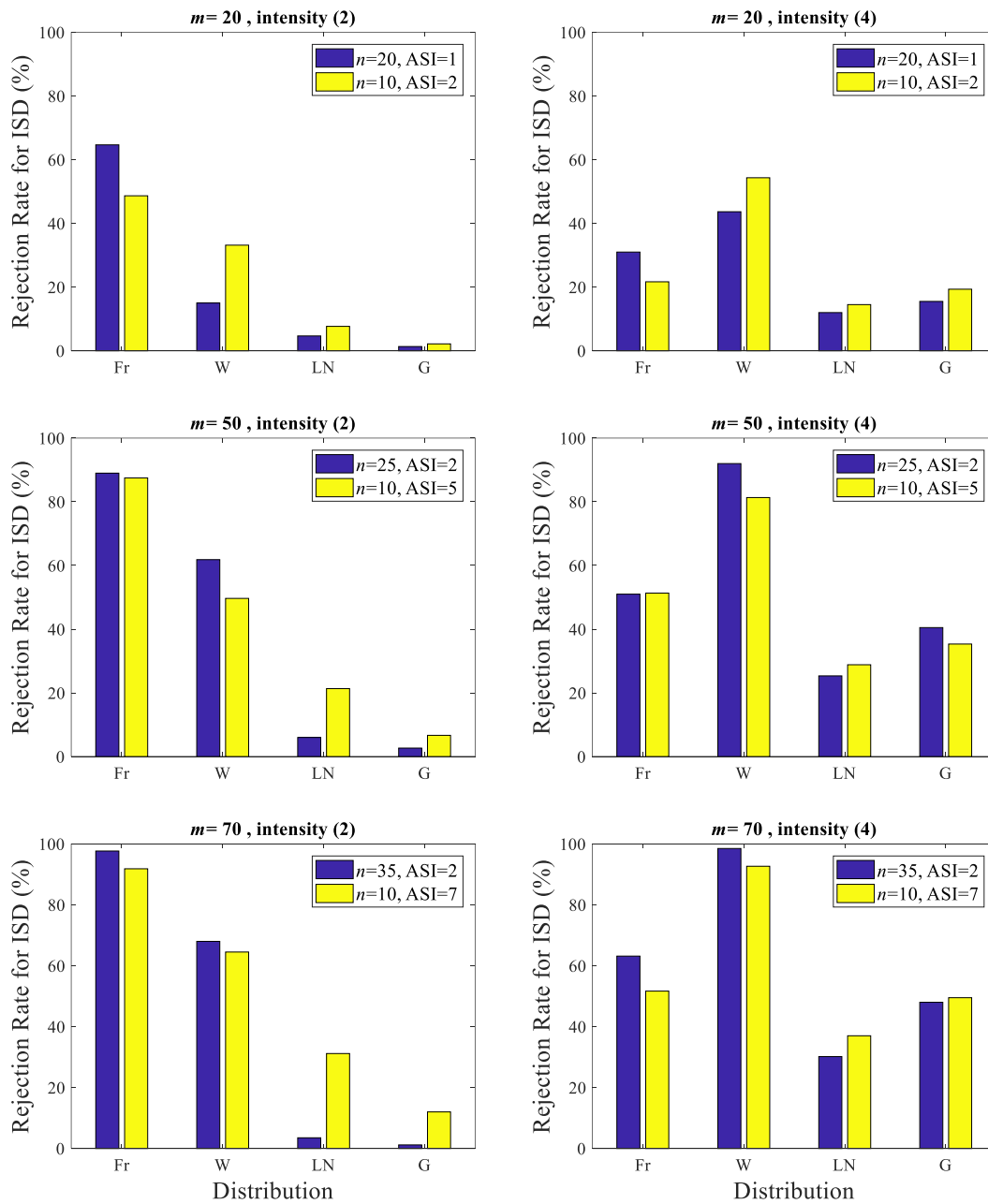


Figure 4.21 Rejection rates of selected ISD samples

It is also worth noting that, for this intensity, the lognormal rejection rates are systematically higher than those of the corresponding Gamma distribution. On the contrary, the Weibull distribution model has rejection rates higher than 50% in many cases, but it still outperforms the Fréchet model, which appears to be the most inappropriate model, given it exhibits the highest rejection rates. Furthermore, the effect of the ASI on the fittings tested for these two distributions does not show any clear trends, while for the larger sample size the rejection rates are similar regardless of the n_ASI combination.

As far as the higher intensity level is concerned, one of the most apparent features observed in the results is that the rejection rates are higher for all distribution models when compared to those obtained for the lower intensity, except for the Fréchet model. Contrary to the almost 95% rejection rates observed for the lowest intensity when using the Fréchet model, the rejection rates for the highest intensity drop to less than 50% (in most of the cases), a value similar to those found for the rest of the distributions due to their increase. The Gamma and the lognormal models still present lower rejection rates, which are in the range of 10-20% for the small sample sizes and reach up to 50% for a sample size of 70 (the largest size presented in Figure 4.21). It is worth mentioning that, for this intensity, the lowest rejection rates are obtained for the lognormal distribution, while the Weibull model appears to be the most inappropriate one. Finally, although the differences are small, the performance of the lognormal model seems to be better when, for a given sample size, the number of ASIs that are involved is lower, while the opposite is observed for the Fréchet model. On the contrary, results obtained for the Gamma and Weibull models do not show any constant trends in this respect.

To provide further details about these rejection rates and their trends, a few selected goodness-of-fit plots are also presented herein, illustrating cases that include all the possible combinations of reject/cannot reject outcomes of the S-W and B-E tests. In particular, Figure 4.22 shows the CDF curves of the lognormal and the Gamma models fitted to four ISD samples of size 50 (25_2 combinations of particular buildings and intensities were used as indicated in Figure 4.22) and the tests' results for a 5% significance level. The discussion concerns the fit of ISD samples using the Gamma and the lognormal distribution models due to their superior performance compared to the rest of the distribution models, and due to the small differences between them in terms of

rejection rates. The goodness-of-fit curves in Figure 4.22 are further accompanied by the Q-Q plots for additional interpretation of the results.

It is observed that, in most of the cases, the lognormal and the Gamma CDF curves are visually similar, thus explaining the small differences between the rejection rates obtained by the two tests. Figure 4.22(a) shows a case where none of the tests rejected the null hypothesis and for which this result is visually consistent with both the CDF and the Q-Q plots obtained for the two models. Focusing on the next two cases, Figure 4.22(b) and (c), it is observed that, although the two CDFs are similar, the tests returned different outcomes. The latter suggests that the two tests might be sensitive against different parts of the distribution (i.e. the tails – lower or upper – or the middle part). More particularly, the case of Figure 4.22(b) exhibits larger differences between the fitting and the empirical data in the region of the upper tail, while the case of Figure 4.22(c) exhibits differences between the fitting and the empirical data that, although smaller than those of the previous case, are spread from the lower to the upper tail of the distribution. Finally, Figure 4.22(d) presents a case where both tests rejected the null hypothesis, which is consistent with a visual observation that indicates the presence of large departures across the entire range of the data.

The rejection rates for the RD are presented in Figure 4.23 following the presentation format used for the ISD rejection rates. The trends observed for the RD are very similar to those discussed for the ISD results. It can be seen from Figure 4.23 that the lognormal and the Gamma models present the lower rejection rates in most of the cases. Furthermore, the Fréchet model shows the higher rejection rate for the lowest intensity, while the Weibull model appears to be the least appropriate for the highest intensity. Another important observation is that, except for the clear differences observed for the lognormal and the Gamma models for $m = 50$ and $m = 70$ for the lowest intensity, no significant effect can be seen with respect to the number of ASIs that is used, since the rejection rates for the different n_{ASI} combinations of a given m size are similar for each model.

The rejection rates for the PFA of the four distribution models are presented in Figure 4.24. Trends for PFA are quite similar to those found for ISD, with lower rejection rates observed for the lognormal and the Gamma models and higher rejection rates observed for the Weibull and the Fréchet models.

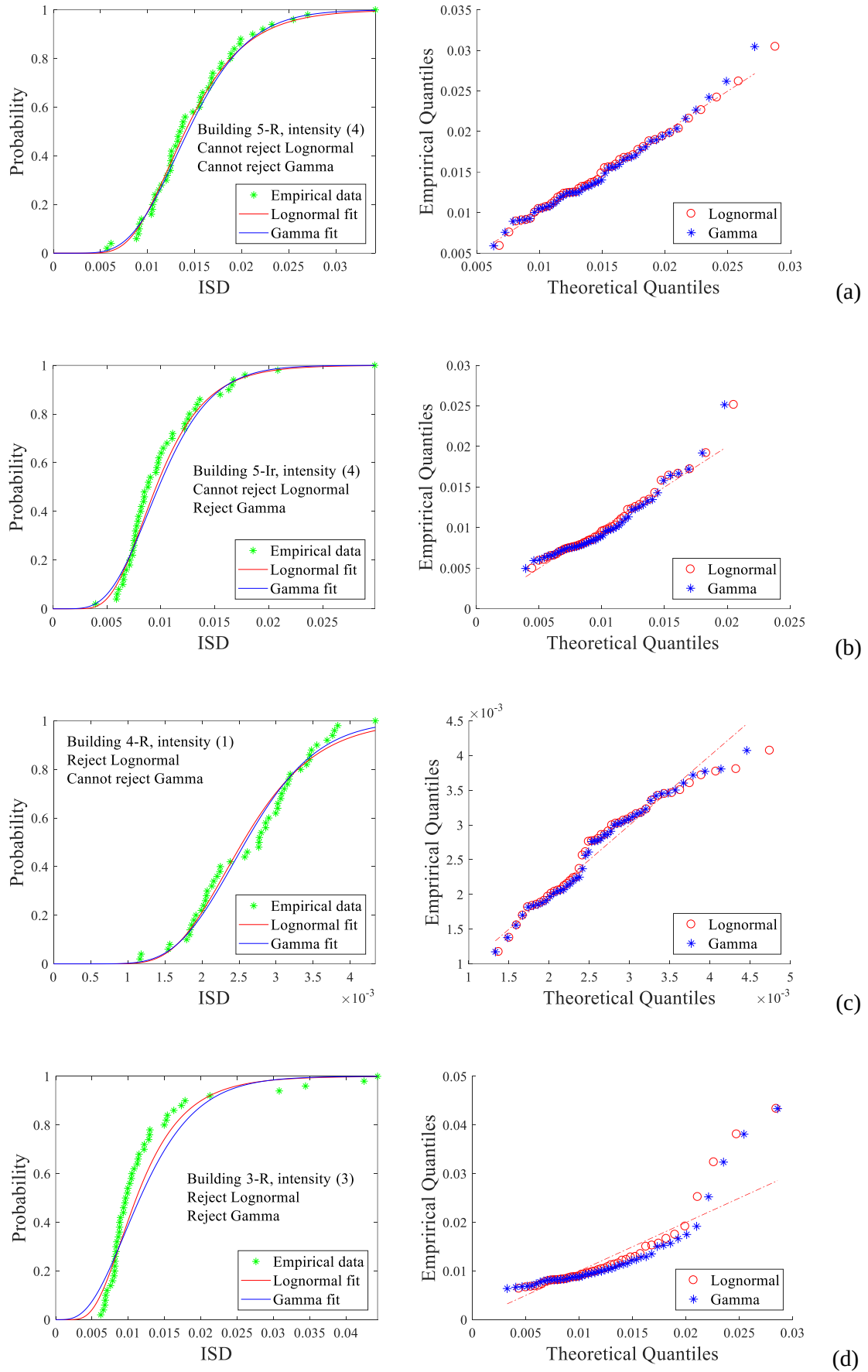


Figure 4.22 Goodness-of-fit plots and Q-Q plots for different tests outcomes

The interesting trend seen for the Weibull and the Fréchet models, where they reverse the order of their rejection rates from one intensity to the other, is also found for the PFA results. Still, the PFA results highlight an additional feature which is the much higher rejection rate of both the Gamma and lognormal models when a higher number of ASIs is used for a given sample size m and for both intensities. This is in contrast with the ISD results for which similar rejection rates were obtained for the higher intensity.

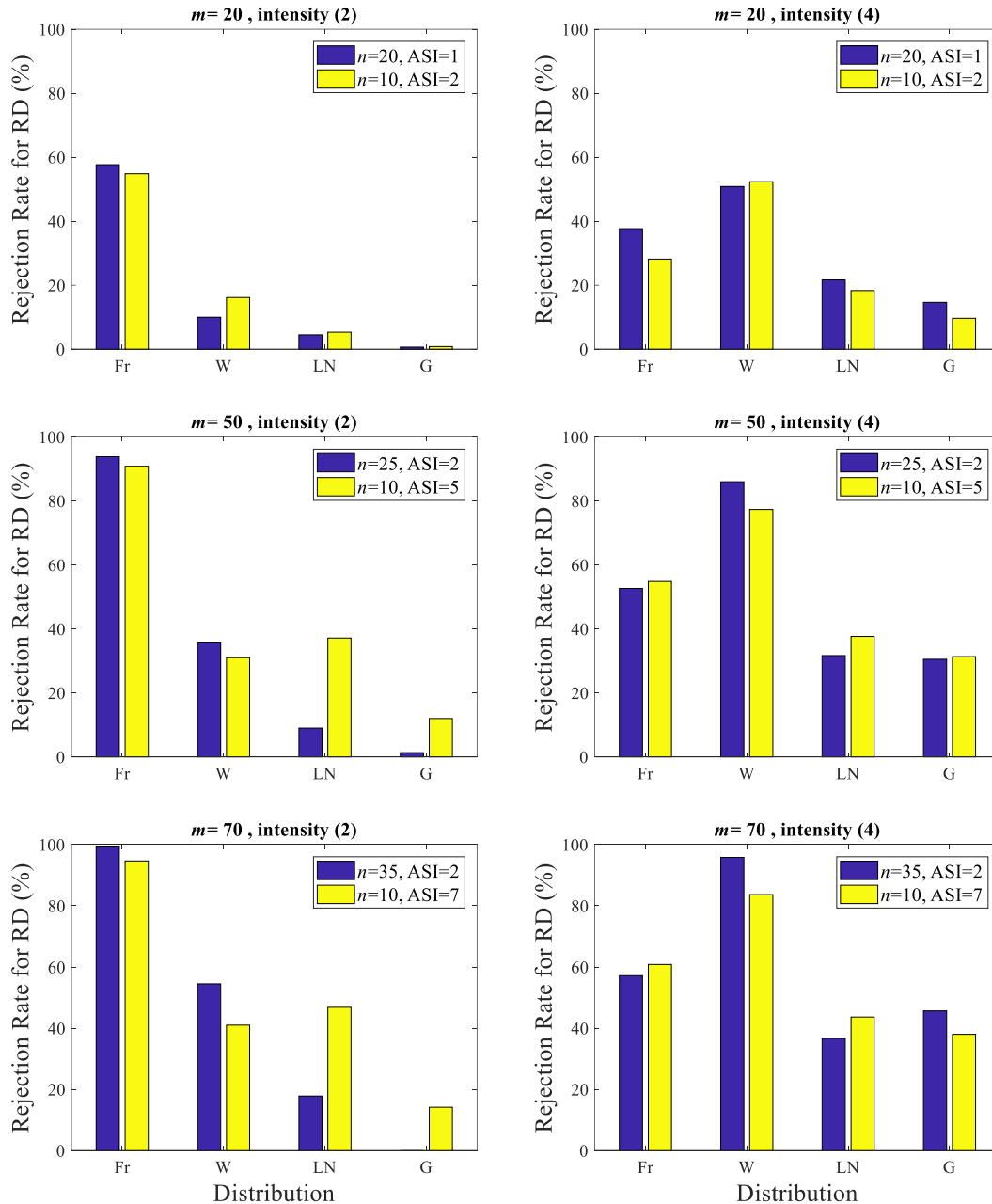


Figure 4.23 Rejection rates of RD selected samples

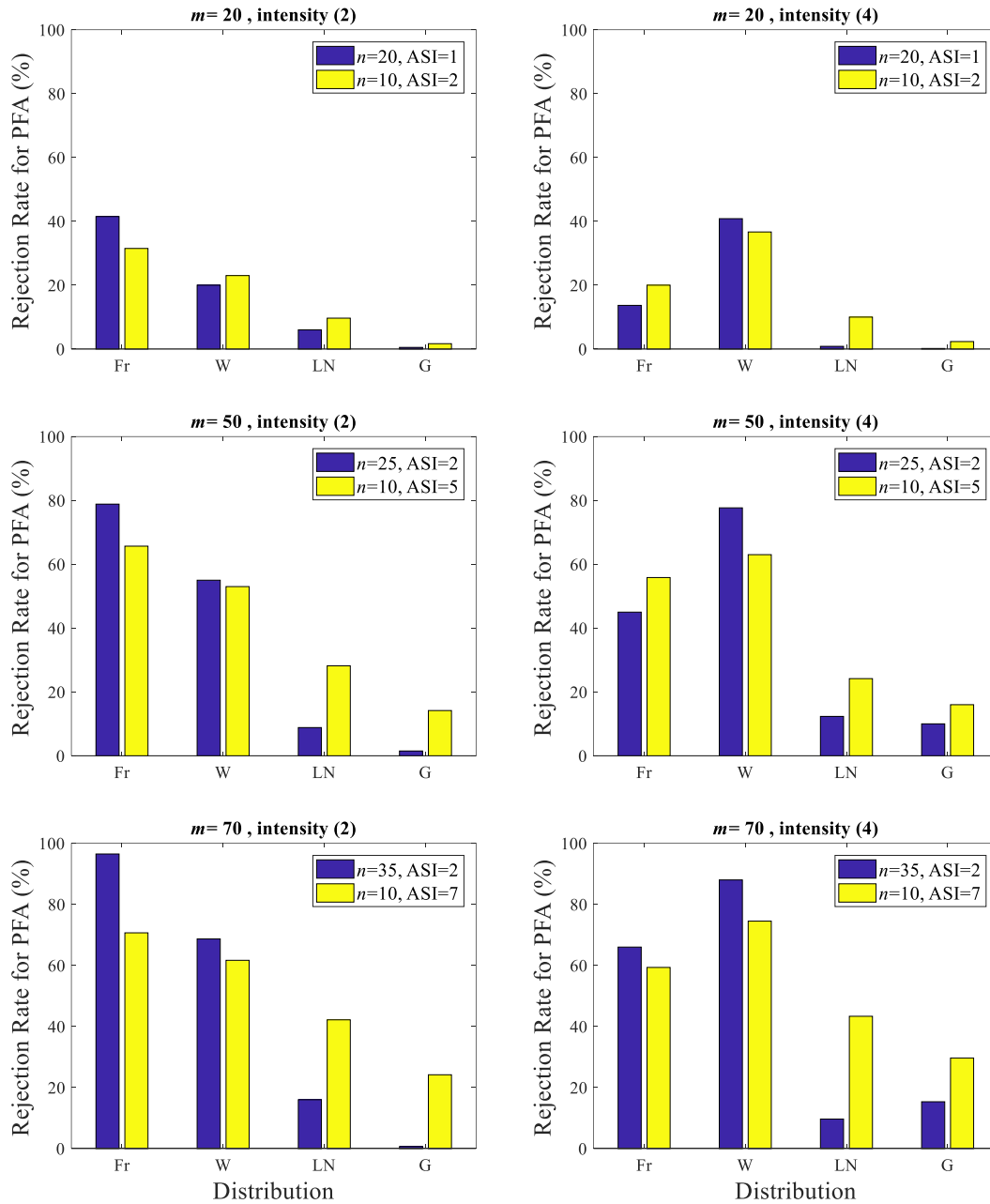


Figure 4.24 Rejection rates of PFA samples

4.4.4 Effect of the ASI

An additional effect that requires further analysis addresses the different rejection rates found for samples of a given EDP involving different n -ASI combinations but with the same size m , thus involving a different number of ASIs. The following figure shows the goodness-of-fit plots of the Gamma and the lognormal distributions of four different ISD samples of the same size created using different n -ASI combinations. The data presented in Figure 4.25 correspond to ISD samples of size $m = 60$. Although the fitted curves of

the lognormal and the Gamma model are similar for all cases presented, the shape of the empirical data changes and starts deviating from the curves when increasing the number of ASIs, which is expected to lead to higher rejection rates of the tests.

The effect of the ASI is also explored in the PFA and RD results. Figure 4.26 and Figure 4.27 show the fitted lognormal and Gamma CDF plots for PFA and RD, respectively, obtained using different combinations of n_{ASI} with $m = 60$. Although the two fitted curves are similar for each n_{ASI} case, the shape of the empirical data also changes when increasing the number of ASIs and appears to deviate from the lognormal shape, showing trends similar to those observed for the ISD.

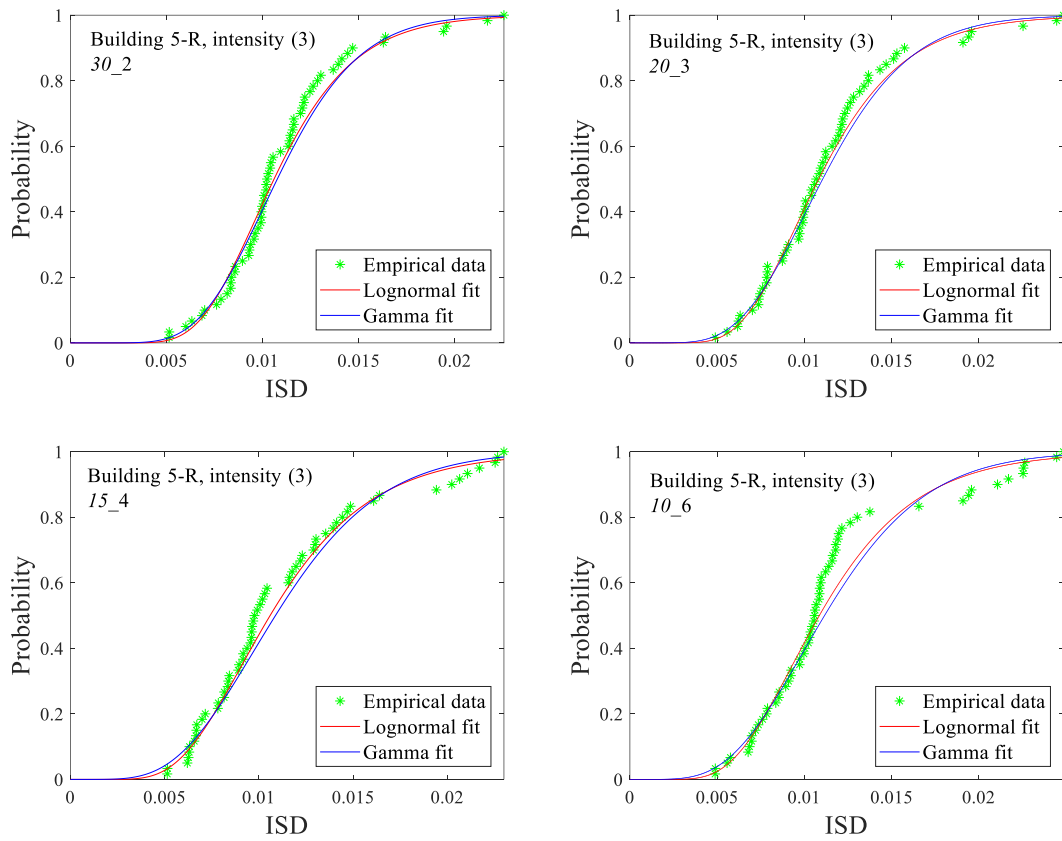
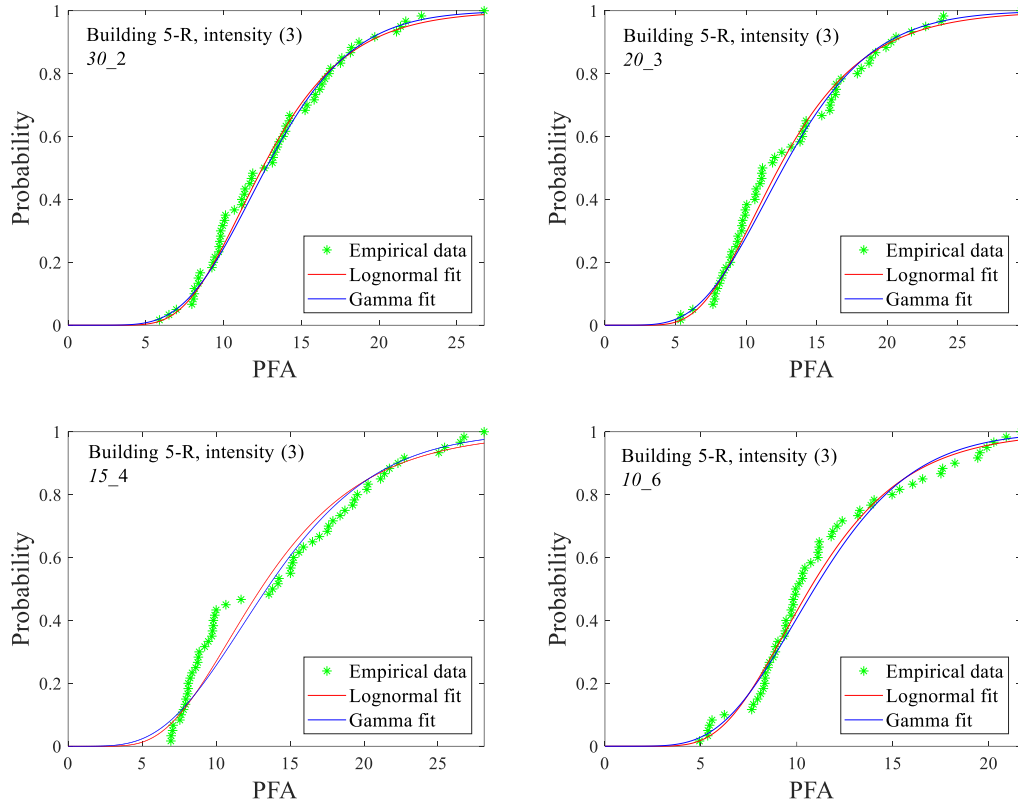
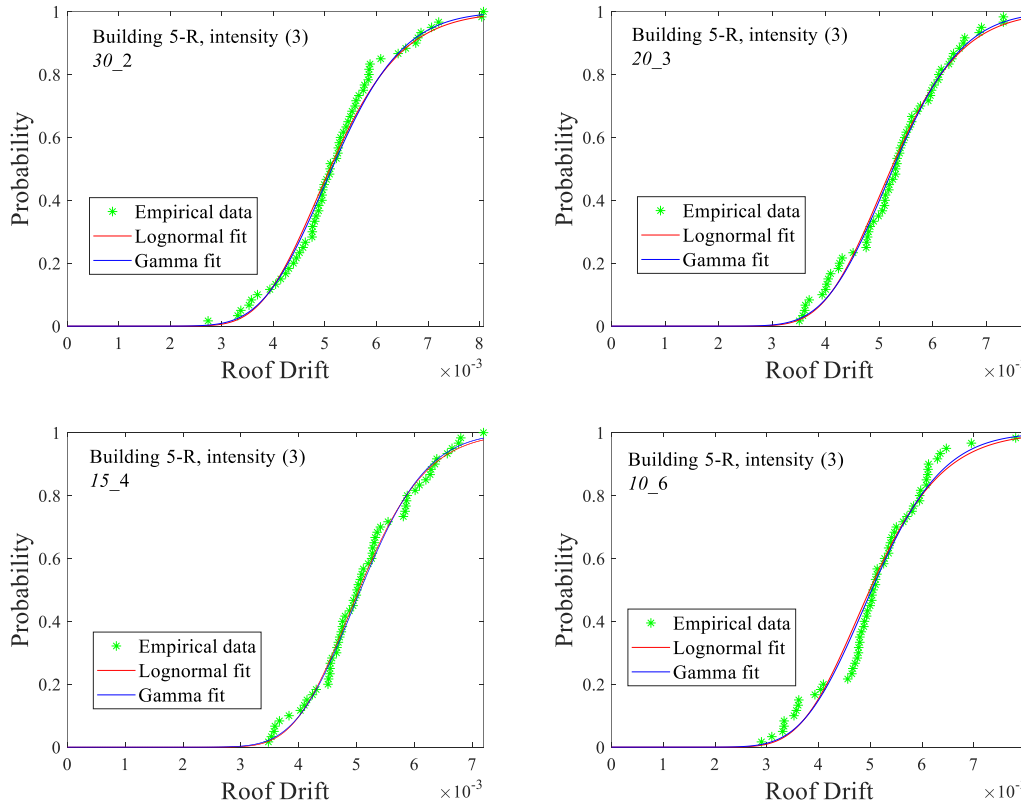


Figure 4.25 ISD goodness-of-fit plots for different n_{ASI} combinations of $m = 60$

Figure 4.26 PFA goodness-of-fit plots for different n_{ASI} combinations of $m = 60$ Figure 4.27 RD goodness-of-fit plots for different n_{ASI} combinations of $m = 60$

To obtain additional insights about the effect of the ASI on the distribution of the empirical data, the kurtosis and the standard deviation (std) of the logarithms of the data are examined. The change of the kurtosis with the increase of the ASI would suggest that the data being added by involving more ASIs is not evenly distributed across the range of the distribution. Particularly, the reduction of the kurtosis would indicate that the new data is located both away from the centre and from the tails (platykurtic distribution), while the increase of the kurtosis would indicate that the data is located both closer to the centre of the distribution and in the tails (leptokurtic distribution). The use of the logarithms of the data provides a simpler way to analyse the data in this case. Since the lognormal model is expected to be adequate in most cases, the $\log(\text{data})$ is expected to be close to symmetric and to follow a normal distribution for which the kurtosis is 3. Both the symmetry of the data and the fixed threshold value facilitate the evaluation of changes in kurtosis due to an increase in the number of ASIs.

To illustrate these issues in the context of the adequacy of the lognormal distribution, Figure 4.28 and Figure 4.29 show the evolution of the std and kurtosis of the logarithms of the ISD data for two cases with the increase of the ASI. The data presented correspond to a particular group of $n = 35$ of the 5-R and the 5-Ir buildings, respectively, and for three number of ASIs, i.e. 1, 6 and 12. The figures show the empirical data as well as the histograms for the three cases of 35_ASI combinations and the corresponding std and kurtosis values. It can be seen that the two cases demonstrate contradictory results. In Figure 4.28 the increase of the ASI leads to a reduction of the kurtosis, while the std experiences a reduction as well. The kurtosis in this case even drops below 3 (the kurtosis of the normal distribution). The more platykurtic shape of the histograms is also apparent with the increase of the ASI in Figure 4.28 (the shape becomes less peaked and more “flat-topped”). On the other hand, Figure 4.29 shows that the increase of the ASI is accompanied by an increase of both the kurtosis and the std. The histograms in this case highlight the thicker tails and the peakedness of the data when more ASIs are involved. It is worth mentioning that, in both cases, differences between the 35_6 and the 35_12 combinations are much smaller than those found when comparing with the 35_1 combination.

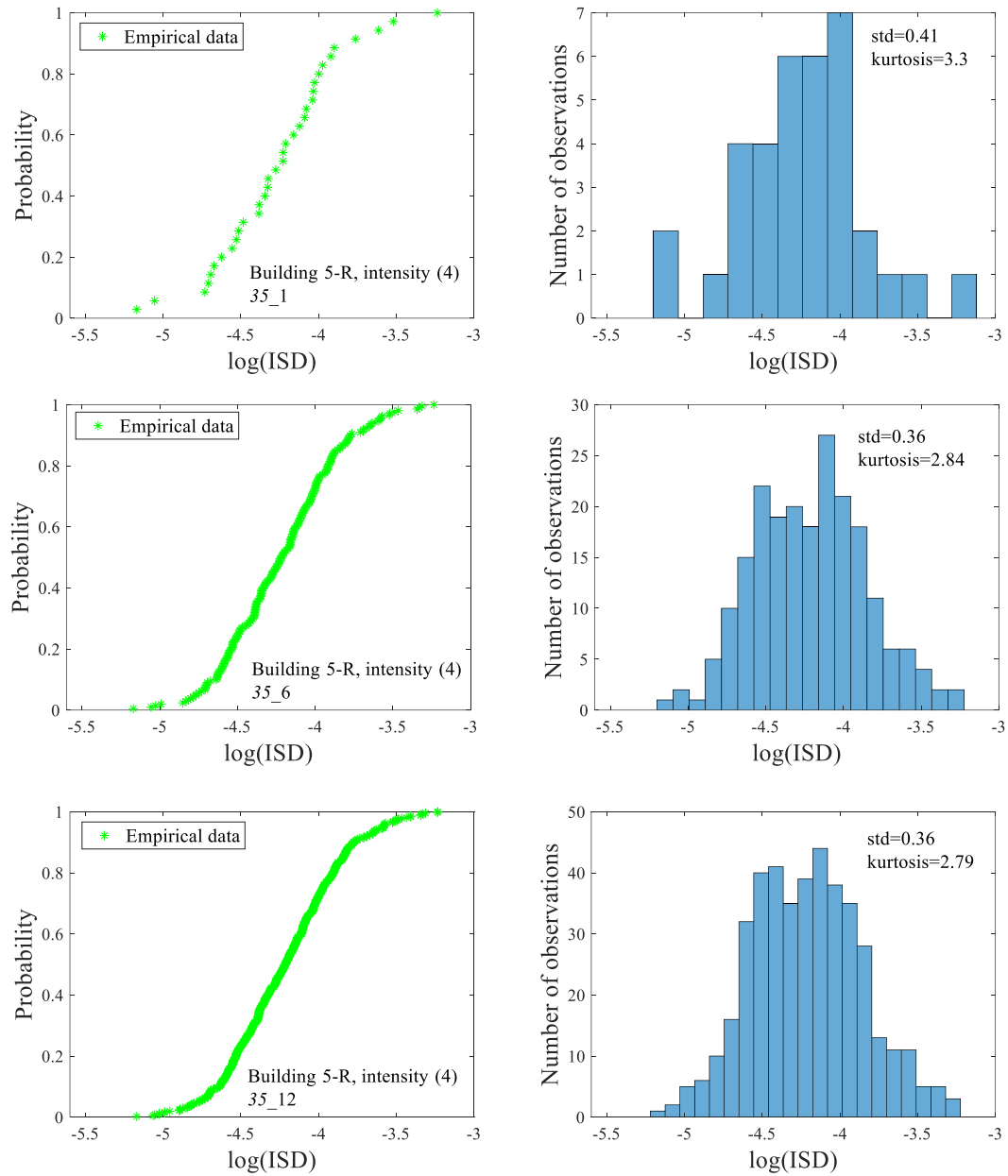


Figure 4.28 Empirical data and histograms of the log(ISD) for $n_{\text{ASI}} = 35\_1$, 35_6 and 35_12. Case 1 (5-R).

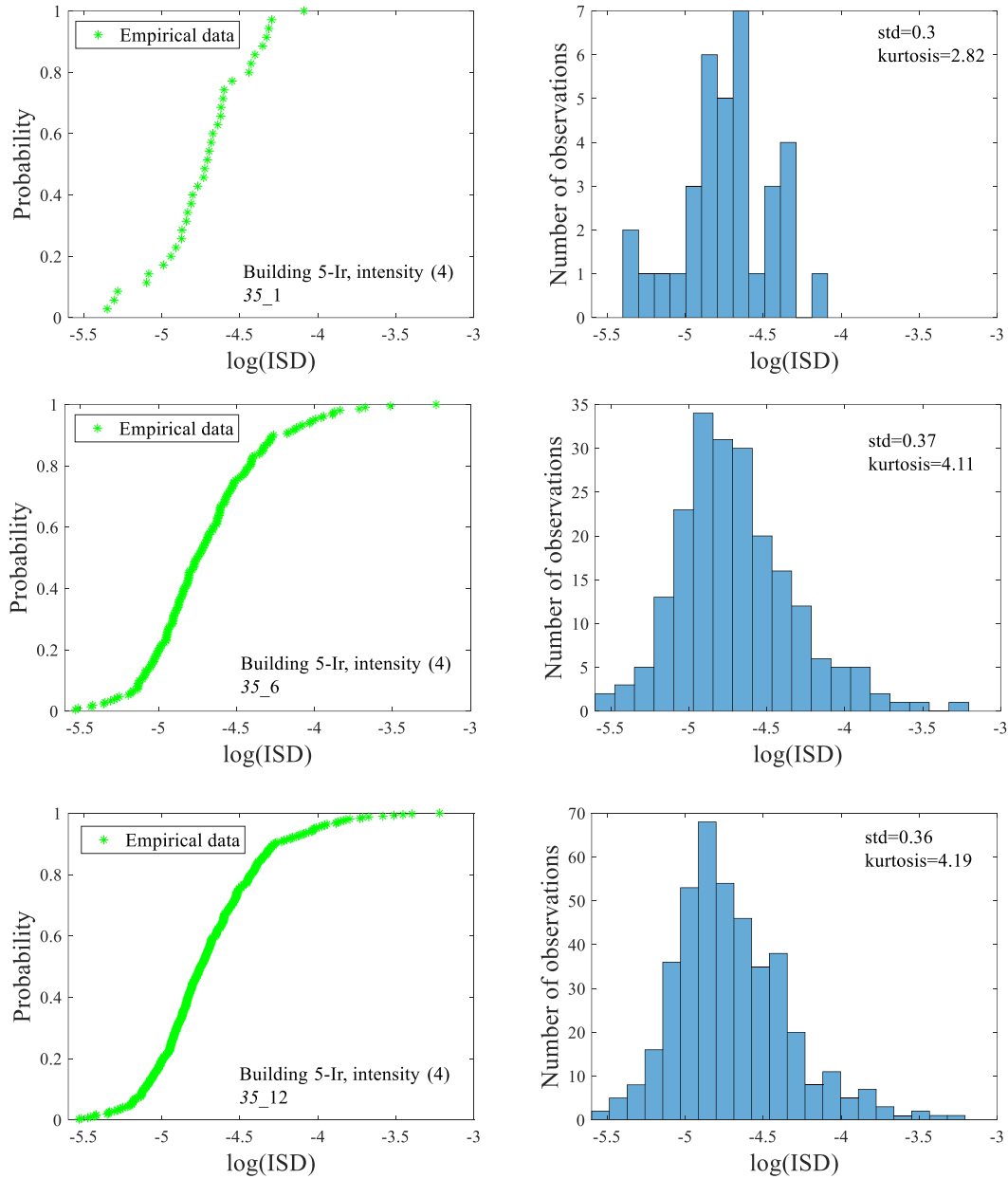


Figure 4.29 Empirical data and histograms of the log(ISD) for $n_{\text{ASI}} = 35\_1, 35\_6$ and 35_12 . Case 2 (5-Ir).

To achieve a more comprehensive view of the change of the distribution shape with the increase of the ASI, the kurtosis and the std of the logarithms of the ISD data are presented as a ratio of the n_{12} over the n_1 combination in the following figures for all 100 GM groups of selected cases (building, intensity and n). Two GM group sizes are presented for each building for comparison. Ratios larger than 1 indicate that the kurtosis/std increases with the increase of the ASI, ratios lower than 1 indicate a reduction of the kurtosis/std with the increase of the ASI, while ratios equal to 1 suggest no changes. Figure 4.30 to Figure 4.34 show that the effect of the ASI does not present any constant

trends on the kurtosis and on the std of the ISD data. As such, on average, the kurtosis can increase, e.g. 3-Ir, 5-Ir, 4-Ir, or decrease, e.g. 5-R, with the ASI or even remain uninfluenced, e.g. 3-R, by the ASI. Similarly, the std experiences no constant trends either and, on average, it appears to increase, e.g. 5-Ir, 3-Ir, or decrease, e.g. 3-R, 5-R, 4-Ir, with the ASI. The level of variation of the ratios depends on the GM group size, with larger sizes leading to lower variability of the ratios. The previous observations suggest that the effect of the ASI can be building- and/or GM group-specific.

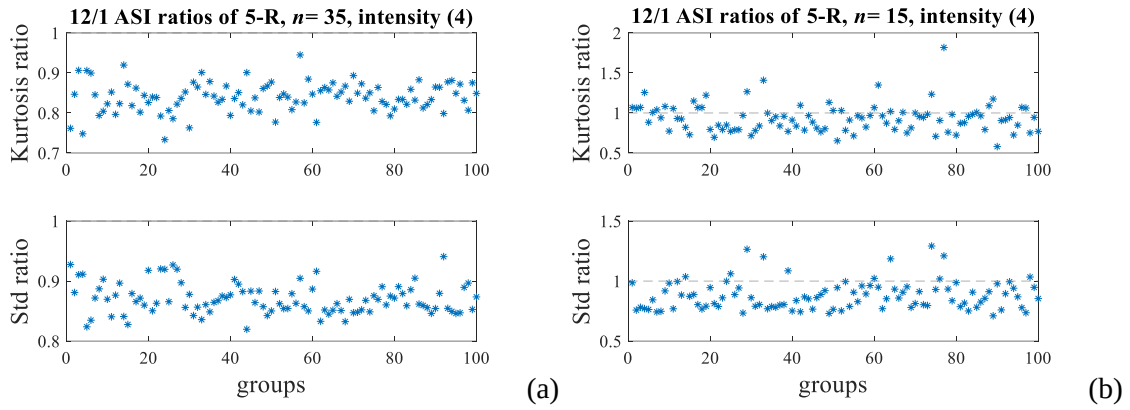


Figure 4.30 Kurtosis and std ratios of n_{12} to n_1 for $n = 35$ (a) and $n = 15$ (b) of 5-R.

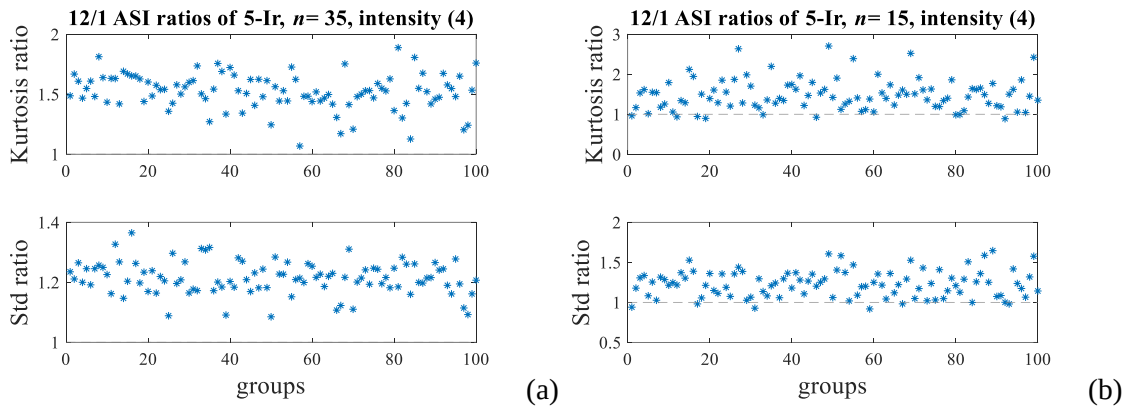


Figure 4.31 Kurtosis and std ratios of n_{12} to n_1 for $n = 35$ (a) and $n = 15$ (b) of 5-Ir.

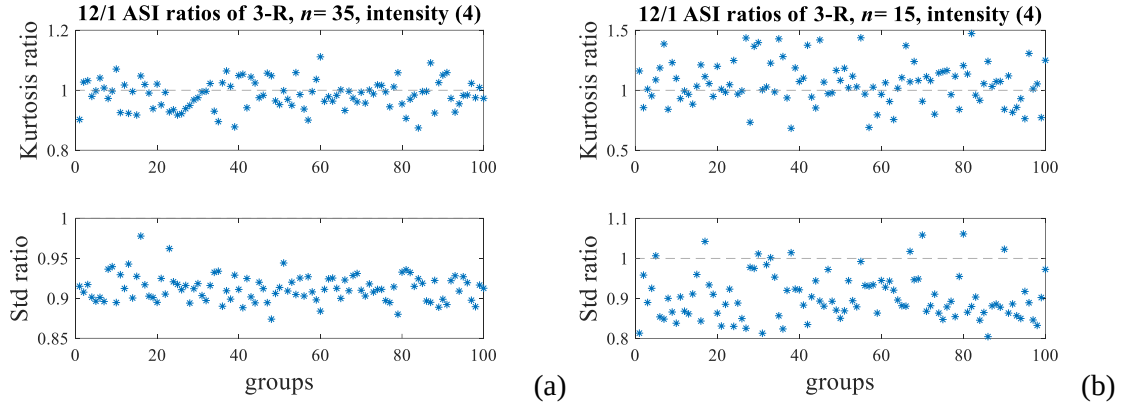


Figure 4.32 Kurtosis and std ratios of n_{12} to n_1 for $n = 35$ (a) and $n = 15$ (b) of 3-R.

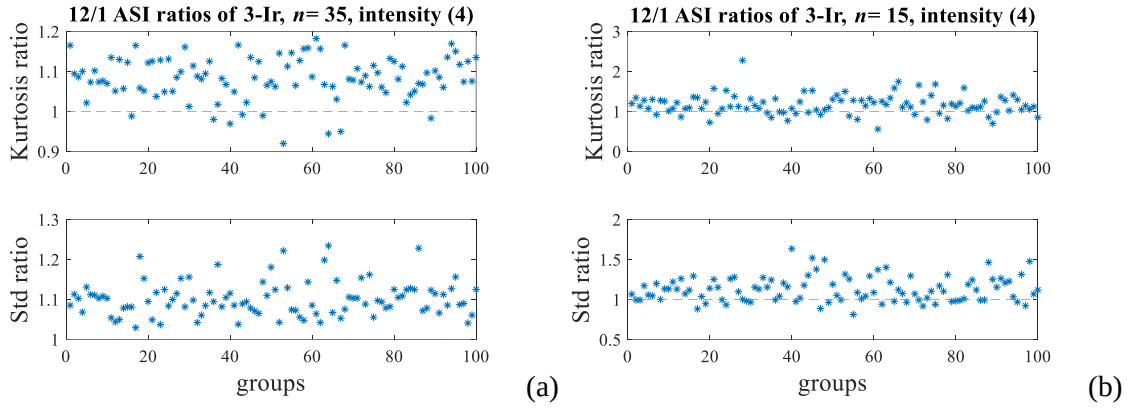


Figure 4.33 Kurtosis and std ratios of n_{12} to n_1 for $n = 35$ (a) and $n = 15$ (b) of 3-Ir.

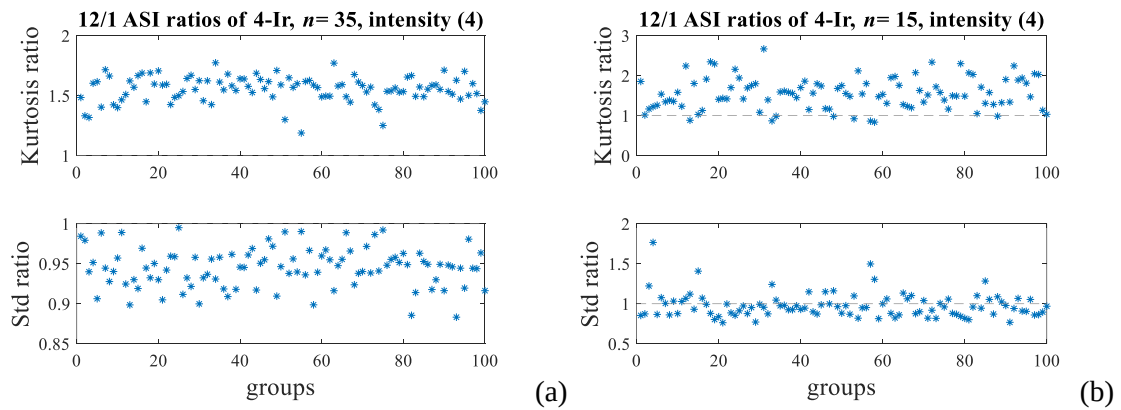


Figure 4.34 Kurtosis and std ratios of n_{12} to n_1 for $n = 35$ (a) and $n = 15$ (b) of 4-Ir.

Focusing on buildings 5-Ir and 3-R that were analysed using the same GM groups and the same intensities (see Section 3.4.2 for details), it can be seen that the ratios are significantly different. The latter observation suggests that the increase of the ASI can have different effects depending on the building characteristics. Furthermore, in order to address the possible influence of the inelastic response of the buildings, the results of building 4-R building are presented in Figure 4.35 for intensities (4) and (3). Referring again to Section 3.4.2, it can be seen that the structural analysis for intensities (3) and (4) was performed by applying a different scale factor on the same GM group. As a result, the comparison of the ratios of Figure 4.35(a) and (b) highlights the effect of the inelastic response of the building. In fact, the referred ratios appear to differ, indicating that the effect of the ASI depends also on the level of inelasticity of the structure. The presented results are not exhaustive of the subject but suggest that further investigation is required on this topic. In particular, buildings with different configurations analysed with different GM groups and intensity levels should be involved in order to determine more precisely the effect of the ASI on the shape of the EDP data distribution.

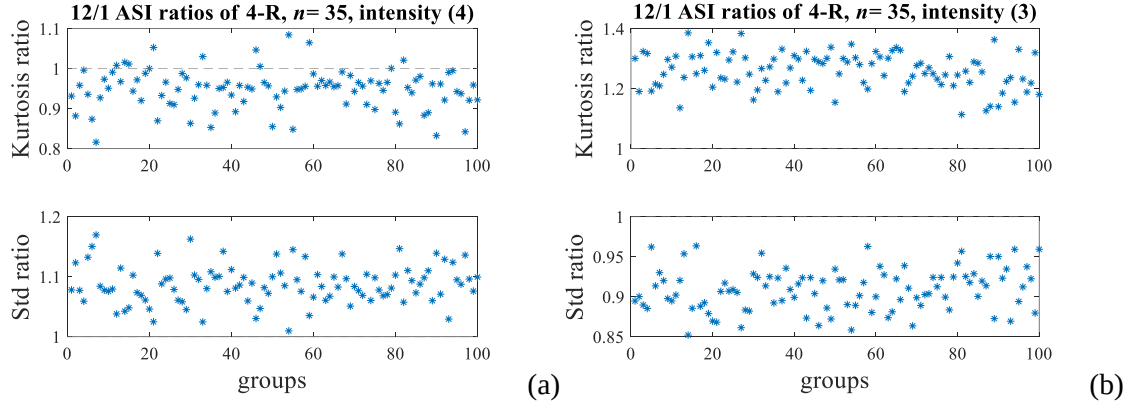


Figure 4.35 Kurtosis and std ratios of n_{12} to n_1 for int(4) (a) and int(3) (b) of 4-R.

4.4.5 Additional remarks

Based on the evaluation of the performance of the four distribution models using the goodness-of-fit tests, the following remarks can be made:

- Lower sample sizes were, as expected, seen to lead to lower rejection rates.

- The rejection rates increased with the seismic intensity for all models examined, with the exception of the Fréchet model, which appeared to fit better for data samples of higher intensities.
- The lognormal and the Gamma models showed the lowest rejection rates for all cases examined. The different cases involved various n_{ASI} combinations and sample sizes, the different EDPs and two seismic intensities.
- The Weibull model exhibited the highest rejection rates for the higher intensity (higher inelastic response) for all the examined EDPs, while the same was observed for the Fréchet model for the lower intensity. The behaviour of these two models was judged mostly inadequate, with the exception of the Fréchet model for smaller sizes and for the higher intensity. In that case, the Fréchet model exhibited rejection rates similar to those of the lognormal and the Gamma models.
- The selected number of ASIs was seen to affect the rejection rates in many cases. This was particularly obvious for the PFA data, for which higher rejection rates were obtained for the Gamma and the lognormal models when more ASIs were used. The opposite effect was observed for the Weibull and the Fréchet models, though. Smaller differences were found for the ISD and the RD, particularly for the higher intensity level.
- A closer analysis to the effect of the ASI showed that increasing the number of ASIs changes the shape of the empirical data to an extent that models such as the lognormal and the Gamma are no longer valid. However, the aforementioned changes due to the number of ASIs did not show any constant trends. Instead, they appeared to be building-, intensity- and/or GM group-specific

4.5 Conclusions

The combined effect of the number of ASIs and of the GM group size was examined on the probabilistic modelling of seismic demand of RC buildings. More particularly, samples of selected EDPs were analysed with respect to the type of statistical model that best fits the data. Six RC buildings were analysed for this purpose using different sizes of GM groups applied along 1 to 12 ASIs and for four intensity levels. Different statistical models were then fitted to the EDP samples and the performance of each model was evaluated in two subsequent phases. Initially, the adequacy of six statistical models was evaluated through Q-Q plots using the R^2 and RMSE measures of their linear regression.

Next, the best four distributions, i.e. the distributions that presented the best fitting according to the R^2 and RMSE measures, were further analysed using specific goodness-of-fit tests. The examined EDPs are the maximum ISD and PFA across the height of the buildings and the maximum RD.

The R^2 and RMSE comparisons showed that the Gumbel model is the most inappropriate to characterize the demand distribution of all EDPs and intensities. The normal and the Weibull distributions exhibited a good fitting performance for linear and low inelastic structural response. For higher inelastic structural response, however, the fitting of these distributions was characterized by significant variability and, especially for the normal model, by a much lower performance. The lognormal and the Gamma distributions showed adequate fitting performance for all cases, and with a variability that generally increased when increasing the nonlinearity of the structural behaviour. Particularly interesting findings were also observed in cases fitted using the Fréchet model, which showed very low performance for most of the cases, but exhibited a good performance in particular cases where all the alternative models were inappropriate.

Based on the previous, mainly qualitative, results, the lognormal, Gamma, Weibull and Fréchet models were further examined using appropriate goodness-of-fit tests. Since the tests are sensitive for very small, e.g. lower than 25 and for very large, e.g. larger than 150, sample sizes, the performance of the models was evaluated using specific combinations of number of ASIs and GM group sizes. Furthermore, EDP data for two intensity levels associated with differences in structural behaviour were separately examined.

The lower rejection rates were obtained for the Gamma and the lognormal model, in agreement with the conclusions reached using the R^2 and the RMSE results. The higher rejection rates were observed for the Weibull model that was then considered to be the least appropriate. The Fréchet model also presented high rejection rates, especially for elastic structural response. However, for the highest intensity, where larger inelasticity was observed, and especially when considering a higher number of ASIs, the rejection rates of the Fréchet distribution decreased significantly, reaching values below 50%. It is worth noting that, for the same cases, the lognormal and Gamma models exhibited similar rejection rates. The number of ASIs was seen to affect the rejection rates. In particular, the lognormal and the Gamma models exhibited higher rejection rates when a larger number of ASIs was involved in the sample. This was highlighted by the goodness-of-fit

plots that showed a change in the shape of the empirical data with the increase in the number of ASIs. Still, no specific trend was able to be identified regarding the effect of the number of ASIs on the shape of the empirical data.

Overall, it is concluded that the most appropriate model to fit EDP data, such as ISD, PFA and RD, coming from any combination of number of ASIs and number of ground motions is the lognormal. Regarding the Gamma model, although the rejection rates that were found are particularly low (similar or even lower than those found for the lognormal model), the lower reliability of the test found in the power analysis indicates that it should be used with caution. Nevertheless, the similarity of the alternative distributions for the examined parameter range is also an indication that alternative models could be used without significant consequences. Therefore, it is also suggested to examine the alternative models, i.e. the lognormal and the Gamma, using a visual inspection of the empirical data, such as using CDF or Q-Q plots. Furthermore, especially for higher intensities and when a larger number of ASIs is used, the Fréchet model could also be examined as an alternative.

Chapter 5

5 How is collapse risk of RC buildings affected by the angle of seismic incidence?

Based on:

Skoulidou, D., Romão, X., & Franchin, P. (2019). How is collapse risk of RC buildings affected by the angle of seismic incidence?. *Earthquake Engineering & Structural Dynamics*, 48(14), 1575-1594. DOI: 10.1002/eqe.3214

Abstract

Despite the undeniable influence of the angle of seismic incidence on seismic demand, limited research has been performed to determine its effect when assessing the probabilistic seismic performance of a structure. Since the ground motion group size has a well-known significant effect on the variability of collapse risk, the joint effect of both the ground motion group size and the angle of seismic incidence on the estimation of collapse risk is investigated. Results show that the two variables have a different effect on the collapse risk estimates, with one affecting the bias in the expected value of the estimates and the other their variability. In order to ensure acceptable levels of variability and bias, practical proposals are made regarding the minimum number of records and angles of seismic incidence that should be considered.

5.1 Introduction

Performance-Based Earthquake Engineering (PBEE) guidelines, such as FEMA P-58 (2012), provide guidance to perform the probabilistic seismic performance assessment of

a structure. One of the main advantages of PBEE lies in its ability to explicitly consider different sources of uncertainty that affect the assessment problem. Common sources of uncertainty are those associated with the seismic input, expressed by the seismic hazard uncertainty (e.g. see Abrahamson and Bommer 2005) and the record-to-record variability (e.g. see Haselton et al. 2009), the structural modelling, such as the variability of geometric properties, and material constitutive laws and members response models (e.g. see FEMA P-58 2012; Liel et al. 2009; Franchin et al. 2018), the damage definitions and the economic losses (e.g. see Lee and Mosalam 2006). In this context, the present study focuses on assessing the uncertainty associated with the direction of application of the input seismic action with respect to the building's structural axes, usually termed angle of seismic incidence (ASI). Despite the early recognition of its importance (Penzien and Watabe 1974) and the several studies performed addressing this issue, there is still a lack of consensus on how to account for the ASI in seismic performance assessment.

The uncertainty associated to the ASI stems from the fact that the orientation of a building's structural axes with respect to the location of future fault ruptures is unknown, particularly when the former is not located in the vicinity of a known fault rupture. This in turn enables the seismic action to act on the building from any direction, i.e. at any ASI, and can have significant effects on structural demand (e.g. see (Lopez and Torres 1997; Kostinakis et al. 2013; Magliulo et al. 2014; Cantagallo et al. 2015; Fontara et al. 2015)). In this context, one aspect that was focused by past research (Smeby and Der Kiureghian 1985; Menun and Der Kiureghian 1998; Lopez et al. 2000; Athanatopoulou 2005; Faggella et al. 2018) is the identification of the ASI that causes the most unfavourable effect on the building, also termed the critical ASI (ASI_{crit}). However, when structural performance is assessed in probabilistic terms, using the ASI_{crit} is seen to lead to an overly conservative demand assessment (Beyer and Bommer 2007; Stewart et al. 2011). In light of this, some researchers have been proposing alternative approaches to obtain adequate probabilistic demand distributions while accounting for the randomness of the ASI. For example, Rigato and Medina (2007) suggested the idea that an adequate number of GMs and number of ASIs could be defined to estimate inelastic demand with a predefined confidence level. In a loss assessment framework, Lagaros (2010a) analysed the effect of the ASI by sampling it from a uniform distribution and concluded that 20 ground motions applied along 5 random ASIs are adequate to account for the randomness of both the ground motion and the ASI when evaluating the life-cycle cost of a building.

Lagaros (2010b) also incorporated the effect of the ASI in a probabilistic framework involving Multidirectional Incremental Dynamic Analysis (MIDA). Given the difficulty of defining the ASI_{crit} during MIDA, the author suggested that, by combining 30 records with an equal number of ASIs using specific sampling techniques, a good approximation to fragility curves determined with a large number of records and randomly sampled ASIs could be obtained. More recently, Giannopoulos and Vamvatsikos (2018) analysed structural demand distributions obtained from IDA and showed that, in a scenario where only a fixed number of analyses is performed, the number of ground motions has a larger influence on obtaining an accurate structural demand distribution when compared to that of the number of ASIs being considered. Although not exhaustive, this literature review emphasizes the lack of consensus that still exists regarding the effects of the ASI and on ways to account for them. This lack of consensus is further emphasized by the inconsistent and sometimes vague standard provisions on this respect, with some of the standards ignoring the effect of the ASI (e.g. see EAK 2003; NTC 2018) and others making specific reference to the ASI_{crit} but without providing clear guidelines to determine it (e.g. see (N1998-1 2004; EN1998-3 2005; ASCE/SEI 2017; NZSEE 2017; ICC 2018)).

In light of this, the present study assesses the effect of the randomness of the ASI within the context of PBEE focusing on the collapse risk of reinforced concrete (RC) buildings subjected to far-field ground motions. It is noted that only far field ground motions are considered herein since the referred randomness cannot be justified in near-field scenarios (Beyer and Bommer 2007). Given that collapse risk is determined based on structural demand data obtained from nonlinear time history analyses, the study integrates the effects of the ASI and of the ground motion group size, but analyses the influence of these two factors separately. Based on the results obtained from the analysis of six buildings with different height and regularity, the ASI and the ground motion group size are shown to affect collapse risk in distinct ways. Furthermore, results also show that the uncertainty introduced by the ASI can be accounted for by just considering two ASIs per ground motion.

5.2 Methodology

The seismic collapse risk of low- to mid- rise RC buildings with infilled frames is examined considering the effect of the size of the ground motion group and the effect of the number of ASIs. Six benchmark buildings with and without structural irregularities

are modelled and subjected to nonlinear dynamic analysis with bidirectional ground motions in groups of size n ranging from 10 to 40, applied along 1 to 12 ASIs and for 20 intensity levels. Ground motion groups of size equal to 40 are initially selected for each building in order to be compatible with a response spectrum that represents the seismic scenario (see details in Section 5.3.2). Subsequently, the groups of 40 records are regrouped to create ground motion groups of smaller sizes equal to 10, 15, 20, 25, 30 and 35, according to a regrouping procedure that maintains the compatibility of all new groups with the seismic scenario (see details in Section 5.3.2). This regrouping procedure yields a total number of 100 groups for each new size n , which provides a large enough sample of data for the statistical post-processing of the results.

After selecting the ground motions and applying the regrouping procedure, the structural analyses of the buildings are performed accounting for the effect of the ASI. As such, all the ground motions of the 100 groups of each new size n are applied to the buildings along different numbers of ASIs. The ASI of a bidirectional ground motion is defined by the angle between one preselected component of the ground motion and a structural axis. Since only far-field records are considered in the current study, the reference ground motion component is defined arbitrarily and the angle it forms with structural axis X of the building defines the ASI. Given that the structural axis X of a building can have a random orientation with respect to the reference ground motion component, all ASIs are considered to be equally likely within the range $[0^\circ \ 180^\circ)$, assuming an angle step of 15° . It is noted that, although the responses obtained for supplementary angles such as 0° and 180° may not necessarily be the same, differences are nonetheless expected to be minor for the cases analysed in the present study. In order to be consistent with current practice, when a single ASI is used, the direction of application of the seismic action coincides with the structural axes of the building, i.e. all ground motions of a group are applied along 0° . When multiple ASIs are used, the ASIs equidistantly cover the total angle range, with the first ASI always being 0° . For instance, using two ASIs corresponds to using angles 0° and 90° , while three ASIs corresponds to using angles 0° , 60° and 120° . The overall procedure leads to structural demand results of 100 groups of ground motions for each combination of group size n and number of ASIs that are then used for further post-processing.

Following the dynamic analyses of the structures, seismic collapse is then determined, with collapse being conditioned by three criteria, as described in detail in Section 5.3.4.

Collapse fragility curves are derived for all 100 groups of each combination of group size n and number of ASIs and are subsequently integrated with the derivative of the seismic hazard curve to determine the collapse risk. The effect of the ASI on the collapse risk is initially examined for each individual size n , by comparing the collapse risk computed using less than 12 ASI to the collapse risk of the same size n obtained when all 12 ASIs are considered. Subsequently, the joint effect of the number of ASIs and of the group size n is determined by comparing all previously defined combinations with the reference scenario involving the 40 ground motions applied along all 12 ASIs.

5.3 Modelling

5.3.1 Buildings and Structural Modelling

5.3.1.1 General characteristics of the buildings

Six RC buildings with infilled frame systems are analysed. The selected buildings have configurations ranging from low- to mid-rise buildings with and without in-plan irregularities. The plan view of a typical storey of the 3-storey irregular (3-Ir), the 4-storey irregular (4-Ir) and the 5-storey irregular (5-Ir) is presented in Figure 5.1, along with the design details of the end sections of the elements. The height of the first storey is 3.5 m and the height of all upper storeys is 3.0 m. Similarly, Figure 5.2 shows the plan view of a typical storey of the 3-storey regular (3-R), the 4-storey regular (4-R) and the 5-storey regular (5-R) building and the design details of the end section of the elements. All regular buildings have a unique storey height equal to 3.0 m. The concrete strength f_c and the yield strength of the reinforcing steel f_y are equal to 25 MPa and 500 MPa, respectively, for all buildings.

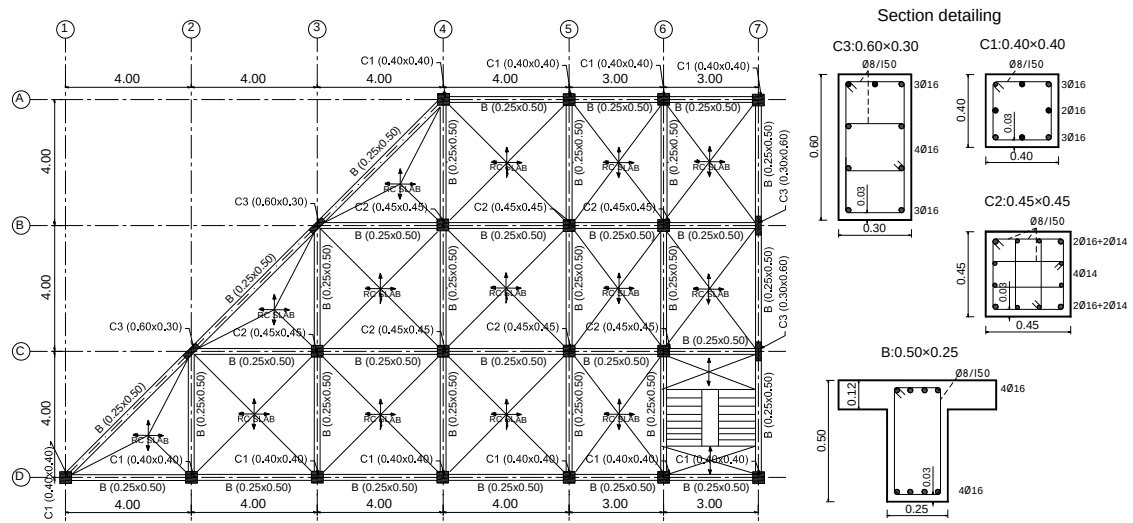


Figure 5.1 Plan view of a typical storey of the 3-Ir, 4-Ir and 5-Ir buildings and the design details of the end sections of the elements (all dimensions are in m).

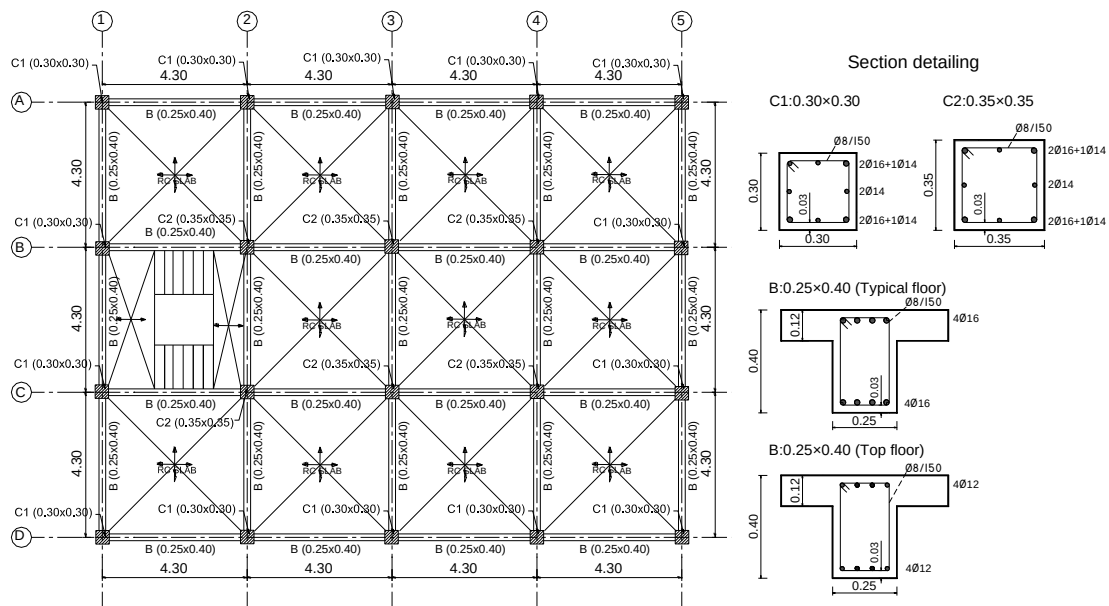


Figure 5.2 Plan view of a typical storey of the 3-R, 4-R and 5-R buildings and the design details of the end sections of the elements (all dimensions are in m).

All buildings are defined as existing buildings located in Lisbon, Portugal, designed for gravity loads and with a minimum lateral strength to account for wind. Specifically, a non-structural permanent load of 4 kN/m^2 is considered to be uniformly distributed on all

floors of both building categories, additional to the slab self-weight (all two-way solid slabs have a thickness of 0.12 m). Additionally, the quasi-permanent component of a uniform live load of 3 kN/m^2 is also assigned to all floors, except to the top storey floor where this load is 1 kN/m^2 . Staircases are modelled only as permanent and quasi-permanent loads, which are transferred to the supporting beams and applied uniformly. The corresponding loads are 7.75 kN/m and 8.60 kN/m for the permanent and the quasi-permanent loads, respectively. Masonry infills are present in all peripheral frames and represent a permanent uniform load of 7 kN/m . For modelling simplification purposes, a unique beam section with symmetric reinforcement configuration is used for each floor.

5.3.1.2 Modelling of the structural members

Three-dimensional models of the buildings are simulated in the OpenSees (OS) computer software (McKenna 2011) considering mean values of all material and geometrical properties. A lumped plasticity approach is adopted to simulate the inelastic behaviour of both beams and columns. Phenomenological hysteresis laws are assigned in rotational springs located on both ends of all structural elements to simulate inelastic flexural behaviour. Two independent springs are assigned to each end of the columns, one for each orthogonal direction, while one spring is assigned to each end of the beams modelling the in-plane flexural behaviour. Due to the nature of the selected inelastic modelling approach, biaxial moment interaction or axial force moment interaction are not considered when modelling the behaviour of columns (Ricci et al. 2018). The hysteretic flexural behaviour is defined by a moment-rotation law that is simulated using the *hysteretic* material provided by OS. The properties of the backbone curve of all columns are determined considering the axial force due to vertical loads of the seismic combination ($G+0.3Q$), where G and $0.3Q$ stand for all permanent and quasi-permanent loads, respectively. The yielding moment (M_y) and the yielding rotation capacity (θ_y) are determined according to Panagiotakos and Fardis (2001). The capping (θ_{cap}) and the ultimate (θ_u) rotation capacities are computed according to Haselton et al. (2008) along with the corresponding capping (M_{cap}) and residual (M_r) moments, where the latter is considered to be 20% of M_{cap} (see the backbone curve in Figure 5.3(a)). Stiffness, strength and unloading stiffness degradation are considered in the hysteresis curves as shown in Figure 5.3(a). The unloading stiffness degradation is implemented in OS via the *beta* factor of the *hysteretic* material. A *beta* factor of 0.75 is used for the columns and of 0.85 for the beams in order to introduce higher degradation for the latter (Dolšek and Fajfar

2005). The *hysteretic* material parameters related to pinching and to damage due to ductility and dissipated energy are all set equal to zero. The springs are connected in-series with a linear elastic interior element. A stiffness modification factor equal to 10 is applied according to Zareian and Medina (2010) to account for the effect of the series connection on the total stiffness of the member. The possibility of shear failure is analysed in the post-processing phase of the analysis. Beam-column node failure is not considered and rigid offsets are modelled at all joints.

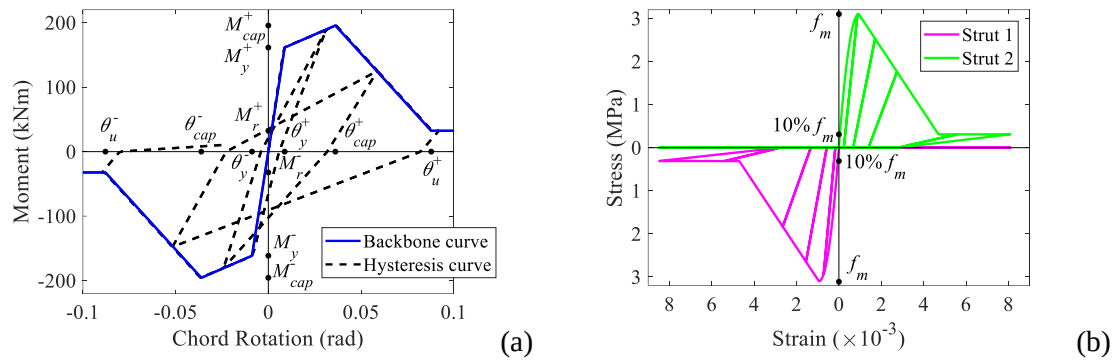


Figure 5.3 Moment-rotation backbone curve and hysteresis loop simulation for a column (a) and stress – strain relationship of the diagonal struts (in compression) representing the infills, measured in the diagonal direction (b).

5.3.1.3 Modelling of the infills

Masonry infills are considered in all peripheral frames and are modelled with a single strut active only in compression. Two diagonal struts (strut 1 and strut 2 as shown in Figure 5.3(b)) are used to simulate one infill and are connected to the beam-column joints. The equivalent area of each strut is established based on the maximum lateral force of the infill (Žarnić and Gostič 1997), transformed to the direction of the diagonal, and on the masonry compressive stress f_m . Maximum strength is assumed to be reached at an interstorey drift of 0.2% (Dolšek and Fajfar 2008). The lateral displacement of each infill is transformed into the diagonal displacement for the subsequent definition of the strain of the strut. The parameters obtained, i.e. the maximum stress and strain, are used to define the masonry material with zero tensile strength simulated by the *Concrete01* constitutive model (Ricci et al. 2018; Noh et al. 2017; Mohamed and Romão 2018) (Figure 5.3(b)). The stress f_m is equal to 3.10 MPa and all infills have a thickness of 0.15m. Additionally, a residual stress equal to 10% of the maximum stress is considered for numerical stability, which is reached at an interstorey drift five times the interstorey drift at maximum strength.

5.3.1.4 Additional modelling considerations and dynamic properties of the buildings

Geometric nonlinearities are incorporated in the form of P- Δ effects. Furthermore, soil-structure interaction phenomena are not modelled and all structures are considered fixed to the ground. The first and the second fundamental periods of vibration of the six buildings, with and without the masonry infills, are presented in Table 5.1. The terms infilled and bare structure refer to the structure with and without the infills, respectively. Finally, the average of the first two periods of the infilled structure and the first two periods of the bare structure T^* is determined (also shown in Table 5.1) to be used for the ground motion selection. Involving periods of vibration of the bare structure in the definition of T^* , is conceptually similar to accounting for the apparent period elongation of the structure after yielding and failure of the infills.

Table 5.1 First and second mode periods of vibration of the buildings.

Period\Building	3-R (s)	4-R (s)	5-R (s)	3-Ir (s)	4-Ir (s)	5-Ir (s)
T_1, T_2 (infilled)	0.31,0.25	0.41,0.31	0.52,0.39	0.21,0.15	0.29,0.20	0.37,0.26
T_1, T_2 (bare)	0.73,0.72	0.96,0.93	1.18,1.15	0.39,0.35	0.55,0.47	0.70,0.60
T^*	0.50	0.66	0.82	0.27	0.38	0.48

5.3.2 Seismic hazard, ground motion selection and regrouping procedure

The ground motion record selection is performed using the recently developed SelEQ software (Macedo and Castro 2017) using a Conditional Spectrum (CS) in terms of the 5% damped spectral acceleration (Lin et al. 2013a) as the target spectrum. First, the probabilistic seismic hazard analysis of the site is performed using the open source software OpenQuake (Pagani et al. 2014)) and the seismic hazard model is developed within the SHARE project (Woessner et al. 2015). The GM prediction equations proposed by Atkinson and Boore (2006) and Akkar and Bommer (2010) were used with a weight of 70% and 30%, respectively. The geometric mean spectral acceleration is employed for the definition of the seismic hazard in order to ensure compatibility with the spectral acceleration used in the ground motion selection (Baker and Cornell 2006), as described next. The annual seismic hazard curve H_{IM} of the benchmark site is determined at T^* for each building and is shown in Figure 5.4(a). Note that, since T^* of the 5-storey irregular building is equal to 0.48s, the seismic hazard curve considered for this case is also the one determined for the 3-storey regular building that has $T^*=0.5$ s.

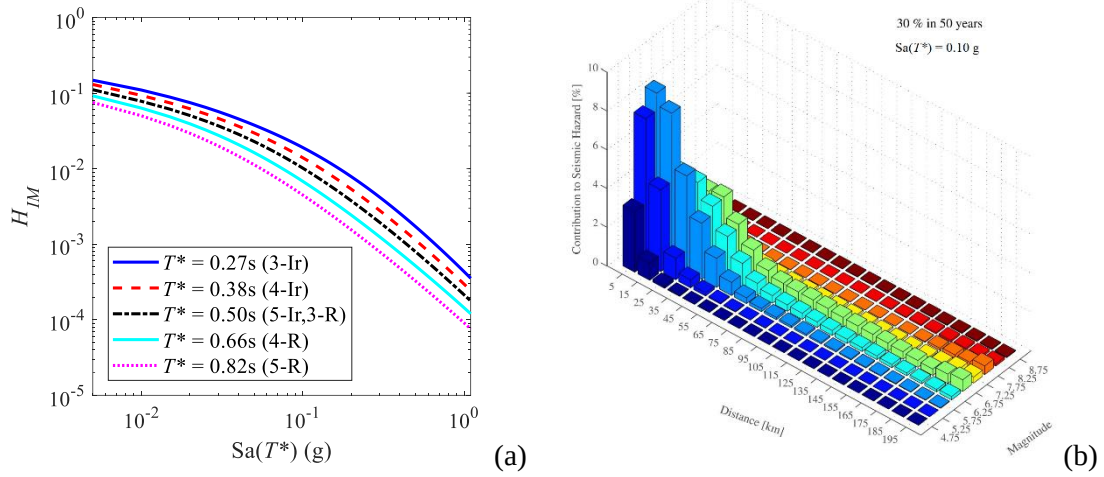


Figure 5.4 Seismic hazard curves for Lisbon and for the periods of interest (a) and seismic hazard disaggregation for $T^* = 0.66$ s and 30% in 50 years.

Disaggregation of the hazard is then performed for four probabilities of exceedance, i.e. 30%, 10%, 5% and 2% in 50 years, at T^* for each building. An illustrative example of the hazard disaggregation is shown in Figure 5.4(b) for the 4-R building and for the 30% probability of exceedance in 50 years. The results are next used to build four CSs using the methodology proposed by Lin et al. (2013a; 2013b; 2013c). A preliminary ground motion record selection is performed using the NGA-WEST2 database (Ancheta et al. 2014) based on seismological and strong motion parameters, such as magnitude, epicentral distance and average shear wave velocity. Subsequently, the final selection and scaling of groups of 40 bi-directional records (for each building and for each probability of exceedance) is carried out by ensuring compatibility between the geometric means of the horizontal components of all pairs and the target spectrum, i.e. the CS. This compatibility is ensured by minimizing the difference between the statistics (i.e. the mean and standard deviation of the logarithms of the spectral accelerations) of the group and the same statistics of the target spectrum, within a period range of $\min(0.1s, 0.2T^*)$ and $1.5T^*$, as described in Macedo and Castro (2017). An additional criterion is included in the process which involves minimizing also the skewness of the logarithms of the spectral accelerations to a value close to zero for the same period range. This criterion is based on the fact that the spectral accelerations are expected to follow a multivariate lognormal distribution (Jayaram and Baker 2008) and therefore, the logarithms of the spectral accelerations are expected to follow a normal distribution. Overall, 4 groups of 40 records are selected for each building, each corresponding to a predefined probability of

exceedance (see Appendix B for a complete list of the GMs). To illustrate the results of the ground motion selection process, Figure 5.5 shows the geometric mean spectra of the selected 40 records along with the CS for the four probabilities of exceedance for the 3-R building. Furthermore, in order to highlight the directionality characteristics of the selected groups of GMs, and particularly the fact that they are representative of far-field records, polar plots of the $S_a(T^*)$ of all groups are presented in Appendix C.

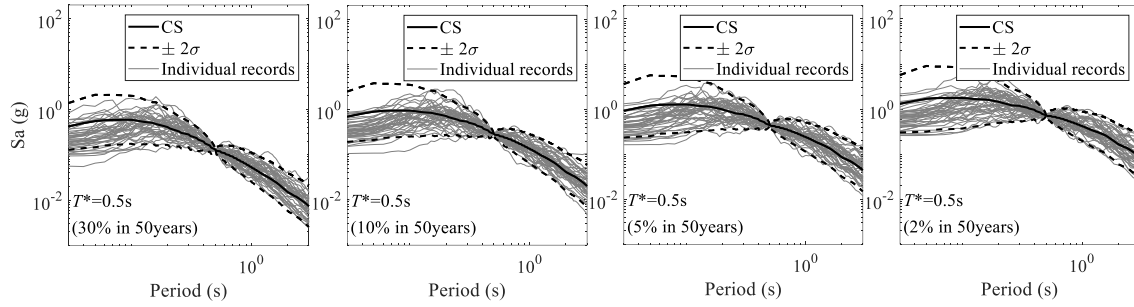


Figure 5.5 The CS with the geometric mean spectra of the 40 records for each probability of exceedance, $T^* = 0.50s$.

Each initial group of size $n = 40$ is then regrouped to create ground motion groups of smaller size $n = 10, 15, 20, 25, 30$ and 35 . The total number of combinations of m items taken k at a time is given by the binomial coefficient:

$$\binom{m}{k} = \frac{m!}{(m-k)!k!} \quad (5.1)$$

The binomial coefficient for the requested group sizes is a number between 6.6×10^5 and 1.4×10^{11} . However, not all combinations are admissible, since groups must represent a seismic scenario consistent with that of the reference group of size 40. Consistency is assumed to be verified when a ground motion group of size n meets three criteria, consistent with the criteria used for the initial selection as described previously. These criteria are applied between the statistics of each new group and the target spectrum, where the target spectrum now corresponds to the spectrum of the 40 ground motions. The spectrum of the 40 ground motions, which is necessarily close, within a tolerance, but not coincident with the corresponding target CS, is selected as the target spectrum for the regrouping procedure, aiming to avoid additional error introduced in the comparison of the reference demand with the demand of the smaller group sizes. Regarding the mean and the standard deviation of the logarithms of the spectral accelerations, a percentage

error of 10% is allowed. With respect to the skewness of the logarithms of the spectral accelerations, the standard skewness error is satisfied with a significance level of 5% (Tabachnick 1996). A total number of 100 admissible groups are created for each new group size n , to describe the uncertainty associated with a finite group size, and illustrative examples of the geometric mean spectra of two groups of size $n = 35$ and $n = 15$ are presented in Figure 5.6 (a) and (b), respectively. The groups presented are representative of large and small size n , respectively, and were obtained from the group with $T^*=0.5$ s, i.e. for the 3-R building, and for a 2% in 50 years probability of exceedance.

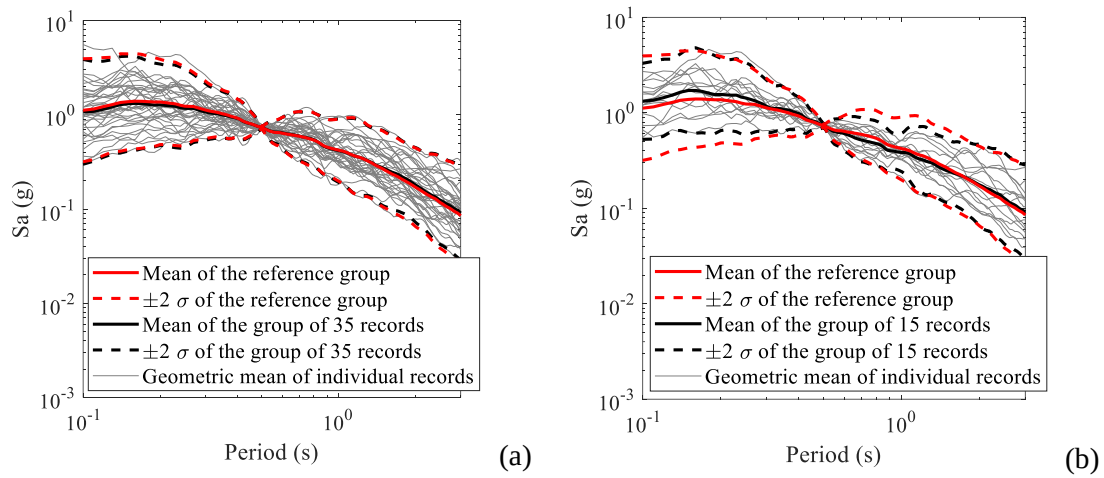


Figure 5.6 The individual geometric mean spectra of all ground motions of a group (one out of the 100 generated) of size $n = 35$ (a) and $n = 15$ (b), along with their average and the standard deviation for the 3-R building. The average of the reference group of 40 records and the standard deviation are also shown for comparison.

5.3.3 Probabilistic demand model

The response of the six 3D RC building models presented in Section 5.3.1 is determined using multiple stripe analysis (MSA) (Jalayer and Cornell 2009) considering the ground motion groups defined in Section 5.3.2. Following a procedure similar to the one proposed by Christovasilis et al. (2014), the four ground motion groups selected for each building are scaled up and down to span a total of 20 intensity levels. The spectral accelerations $Sa(T^*)$ and the corresponding scale factors (SF) for all groups of records are summarised in Table 5.2. An illustrative example of this procedure is presented in Figure 5.7, which shows the scaled average geometric mean response spectra for the case of $T^*=0.5$ s. All 40 ground motions are applied considering 12 different ASIs, ranging from 0° to 165° in steps of 15° , resulting in a total of (20 intensities $\times$ 40 ground motions $\times$ 12 ASIs) 9600 nonlinear 3D analyses per building (57600 total analyses).

Table 5.2 Scale factors used for the MSA procedure and the corresponding spectral accelerations.

Probability of exceedance	Scale factor	Sa($T^*=0.27$ s) g	Scale factor	Sa($T^*=0.38$ s) g	Scale factor	Sa($T^*=0.50$ s) g	Scale factor	Sa($T^*=0.66$ s) g	Scale factor	Sa($T^*=0.82$ s) g
60% in 50 years	0.47	0.10	0.41	0.07	0.38	0.05	0.41	0.04	0.40	0.03
45% in 50 years	0.75	0.16	0.71	0.12	0.77	0.10	0.72	0.07	0.67	0.05
30% in 50 years	1.00	0.21	1.00	0.17	1.00	0.13	1.00	0.10	1.00	0.07
15% in 50 years	1.41	0.30	1.41	0.24	1.54	0.20	1.45	0.14	1.48	0.11
13% in 50 years	0.85	0.38	0.86	0.31	0.86	0.25	0.82	0.18	0.82	0.14
10% in 50 years	1.00	0.45	1.00	0.36	1.00	0.29	1.00	0.22	1.00	0.17
7% in 50 years	1.14	0.51	1.14	0.41	1.21	0.35	1.19	0.26	1.23	0.21
6% in 50 years	0.89	0.58	0.87	0.47	0.91	0.40	0.89	0.30	0.90	0.24
5% in 50 years	1.00	0.65	1.00	0.54	1.00	0.44	1.00	0.34	1.00	0.27
4% in 50 years	1.15	0.75	1.17	0.63	1.20	0.53	1.22	0.41	1.21	0.32
3% in 50 years	1.30	0.85	1.31	0.71	1.36	0.60	1.39	0.47	1.39	0.37
2.3% in 50 years	0.92	0.95	0.92	0.80	0.92	0.67	0.91	0.52	0.91	0.42
2% in 50 years	1.00	1.03	1.00	0.87	1.00	0.73	1.00	0.57	1.00	0.46
1.7% in 50 years	1.08	1.10	1.08	0.94	1.08	0.79	1.10	0.63	1.09	0.50
1.5% in 50 years	1.06	1.20	1.16	1.01	1.16	0.85	1.20	0.69	1.17	0.54
1.3% in 50 years	1.16	1.35	1.32	1.15	1.30	0.95	1.39	0.80	1.37	0.63
1.1% in 50 years	1.31	1.50	1.49	1.30	1.44	1.05	1.57	0.90	1.54	0.71
0.9% in 50 years	1.45	1.60	1.61	1.40	1.58	1.15	1.71	0.98	1.70	0.78
0.7% in 50 years	1.55	1.70	1.72	1.50	1.71	1.25	1.83	1.05	1.83	0.84
0.5% in 50 years	1.65	1.80	1.84	1.60	1.85	1.35	1.97	1.13	1.96	0.90

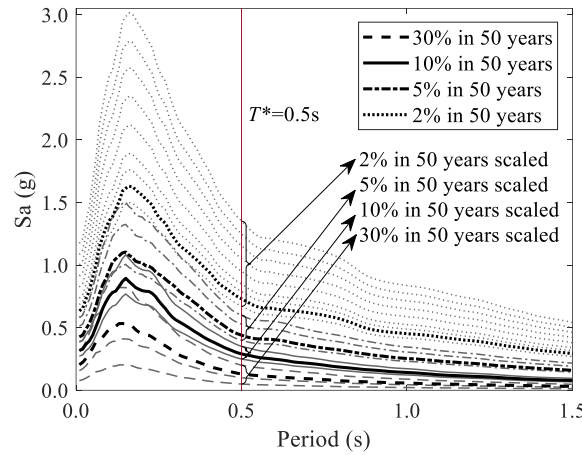


Figure 5.7 Average acceleration response spectra of the 40 ground motions for the case of $T^* = 0.5s$ scaled according to Table 5.2.

5.3.4 Probabilistic capacity model

Collapse of the buildings is associated to the occurrence of one of the following three conditions: 1) exceedance of the ultimate chord rotation in a column (flexural failure), 2) shear failure in a column or 3) numerical failure of the model. Both conditions 1) and 2) involve component-level failure mechanisms indicating the loss of load bearing capacity of a column. Both conditions are associated with the onset of localised collapse, which likely act as a trigger for global collapse. The ultimate chord rotation and shear capacities are evaluated for each column according to the models proposed by Grammatikou et al. (2018) and Biskinis et al. (2004), respectively, and are defined in Equations (5.2), (5.3), (5.4) and (5.5). Numerical failure occurs when interstorey drifts increase without bound and lead to non-convergence of the numerical algorithm and thus to the termination of the analysis. The development of a soft storey (a system-level failure mechanism) is a typical example that often leads to numerical failure since it usually generates dynamic instability of the model. The modelling approaches selected to simulate the nonlinear behaviour of the buildings (see Section 5.3.1.2) are able to prevent early numerical failures and to ensure that the analysis will converge even for cases of large spread of plasticity. Finally, since the selected buildings are not seismically designed, a weak column–strong beam behaviour is expected to occur and beam failure is not analysed.

The selected model defining the ultimate chord rotation capacity for members that were not seismically designed is given by (Grammatikou et al. 2018):

$$\theta_u = \theta_y + \theta_u^{pl} \quad (5.2)$$

where,

$$\theta_y = \varphi_y \frac{L_s + z}{3} + \frac{\varphi_y d_b f_y}{8\sqrt{f_c}} + 0.0019 \left(1 + \frac{0.625h}{L_s} \right) \quad (5.3)$$

and

$$\theta_u^{pl} = 0.016 (\min(50; f_c))^{0.1} 0.2^{\min(0.7; v)} \left(\frac{\max(0.01; \omega')}{\max(0.01; \omega_{tot} - \omega')} \right)^{0.25} \left(\min(9; \frac{L_v}{h}) \right)^{0.35} 24^{(\alpha \rho_s f_{yw} / f_c)} \quad (5.4)$$

In Equations (5.2) to (5.4), θ_y and θ_u^{pl} correspond to the yielding rotation and the plastic chord rotation, respectively. The factor φ_y is the yielding curvature, L_v is the shear span, z is the length of the internal lever arm (considered equal to $0.9 \times d$, with d being the structural depth), d_b is the tension bar diameter and h is the section depth. Parameter v is the normalised axial force, ω_{tot} and ω' are the mechanical reinforcement ratios of the tension and compression longitudinal reinforcements, respectively, α is the confinement effectiveness factor, ρ_s is the ratio of the transverse steel area parallel to the direction of loading and f_{yw} is the yield stress of the transverse reinforcement equal to 500 MPa.

The selected model defining the shear capacity V_R of the members is given by (Biskinis et al. 2004):

$$V_R = \frac{h-x}{2L_v} \min(N; 0.55A_c f_c) + (1 - 0.05 \min(5; \mu_\Delta^{pl})) \left[0.16 \max(0.5; 100\rho_{tot}) \left(1 - 0.16 \min(5; \frac{L_v}{h}) \right) \sqrt{f_c} A_c + V_w \right] \quad (5.5)$$

where A_c is the area of the cross section (equal to $b_w \times d$, with b_w being the width of the section) and ρ_{tot} is the total longitudinal reinforcement ratio. The factor x represents the depth of the compression zone and the term $h-x$ is considered to be equal to $2h/3$ (Romão et al. 2010). According to Mpampatsikos et al. (2008), the shear span L_v can be taken as half of the length of the member and the compressive axial force N as the value obtained from gravity loads. The factor μ_Δ^{pl} is the chord rotation ductility demand obtained from the analysis and V_w is the transverse reinforcement contribution to shear resistance given by:

$$V_w = \rho_w b_w z f_{yw} \quad (5.6)$$

where ρ_w is the transverse reinforcement ratio.

5.3.5 Collapse fragility and risk model

As referred before, the effect of the ASI is analysed in terms of its impact on collapse risk, considered herein as being equivalent to the mean annual collapse rate λ_c ⁵ of a given structure which is defined by the integral:

$$\lambda_c = \int_0^\infty P(C|IM=im) |dH_{IM}(im)| \quad (5.7)$$

where $P(C|IM=im)$ and $dH_{IM}(im)$ are the collapse fragility and the derivative of the annual seismic hazard curve, respectively, for an intensity IM equal to im . The collapse fragility represents the probability of collapse $P(C)$ of the structure given the im , and it is herein expressed by the cumulative distribution function (CDF) of a lognormal distribution:

$$P(C|IM=im) = \Phi\left(\frac{\log(im / \theta_c)}{\beta_c}\right) \quad (5.8)$$

where Φ is the standard normal CDF with parameters $\log(\theta_c)$ and β_c . Among the different methodologies that are currently available for deriving numerical simulation-based fragility functions (e.g., see (Baker 2015; Bakalis and Vamvatsikos 2018)), the EDP-based approach is considered the most suitable in the present study. According to the EDP-based approach, the total number of analyses z_{IM} at a given IM level is partitioned into the number of analyses that lead to collapse c_{IM} and the complementary number of analyses for which collapse doesn't occur. Subsequently, the ratios indicating the probability that a record will cause collapse of the structure at a given IM level:

$$p_{IM} = \frac{c_{IM}}{z_{IM}} \quad (5.9)$$

are determined for each IM level. By considering the collapse and no collapse observations produced by each record to be independent, the binomial distribution can be

⁵ Assuming that both hazard and fragility are memoryless, collapse can be considered to be a Poisson process and the annual probability of collapse, i.e. the risk of collapse, can be derived by: $P_c = 1 - e^{-\lambda_c}$, which in case of small collapse rates, can be considered equal to the collapse rate without significant error.

used to express the probability of having z_{IM} collapses in c_{IM} records at each IM level. The collapse fragility function parameters of Equation (5.8) can then be determined using the maximum likelihood estimation procedure suggested by Baker (2015). It should be noted that this procedure for deriving the parameter estimates assumes that all observations are independent. Although this assumption holds true for a given IM level since only unique records are used, when the same ground motions are used for multiple IM levels this independency condition is violated. However, as referred by Baker (2015), violating this condition only causes minor numerical discrepancies which allows the issue to be neglected in the present study. Similarly, possible violations introduced by the use of more than one ASIs of the same ground motion for a given IM level are also neglected. As previously stated, the IM for the present study is defined by $Sa(T^*)$ and 20 IM levels are used to evaluate the fragility function. The fragility function is then numerically integrated with the derivative of the annual seismic hazard curve dH_{IM} as per Equation (5.7) yielding the mean annual collapse rate of the structure (simply termed collapse risk hereon).

5.3.5.1 Sources of uncertainty considered when assessing collapse risk

Sources of uncertainty related only to the seismic input are taken into account, while the rest of the parameters involved, e.g. material properties, dimensions and member capacities, are considered with their mean values. More specifically, uncertainty stems from record-to-record variability and ASI-to-ASI variability, i.e. variability of the obtained parameters related to the finite number of records and of ASIs used to determine the demand distribution. It can be thus said that the obtained fragility function parameters are estimates $\{\hat{\theta}_c, \hat{\beta}_c\}$ of the true parameters $\{\theta_c, \beta_c\}$ due to the finite number of records (Baker and Cornell 2008; Iervolino 2017) and the finite number of ASIs. Following the same rationale, the risk values that are obtained are estimates $\hat{\lambda}_c$ of the true risk λ_c ⁶. The uncertainty of the estimates will be examined using the proposed *regrouping* procedure described in Section 5.3.2. The proposed procedure examines both the effect of using a finite ground motion group and a finite number of ASIs, determining the variability of the parameter estimates as well as the bias of the expected value with respect to a

⁶ Parameter values referred as “true” are considered as such given the assumptions made regarding aspects such as structural or capacity modelling. These additional sources of uncertainty were not considered to better isolate the effect of the ASI. However, if they were considered, values identified as “true” in the presented study would have to be seen as estimates also.

reference value. Although the true parameters are inherently unknown, the reference parameters considered in the current study were obtained using 40 records and 12 ASIs. The parameters examined are the collapse fragility parameters and the corresponding collapse risk.

5.4 Fragility and collapse risk assessment results

5.4.1 Initial remarks about the approach followed to analyse the results

The collapse fragility curves are determined according to Equation (5.8) for each building and for all 100 groups of each combination of ground motion group size n (≤ 35) and number of ASIs. Similarly, for each building and for the ground motion group size n equal to 40, one fragility curve is determined considering all 12 ASIs, which corresponds to the reference case. The total number of dynamic analyses z_{IM} involved in Equation (5.9) for a given IM level is the product between the number of records n of the group and the number of ASIs. The collapse risk $\hat{\lambda}_c$ is subsequently calculated according to Equation (5.7) for all the previously referred cases. To facilitate the discussion of the results, an illustrative example is presented in Figure 5.8 showing the results corresponding to the 100 ground motion groups of size n equal to 20 and 1 ASI for the 3-R building. The 100 generated fragility curves are presented in Figure 5.8 (a), highlighting the parameter $\hat{\theta}_c$ of each curve with the black dots. An empirical distribution function is assigned to the $\hat{\theta}_c$ values and a kernel density smoothing approach is used to define a kernel probability density function (kpdf). The kpdf is also shown in Figure 5.8 (a), while the corresponding kernel cumulative distribution function (KCDF) is shown in Figure 5.8 (b). Similarly, a KCDF is also defined for the 100 values of $\hat{\beta}_c$, which is shown in Figure 5.8 (c). Finally, Figure 5.8 (d) shows the KCDF that is then obtained for the 100 values of $\hat{\lambda}_c$, which are determined by integrating the 100 fragility curves with the derivative of the seismic hazard curve (also shown in Figure 5.8 (a) for completeness). KCDFs are presented to enhance the visual evaluation and comparison of the distributions of the previously defined parameters obtained using different combinations of n and ASIs, as will be shown in the following Section.

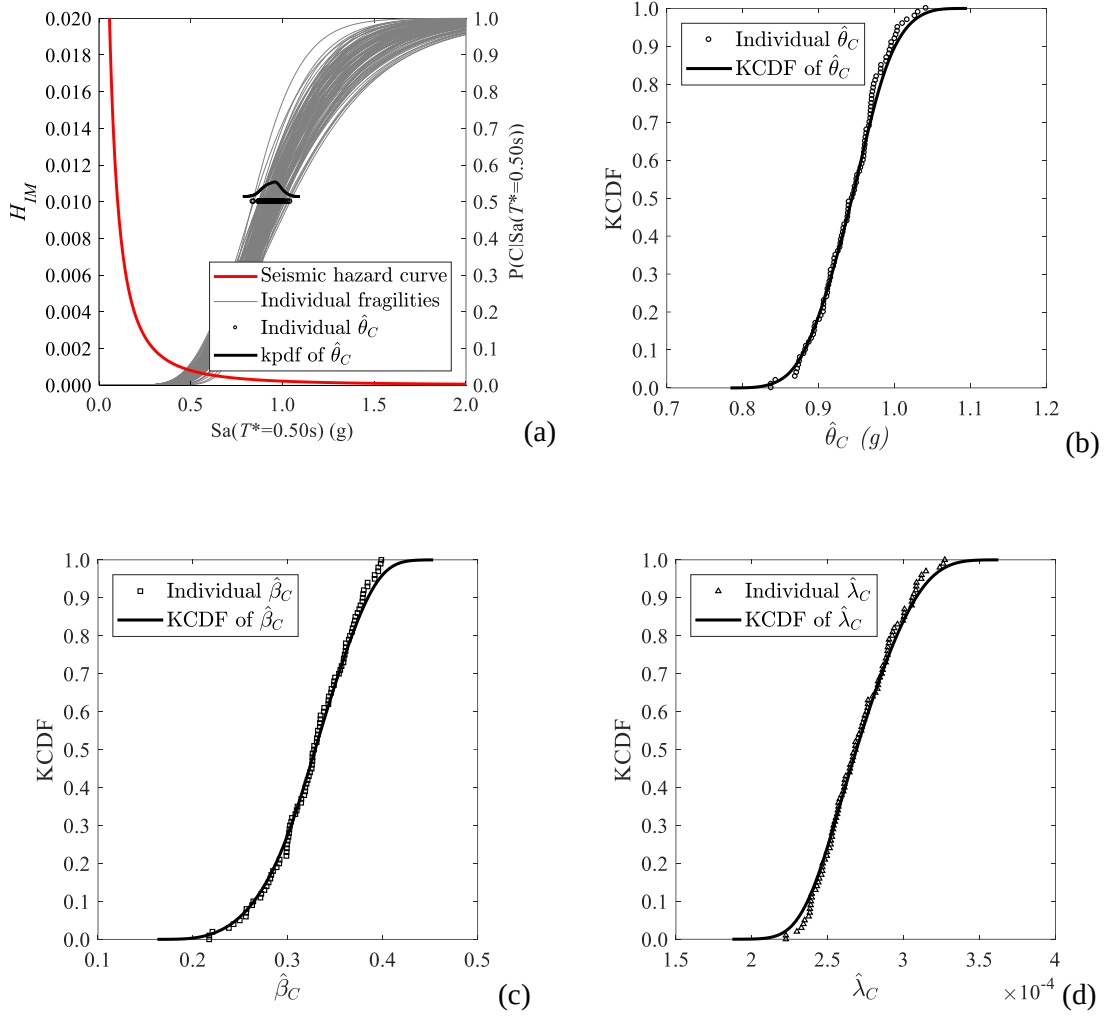


Figure 5.8 The 100 individual fragility curves of the 3-R building for size $n = 20$ and 1 ASI along with the seismic hazard curve (a), the KCDF of $\hat{\theta}_C$ (b), the KCDF of $\hat{\beta}_C$ (c) and the KCDF of $\hat{\lambda}_C$ (d).

Although the main purpose of the current study is to analyse the effect of the ASI on collapse risk, relevant findings addressing the intermediate steps, i.e. the collapse fragilities, are also presented since they provide the basis of the final conclusions. In the following, the notations $\hat{\theta}_{C(n\_ASI)}$, $\hat{\beta}_{C(n\_ASI)}$ and $\hat{\lambda}_{C(n\_ASI)}$ will be used to refer to the $\hat{\theta}_C$, $\hat{\beta}_C$ and $\hat{\lambda}_C$, respectively, of a specific combination of ground motion group size n and number of ASIs.

5.4.2 Preliminary findings about the influence of the ASI on the collapse fragility and the collapse risk

A preliminary evaluation of the effect of the ASI when considering different ground motion group sizes n is performed by comparing the previously presented KCDFs

obtained for different numbers of ASIs. The KCDFs of the 5-Ir building for $\hat{\theta}_c$, $\hat{\beta}_c$ and $\hat{\lambda}_c$ are shown in Figures 5.9, 5.10 and 5.11, respectively, considering three sizes n (35, 25 and 15) and a number of ASIs from 1 to 12. Results for n equal to 35 are representative of larger ground motion group sizes (e.g. 30, 35), while results for n equal to 15 are more appropriate to characterise smaller ground motion group sizes (e.g. 10, 15). The intermediate case of n equal to 25 is also included for completeness. Considering the case of 12 ASIs as the more exact case for a given size n ($\hat{\theta}_{c(n_{12})}$, $\hat{\beta}_{c(n_{12})}$ and $\hat{\lambda}_{c(n_{12})}$), general trends can be deduced regarding the effect of the number of ASIs according to the size n .

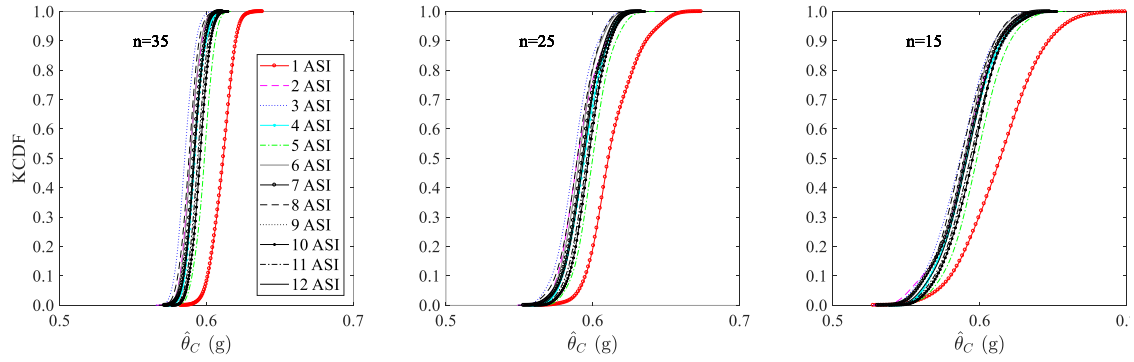


Figure 5.9 KCDF of $\hat{\theta}_c$ of the 5-Ir building for all 100 ground motion groups of n equal to 35, 25, 15 and for 1 to 12 ASIs.

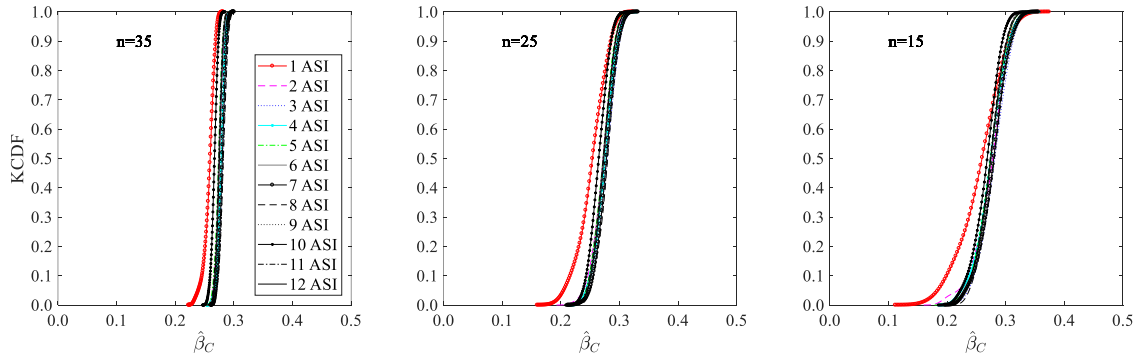


Figure 5.10 KCDF of $\hat{\beta}_c$ of the 5-Ir building for all 100 ground motion groups of n equal to 35, 25, 15 and for 1 to 12 ASIs.

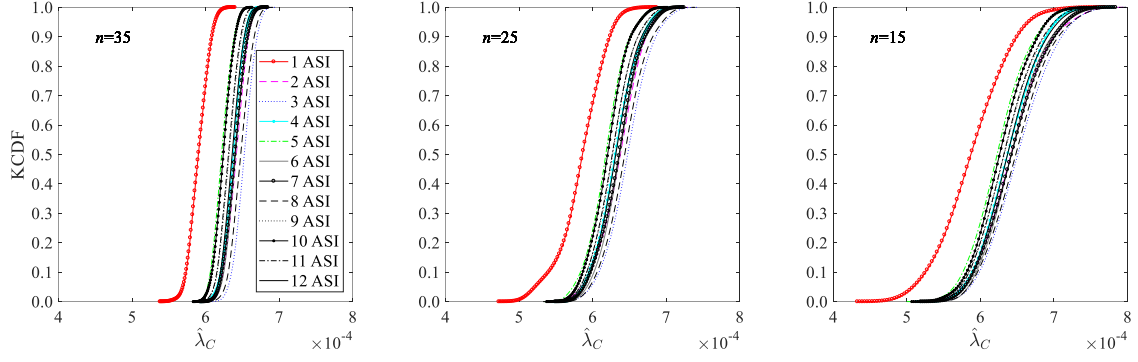


Figure 5.11 KCDF of $\hat{\lambda}_c$ of the 5-Ir building for all 100 ground motion groups of n equal to 35, 25, 15 and for 1 to 12 ASIs.

Focusing on $\hat{\theta}_c$, it is observed that its variability decreases for all ASIs when the ground motion group size n increases (i.e. the KCDFs become more vertical). However, for a given size n , the variability of $\hat{\theta}_c$ is not significantly influenced by the use of different numbers of ASIs (i.e. the KCDFs have the same shape and are relatively close to each other), except in the case when only 1 ASI is involved. For this building (5-Ir), observations exhibit a larger spread, especially for smaller ground motion group sizes. It is also noted that increasing the number of ASIs, the distribution of $\hat{\theta}_c$ oscillates around the more refined case (12 ASIs), regardless of the size n . Again, the largest (always leftward for the 5-Ir building) shift occurs for the case where only 1 ASI is involved. Similar remarks can be made for the distribution of $\hat{\beta}_c$ with respect to the effect of the size n and the number of ASIs, i.e. the decrease of the variation of $\hat{\beta}_c$ for larger values of n and simultaneously the shift of the distribution of $\hat{\beta}_c$ when a different number of ASIs is involved. Nevertheless, the shift in this case appears to be much smaller. Finally, with respect to $\hat{\lambda}_c$, which is a combination of the previous two parameters and of the seismic hazard, the number of ASIs and the size n appear to have similar effects on the KCDFs as those previously discussed for $\hat{\theta}_c$, indicating that those effects propagate to $\hat{\lambda}_c$.

To complement these results, similar plots are presented in Figure 5.12 for the rest of the examined buildings, illustrating the KCDFs of $\hat{\lambda}_c$ for an intermediate ground motion group size n equal to 25 and for all the numbers of ASIs. The results of Figures 5.11 and 5.12 show that considering a different number of ASIs has a larger effect on the 3-R, 4-Ir and the 5-Ir buildings, while hardly influencing the rest of the buildings. This observation

goes against the intuitive expectation that the larger the irregularity of the structural configuration, the larger the influence of the ASI should be. In fact, the correlation between the effect of the ASI and the structural configuration appears to be random and more likely to depend on the vibration periods of the building as well as on the ground motion characteristics. The results obtained for the 3-R and the 5-Ir buildings are further analysed in the following section since their collapse risk was found to exhibit larger variations when considering a different number of ASIs. The results obtained for the remaining buildings are omitted for the sake of brevity.

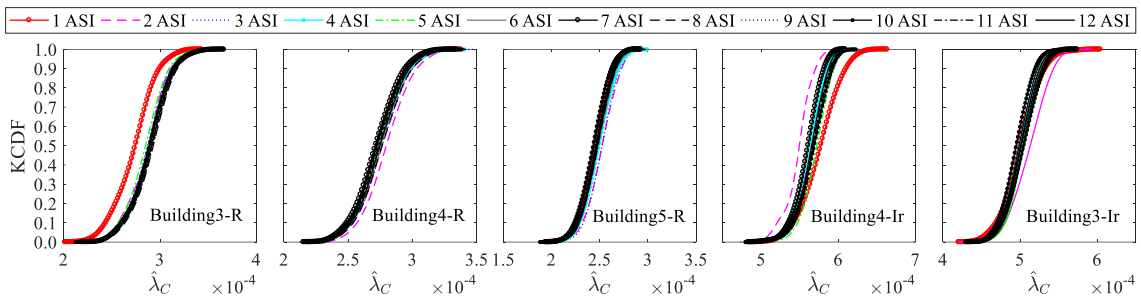


Figure 5.12 KCDFs of $\hat{\lambda}_c$ of the 3-R, 4-R, 5-R, 3-Ir and 4-Ir buildings for all 100 ground motion groups of n equal to 25 and for 1 to 12 ASIs.

5.4.3 Overall effect of the ASI on collapse risk

The influence of considering different number of ASIs on $\hat{\lambda}_c$ was illustrated in the previous Section for different values of the ground motion group size n . To quantify more clearly this influence, Figures 5.13 and 5.14 present the evolution of the median and of the coefficient of variation (CoV) of $\hat{\lambda}_c$ with the increase in the number of ASIs, for different ground motion group sizes n and for the 3-R and 5-Ir buildings, respectively. The median values of $\hat{\lambda}_c$ that are represented hereon are normalised by the reference risk value $\hat{\lambda}_{c(40\_12)}$ of each building to enhance more clearly the effect of both the number of ASIs and the ground motion group size n .

The evolution of the normalised median $\hat{\lambda}_c$ presented in Figures 5.13 and 5.14 exhibits larger fluctuations when a low number of ASIs is involved and, as this number increases, ends up converging to a specific value that depends on the size n . The initial fluctuation reflects the combined effect of both the number of ASIs and the size n on the median $\hat{\lambda}_c$ and can yield values that have a variation larger than +/-5% of the reference value.

Nevertheless, in all buildings, the variation is always less than $\pm 10\%$. When increasing the number of ASIs for a given ground motion group size n , the normalised median $\hat{\lambda}_C$ converges towards the value defined by $\hat{\lambda}_{C(n_{12})}$, i.e. the median $\hat{\lambda}_C$ obtained when considering the maximum number of ASIs. Convergence to $\hat{\lambda}_{C(n_{12})}$ is assumed to be achieved when the ratio $\hat{\lambda}_{C(n_{ASI})} / \hat{\lambda}_{C(n_{12})}$ is in the range between 0.95 and 1.05. The number of ASIs that marks the onset of this convergence is shown in Figures 5.13 and 5.14 by the vertical grey dashed line and appears to be 2 ASIs for both buildings. In light of these results, it is thus inferred that increasing the number of ASIs above a specific value only has negligible effects on the median $\hat{\lambda}_C$. Moreover, it is observed that, in most cases, the normalised $\hat{\lambda}_{C(n_{12})}$ values are close to, but not equal to, one, even though $\hat{\lambda}_{C(n_{12})}$ fully accounts for the effect of the ASI. This result is seen to derive from the fact that $\hat{\lambda}_{C(n_{12})}$ is only an estimate of the reference risk value $\hat{\lambda}_{C(40_{12})}$, due to the lower number of ground motions involved, thus reflecting the independent effect of the ground motion group size. Therefore, it can be concluded that bias resulting from the use of a low number of ground motions cannot be compensated by increasing the number of ASIs (which also increases the number of runs). This bias is generally larger for smaller ground motion group sizes n and may lead to either an over- or underestimation of the reference value $\hat{\lambda}_{C(40_{12})}$. The largest biases found across the six buildings are those observed in Figure 5.13 for n equal to 10. For all the other cases, the median $\hat{\lambda}_{C(n_{12})}$ is always within $\pm 5\%$ of the reference $\hat{\lambda}_{C(40_{12})}$.

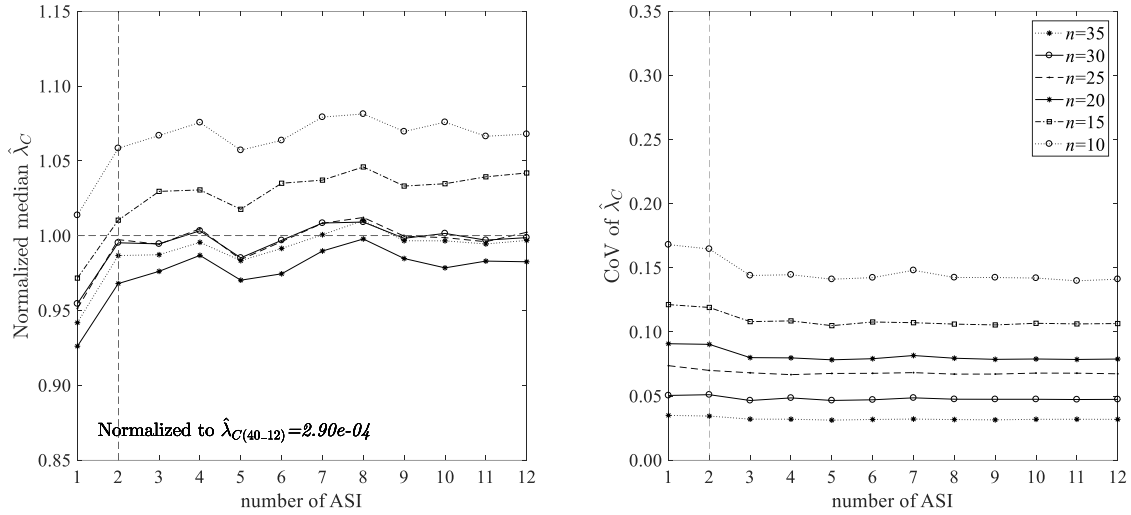


Figure 5.13 The evolution of the median $\hat{\lambda}_C$ normalised by $\hat{\lambda}_{C(40-12)}$ (left) and the CoV of $\hat{\lambda}_C$ (right) with the number of ASIs for different ground motion group sizes n for the 3-R building.

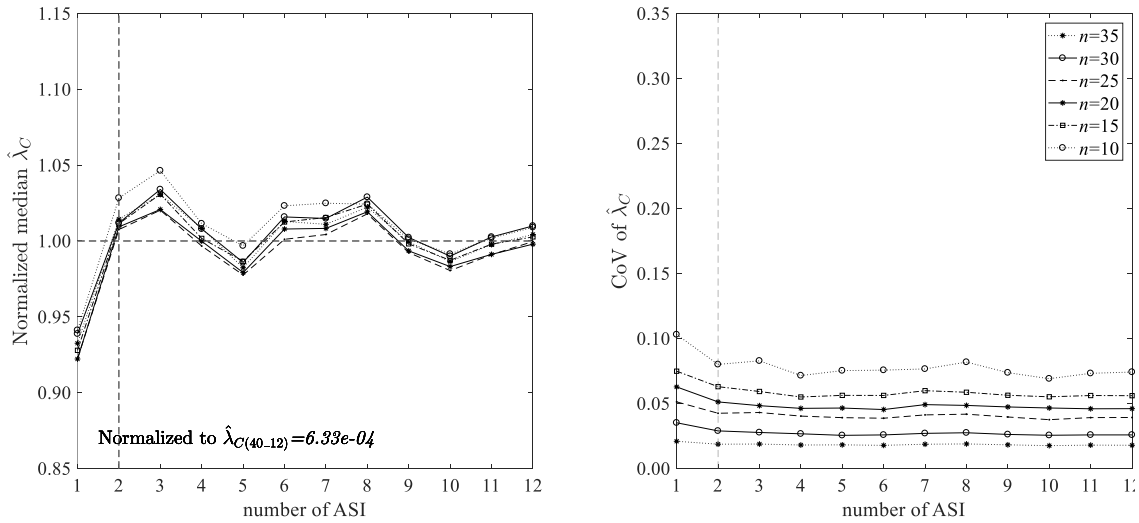


Figure 5.14 The evolution of the median $\hat{\lambda}_C$ normalised by $\hat{\lambda}_{C(40-12)}$ (left) and the CoV of $\hat{\lambda}_C$ (right) with the number of ASIs for different ground motions group sizes n for the 5-Ir building.

The variation of $\hat{\lambda}_C$ is represented by the CoV of $\hat{\lambda}_C$ whose evolution with the number of ASIs and the ground motion group size is shown on the right graph of Figures 5.13 and 5.14. The influence of the ASI on the CoV of $\hat{\lambda}_C$ seems to be negligible in most cases, in agreement with the preliminary remarks of Section 5.4.2. In particular, for all buildings and for sizes n greater than 20, the number of ASIs has practically no effect on the CoV

of $\hat{\lambda}_c$, while for ground motion groups of size 10, 15 and 20 using 1 to 3 ASIs seems to slightly increase the CoV of $\hat{\lambda}_c$. Contrary to the effect of the number of ASIs, the size n significantly affects the variation of $\hat{\lambda}_c$ since a larger value of the CoV is obtained when n is reduced. Quantitatively, however, it is noted that all buildings exhibit a small variation of $\hat{\lambda}_c$, as the CoV of $\hat{\lambda}_c$ is lower than 10% in most cases. For cases involving smaller ground motion group sizes, the CoV of $\hat{\lambda}_c$ can be larger than 10%-15% but it is always well below 20%. This low level of variability can be the result of having early cases of building collapse due to shear failure but also of having more severe criteria for ground motion record selection.

Even though the variation of $\hat{\lambda}_c$ is only marginally influenced when the effect of the number of ASIs is neglected (i.e. when only 1 ASI is considered), the median $\hat{\lambda}_c$ may experience a bias of more than 5% in that case. The consequences of this observation and particularly of its effect combined with the variation due to the finite ground motion group size n are illustrated in Figures 5.15 and 5.16. The boxplots of Figures 5.15 and 5.16 provide a clear representation of the distribution of the 100 $\hat{\lambda}_c$ values obtained for each ground motion group size n and for three specific cases of the number of ASIs, for the 3-R and 5-IR building, respectively. The three cases of the number of ASIs that are presented for each building correspond to: a) 1 ASI, which represents the current practice, b) 12 ASIs, which is the most accurate/discretised scenario for each ground motion group size n in the current study, and c) 2 ASIs, which is the lowest number of ASIs that was seen to be adequate to represent the median $\hat{\lambda}_{C(n\_12)}$ based on the results of Figures 5.13 and 5.14. The fourth boxplot shown in Figures 5.15 and 5.16 for each ground motion group size, noted as 1_{rnd} , is discussed later and is only presented herein due to length restriction reasons. All the represented values are normalised to the reference $\hat{\lambda}_{C(40\_12)}$ value of each building, the whiskers of the boxplots represent the total range of the $\hat{\lambda}_c$ observations and the maximum percentage of over- and underestimation of $\hat{\lambda}_{C(40\_12)}$ is plotted near the whiskers.

For small ground motion group sizes ($n = 10$ or 15), the amount of over- or underestimation due to both the ground motion group size and the use of only 1 ASI (boxplot of $\hat{\lambda}_{C(10\_1)}$, $\hat{\lambda}_{C(15\_1)}$) is generally high. It is higher than 20% in all cases and can

be up to 39%, as shown in Figure 5.15 for the 3-R building. For the same ground motion group sizes ($n = 10$ or 15), considering a larger number of ASIs (i.e. 12) does not necessarily lead to better results, given the bias exhibited by the median $\hat{\lambda}_{C(n_{12})}$. The overestimation of the reference value can be up to 42% and 39% when 12 and 1 ASI are used, respectively, and, at the same time, the underestimation may reach 24% and 31% when 12 and 1 ASI are used, respectively. Hence, the effect of the small ground motion group size appears to govern the accuracy of the estimate of $\hat{\lambda}_c$, suggesting the need to use ground motion groups with more than 15 records. This suggestion is in agreement with similar studies (Lagaros 2010a; Lagaros 2010b) that focused on different demand parameters.

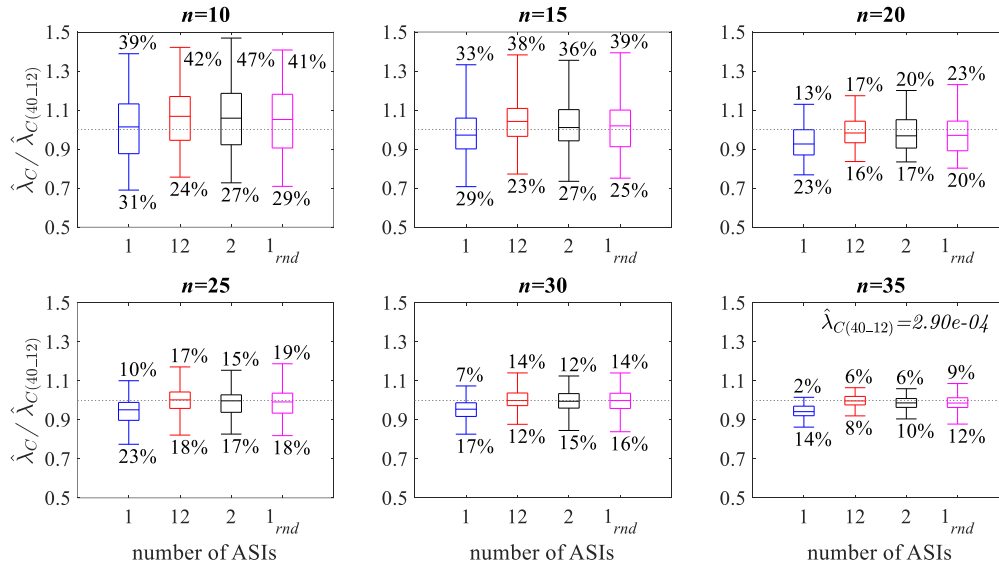


Figure 5.15 Boxplots of $\hat{\lambda}_{C(n_{1})}$, $\hat{\lambda}_{C(n_{2})}$, $\hat{\lambda}_{C(n_{12})}$ and $\hat{\lambda}_{C(n_{1\_rnd})}$ normalised by $\hat{\lambda}_{C(40-12)}$ for the 3-R building.

For larger ground motion group sizes, the variation around the median $\hat{\lambda}_c$ remains unaltered with the increase in the number of ASIs, in agreement with the CoV values presented in Figures 5.13 and 5.14. Furthermore, the bias of the median $\hat{\lambda}_{C(n_{12})}$ becomes negligible since the normalised median $\hat{\lambda}_{C(n_{12})}$ is almost unity. Accounting only for the variation of $\hat{\lambda}_c$ due to the limited ground motion group size, i.e. using all 12 ASIs, results in under- or overestimation of the $\hat{\lambda}_{C(40-12)}$ whose magnitude is between 4% and 18% and reduces with the increase of the size n . Neglecting the effect of the number of ASIs, however, results in a variation of $\hat{\lambda}_c$ around a biased median $\hat{\lambda}_{C(n_{1})}$, therefore leading to

larger levels of over- or underestimation with respect to the reference value. An extreme case of this effect can be seen to occur for the 5-Ir building (Figure 5.16) and n equal to 35. Even though the variation for all different numbers of ASIs is small, i.e. the CoV of $\hat{\lambda}_c$ is less than 5% (Figure 5.14), the oscillation of $\hat{\lambda}_c$ around a biased median leads to a larger underestimation of $\hat{\lambda}_{C(40\_12)}$ with zero probability of attaining the reference risk value. This underestimation, although small in absolute terms, is three times the level of underestimation that is obtained when the effect of the ASI is considered, i.e. by involving 12 ASIs (Figure 5.14). Therefore, while considering 1 ASI is insufficient to obtain an unbiased estimate of the collapse risk, when using 2 ASIs (0° and 90°), this bias appears to be captured adequately.

Although this solution leads to adequate collapse risk estimates, by reducing the bias and the dispersion of these estimates, it requires twice the number of dynamic analyses that are involved in a situation where the effect of the ASI is not accounted for and only the ground motion group size n is used to control the fidelity of the analyses. In order to reduce the referred number of dynamic analyses while maintaining the desired ground motion group size n and considering the effect of the ASI, an alternative approach is also examined herein. This alternative approach is based on the shown sufficiency of considering 2 ASIs, i.e. 0° and 90° , and on a special property of the component selected to represent the bi-directional ground motions, i.e. the geometric mean response spectrum. The geometric mean is known to depend on the ground motion orientation. Nevertheless, a bi-directional record rotated by 90° is a new bi-directional record with a geometric mean identical to that of the original record. Consequently, in groups involving only 1 ASI, the ASIs of 0° and 90° can be used interchangeably without changing the spectral properties that are used for spectral matching. Based on this rationale, new boxplots are created using only 1 ASI randomly selecting 0° or 90° for each record within the same group. These boxplots are presented in Figures 5.15 and 5.16 for each group size and are termed 1_{rnd} , where *rnd* stands for random. It can be seen that randomizing the ASI leads to median collapse risk estimates similar to the median $\hat{\lambda}_{C(n\_2)}$, which were shown to be unbiased, but with a dispersion that is generally larger when compared to that of $\hat{\lambda}_{C(n\_2)}$. This increase of the dispersion appears to be case-specific and can be either negligible, e.g. for the 3-R building and for most sizes n shown in Figure 5.15, or it can lead to dispersion values that are more than twice those obtained using 2 ASIs, as shown in Figure 5.16 for the 5-Ir building and for sizes n equal to 30 and 35. Finally, it is worth

noting that using a random ASI (i.e. by selecting 0° or 90°) is in line with the main suggestions of Giannopoulos and Vamvatsikos (2018).

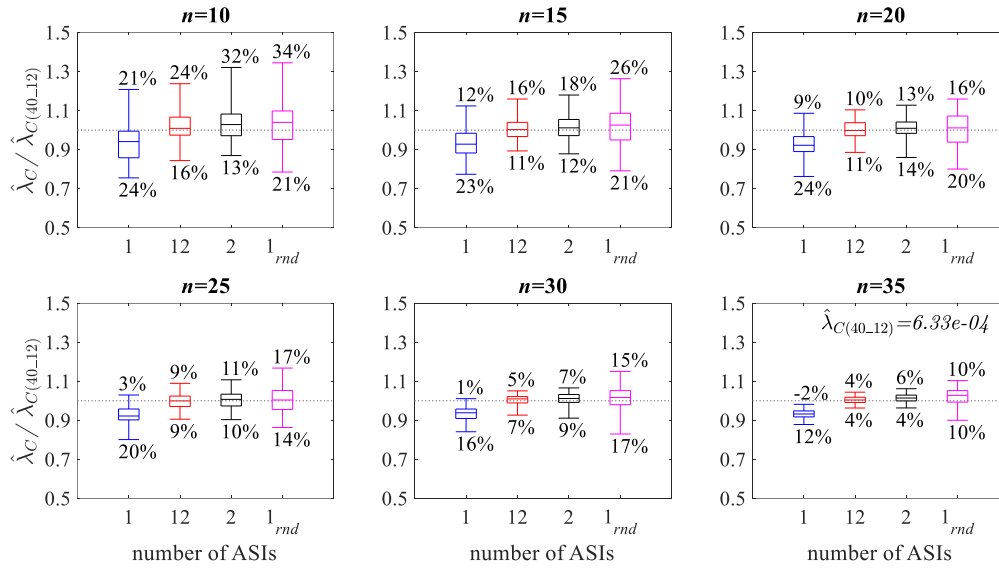


Figure 5.16 Boxplots of $\hat{\lambda}_{C(n-1)}$, $\hat{\lambda}_{C(n-2)}$, $\hat{\lambda}_{C(n-12)}$ and $\hat{\lambda}_{C(n-1_{rnd})}$ normalised by the $\hat{\lambda}_{C(40-12)}$ for the 5-Ir building.

5.5 Conclusions

The effect of the number of ASIs and of the ground motion group size on the seismic collapse risk was quantified based on the results obtained for six low- and mid-rise, regular and irregular, RC buildings with infilled frame systems subjected to MSA using far-field ground motions. The ground motion group size was found to have a significant effect on the variability of the collapse risk, causing a reduction of the collapse risk variation when it increases. The uncertainty introduced in the collapse risk estimate when 15 ground motions or less were considered was found to be significant and no improvement was observed by increasing the number of ASIs. It is therefore suggested to use a minimum of 20 ground motions to determine collapse risk. The influence of the number of ASIs on the risk variation was significantly lower when compared to that of the ground motion group size and it only appeared to have an effect when small ground motion group sizes were considered, i.e. smaller than 25. Despite the smaller effect the number of ASIs had on the variability of collapse risk estimates, using only 1 ASI introduces a bias in the median risk estimate. This bias was found to be up to 10% in some cases, leading to an over- or underestimation of the collapse risk. This effect was seen to

be independent of the ground motion group size and could be particularly important for larger group sizes where the probability of obtaining the reference risk value could actually be zero. Nevertheless, for ground motion group sizes larger than 15, this bias could be adequately corrected by considering 2 ASIs only, i.e. 0° and 90° , while using more ASIs only offered minor additional contributions.

In the overall, the results obtained for the buildings that were analysed indicate that using a group of 20 or more ground motions in combination with 2 ASIs (0° and 90°) will provide an adequate (i.e. unbiased) estimate of the collapse risk, regardless of the regularity of the building's in-plan configuration. The proposed minimum number of ground motions will prevent the occurrence of large variations around the median due to the limited sample size, while the 2 ASIs will remove the bias of the median estimate of collapse risk. An alternative approach was also explored involving only 1 ASI, randomly selecting 0° or 90° , to reduce the computational cost related to the number of dynamic analyses. The results showed that this procedure reduces the bias of the median, but it can be associated with a larger dispersion of the risk estimates. It is also noted, that the presented results were obtained considering only far-field effects. Furthermore, the current research focused only on uncertainties related to the seismic action, while additional uncertainties, such as structural modelling uncertainties and capacity model error, were disregarded. Although the main conclusions drawn from the presented results are not expected to change, the evaluation of the effect of the ASI on the collapse risk including these additional effects should be the focus of future research.

Chapter 6

6 Are seismic losses affected by the angle of seismic incidence?

Based on:

Skoulidou, D., & Romão, X. (2020). Are seismic losses affected by the angle of seismic incidence?. *Bulletin of Earthquake Engineering*. Submitted for publication

Abstract

The proposed study examines the propagation of uncertainty associated to the angle of seismic incidence of a group of ground motions, as well as to that of the ground motion group size, to the expected seismic loss estimates of RC buildings. Six buildings with infilled frame systems are studied, representative of non-seismically designed structures typical of southern European practice. Their monetary losses are computed using a performance-based earthquake engineering framework, which integrates the site-specific seismic hazard, structural response, damage to building components and contents, and the resulting costs (repair, demolition, reconstruction). To estimate monetary losses, an inventory of the structural and non-structural elements is compiled and existing damage and cost data/functions are employed. Seismic loss estimates account for both the probability of collapse and the probability of demolition as a result of excessive residual storey drifts. Multiple stripe analysis is performed for this purpose using numerical models capable of simulating the relevant limit states up to collapse. Results are presented in terms of expected losses conditional to the ground motion intensity and expected

annual losses, which are determined by integrating the former with the seismic hazard curve of the site. Results show that the comparative effect of the ground motion group size in the expected annual losses is much more important when compared to that of the angle of seismic incidence, whose influence is shown to be trivial.

6.1 Introduction

Seismic loss assessment methodologies have received a growing interest over the past 20 years as a method to manage the consequences of earthquakes on structures and infrastructures (e.g. see (Porter et al. 2002; Aslani and Miranda 2005; Ramirez and Miranda 2012; Günay and Mosalam 2013; Hasik et al 2018), among others). Modern Performance Based Earthquake Engineering (PBEE) concepts were first developed following high-impact earthquakes that occurred in the mid-1990s and the first generation of PBEE procedures (SEAOC 1995) established a framework that was able to integrate the performance of structures and infrastructures at several seismic hazard levels. Despite their pioneering format, early PBEE procedures involved a number of shortcomings, including the lack of connection between global (system-level) and local (element-level) performance criteria and the deterministic nature of their definition (Günay and Mosalam 2013). These issues then led to the development of the improved framework for seismic loss assessment proposed by the Pacific Earthquake Engineering Research Centre (PEER) in the early 2000s (Cornell and Krawinkler 2000) that formed the basis for most current building-specific assessment methodologies.

The PEER-PBEE framework comprises four sequential stages of analysis (hazard analysis, structural analysis, damage analysis and loss analysis) and contains provisions to perform probabilistic seismic loss assessments for individual structures (e.g. see Miranda and Aslani (2003) or Günay and Mosalam (2013) for a summarised description of the methodology). The main advantages of the framework involve the straightforward inclusion of uncertainties in all steps of the assessment procedure, as well as the ability to express the final assessment outputs in terms that are relevant for both engineers and stakeholders. In addition, performing full probabilistic seismic safety assessments of individual buildings has been further facilitated by the development of appropriate software and data. The Global Earthquake Model initiative, founded in 2009, and the PACT tool provided by FEMA P58 (2012) are examples of such contributions, where the former provides hazard and risk computation software and the latter provides fragility

and consequence functions appropriate for the US building inventory, but that can also be customised by the user (e.g. see Cardone and Perrone (2017)).

One of the possible final outputs of a seismic loss assessment procedure is often expressed by the Expected Annual Loss (EAL). The EAL can be used as an economic metric for the damaging effect of future earthquakes on a building and can be employed for multiple purposes such as evaluating the need for strengthening measures or the efficiency of a certain retrofit solution (e.g. see Sousa and Monteiro (2018)). A continuously growing number of studies have been produced using this methodology leading to benchmarking and to further improvements of the methodology (e.g. Haselton et al (2008)). Applications of this methodology to steel (Hwang and Lignos 2017) and RC buildings of various systems (Caruso et al 2019; O'Reilly et al 2018; Romano et al 2018) can be found, as well as applications to other types of structures, such as bridges (Zanini et al. 2017; Yoon et al. 2019). Nevertheless, further developments/improvements in several aspects of the methodology can still be expected in the future. In this direction, Calvi et al. (2014) discussed several limitations of the methodology, highlighting the large uncertainties related to the adopted loss model (number of damageable assemblies and distribution of repair costs) and the building replacement cost. Additionally, the influence of indirect losses has also been pointed out by Cardone et al. (2019) as a factor that requires further consideration in this methodology, emphasising the existence of additional barriers within current practice, as well as the need for additional data.

One of the variables responsible for introducing a significant amount of uncertainty in the probabilistic seismic assessment is the input seismic action. The parameters related to the seismic action are associated to the Intensity Measure (IM) used for the ground motion (GM) selection, the number of GM that are selected for the assessment (i.e. the GM group size) and the direction of application of the GMs with respect to the building (i.e. the Angle of Seismic Incidence - ASI). Studies addressing the effect of the input seismic action in the probabilistic seismic assessment procedure have mainly focussed on the PEER-PBEE methodology stages of hazard, structural and damage analysis, while relevant studies on its effect on the EAL are limited. The effect of the GM group size and the ASI on the variability of specific Engineering Demand Parameters (EDPs) was studied by Giannopoulos and Vamvatsikos (2018) and Chapter 3 of the present dissertation, where the former also addressed the effect of the IM. The collapse risk variability was also studied with respect to the GM group size (Iervolino 2017) and with

respect to the combined effect of the GM group size and the ASI (Chapter 5). To the authors' knowledge, only one study addresses the effect of the GM group size and the ASI on loss estimates, and it proposes the use of 30 GMs applied in a random ASI in order to account for the uncertainty of both effects (Lagaros 2010a).

Given the limited contributions addressing the effect of the ASI on estimates of the EAL for individual buildings, the present study provides a comprehensive analysis on the relevance of this effect using the PBEE-PEER framework. Since past research has shown that the effect of the GM group size is important for the estimation of the parameters associated to the various stages of the framework, the combined effect of the ASI and the GM group size on the EAL is examined. The analysis is based on the results obtained for six RC buildings that provide relevant insights on the optimum combination of number of ASIs and GM group size for the EAL estimation.

6.2 Methodology

The PEER-PBEE framework is applied herein for the seismic loss assessment of six existing RC buildings to analyse the effects of the GM group size and the number of ASIs involved. The framework comprises four stages of analysis (hazard analysis, structural analysis, damage analysis and loss analysis), which are connected using the total probability theorem. The end product of the framework is the probability of exceedance of a decision variable DV that is often expressed in terms of economic losses due to the repair/replacement of the structure, casualties or downtime. In the present analyses, only the first DV is examined, namely in terms of direct economic losses. Furthermore, since the purpose of the study is to determine the effect of the input seismic action, no uncertainty is considered in the economic losses and thus the expected value of the DV , $E(DV)$, is determined. The $E(DV|IM)$, which represents the $E(DV)$ conditional to the IM intensity level is given by:

$$E(DV | IM_m) = E(DV | NC \cap R, IM_m)P(NC \cap R | IM_m) + E(DV | NC \cap D)P(NC \cap D | IM_m) + E(DV | C)P(C | IM_m) \quad (6.1)$$

where the three individual loss-related components are analysed hereafter. Component $E(DV | NC \cap R, IM_m)$ expresses the expected loss of the building given that no collapse has occurred and the building can be repaired, for the occurrence of the seismic intensity

IM_m . Component $E(DV | NC \cap D)$ is the expected loss of the building given that no collapse has occurred and the building needs to be demolished, for the occurrence of the seismic intensity IM_m , and $E(DV | C)$ corresponds to the expected loss of the building when collapse occurs at the IM_m intensity. For simplicity, the expected loss corresponding to having to demolish and reconstruct the building, as well as the expected loss of the collapsed building, are taken equal to the replacement cost of the building (see Section 4.3).

The weighting factors attributed to each cost-related component in Eq. (6.1) correspond to: (i) $P(NC \cap R | IM_m)$, the probability that the building can be repaired given that no collapse has occurred at the seismic intensity IM_m , (ii) $P(NC \cap D | IM_m)$, the probability that the building should be demolished given that no collapse has occurred at the seismic intensity IM_m , and (iii) $P(C | IM_m)$, the probability of collapse of the building given the intensity IM_m . Component $P(C | IM_m)$, or the collapse fragility function, is determined herein according to Baker (2015), considering the number of analyses that lead to collapse over the total number of analysis at each IM and fitting a lognormal model over the range of IMs by the maximum likelihood estimation method. Collapse of the building is considered herein as being defined by the numerical collapse of the model.

The probability of having to demolish the building given that no collapse has occurred at a given IM is determined according to:

$$P(D | NC, IM_m) = \int_0^\infty P(D | RISD) dP(RISD | NC, IM_m) \quad (6.2)$$

where $P(D | RISD)$ is the probability of having to demolish the building conditioned on the maximum residual interstorey drift ratio (RISD) and $P(RISD | NC, IM_m)$ is the probability of exceeding a predefined level of RISD, obtained from the analysis results given the building has not collapsed for IM_m . Component $P(D | RISD)$ is assumed to follow a lognormal distribution with median of 1.5% and a logarithmic standard deviation of 0.3 (Ramirez and Miranda 2012). The terms $P(NC \cap D | IM_m)$ and $P(NC \cap R | IM_m)$ are calculated from:

$$P(NC | IM_m) = 1 - P(C | IM_m) \quad (6.3)$$

$$P(R | NC, IM_m) = 1 - P(D | NC, IM_m) \quad (6.4)$$

$$P(NC \cap R | IM_m) = P(R | NC, IM_m)P(NC | IM_m) \quad (6.5)$$

$$P(NC \cap D | IM_m) = P(D | NC, IM_m)P(NC | IM_m) \quad (6.6)$$

Finally, the expected annual value of the DV is obtained by the convolution of Eq. (6.1) with the annual seismic hazard curve:

$$E(DV) = \sum_m E(DV | IM_m) p(IM_m) \quad (6.7)$$

where $E(DV)$ corresponds to the expected annual direct economic losses in the present research, simply denoted as EAL. The results of Eqs. (6.1) and (6.7) are determined for the six buildings using bidirectional GMs in groups of size n ranging from 10 to 40, applied along 1 to 12 ASIs. GM groups of size equal to 40 are initially selected for each building in order to be compatible with a response spectrum that represents the seismic scenario. Subsequently, the groups of 40 records are regrouped to create GM groups of smaller sizes equal to 10, 15, 20, 25, 30 and 35, according to a regrouping procedure (presented in Appendix A) which takes into account the requirements needed to maintain the compatibility with the seismic scenario for all the new GM groups. The overall procedure leads to 100 groups for each combination of GM group size n and number of ASIs, and to an equal number of EAL values, which provides adequate data for statistical post-processing. The notation n_ASI will be used to refer to results obtained from a given combination of a GM group of size n applied along ASI number of angles.

6.3 Description of the selected case studies

6.3.1 General characteristics of the buildings

The analysed buildings comprise six RC buildings with infilled frame systems designed for gravity loads only and assumed to be located in Lisbon, Portugal. The building configurations range from low- to mid-rise buildings with and without in-plan irregularities, aiming to cover a wide variety of buildings. The plan view of a typical storey of the 3-storey irregular (3-Ir), the 4-storey irregular (4-Ir) and the 5-storey irregular (5-Ir) buildings is presented in Figure 6.1, along with design details. Similarly, Figure 6.2 shows the plan view of a typical storey of the 3-storey regular (3-R), the 4-

storey regular (4-R) and the 5-storey regular (5-R) buildings and the design details. Further information regarding the building characteristics, the material properties and the loading assumptions can be found in Chapter 3.

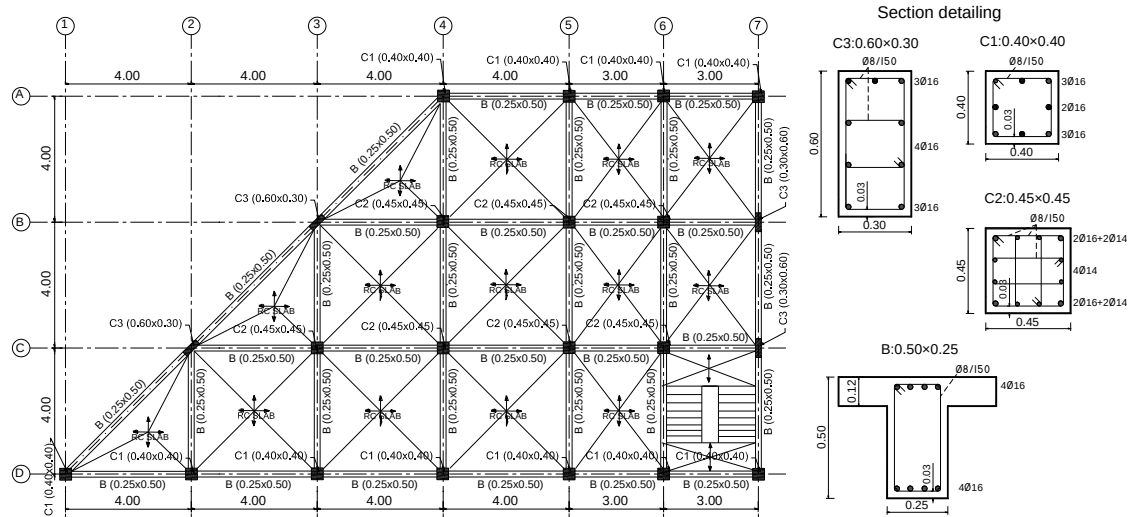


Figure 6.1. Plan view of a typical storey of the 3-Ir, 4-Ir and 5-Ir buildings and design details (all dimensions are in m).

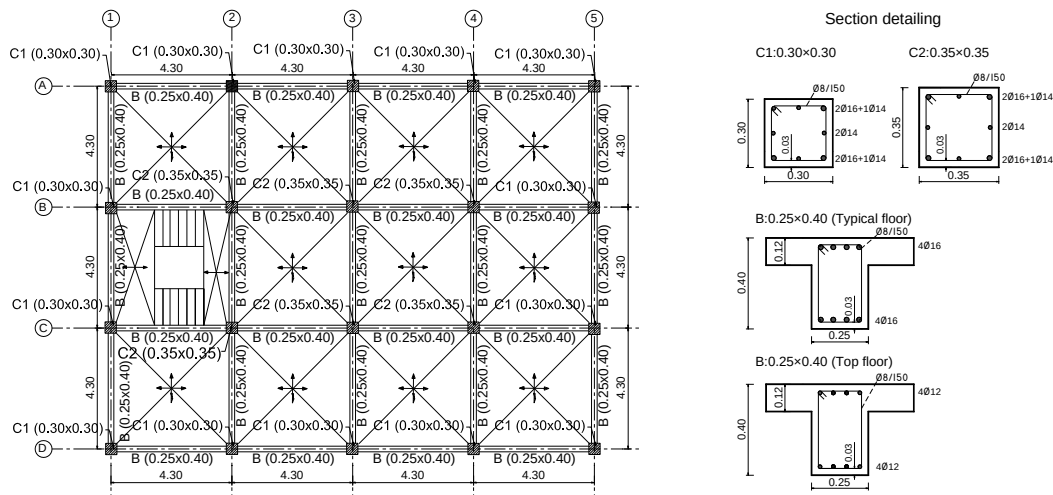


Figure 6.2. Plan view of a typical storey of the 3-R, 4-R and 5-R buildings and design details (all dimensions are in m).

6.3.2 Structural modelling

Three-dimensional models of the buildings are simulated in the OpenSees (OS) computer software (McKenna 2011) considering mean values of all material and geometrical properties. A brief presentation of the modelling approach considered for the structural and non-structural elements is provided herein, and the reader can refer to Chapter 3 for additional information.

6.3.2.1 Modelling of the structural elements

The inelastic behaviour of beams and column is simulated using a lumped plasticity approach. Rotational springs located on both ends of all structural elements are assigned with phenomenological hysteresis laws to simulate inelastic flexural behaviour. The hysteretic flexural behaviour is defined by a moment-rotation law defined by the *hysteretic* material provided by OS. The yielding moment (M_y) and the yielding rotation capacity (θ_y) of the backbone curve are computed using the proposals by Panagiotakos and Fardis (2001). The capping (θ_{cap}) and the ultimate (θ_u) rotation capacities are established according to Haselton et al. (2008) along with the corresponding capping (M_{cap}) and residual (M_r) moments, where the latter is considered to be 20% of M_{cap} (see the backbone curve in Figure 6.3(a)). The effects of stiffness, strength and unloading stiffness degradation on the hysteresis curves are considered as shown in Figure 6.3(a). The springs are connected in-series with a linear elastic interior element. A stiffness modification factor equal to 10 is applied according to Zareian and Medina (2010) to account for the effect of the series connection on the total stiffness of the element.

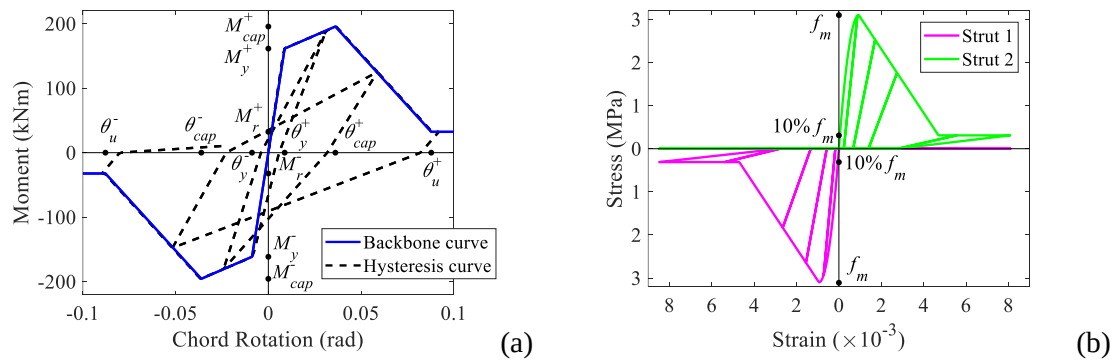


Figure 6.3. Moment-rotation backbone curve and hysteresis loop simulation for a column (a) and stress – strain relationship of the diagonal struts (in compression) representing the infills, measured in the diagonal direction (b)

6.3.2.2 Modelling of the infills

Masonry infills are considered in all peripheral frames and single diagonal struts active only in compression is selected to simulate their behaviour. Two struts (strut 1 and strut 2 as shown in Figure 6.3(b)) are used to model one infill and are connected to the beam-column joints. The equivalent area of each strut is defined using the maximum lateral force of the infill (Žarnić and Gostič 1997) and on the masonry compressive stress f_m . Maximum strength is assumed to be reached at an interstorey drift of 0.2% (Dolšek and Fajfar 2008). The parameters obtained, i.e. the maximum stress and strain, are used to define the masonry material simulated by the *Concrete01* constitutive model (Figure 6.3), (Noh et al. 2017; Mohamed and Romão 2018). The stress f_m is considered to be 3.10 MPa and all infills have a thickness of 0.15m.

6.3.2.3 Additional considerations and dynamic properties of the buildings

Geometric nonlinearities are incorporated in the form of P-Δ effects and all structures are considered fixed to the ground. The first and the second fundamental periods of vibration of the six buildings, T_1 and T_2 , are presented in Table 6.1. In Table 6.1 the terms infilled and bare structure refer to the structure with and without the simulation of the infills, respectively. Finally, the average of the first two periods of the infilled structure and the first two periods of the bare structure T^* is determined (also shown in Table 6.1) to be used for the ground motion selection. It is noted that involving periods of vibration of the bare structure in the definition of T^* , is conceptually similar to accounting for the apparent period elongation of the structure after yielding and failure of the infills.

Table 6.1. First and second mode periods of vibration of the buildings

Period\Building	3-R (s)	4-R (s)	5-R (s)	3-Ir (s)	4-Ir (s)	5-Ir (s)
T_1, T_2 (infilled)	0.31,0.25	0.41,0.31	0.52,0.39	0.21,0.15	0.29,0.20	0.37,0.26
T_1, T_2 (bare)	0.73,0.72	0.96,0.93	1.18,1.15	0.39,0.35	0.55,0.47	0.70,0.60
T^*	0.50	0.66	0.82	0.27	0.38	0.48

6.4 Fragility and consequence functions

6.4.1 Identification of the damageable inventory

In order to compute realistic loss estimations for the buildings under analysis, their architectural layouts were also developed. The architectural layouts of the buildings are provided in Figures 6.4 and 6.5 for the in-plan regular and irregular buildings,

respectively, in which the exact location of the non-structural damageable components are shown (i.e. stairs, infill walls, windows and doors), as well as the usage of each room.

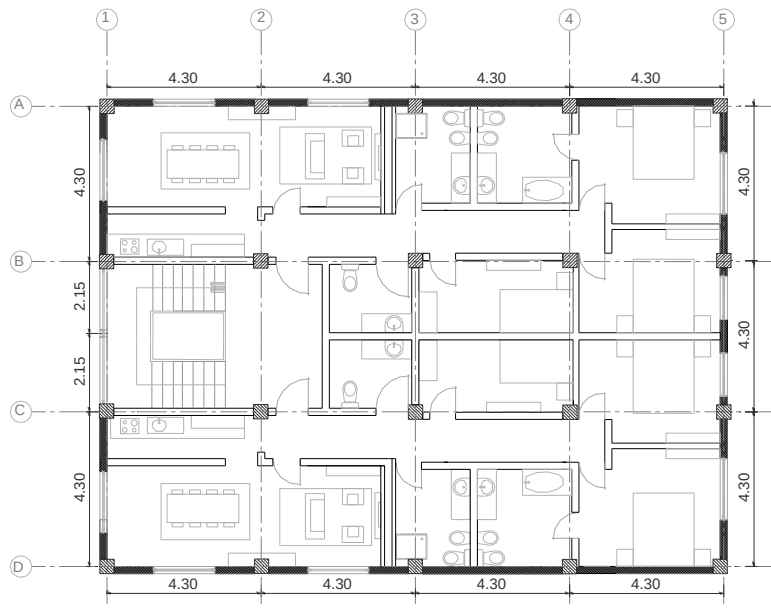


Figure 6.4 Architectural layout of the regular buildings (all dimensions are in m).

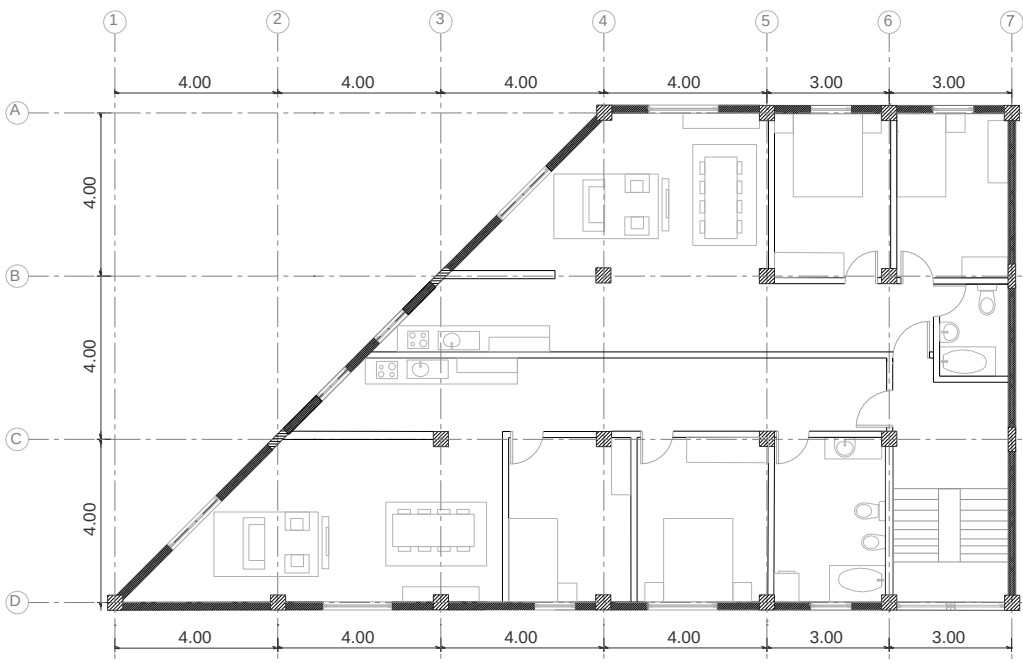


Figure 6.5 Architectural layout of the irregular buildings (all dimensions are in m).

The buildings' inventory, comprising both structural and non-structural damageable components, is presented in Tables 6.2 and 6.3 for the regular and the irregular buildings, respectively, along with the quantities per storey. For each component, the quantities are expressed in units based on the selected method of repair (presented in the next section).

The electrical wiring and piping utilities are considered as a unity within infill walls. Regarding their position in plan, all infill walls are considered to have electrical wirings, while only infill walls of bathrooms and kitchens are considered to have piping utilities.

Table 6.2 List and quantities of damageable components per storey of the regular buildings

Component	Unit	Quantity/storey	Unit	Quantity/storey	Unit	Quantity/storey
Columns	ea [†]	20	m ²	65.52		
Beams	ea	31	m ²	160.42		
Infill walls	ea	39	m ²	360.2	m	367.2
Wooden doors	ea	16	m ²	35.2		
Aluminium windows	ea	12	m ²	27		
Suspended ceiling	m ²	221.88				
Stairs	ea	1	m ²	17.14	m	8
Electrical Wiring*	ea	39				
Piping utilities*	ea	19				

*Presented as a unity in each infill wall

[†]Each

Table 6.3 List and quantities of damageable components per storey of the irregular buildings (the number in parenthesis indicates the results for the first storey)

Component	Unit	Quantity/storey	Unit	Quantity/storey	Unit	Quantity/storey
Columns	ea [†]	22	m ²	95.5 (114.6)		
Beams	ea	36	m ²	187.53		
Infill walls	ea	32	m ²	261.02 (320.4)	m	267.58 (325.38)
Wooden doors	ea	9	m ²	15.48		
Aluminium windows	ea	13	m ²	29.25		
Suspended ceiling	m ²	192				
Stairs	ea	1	m ²	12.45 (12.88)	m	5.8 (6.2)
Electrical Wiring*	ea	32				
Piping utilities*	ea	15				

*Presented as a unity in each infill wall

[†]Each

6.4.2 Damage fragility function parameters

For each damageable component, the probability of occurrence of a given damage state (DS) conditioned to a series of increasing levels of the response defined in terms of an EDP is defined by a fragility function. These fragility functions are defined for a number of sequential DSs and are assumed to be expressed by lognormal distribution functions. The EDPs selected to express damage depend mainly on literature availability and different EDPs are selected to associate damage for the different damageable components. For structural components such as columns and beams, the maximum section rotation is used, while for the stairs the maximum interstorey drift ratio (ISD) is selected. For most non-structural components, e.g. infill walls, doors, electrical wiring, etc., the ISD is also used, whereas the peak floor acceleration (PFA) is only selected for the suspended ceiling. A list of the DSs used for all components, the associated description of the damage, the EDPs, the lognormal function parameters and their corresponding reference are given in Table 6.4. The damageable components presented in Table 6.2 that do not appear in Table 6.4 (i.e. doors, windows, electrical wiring and piping utilities) are located in the infill walls and their fragility function is assumed to be equal to that of the infill they belong to.

Table 6.4 Damage fragility function parameters for the components of the building.

Component	Damage State	Description of Damage	Fragility Function Parameters			Reference
			EDP used	Median	Dispersion *	
Column	DS1	Cracking ($w > 0.8\text{mm}$) Initiation of yielding	Rot (rad) [†]	0.008	0.5	Aslani and Miranda (2005), ASCE/SEI 41-17 (2017)
	DS2	Spalling	Rot (rad) [†]	0.0175	0.5	
	DS3	Buckling of bars	Rot (rad) [†]	0.043	0.5	
	DS4	Element out of position	Rot (rad) [†]	0.058	0.5	
Beam	DS1	Cracking ($w > 0.8\text{mm}$) Initiation of yielding	Rot (rad) [†]	0.01	0.5	Aslani and Miranda (2005), ASCE/SEI 41-17 (2017)
	DS2	Spalling	Rot (rad) [†]	0.02	0.5	
	DS3	Buckling of bars	Rot (rad) [†]	0.056	0.5	
	DS4	Element out of position	Rot (rad) [†]	0.062	0.5	

Table 6.4 Damage fragility function parameters for the components of the building.
(Continued)

Component	Damage State	Description of Damage	Fragility Function Parameters			Reference
			EDP used	Median	Dispersion *	
Infill interior w/ opening	DS1	Separation of the infill from the frame	Rot (rad) [†]	0.00075	0.5	Cardone and Perrone (2017)
	DS2	Extensive diagonal cracking (1mm<w<2mm)	ISD [‡]	0.002	0.5	
	DS3	In-plane or out-of-plane collapse	ISD [‡]	0.005	0.4	
	DS4	In-plane or out-of-plane collapse	ISD [‡]	0.005	0.4	
Infill interior w/o opening	DS1	Separation of the infill from the frame	ISD [‡]	0.0015	0.5	Cardone and Perrone (2017)
	DS2	Extensive diagonal cracking (1mm<w<2mm)	ISD [‡]	0.004	0.5	
	DS3	In-plane or out-of-plane collapse	ISD [‡]	0.01	0.4	
	DS4	In-plane or out-of-plane collapse	ISD [‡]	0.01	0.4	
Infill exterior w/ opening	DS1	Separation of the infill from the frame	ISD [‡]	0.001	0.5	Cardone and Perrone (2017)
	DS2	Extensive diagonal cracking (1mm<w<2mm)	ISD [‡]	0.003	0.5	
	DS3	Corner crushing, brick spalling, detachment of plaster	ISD [‡]	0.0075	0.4	
	DS4	In-plane or out-of-plane collapse	ISD [‡]	0.0175	0.35	
Infill exterior w/o opening	DS1	Separation of the infill from the frame	ISD [‡]	0.0015	0.5	Cardone and Perrone (2017)
	DS2	Extensive diagonal cracking (1mm<w<2mm)	ISD [‡]	0.004	0.5	
	DS3	Corner crushing, brick spalling, detachment of plaster	ISD [‡]	0.01	0.4	
	DS4	In-plane or out-of-plane collapse	ISD [‡]	0.0175	0.4	

Table 6.4 Damage fragility function parameters for the components of the building.
(Continued)

Component	Damage State	Description of Damage	Fragility Function Parameters			Reference
			EDP used	Median	Dispersion *	
Suspended Ceiling	DS1	Light damage	PFA [§] (g)	0.9	0.4	FEMA P-58 (2012)
	DS2	Extensive damage	PFA [§] (g)	1.5	0.4	
	DS3	Total collapse	PFA [§] (g)	2.2	0.4	
Stairs	DS1	Light damage	ISD [‡]	0.01	0.4	FEMA P-58 (2012)
	DS2	Extensive damage	ISD [‡]	0.0175	0.4	
	DS3	Total collapse	ISD [‡]	0.0322	0.4	

*Standard deviation of the log of the EDP data

[†]The Square root of sum of squares (SRSS) rotation is used for the columns

[‡]The ISD in the direction of the element is used

[§]The maximum PFA among the two X and Y directions is used

6.4.3 Consequence data

Each damage state, as described in Section 6.4.2, is associated to a method of repair and a corresponding monetary cost. The methods of repair used in the present study, as well as the corresponding costs, were selected in order to reflect up-to-date Portuguese practice. In order to achieve that, a series of meetings with practice engineers from several construction companies were performed, involving exchange of knowledge and experience. Table 6.5 aggregates the obtained information and the data used for the loss analysis of the examined buildings.

Table 6.5 Loss function parameters for the performance-based assessment of the buildings*

Component	Damage State	Repair Action	Unit	Unit Cost (€)	Reference
Column	DS1	Epoxy resin injection [†]	ea	300	Expert elicitation
		Coating [‡]	m ²	39	
		Paint	m ²	13.5	
	DS2	RC jacketing (10cm)	m ³	960	
		Coating	m ²	39	
		Paint	m ²	13.5	
	DS3	Replacement	ea	600	

Table 6.5 Loss function parameters for the performance-based assessment of the buildings* (Continued)

Component	Damage State	Repair Action	Unit	Unit Cost (€)	Reference
Beam	DS4	Coating	m ²	39	Expert elicitation
		Paint	m ²	13.5	
		Replacement	ea	600	
		Coating	m ²	39	
		Paint	m ²	13.5	
	DS1	Epoxy resin injection [§]	ea	300	
		Coating [†]	m ²	39	
		Paint	m ²	13.5	
	DS2	RC jacketing (10cm) ^{**}	m ²	1020	
		Coating	m ²	39	
		Paint	m ²	13.5	
	DS3	Replacement	ea	700	
		Coating	m ²	39	
		Paint	m ²	13.5	
	DS4	Replacement	ea	700	
		Coating	m ²	39	
		Paint	m ²	13.5	
Infills	DS1	Repoint mortar	m	9	Expert elicitation
		Paint ^{††}	m ²	13.5	
		Repair wire	ea	0.20·DS4	
		Repair pipes ^{‡‡}	ea	0.20·DS4	
	DS2	Treatment of the cracked zones ^{††}	m ²	27	
		Paint ^{††}	m ²	13.5	
		Repair wires	ea	0.25·DS4	
		Repair pipes ^{‡‡}	ea	0.25·DS4	
	DS3	Demolition	m ²	27	
		New wall ^{§§}	m ²	42	
		Coating ^{††}	m ²	18	

Table 6.5 Loss function parameters for the performance-based assessment of the buildings* (Continued)

Component	Damage State	Repair Action	Unit	Unit Cost (€)	Reference
		Paint ^{††}	m ²	13.5	Expert elicitation
		Replace wooden door ^{‡‡}	m ²	675	
		Replace aluminium window ^{‡‡}	m ²	1050	
		New blinds ^{‡‡}	m ²	45	
		Remove blinds ^{‡‡}	ea	81	
		Repair wires	ea	0.80·DS4	
		Repair pipes ^{‡‡}	ea	0.80·DS4	
		Demolition	m ²	27	
		New wall ^{§§}	m ²	42	
		Coating ^{††}	m ²	18	
	DS4	Paint ^{††}	m ²	13.5	
		Replace wooden door ^{‡‡}	m ²	675	
		Replace aluminium window ^{‡‡}	m ²	1050	
		New blinds ^{‡‡}	m ²	45	
		Remove blinds ^{‡‡}	ea	81	
		Replace wires	ea	285	
		Replace pipes ^{‡‡}	ea	408	
	DS1	Repair		0.12·DS3	
	DS2	Repair		0.36·DS3	
	DS3	Remove	m ²	4.5	
		New Ceiling	m ²	66	
		Coating	m ²	40.5	
		Paint	m ²	15	
Suspended Ceiling	DS1	Repair		0.30·DS3	Expert elicitation
	DS2	Repair		0.50·DS3	
Stairs	DS1	Repair		0.30·DS3	Expert elicitation
	DS2	Repair		0.50·DS3	

Table 6.5 Loss function parameters for the performance-based assessment of the buildings* (Continued)

Component	Damage State	Repair Action	Unit	Unit Cost (€)	Reference
		New RC stairs	ea	1100	
		Coating	m ²	39	
		Paint	m ²	13.5	
	DS3	Covering	m ²	45	
		Finishing	m ²	105	
		Handrail	m	105	

*All costs include administration costs and the margin of profit, applied as a uniform amplification factor equal to 1.5. Furthermore, all costs were also amplified by a factor 2 to account for the increase in prices and costs related to displaced workers in an emergency or post-disaster recovery situation. Costs do not include tax.

†Assuming 4 cracks per element end and face, with a length of approximately 0.1m and a cost of 90€/crack

‡Applied over a length of approximately 0.5m starting from the joint

§Assuming 4 cracks per element end and face (only sides), with a length of approximately 0.2m and a cost of 90€/crack

**Considering the area of the beam width and twice the beam height minus the slab height

††To be applied on both sides of the infill.

‡‡Where applicable.

§§1.3 times the value to account for brick waste during construction

For all repair actions related to extensive repair works or replacement of beams and columns, the cost of replacing the coating and the finishing of the area adjacent to the damaged element is also added. The respective costs were taken equal to 45 €/m² and 105 €/m², that include all the amplification factors presented in Table 6.5. Finally, the building replacement cost corresponds to the construction cost of a new building increased by a factor of 1.2 that accounts for demolition needs and the removal of debris. For low- to mid- rise RC frame buildings with masonry infills, an average price of 800€/m² is adopted for new buildings based on data received from the referred Portuguese practice engineers.

6.5 Seismic action and additional modelling techniques

6.5.1 Seismic hazard, ground motion selection and regrouping procedure

The GM record selection is performed using the SeleEQ software (Macedo and Castro 2017) using a Conditional Spectrum (CS) in terms of the 5% damped spectral acceleration (Baker 2010) as the target spectrum. First, the probabilistic seismic hazard analysis of the

site was performed using the open source software OpenQuake (Pagani et al. 2014) and the seismic hazard model developed within the SHARE project (Woessner et al. 2015). The GM prediction equations proposed by Atkinson and Boore (2006) and Akkar and Bommer (2010) were used with a weight of 70% and 30%, respectively. The annual seismic hazard curve H_{IM} of the benchmark site was determined at T^* for each building and is shown in Figure 6.6. Note that since T^* of the 5-Ir building is equal to 0.48s, the seismic hazard curve considered for this case is also the one determined for the 3-R building that has $T^* = 0.50$ s.

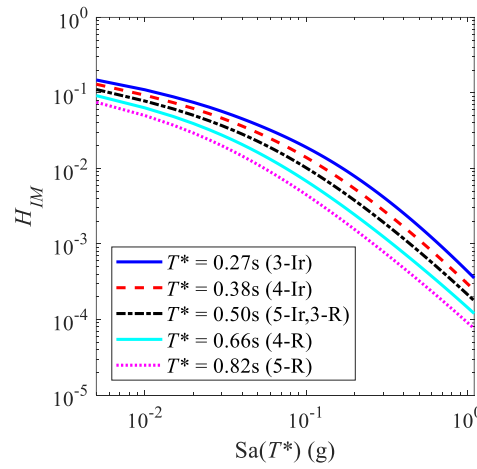


Figure 6.6. Seismic hazard curves for Lisbon and for the periods of interest

Disaggregation of the hazard was then performed for four probabilities of exceedance, i.e. 30%, 10%, 5% and 2% in 50 years, at T^* for each building. The results were next used to build four CSs using the methodology proposed by Lin et al. (2013a; 2013b; 2013c). A preliminary ground motion record selection was performed using the NGA-WEST2 database (Ancheta et al. 2014) based on seismological and strong motion parameters (see Chapter 3 for additional details). Subsequently, the final selection and scaling of groups of 40 bi-directional records (for each building and for each probability of exceedance) was carried out by ensuring compatibility between the target spectrum, i.e. the CS, and the average of the geometric means of the horizontal components of all pairs. This compatibility was ensured by minimising the difference between statistics of the target spectrum (i.e. the mean and standard deviation of the logarithms of the spectral accelerations) and the same statistics of the group, within a period range of $\min(0.1s, 0.2T^*)$ and $1.5T^*$, as described in Macedo and Castro (2017). An additional criterion was included in the process which involves minimising also the skewness of the logarithms

of the spectral accelerations to a value close to zero for the same period range. Overall, 4 groups of 40 records are selected for each building, each corresponding to a predefined probability of exceedance (see Appendix B for a complete list of the GMs). Figure 6.7 illustrates the results of the GM selection process by showing the geometric mean spectra of the selected 40 records along with the CS for the four probabilities of exceedance for the building with $T^* = 0.50$ s. Furthermore, in order to highlight the directionality characteristics of the selected groups of GMs, and particularly the fact that they are representative of far-field records, polar plots of the $Sa(T^*)$ of all groups are presented in Appendix C.

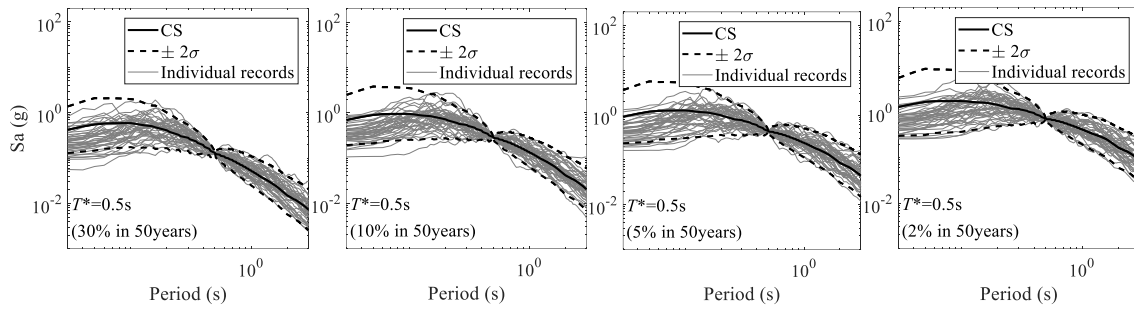


Figure 6.7. The CS with the geometric mean spectra of the 40 records for each probability of exceedance, $T^* = 0.50$ s

Each initial group of size $n = 40$ is then regrouped to create GM groups of smaller size $n = 10, 15, 20, 25, 30$ and 35 by implementing a regrouping procedure that creates groups of GMs representing a seismic scenario consistent with that of the reference group of size 40. A total number of 100 groups are created for each new group size n and a detailed description of the procedure can be found in Appendix A.

6.5.2 Probabilistic demand model

The response of the six 3D RC building models presented in Section 6.3 is determined by performing nonlinear dynamic analysis using the GM groups defined in Section 6.5.1. Following a procedure similar to the one proposed by Christovasilis et al. (2014), the four GM groups selected for each building are scaled up and down to span a total of 20 intensity levels. The aim of this procedure is to use scale factors (SF) close to unity, i.e. to use each group of GMs for probabilities of exceedance similar to the one they were selected for. Although the CS would be different for a different probability of exceedance, no significant changes are expected for close intensities. As a result, the same CS can be used for neighbouring intensities and, as an extent, the scaled groups of GMs

corresponding to these CS can be considered to be hazard compatible. The three groups of GMs corresponding to the 30%, 10% and 5% in 50 years probability of exceedance are scaled 2 to 3 times with a maximum SF equal to 1.54. The groups corresponding to the 2% in 50 years probability of exceedance are scaled 8 times with a maximum scale factor close to 2 in order to achieve maximum intensity for all buildings that corresponds to 0.5% probability of exceedance in 50 years. The spectral accelerations $S_a(T^*)$ and the corresponding SF for all groups of records are summarised in Table 6.6. All 40 GMs are applied considering 12 different ASIs, ranging from 0° to 165° in steps of 15° , resulting in a total of $(20 \text{ intensities} \times 40 \text{ GMs} \times 12 \text{ ASIs})$ 9600 nonlinear 3D analyses per building (57600 total analyses).

Sources of uncertainty related only to the seismic input are taken into account in the definition of the demand model, while the rest of the parameters involved, i.e. material properties, dimensions and member capacities, are considered with their mean values. More specifically, the considered uncertainty stems from record-to-record variability and ASI-to-ASI variability, i.e. variability of the obtained parameters related to the finite size of the GM groups and of the number of ASIs used to determine the demand distributions. As far as the overall framework is concerned, uncertainty is also involved in the definition of the fragility parameters and consequence functions of the assemblies. Although the latter has received significant attention and criticism regarding its effect on the EAL (Calvi et al. 2014), its influence is not relevant for the purpose of the present study since it is considered as a constant throughout the parametric analysis. It can be thus said that the obtained EAL values are estimates of the true, yet unknown, statistics due to the finite number of GMs and the finite number of ASIs. The uncertainty of the estimates will be examined using the proposed *regrouping* procedure described in Section 6.5.1 and in Appendix A. The proposed procedure examines both the effect of using a finite GM group and a finite number of ASIs, determining the variability of the estimates as well as the bias of the expected value with respect to a reference value. Although the true statistics are inherently unknown, the reference EAL considered in the current study is obtained using 40 records and 12 ASIs.

Table 6.6. Spectral accelerations used and the corresponding scale factors

Probability of exceedance	Scale factor	Sa($T^*=0.27$ s) g	Scale factor	Sa($T^*=0.38$ s) g	Scale factor	Sa($T^*=0.50$ s) g	Scale factor	Sa($T^*=0.66$ s) g	Scale factor	Sa($T^*=0.82$ s) g
60% in 50 years	0.47	0.10	0.41	0.07	0.38	0.05	0.41	0.04	0.40	0.03
45% in 50 years	0.75	0.16	0.71	0.12	0.77	0.10	0.72	0.07	0.67	0.05
30% in 50 years	1.00	0.21	1.00	0.17	1.00	0.13	1.00	0.10	1.00	0.07
15% in 50 years	1.41	0.30	1.41	0.24	1.54	0.20	1.45	0.14	1.48	0.11
13% in 50 years	0.85	0.38	0.86	0.31	0.86	0.25	0.82	0.18	0.82	0.14
10% in 50 years	1.00	0.45	1.00	0.36	1.00	0.29	1.00	0.22	1.00	0.17
7% in 50 years	1.14	0.51	1.14	0.41	1.21	0.35	1.19	0.26	1.23	0.21
6% in 50 years	0.89	0.58	0.87	0.47	0.91	0.40	0.89	0.30	0.90	0.24
5% in 50 years	1.00	0.65	1.00	0.54	1.00	0.44	1.00	0.34	1.00	0.27
4% in 50 years	1.15	0.75	1.17	0.63	1.20	0.53	1.22	0.41	1.21	0.32
3% in 50 years	1.30	0.85	1.31	0.71	1.36	0.60	1.39	0.47	1.39	0.37
2.3% in 50 years	0.92	0.95	0.92	0.80	0.92	0.67	0.91	0.52	0.91	0.42
2% in 50 years	1.00	1.03	1.00	0.87	1.00	0.73	1.00	0.57	1.00	0.46
1.7% in 50 years	1.08	1.10	1.08	0.94	1.08	0.79	1.10	0.63	1.09	0.50
1.5% in 50 years	1.06	1.20	1.16	1.01	1.16	0.85	1.20	0.69	1.17	0.54
1.3% in 50 years	1.16	1.35	1.32	1.15	1.30	0.95	1.39	0.80	1.37	0.63
1.1% in 50 years	1.31	1.50	1.49	1.30	1.44	1.05	1.57	0.90	1.54	0.71
0.9% in 50 years	1.45	1.60	1.61	1.40	1.58	1.15	1.71	0.98	1.70	0.78
0.7% in 50 years	1.55	1.70	1.72	1.50	1.71	1.25	1.83	1.05	1.83	0.84
0.5% in 50 years	1.65	1.80	1.84	1.60	1.85	1.35	1.97	1.13	1.96	0.90

6.6 Seismic loss results

By applying the methodology presented in Section 6.2, using the appropriate data described in Sections 6.3 to 6.5, the expected loss of the six buildings is determined for the different combinations of GM group size and number of ASIs. A preliminary

discussion of the seismic losses of the reference cases is initially performed, followed by the comparative assessment of the effect of the ASI and the GM group size on the expected losses.

6.6.1 Seismic losses of the reference cases

6.6.1.1 Expected annual losses

The normalised EAL of the reference case of the six buildings is shown in Figure 6.8 as a summation of the disaggregated normalised EAL components, which are presented in the form of stacked bar plots and are discussed in the following section. The EAL values of the six buildings range from 0.10% to 0.26% of the buildings' replacement cost. These EAL values appear to be low at first glance considering that the analysed buildings are non-seismically designed. However, the vulnerability of the buildings is only one of the contributors to the calculation of the EAL, the second one being the seismic hazard of the site. Existing studies have shown that lower seismic hazard generates lower EAL values (O'Reilly et al 2018; Sousa and Monteiro 2018) for the same structure. In fact, the six buildings analysed in the present study are located in Lisbon, a site that is characterised by moderate seismic hazard, as expressed by the seismic hazard curves presented in Section 6.5.1. Furthermore, similar EAL values, i.e. 0.12%, were reported by Caruso et al (2019) for a building located in the same region, while Sousa and Monteiro (2018) reported values in the same range for buildings located in low and medium seismicity areas in Italy.

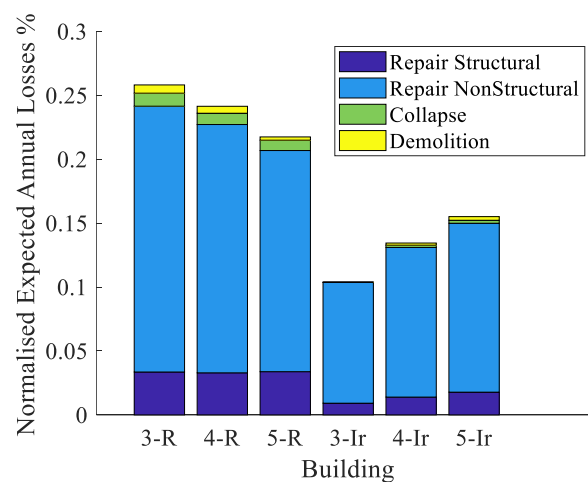


Figure 6.8 Total and disaggregated EAL of the reference cases normalised by the corresponding building replacement cost.

It is important to note herein that a comparison of the obtained results in absolute terms should only be performed with buildings located in Europe and with studies using European fragility and loss assembly functions. Since the loss assembly functions have been found to significantly affect the overall losses (Porter et al. 2002), comparing these results with those of studies that involve data from other sources was considered to be inadequate. Nevertheless, results from such studies can be used to analyse and compare the relative value of different loss components to extract relevant trends. Moreover, it is also noted that the two building configurations (regular and non-regular) result in buildings with different strength, stiffness and ductility characteristics and as an extent in different fragility characteristics. The capacity (Pushover) curves corresponding to the 5-R and the 5-Ir buildings along structural axis X and Y, separately, with and without the infill walls, presented in Figure 6.9, further highlight this fact and provide explanation why the buildings with the irregular configuration experienced lower losses. It can be seen that the irregular building experiences higher strength, stiffness and ductility compared to its regular counterpart, thus explaining the lower losses presented above.

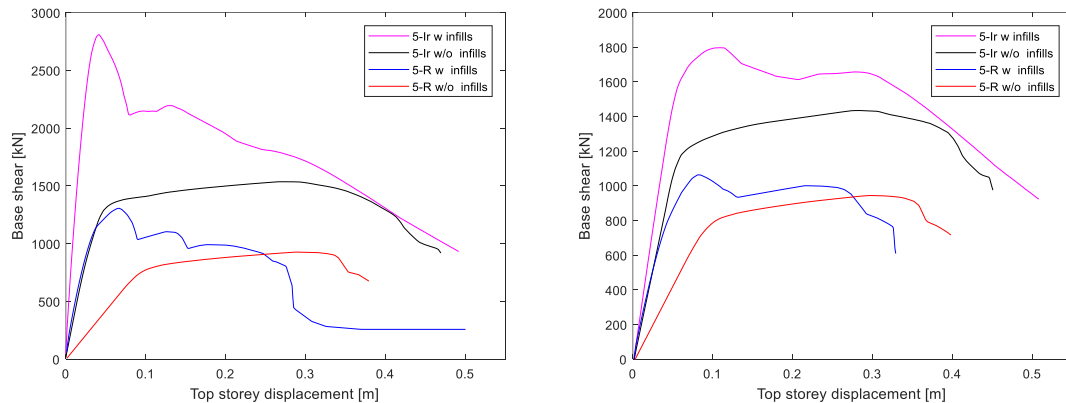


Figure 6.9 Capacity curves comparison of the 5-R and the 5-Ir buildings along axis X (left) and Y (right) with and without infill walls. (A triangular distribution of lateral forces was assumed).

6.6.1.2 Disaggregated losses

Additional observations and comments can be made by analysing the disaggregated losses. Figure 6.8 shows the normalised EAL of each component contributing to the total normalised EAL for the reference case of the six buildings analysed. It can be seen that the highest contribution is that of losses due to repair (i.e. the sum of losses due to the repair of structural and non-structural components), that correspond to more than 60%. Among these, losses associated to the repair of non-structural components have the

largest contribution, which is in agreement with existing research results (e.g. see (Caruso et al. 2019; Aslani and Miranda 2005)). Losses due to collapse and demolition are significantly lower and their contribution varies among the buildings.

Further insights can be obtained by analysing the disaggregated losses as a function of the seismic intensity shown in Figure 6.10, in which the normalised expected losses of the 3-R and 5-Ir buildings for the reference case are plotted. The trends observed for the 3-R building are representative of those found for the three regular buildings analysed herein, while the trends observed for the 5-Ir building are representative of those found for the three irregular buildings. Focusing on the 3-R building, Figure 6.10 (a) shows that losses due to the repair of structural and non-structural components outweigh the rest of the loss components for the lower and medium intensities, while losses due to collapse and demolition are the governing loss contributions for higher intensities. By considering the weighting factors assigned to each loss component by the hazard curve during the EAL calculation, expressed by Eq. (6.7) (higher probabilities, thus higher weights, for lower intensities), it can be seen that the repair losses are the main EAL contributor. Although losses due to collapse and demolition increase with the seismic intensity, the concurrent reduction of the seismic hazard leads to a smaller overall contribution of these components in the aggregated loss, i.e. in the EAL.

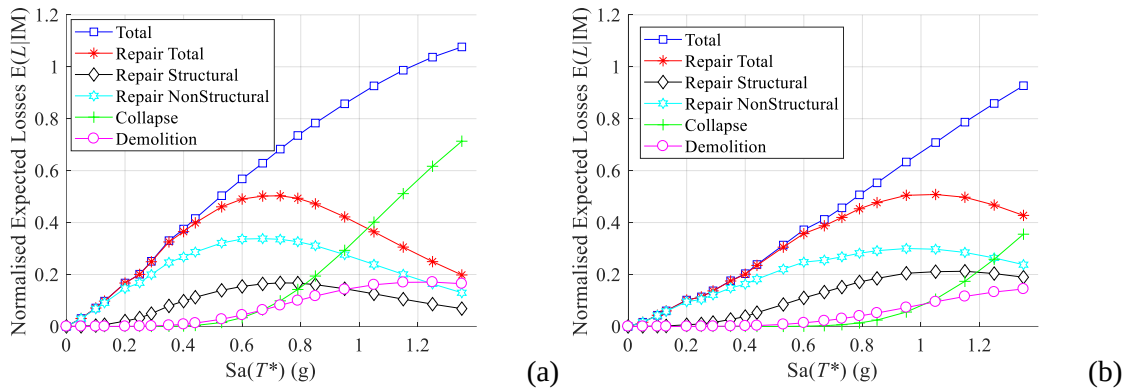


Figure 6.10 Expected loss of the 3-R (a) and 5-Ir (b) buildings as a function of the ground motion intensity, normalised by the replacement cost of the building (Reference case 40_12).

For the intensity with a 30% in 50 years probability of exceedance ($Sa(T^*) = 0.13g$), Figure 6.10 (a) shows that total losses are about 10% and are all associated to damage in non-structural components. For the intensity with a 10% in 50 years probability of exceedance ($Sa(T^*) = 0.29g$), the total losses are in the range 22%-24% and are mostly associated to non-structural components; only a small percentage (2%) is due to damage

in structural components. For higher intensity levels, loss due to demolition also contributes to the total loss, while the contribution of collapse increases significantly and reaches 70%-80% for the maximum intensity level that is considered. Finally, the total expected losses reach to up to 1.1 times the cost of a new building for the maximum intensity level that is considered, which is about twice the intensity that corresponds to the 2% in 50 years probability of exceedance ($Sa(T^*) = 0.73g$).

As far as the 5-Ir building is concerned (Figure 6.10 (b)) results show that, with respect to the 3-R building, a higher intensity is required to induce the same level of loss. As a result, the highest intensity used herein, i.e. twice the intensity of the 2% in 50 years scenario earthquake, led to expected losses lower than the cost of the new building (losses reach up to 93% of the cost of a new building). Nonetheless, for the lower intensities, only loss due to repair of the non-structural components is observed, while structural losses also appear as the intensity increases. Namely, for the intensity with a 10% in 50 years probability of exceedance ($Sa(T^*) = 0.29g$), losses attributed to non-structural components of about 12% can be reported, while only a small percentage (1.5%) is due to damage in structural components. Similar to the case of the 3-R building, loss due to repair of the non-structural components outweighs loss due to structural components for the whole range of intensities. Losses due to demolition and collapse also contribute for larger intensities, leading to 50% of the cost of a new building for the highest intensity used herein when combined.

6.6.2 Results considering the effect of the GM group size and the ASI

6.6.2.1 Expected annual loss results

After discussing the general trends of the expected losses and the EAL of the buildings, the effect of the GM group size and of the ASI is examined in this section. As presented in Section 6.2, the EAL of each building is determined for GM groups of size 10 to 40, and considers 100 groups for each size lower than 40. Furthermore, all GMs of a given group are applied along 1 to 12 ASIs resulting in 100 EAL values for each n_ASI combination. To illustrate this type of result, Figure 6.11 shows the normalised EAL of the 3-R building for all n_ASI combinations, while results obtained for the rest of the buildings are shown afterwards. The bar plots correspond to the median value of the normalised EAL for a given n_ASI combination (EAL_{median}), while the whiskers show the range of EAL values around the median. The normalisation is performed using the building's replacement cost.

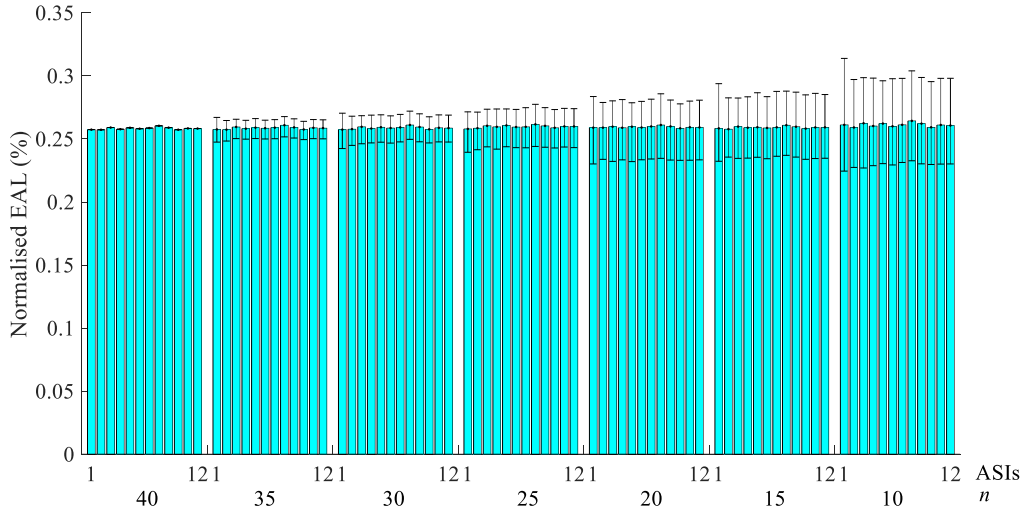


Figure 6.11 Normalised EAL of the 3-R building for all combinations of n_{ASI} .

By comparing the $\text{EAL}_{\text{median}}$ of all n_{ASI} combinations to the reference EAL, i.e. the EAL_{40_12} , one of the most prominent observations that can be made is that the $\text{EAL}_{\text{median}}$ does not exhibit any significant changes with the number of ASIs or with the GM group size. On the other hand, the variability of the EAL, expressed by the represented range, can be seen to change significantly depending on the GM group size. This variation can be analysed by comparing the results of the several n_{12} combinations, in which the uncertainty related to the ASI is fully accounted for and the pure effect of the GM group size on the EAL range can be seen. As such, it is observed that by increasing the GM group size, the referred range decreases. Particularly for the 3-R building, the range for the 10_12 combination is $[-0.0305\%, 0.0374\%]$, while for the 35_12 combination it is $[-0.0085\%, 0.0066\%]$, i.e. more than four times lower.

As opposed to the effect of the GM group size, the number of ASIs for a particular GM group size appears to have a much smaller effect on the EAL range. A slightly larger range of EAL values can be observed in cases where only 1 ASI is used for almost all GM group sizes n when compared to that of the n_{12} case. Furthermore, a fairly constant range is observed for the rest of the n_{ASI} combinations, again when compared to that of the n_{12} case. Although, no other studies have dealt with the combined effect of the ASI and GM groups size on the EAL, this observation, i.e. the larger effect of the GM group size comparatively to that of the ASI, is in agreement with conclusions reached in similar research dealing with other PBEE-related results, such as the variability of EDPs (Giannopoulos and Vamvatsikos 2018; Chapter 3) and collapse risk (Chapter 5).

Considering both the effect of the ASI and of the GM group size, when a small number of GMs is used (e.g. 10 GMs), the over- or under-estimation of the normalised EAL was found to be around 20% for most buildings studied, but more extreme cases can also be found. An example of these more extreme situations is presented in Figure 6.12 that shows the normalised EAL for the 3-Ir building and all n_{ASI} combinations. By examining the results obtained for the 10_1 combination in particular, it can be seen that a single EAL value is 340% higher than the reference value. The latter observation is an example of how a small number of observations can induce an unpredictably high variability in the outputs of a probabilistic analysis. It is also highlighted that, although using more than 1 ASIs was shown to have negligible effects on the variability of the EAL, it can provide additional data that appear to act beneficially against outliers in cases where the number of observations is small. As an example, Figure 6.12 shows that the range of the 10_2 case is similar to that of the 10_12 case, i.e. the outlier value that occurred in the results of the 10_1 combination disappears by considering an additional ASI. In any case, increasing the number of observations (and the number of structural analysis) by increasing number of GMs is a preferable option as it decreases both the likelihood of the previously mentioned outliers and the EAL range. For example, in Figure 6.12, the overestimation of the normalized EAL with respect to the reference value decreases to less than 4% for the 20_1 combination when compared to the overestimation of about 7% found for the 10_2 combination, that requires the same number of structural analyses. Furthermore, the 15_1 combination leads to about 5% overestimation of the normalised EAL, which is a lower level of overestimation when compared to that of the 10_2 combination, but involves a lower number of structural analyses and still has no outliers in the data. In light of this, it can be said that using a large enough GM group size is essential to reduce variability.

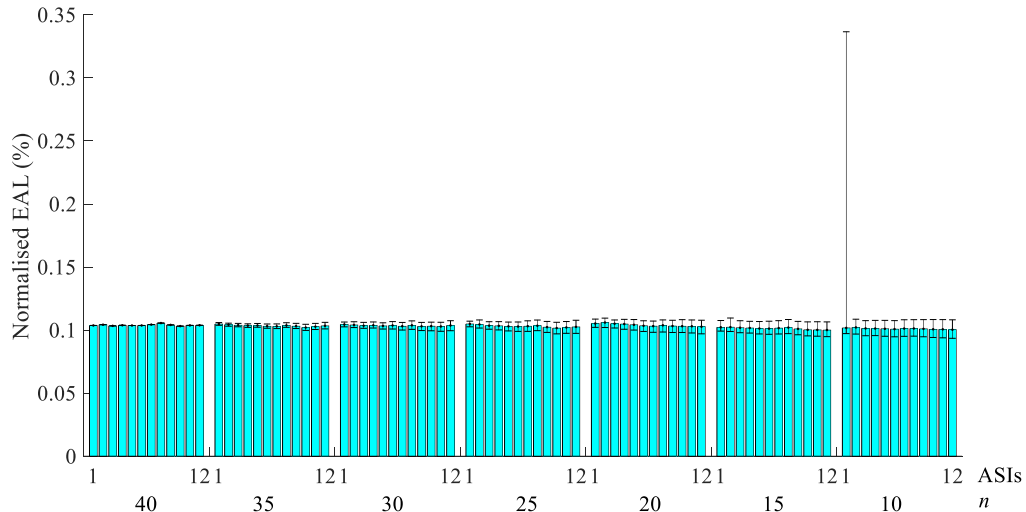


Figure 6.12 Normalised EAL of the 3-Ir building for all combinations of n_{ASI} .

Since no significant changes were observed on the median and the range of the normalised EAL with respect to the ASI, Figure 6.13 only shows results for the remaining buildings (4-R, 5-R, 4-Ir, 5-Ir) for all n sizes but for the following n_{ASI} combinations: n_1 , n_2 and n_{12} . For the buildings analysed herein, using 25 GMs led to a 15% over- or under-estimation of the reference value, which drops to 10-12% when considering 30 GMs. Furthermore, avoiding the use of small groups of GMs, i.e. with 10 GMs or less according to the results of the examined buildings, is important to avoid the possible occurrence of extreme outliers. Using more than 1 ASI, however, does not seem to provide additional benefits for the reliable estimate of EAL, unless for cases where the GM group size is too small to provide adequate data for probabilistic analysis.

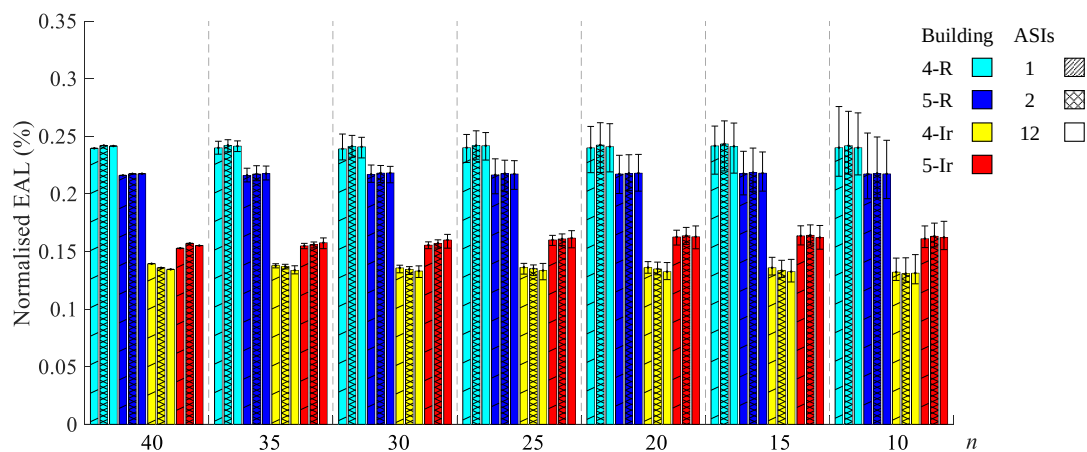


Figure 6.13 Normalised EAL of the 4-R, 5-R, 4-Ir and 5-Ir buildings for all n_1 , n_2 and n_{12} combinations.

6.6.2.2 Disaggregated losses

The effects of the GM group size and the number of ASIs are examined in this section with respect to the disaggregated EAL. The cyan bar plots in Figure 6.14, Figure 6.15, Figure 6.16 and Figure 6.17 show the normalised EAL_{median} values due to the repair of structural components, the repair of non-structural components, demolition and collapse, respectively, for the 3-R building. Observations regarding the disaggregated EAL_{median} values are similar to those made for the total EAL_{median} values, i.e. no significant variations of the values are seen due to the GM group size or the number of ASIs. Combining this information with the observations made in Section 6.6.1 shows that the largest contribution to the EAL comes from the repair of non-structural components, whereas EAL due to collapse and demolition are much lower for all n_ASI combinations.

As far as the variability of the different EAL components is concerned, observations similar to those made regarding the total EAL can also be established with respect to the effect of the GM group size on the EAL range. Hence, the range appears to reduce with the increase of the GM group size for all the EAL components. On the other hand, the effect of the ASI on this range, for a given GM group size, appears to vary among the different components. Although the variation is smaller when compared to that resulting from the effect of the GM group size, it induces a fluctuation of the EAL values when the number of ASIs is small, that then becomes insignificant for cases involving a larger number of ASIs. The latter can be observed in the uneven whiskers of the bar plots corresponding to a specific n and different number of ASIs in Figure 6.15 and Figure 6.16, especially for small n sizes (i.e. 10 and 15). In order to quantify the effect of the ASI on the variability of the different EAL components for each GM group size, an additional measure is employed. This measure represents the median of the absolute deviations of the data with respect to its median, MAD, and is expressed by

$$MAD = \text{median}(|EAL_i - EAL_{median}|) \quad (6.8)$$

where EAL_i represents the individual EAL observations of a given n_ASI combination. The MAD was selected due to its robustness that makes it suitable for datasets with a few outliers. The MAD values that correspond to a given GM group size and 1–11 ASIs are normalised by the MAD value obtained for that GM size and the 12 ASIs. The results obtained for the different MAD values are presented by the red bar plots in Figure 6.14, Figure 6.15, Figure 6.16 and Figure 6.17, where values close to 1 indicate a minor

influence of the ASI on the EAL variability. The ASI can be observed to have an influence on the EAL for all GM group sizes. A larger influence is seen when a smaller number of ASIs is involved, i.e. 1, 2 or 3, but the MAD starts converging to 1 when increasing the number of ASIs. For most of the examined buildings, the largest influence was found in the EAL due to demolition, while the smallest effect was observed in the EAL due to collapse. Nonetheless, a rather low effect of the ASI can be observed overall.

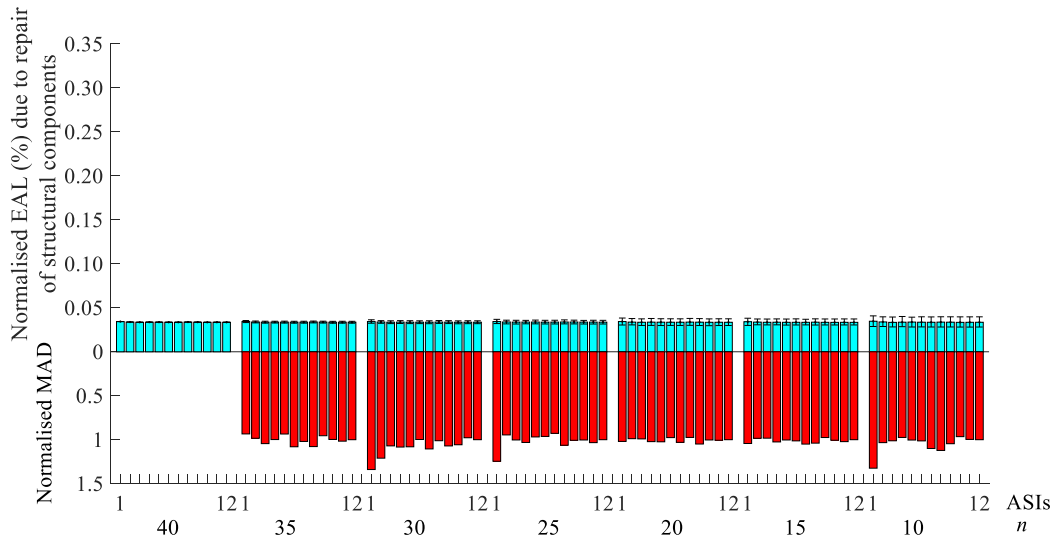


Figure 6.14 Normalised EAL due to repair of structural components and normalised MAD of the 3-R building for all combinations of n_{ASI} .

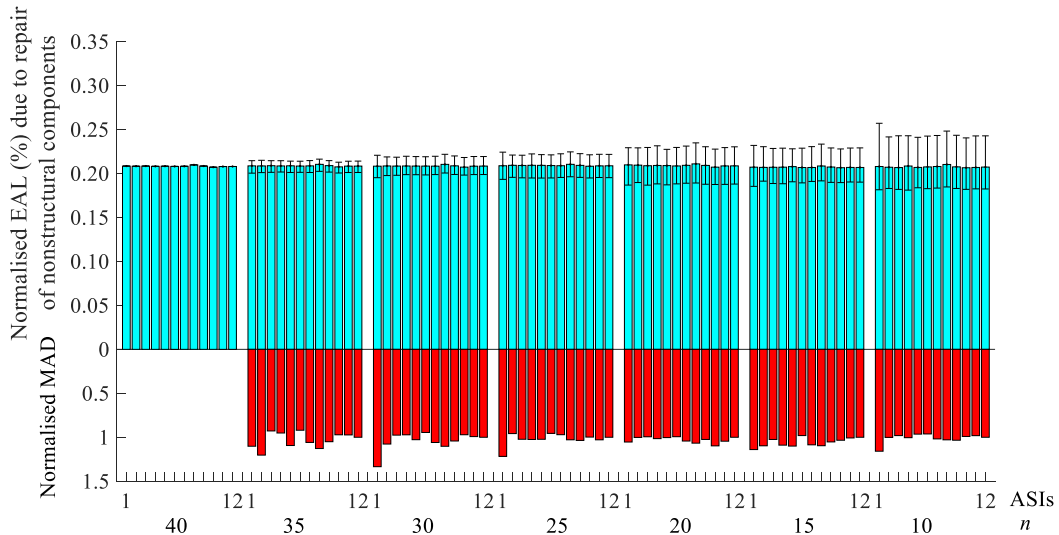


Figure 6.15 Normalised EAL due to repair of non structural components and normalised MAD of the 3-R building for all combinations of n_{ASI} .

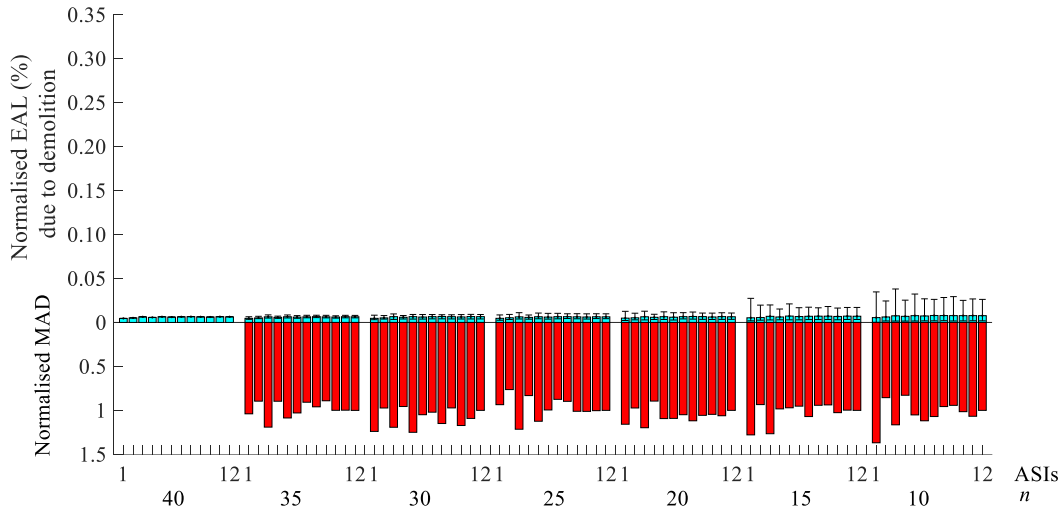


Figure 6.16 Normalised EAL due to demolition and normalised MAD of the 3-R building for all combinations of n_{ASI} .

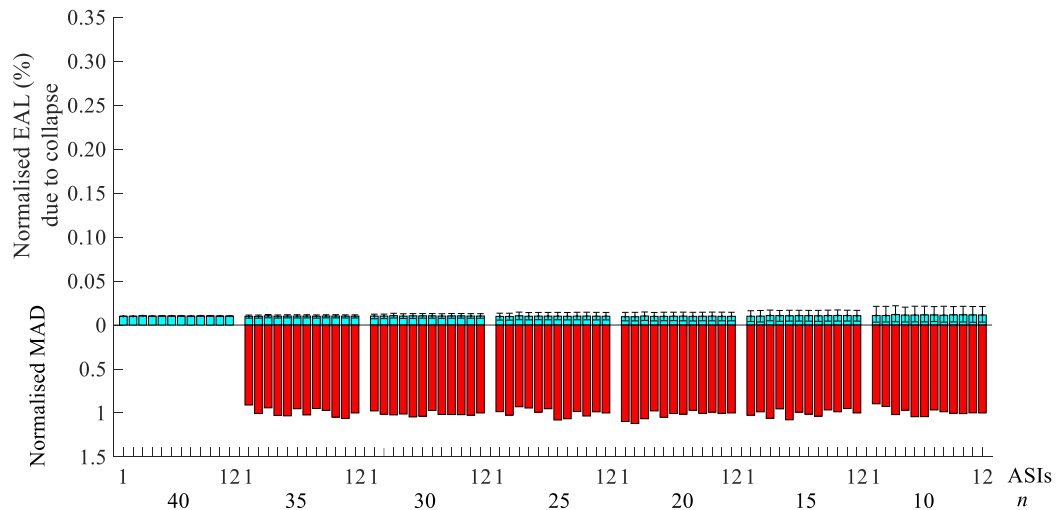


Figure 6.17 Normalised EAL due to collapse and normalised MAD of the 3-R building for all combinations of n_{ASI} .

6.7 Conclusions

The effect of the GM group size and of the ASI was analysed on the EAL of six RC buildings using the PEER-PBEE framework. An inventory of the buildings' components was defined, along with discrete damage states for each component, conditional to a response parameter, and a method of repair corresponding to a monetary value, conditional to a damage state. Additional to the loss due to repair, the loss due to collapse and demolition (in case the buildings experience excessive residual deformation) were

accounted for. The building response was computed using multiple stripe analysis using hazard-compatible groups of GMs of various sizes that were applied using different ASIs.

The results obtained for the six buildings showed that the EAL_{median} was not significantly influenced by the GM group size or the ASI. For a given building, this comparison was performed using the EAL_{median} of all n_ASI combinations and the EAL of the reference case corresponding to the 40_12 combination, that was considered the most exact result of the present study. The GM group size was found to have a significant influence on the EAL variability, where the latter reduces with the increase of the former. The effect of the ASI on the variability, on the other hand, was shown to be much smaller and to induce only minor fluctuations on the variability of the total EAL and of the EAL of each individual component. As a result, the use of only 1 ASI is considered to be adequate for the estimation of the EAL. The importance of having a large enough number of observations for probabilistic analysis was also emphasised, since extreme outliers were observed when only 10 GMs were used applied along 1 ASI. In that case, outliers were seen to disappear by increasing the number of ASIs to at least 2 or, more efficiently, by increasing the number of GMs to at least 15. Overall, for the analysed buildings, the use of 15 GMs along 1 ASI was found to be adequate for the EAL estimation. Yet, depending on the required level of precision, using a larger GM group size could be advantageous since it was shown to reduce the EAL variability. Increasing the number of ASIs, however, did not provide additional benefits.

Chapter 7

7 Critical orientation of earthquake loading for building performance assessment using Lateral Force Analysis

Based on:

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Abstract

The orientation of the seismic action is known to have a non-negligible effect on the seismic behaviour of buildings. However, modern earthquake-related standards only partially cover the effect of this factor and results of practical utility on this topic are still limited. To address this issue, a methodology enabling the determination of the critical angle of seismic incidence in the context of Lateral Force Analysis is proposed. The analytical methodology determines the critical angle of seismic incidence for the storey displacements and the interstorey drifts of buildings that conform with standard-based provisions for linear static analysis. The applicability of the methodology is illustrated for three buildings comprising different typologies in plan and in elevation. The accuracy of the analytical results obtained is validated by performing a parametric Lateral Force Analysis of the buildings for different orientations of the bidirectional seismic action.

List of symbols

α , ASI	Angle of seismic incidence
α_{01}	Angle between the structural axis X and the <i>optimum principal axis</i> I_{opt}
α_{02}	$\alpha_{01} + 90^\circ$
α_{crit} , ASI _{crit}	Critical angle of seismic incidence
β	Percentile used for the percentage combination rule, 30% or 40%
$\theta_{i,F}$	Rotation about axis i due to F
λ	Modal mass correction factor
ω	Angle between the structural axis X and the principal axis I
ω	Angle between the structural axis X and the <i>fictitious principal axis</i> I
A	Arbitrary point on a diaphragm
a_g	Design ground acceleration on type A soil
C_S	Elastic centre
C_S	<i>Fictitious elastic centre</i>
dof/s	Degree/s of freedom
F	Module of force vector $\underline{F}$
$\underline{F}^{''1''}$	Unit force vector
f_i	Flexibility coefficient along axis i
$F_{i,F}^A$	Component of F along axis i applied on A
$\underline{f}_i, \underline{m}_i$	N-dimensional vector of forces and moments, respectively, along and about axis i
$[\underline{F}_m \ \underline{F}_s]^T$	Force vector acting on the master and on the slave dofs
$\underline{F}_R$	Equivalent force vector acting on the master dofs
$\underline{F}_R^{''1''}$	Equivalent force vector of a unit force acting on the master dofs
I, II, III	Principal axes
I, II, III	<i>Fictitious principal axes</i>
$I_{opt}, II_{opt}, III_{opt}$	<i>Optimum principal axes</i>
ISD ^A	Interstorey drift of point A
K_I, K_{II}, K_{III}	Principal stiffness coefficients
K_I, K_{II}, K_{III}	<i>Fictitious Principal stiffness coefficients</i>
k_I, k_{II}, k_{III}	Numerical coefficients associated with the principal directions of the torsionally coupled single storey system
K_{ij}	Stiffness matrix coefficients with i = X, Y, Z and j = X, Y, Z defined on (X, Y, Z)
k_{ij}	Coefficients functions of k_n with i = x, y, z and j = x, y, z defined on (X, Y, Z)
$\underline{K}_{ij}$	N×N stiffness matrix with i = x, y, z and j = x, y, z defined on (X, Y, Z)

$\begin{bmatrix} \underline{K}_{mm} & \underline{K}_{ms} \\ \underline{K}_{sm} & \underline{K}_{ss} \end{bmatrix}$	Stiffness matrix of a structure rearranged so that $\underline{K}_{mm}$ is a matrix of the master dofs and $\underline{K}_{ss}$ the matrix of the slave dofs
k_n	Numerical coefficient of isotropic buildings
$\underline{K}_0$	Stiffness matrix of order N of the N-storey plane frame
$\underline{K}_R$	Condensed stiffness matrix that corresponds to the master dofs
K_{unc}	Uncoupled stiffness of the building
m	Total mass of a building
$M_{i,F}$	Moment about axis i due to F
N	Number of storeys
$O(X, Y, Z)$	Coordinate system of three orthogonal axes X, Y, Z that intersect the diaphragm at point O
S	Soil factor of the EC8 response spectrum
$S_e(T)$	Elastic spectral acceleration at period T
T_B, T_C, T_D	Corner periods of the EC8 response spectrum
T_I, T_{II}	Principal periods
T_{Iopt}, T_{IIopt}	<i>Optimum principal periods</i>
T_{unc}	Uncoupled fundamental period of vibration
$T_{unc.opt}$	<i>Optimum uncoupled fundamental period of vibration</i>
u^A	Resultant displacement of A
$u_{i,F}^A$	Displacement of A on axis i due to F
$\underline{u}_i, \underline{\theta}_i$	N-dimensional vector of displacements and rotations, respectively, along and about axis i
$[\underline{u}_m \ \underline{u}_s]^T$	Displacement vector of the master and the slave dofs
x_A, y_A	Coordinates of A on the structural reference system
x_A', y_A'	Coordinates of A on the principal reference system
x_A'', y_A''	Coordinates of A on the <i>fictitious principal reference system</i>

7.1 Introduction

Following the traditional practice of analysing planar building structures, the seismic analysis of tridimensional (3D) buildings under multidirectional ground motions is usually performed by applying the horizontal loading representing the earthquake action along the main orthogonal axes of the structure, e.g. see (EAK 2003; CEN 2004; Regulatory Guide 1.92 2006; NTC 2018; ASCE 2014). This approach establishes that multidirectional earthquake loading consists of two simultaneous orthogonal components. Furthermore, it also assumes that considering only this angle of seismic

incidence (i.e. the angle between the axes of the earthquake loading and the reference axes of the structure) leads to the critical demand (i.e. the maximum demand considering all possible angles of seismic incidence). However, past research has shown that different engineering demand parameters (EDP) reach their maximum value for different angles of seismic incidence. Furthermore, some of these incidence angles may not coincide with the main structural axes, even for structures that possess two main orthogonal structural axes, e.g. see (Athanatopoulou 2005; Rigato and Medina 2007). Available results (e.g. see (Kostinakis et al. 2013; Magliulo et al. 2014; Fontara et al. 2015)) indicate that the angle of seismic incidence that maximizes structural response in terms of a given EDP depends on several factors. Among others, the frequency content and the intensity of the seismic action as well as the strength/ductility properties of the structure appear to be of significant importance.

In light of this, several studies have addressed the influence of the angle of seismic incidence on the maximum demand of structures under earthquake loading. The maximum value of an EDP due to a horizontal earthquake loading applied along an arbitrary pair of orthogonal directions (i.e. not necessarily the main axes of a structure) is termed final demand hereon. As a result, the maximum of all possible final demands is the critical demand. The final demand can be evaluated using the well-known directional combination rules (which combine the timewise non-simultaneous peak responses of each direction) or by analysing the evolution of the response in each time step. Focussing on the directional combination rules, standards (e.g. (EAK 2003; CEN 2004; Regulatory Guide 1.92 2006; ASCE 2014; NTC 2018)) currently suggest the use of the square-root-sum-of-squares (SRSS) and β percentage rules, namely the 30% rule proposed by Rosenblueth and Contreras (1977) or the 40% rule proposed by Newmark (1975). In the context of response spectrum analysis (RSA), the SRSS rule has been proven to lead to a final demand that does not depend on the orientation of the seismic action, which is represented by the same response spectrum (shape and intensity) along two horizontal orthogonal axes (Wilson et al. 1995). Therefore, the critical demand is always obtained in this case. On the contrary, when the two response spectra have different intensities, the SRSS rule leads to orientation-dependent final demands (Lopez et al. 2001; Anastasiadis et al. 2002). Nevertheless, the ratio of the two orthogonal response spectrum intensities does not affect the value of the critical angle of seismic incidence, i.e. the orientation of the seismic action that leads to the critical demand. With respect to the percentage

combination rules, the final demand is seen to depend significantly on the orientation of the orthogonal spectra (Wilson et al. 1995; Lopez et al. 2001). Therefore, the critical demand may not be obtained if the earthquake loading is only applied along the main structural axes. Similar observations have been found for time history analysis (THA) by Mac Rae and Mattheis (2000) and Kostinakis et al. (2013). Both studies determined that the final demand obtained from the application of different combination rules, i.e. the SRSS and the 30% or 40% rules, depends on the angle of seismic incidence. For the case of lateral force analysis (LFA), results are, however, limited. Only the SRSS rule was analysed by Morfidis et al. (2008) who found that, unlike for RSA, the final demand (evaluated in terms of required longitudinal reinforcement) depends on the choice of the orthogonal directions.

Since orientation-related standard provisions were found to affect the structural response, methodologies have been developed to determine the critical angle of seismic incidence. With respect to RSA, Menun and Der Kiureghian (1998) proposed the CQC3 combination rule and a procedure to determine the critical demand (as well as the corresponding critical angle). This procedure considers the correlation and the relative intensities of the orthogonal components of the seismic action. A similar study in the time domain developed by Athanatopoulou (2005) provides analytical formulae to determine the critical demand and the corresponding critical angle of seismic incidence based on preliminary responses obtained from linear THA. In this study, the responses are obtained in the time domain using ground motion components having an arbitrary degree of correlation. These two studies introduce new methods to determine the critical angle of seismic incidence in the context of RSA and THA. However, similar procedures for LFA do not exist currently.

The simplicity and the familiarity of the practitioners with the traditional force-based procedures rank LFA as a widely used method of seismic analysis (Toranzo-Dianderas 2010). According to current provisions related to earthquake analysis, the LFA is applicable both for the design of new structures (e.g. (CEN 2004)) and the seismic safety assessment of existing ones (e.g. (CEN 2005)). Therefore, procedures to determine the critical angle of seismic incidence should also be developed for this method of analysis. In light of this, an analytical methodology is presented herein for the determination of the critical angle of seismic incidence in the context of LFA and the seismic safety assessment of existing buildings. The methodology is initially developed for single-storey buildings

and is subsequently extended for multi-storey buildings. Two widely used global demand parameters are selected to express structural performance: storey displacements and interstorey drifts. The methodology is applied to three case-studies comprising buildings with different configurations in plan and in elevation. To validate the accuracy of the analytical results obtained, the buildings are subjected to a parametric LFA for different orientations of the bidirectional seismic action. The angles of incidence determined from the analytical methodology and those obtained from the parametric analysis are then compared to examine the applicability limits of the proposed methodology.

7.2 Description of the methodology

7.2.1 Fundamentals of the methodology

The methodology developed for the determination of the critical angle of seismic incidence, termed ASI_{crit} hereon, lays its foundations on the static behaviour of 3D single-storey buildings with an arbitrary configuration in plan, as described by Roussopoulos (1932). Therefore, basic concepts associated with single-storey structures and their behaviour under static loading are first presented to provide some background for the subsequently developed rationale. The proposed methodology considers the mechanical behaviour of the materials to be linear elastic and the floors to be in-plane rigid and out-of-plane flexible. Furthermore, the vertical elements are considered to be axially rigid. The stiffness of a single-storey building with the aforementioned properties, can be described by a 3×3 matrix with respect to an arbitrary reference system $O(X,Y,Z)$ (Figure 7.1), where X - Y is the horizontal plane at the diaphragm level, Z is the vertical axis and O is their intersection point. Given these considerations, for an arbitrary horizontal loading vector $[F_x \ F_y \ M_z]^T$, the following static equilibrium holds:

$$\begin{bmatrix} K_{xx} & K_{xy} & K_{xz} \\ K_{yx} & K_{yy} & K_{yz} \\ K_{zx} & K_{zy} & K_{zz} \end{bmatrix} \cdot \begin{bmatrix} u_x \\ u_y \\ \theta_z \end{bmatrix} = \begin{bmatrix} F_x \\ F_y \\ M_z \end{bmatrix} \quad (7.1)$$

where K_{ij} are the stiffness matrix coefficients, with $i = X, Y, Z$ and $j = X, Y, Z$, and $[u_x \ u_y \ \theta_z]^T$ is the displacement vector with respect to the $O(X,Y,Z)$ reference system.

Single-storey buildings with these properties always have a principal reference system $C_s(I, II, III)$ (Roussopoulos 1932) that is derived from the initial system by a translation along two perpendicular directions and a rotation ω about the vertical axis Z (Figure 7.1).

The stiffness matrix of the principal reference system $C_S(I, II, III)$, termed principal stiffness matrix, is a diagonal matrix with respect to which the corresponding static equilibrium is decomposed into three uncoupled equations:

$$K_I \cdot u_I = F_I, \quad K_{II} \cdot u_{II} = F_{II}, \quad K_{III} \cdot \theta_{III} = M_{III} \quad (7.2)$$

where K_I, K_{II}, K_{III} are the principal stiffness coefficients and $[u_I \ u_{II} \ \theta_{III}]^T$ and $[F_I \ F_{II} \ M_{III}]^T$ are the displacement and the loading vectors, respectively, with respect to $C_S(I, II, III)$. The coordinates of the elastic centre C_S (which has the properties of the centre of stiffness, centre of torsion and shear centre) are determined from the conditions $K_{I,III} = 0$ and $K_{II,III} = 0$, and the orientation angle ω is determined from $K_{I,II} = 0$ (Roussopoulos 1932). The importance of such a transformation lies in the fact that the translational principal stiffness coefficients K_I and K_{II} correspond to the minimum and maximum uncoupled translational stiffness of the system, respectively, and hence the principal periods T_I and T_{II} , which are determined by:

$$T_I = 2 \cdot \pi \cdot \sqrt{\frac{m}{K_I}} \quad \text{and} \quad T_{II} = 2 \cdot \pi \cdot \sqrt{\frac{m}{K_{II}}} \quad (7.3)$$

correspond to the maximum and minimum uncoupled translational periods of the system, respectively, with m representing the total mass of the structure. Furthermore, it can be proven that the uncoupled translational periods, i.e. the periods that correspond to each uncoupled horizontal direction α with respect to axis X given by:

$$T_{unc}(\alpha) = 2 \cdot \pi \cdot \sqrt{\frac{m}{K_{unc}(\alpha)}} \quad (7.4)$$

lie on an ellipse with a semi major axis T_I and a semi minor axis T_{II} along axis I and II , respectively. The term K_{unc} in Eq. (7.4) corresponds to the stiffness of the structure along the uncoupled horizontal direction α , which is defined as the horizontal direction when the orthogonal translational degrees of freedom (dofs) as well as the torsional dofs of the diaphragm are fixed. As a consequence, the uncoupled periods of the structure, Eq. (7.4), can be expressed from the ellipse's equation in polar coordinates as a function of the arbitrary orientation defined by angle α with respect to axis X :

$$T_{unc}(\alpha) = \frac{T_I \cdot T_{II}}{\sqrt{\cos(\alpha - \omega)^2 \cdot T_{II}^2 + \sin(\alpha - \omega)^2 \cdot T_I^2}} \quad (7.5)$$

The angle ω introduced into Eq. (7.5) aims to provide the expression of the uncoupled periods with respect to axis X, instead of axis I. It is noted that the uncoupled periods may be different from the natural periods given the coupling between translation and torsion, e.g. see Palermo et al. (2013) for an analysis of these differences. However, uncoupled periods are required by the methodology proposed herein to express the dynamic characteristics of a structure along a given direction.

Based on the properties of CS (Anastassiadis 1989), a horizontal static force F passing through C_S causes a translation without rotation of the floor diaphragm. When F is a unit force, the resultant displacement falls on an ellipse with a semi major axis that corresponds to the flexibility coefficient along axis I f_I ($f_I = 1/K_I$) and a semi minor axis that corresponds to the flexibility coefficient along axis II f_{II} ($f_{II} = 1/K_{II}$) (Anastassiadis 1989). In addition, a unit torsional moment M about a vertical axis causes rotation θ_{III} ($\theta_{III} = 1/K_{III}$) of the diaphragm about C_S . Therefore, a horizontal force F applied at any point of the diaphragm, e.g. point O featured in Figure 7.1, with an arbitrary orientation α with respect to axis X causes translation of C_S that falls on an ellipse with a semi major axis equal to $F \cdot f_I$ and a semi minor axis equal to $F \cdot f_{II}$. Also, unless F passes through C_S , F also causes a rotation of the diaphragm around C_S . The overall displacement of the floor due to F (Figure 7.2 (a)) can then be obtained as a superposition of two states of pure translation within the planes I-III and II-III (Figure 7.2 (b)) and of one state of pure rotation about the axis III (Figure 7.2 (c)), expressed by Eq. (7.2).

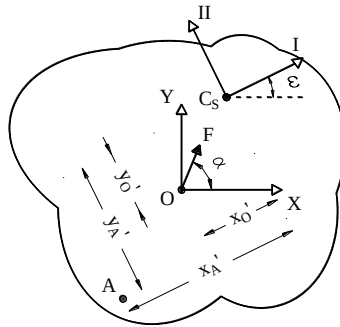


Figure 7.1. Reference systems and coordinates of a single-storey building subjected to a horizontal static force F applied at O with an arbitrary orientation α with respect to structural axis X

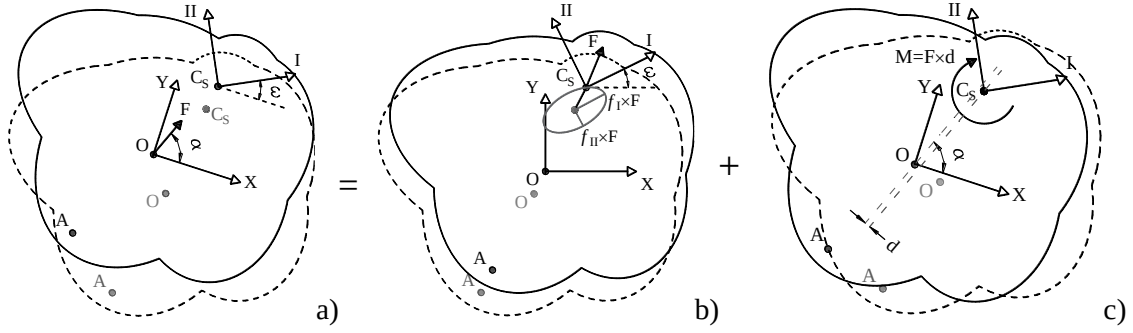


Figure 7.2 Overall displacement of a single-storey building subjected to a horizontal static force F applied at O with an arbitrary orientation α (a) expressed as a combination of a pure translation (b) and a pure rotation (c). The undeformed shape of the diaphragm is shown with the dashed line

Due to linear superposition, the displacement can also be expressed as the displacement caused by a unit force vector $\underline{F}_{1''}$ applied along the same direction inclined of an angle α with respect to the X direction multiplied by the module of vector $\underline{F}$ (namely F), where $\underline{F}_{1''}$ is defined by:

$$\begin{bmatrix} F_{X,1''}(\alpha) \\ F_{Y,1''}(\alpha) \\ M_{Z,1''}(\alpha) \end{bmatrix} = \begin{bmatrix} \cos(\alpha) \\ \sin(\alpha) \\ 0 \end{bmatrix} \quad (7.6)$$

In order to determine the displacement according to Eq. (7.2), $\underline{F}_{1''}$ needs to be projected onto the principal directions I and II . The angle difference between the two coordinate systems is equal to ω and the projection is thus performed according to the following transformation:

$$\begin{bmatrix} F_{I,1''}^{C_s}(\alpha) \\ F_{II,1''}^{C_s}(\alpha) \end{bmatrix} = \begin{bmatrix} \cos\omega & \sin\omega \\ -\sin\omega & \cos\omega \end{bmatrix} \cdot \begin{bmatrix} F_{X,1''}^{C_s}(\alpha) \\ F_{Y,1''}^{C_s}(\alpha) \end{bmatrix} \quad (7.7)$$

The displacement vector of C_s with respect to the principal reference system (I , II , III) due to F applied at O with an arbitrary orientation α is defined by:

$$\begin{bmatrix} u_{I,F}^{C_s}(\alpha) \\ u_{II,F}^{C_s}(\alpha) \\ \theta_{III,F}(\alpha) \end{bmatrix} = F \cdot \begin{bmatrix} F_{I,1''}^{C_s}(\alpha)/K_I \\ F_{II,1''}^{C_s}(\alpha)/K_{II} \\ M_{III,1''}(\alpha)/K_{III} \end{bmatrix} = F \cdot \begin{bmatrix} u_{I,1''}^{C_s}(\alpha) \\ u_{II,1''}^{C_s}(\alpha) \\ \theta_{III,1''}(\alpha) \end{bmatrix} \quad (7.8)$$

where F is the module of the rotating force, $F_{I,1''}^{C_s}$ and $F_{II,1''}^{C_s}$ are obtained by Eq. (7.7) and $M_{III,1''}$ is given by:

$$M_{III,1''}(\alpha) = F_{I,1''}^{C_S}(\alpha) \cdot y_{Cs}' - F_{II,1''}^{C_S}(\alpha) \cdot x_{Cs}' \quad (7.9)$$

where x_{Cs}' , y_{Cs}' are the coordinates of C_S in the $O(X, Y, Z)$ reference system rotated by ω and are defined as the opposite of the coordinates of O with respect to the $C_S(I, II, III)$ reference system, i.e. $x_{Cs}' = -x_O'$ and $y_{Cs}' = -y_O'$, as shown in Figure 7.1. Subsequently, the displacement vector of a generic point of the diaphragm, e.g. point A in Figure 7.1, due to F may then be determined from the displacement of C_S as a function of α based on rigid body kinematics:

$$\begin{bmatrix} u_{I,F}^A(\alpha) \\ u_{II,F}^A(\alpha) \\ \theta_{III,F}(\alpha) \end{bmatrix} = F \cdot \begin{bmatrix} u_{I,1''}^{C_S}(\alpha) - y_A' \cdot \theta_{III,1''}(\alpha) \\ u_{II,1''}^{C_S}(\alpha) + x_A' \cdot \theta_{III,1''}(\alpha) \\ \theta_{III,1''}(\alpha) \end{bmatrix} = F \cdot \begin{bmatrix} u_{I,1''}^A(\alpha) \\ u_{II,1''}^A(\alpha) \\ \theta_{III,1''}(\alpha) \end{bmatrix} \quad (7.10)$$

where x_A' and y_A' are the coordinates of A with respect to the $C_S(I, II, III)$ reference system, as illustrated in Figure 7.1.

7.2.2 Introducing the seismic action

According to current earthquake-related standards, e.g. see (EAK 2003; CEN 2004; Regulatory Guide 1.92 2006; NTC 2018; ASCE 2014), the horizontal seismic action consists of two uncorrelated components considered to act simultaneously along their principal directions (Penzien and Watabe 1974). In the context of LFA, the seismic action is determined by using two acceleration response spectra with identical shapes along two orthogonal directions, which are also considered to have the same intensity. The acceleration response spectrum provided in Eurocode 8 Part 1 (EC8-1) (CEN 2004) is used herein for the development of the proposed procedure but the same approach may be applied for spectra prescribed by different standards. In the context of the seismic safety assessment procedure such as the one prescribed in Eurocode 8 Part 3 (CEN 2005), the considered response spectrum is the elastic acceleration response spectrum, S_e . The shape of the spectrum is divided into four branches, the limits of which are determined by the National Determined Parameters. The second branch corresponds to the constant spectral acceleration region and is defined by the T_B and T_C corner periods. The third branch, defined by the T_C and T_D corner periods, corresponds to the constant spectral velocity region and the fourth, that is defined for periods larger than T_D , corresponds to the constant spectral displacement region.

The spectral acceleration is defined as a function of the structural period, which in LFA corresponds to the fundamental period of vibration in the horizontal direction under consideration, as referred by EC8-1. According to Section 7.2.1, for a specific direction, that fundamental period can be represented by the uncoupled translational period. It was, also, shown by Eq. (7.5) that the uncoupled translational period of single-storey buildings can be expressed as a function of α , $T_{unc}(\alpha)$. Accordingly, the spectral acceleration can also be expressed as a function of the same angle and the resultant base shear along the arbitrary direction α can then be given by:

$$F(\alpha) = S_e(T_{unc}(\alpha)) \cdot m \cdot \lambda \quad (7.11)$$

where m is the total mass of the structure and λ is a modal mass correction factor used in EC8-1 to account for the effective modal mass of the fundamental mode of vibration. The effective modal mass that corresponds to the first mode is on average 15% smaller than the total mass in buildings with at least 3 storeys and translational degrees of freedom in each horizontal direction (CEN 2004). The base shear along the orthogonal direction, expressed by the angle $\alpha+90^\circ$, may be also defined by Eq. (7.11) as a function of the same variable. Therefore, the resultant seismic action consists of two lateral forces expressed as functions of α .

7.2.3 Determination of the critical angle of incidence in single-storey buildings

In the current Section, a procedure is presented to determine the ASI_{crit} in the context of LFA. According to other studies, e.g. (Athanatopoulou 2005; Wilson et al. 1995), the ASI_{crit} depends on the selected EDP, seismic action and directional combination rule. Furthermore, it will be illustrated in a later Section that, for general multi-storey buildings, the ASI_{crit} also depends on the distribution of the base shear across the height. The EDP considered herein is the resultant displacement of an arbitrary point A, i.e. the combined response of the displacement of A along two orthogonal directions using the Pythagorean Theorem. The *final demand* of the resultant displacement, i.e. the combination of the displacements due to the two orthogonal seismic components, is determined using an appropriate combination rule, e.g. the SRSS rule or the β percentage rule, where β can correspond to either 30% or 40% depending on the standard. In the current Section, for illustrative purposes, the base shear is applied only at one point, i.e. at the centre of mass CM of the diaphragm, while the accidental eccentricity is ignored.

The displacement vector of point A due to the base shear applied along an arbitrary orientation α is obtained by introducing the base shear defined by Eq. (7.11) into Eq. (7.10):

$$\begin{bmatrix} u_{I,uniaux}^A \\ u_{II,uniaux}^A \\ \theta_{III,uniaux} \end{bmatrix} = F(\alpha) \cdot \begin{bmatrix} u_{I,"1"}^A(\alpha) \\ u_{II,"1"}^A(\alpha) \\ \theta_{III,"1"}(\alpha) \end{bmatrix} \quad (7.12)$$

where the subscript uniax stands for the unidirectional (uniaxial) excitation. Similarly, the displacement vector of point A due to the base shear along the orthogonal direction can also be defined by Eq. (7.12) just by considering the complementary angle, i.e. $\alpha+90^\circ$. The components of the displacement vector of A along the principal directions due to LFA can then be obtained using an appropriate combination rule, i.e. the percentage β rule:

$$\begin{bmatrix} u_I^A(\alpha) \\ u_{II}^A(\alpha) \\ \theta_{III}(\alpha) \end{bmatrix} = F(\alpha) \cdot \begin{bmatrix} u_{I,"1"}^A(\alpha) \\ u_{II,"1"}^A(\alpha) \\ \theta_{III,"1"}(\alpha) \end{bmatrix} \pm \beta \cdot F(\alpha+90^\circ) \cdot \begin{bmatrix} u_{I,"1"}^A(\alpha+90^\circ) \\ u_{II,"1"}^A(\alpha+90^\circ) \\ \theta_{III,"1"}(\alpha+90^\circ) \end{bmatrix} \quad (7.13)$$

or the SRSS:

$$\begin{bmatrix} u_I^A(\alpha) \\ u_{II}^A(\alpha) \\ \theta_{III}(\alpha) \end{bmatrix} = \begin{bmatrix} \sqrt{(F(\alpha) \cdot u_{I,"1"}^A(\alpha))^2 + (F(\alpha+90^\circ) \cdot u_{I,"1"}^A(\alpha+90^\circ))^2} \\ \sqrt{(F(\alpha) \cdot u_{II,"1"}^A(\alpha))^2 + (F(\alpha+90^\circ) \cdot u_{II,"1"}^A(\alpha+90^\circ))^2} \\ \sqrt{(F(\alpha) \cdot \theta_{III,"1"}(\alpha))^2 + (F(\alpha+90^\circ) \cdot \theta_{III,"1"}(\alpha+90^\circ))^2} \end{bmatrix} \quad (7.14)$$

The expression that defines the resultant horizontal displacement of A (the *final demand*), u^A , due to the simultaneous action of the two orthogonal components of seismic action is then obtained from Eq. (7.13) or Eq. (7.14) using the Pythagorean Theorem:

$$u^A(\alpha) = \sqrt{(u_I^A(\alpha))^2 + (u_{II}^A(\alpha))^2} \quad (7.15)$$

Eq. (7.15) is defined as a function of α and depends on the structural and geometrical characteristics of the building (which affect the value of the principal structural periods), as well as on the shape of the response spectrum. The $F(\alpha)$ and $F(\alpha+90)$ components of Eq. (7.13) and Eq. (7.14) which lead to the final displacement in Eq. (7.15) may take different forms depending on the previously mentioned parameters, i.e. T_I , T_{II} , T_B and T_C .

The explicit derivation of Eq. (7.15) is presented in the Appendix and the only unknown variable it involves is the angle α . Therefore, the angle α that leads to the maximum resultant horizontal displacement of A (the *critical demand*) is determined by maximizing Eq. (7.15) for all non-redundant ASIs:

$$\frac{du^A(\alpha)}{d\alpha} = 0 \quad \rightarrow \quad \alpha_{\text{crit}} \quad (7.16)$$

where α_{crit} corresponds to the ASI_{crit} .

7.3 Extension of the methodology for multi-storey buildings

7.3.1 Buildings with a real elastic axis

The previous Section presented a methodology for the straightforward determination of the ASI_{crit} maximizing the displacement of a given point in a single-storey building. According to the methodology, the building can have an arbitrary configuration in plan as long as the requirements presented in Section 7.2.1 are valid. These requirements are the need to have linear elastic material behaviour, a rigid storey diaphragm and axially rigid columns. In such buildings the uncoupling of the translational and rotational dofs may be performed along a new reference system, termed the principal reference system, defined by the elastic axis and the principal directions. Unlike single-storey buildings, multi-storey buildings, however, do not have these properties in general (Riddel and Vasquez 1984). Nevertheless, there are special categories of multi-storey buildings for which an elastic axis and principal bending planes can be defined. Those belong to one of the following categories of systems (Anastassiadis 1989; Riddel and Vasquez 1984):

- systems with two horizontal axes of in-plan symmetry
- isotropic systems
- ortho-isotropic systems
- complex-isotropic (coaxial) systems

All these systems have principal bending planes (I-III, II-III) and their intersection defines the elastic axis III of the system. Furthermore, the elastic axis of multi-storey buildings possesses properties similar to those of the elastic axis of single-storey buildings, as summarized by Makarios and Anastassiadis (1998). In these systems, the static response may be obtained from the superposition of two states of pure bending within the planes

I-III and II-III and one state of pure torsion about the axis III. The response of such systems may be then determined by analysing an N-storey plane frame along with a torsionally coupled single storey system, where N is the number of storeys.

Generally, for any N-storey building, the following static equilibrium is satisfied with respect to an arbitrary reference system (X,Y,Z) (Anastassiadis 1985):

$$\begin{bmatrix} \underline{K}_{xx} & \underline{K}_{xy} & \underline{K}_{xz} \\ \underline{K}_{yx} & \underline{K}_{yy} & \underline{K}_{yz} \\ \underline{K}_{zx} & \underline{K}_{zy} & \underline{K}_{zz} \end{bmatrix} \cdot \begin{bmatrix} \underline{u}_x \\ \underline{u}_y \\ \underline{\theta}_z \end{bmatrix} = \begin{bmatrix} \underline{f}_x \\ \underline{f}_y \\ \underline{m}_z \end{bmatrix} \quad (7.17)$$

where $\underline{u}_x$, $\underline{u}_y$, $\underline{\theta}_z$ are the N-dimensional vectors of translations u_{xi} , u_{yi} and rotations θ_{zi} with respect to the X_i , Y_i , Z_i axes, respectively, with $i = 1$ to N. Furthermore, $\underline{f}_x$, $\underline{f}_y$, $\underline{m}_z$ are the N-dimensional vectors of forces F_{xi} , F_{yi} and moments M_{zi} with respect to the X_i , Y_i and Z_i axes, respectively, with $i = 1$ to N. Each $\underline{K}_{ij}$ element is an $N \times N$ stiffness matrix with i and j given by the combinations of x, y and z. In isotropic buildings, the horizontal stiffness matrices $\underline{K}_n$ of all elements have the following form (Anastassiadis 1985):

$$\underline{K}_n = k_n \cdot \underline{K}_0, \quad n = 1, 2, \dots \quad (7.18)$$

where k_n is a numerical coefficient and $\underline{K}_0$ is a constant reference matrix of order N (e.g. the horizontal stiffness matrix of an element of the system). Furthermore, it has been proven that matrices $\underline{K}_{ij}$ in Eq. (7.17) have the following form (Anastassiadis 1985; Athanatopoulou et al. 2006):

$$\underline{K}_{ij} = k_{ij} \cdot \underline{K}_0, \quad i, j = x, y, z \quad (7.19)$$

where the coefficients $k_{ij} = k_{ji}$ are calculated as functions of k_n . By taking into consideration Eq. (7.18) and Eq. (7.19), the static equilibrium of the N-storey building with respect to the (X,Y,Z) reference axes of Eq. (7.17) may then be re-written as:

$$\begin{bmatrix} k_{xx}\underline{K}_0 & k_{xy}\underline{K}_0 & k_{xz}\underline{K}_0 \\ k_{yx}\underline{K}_0 & k_{yy}\underline{K}_0 & k_{yz}\underline{K}_0 \\ k_{zx}\underline{K}_0 & k_{zy}\underline{K}_0 & k_{zz}\underline{K}_0 \end{bmatrix} \cdot \begin{bmatrix} \underline{u}_x \\ \underline{u}_y \\ \underline{\theta}_z \end{bmatrix} = \begin{bmatrix} \underline{f}_x \\ \underline{f}_y \\ \underline{m}_z \end{bmatrix} \quad (7.20)$$

The procedure defining the properties of the torsionally coupled system and the N-storey plane frame, which is described analytically in (Anastassiadis 1989; Athanatopoulou et al. 2006), leads then to the following systems of equations:

$$[k_I \cdot \underline{K}_0] \cdot \underline{u}_I = \underline{f}_I, \quad [k_{II} \cdot \underline{K}_0] \cdot \underline{u}_{II} = \underline{f}_{II}, \quad [k_{III} \cdot \underline{K}_0] \cdot \underline{\theta}_{III} = \underline{m}_{III} \quad (7.21)$$

where the terms $\underline{f}_I$, $\underline{f}_{II}$ and $\underline{m}_{III}$ are the N-dimensional vectors of loads and $\underline{u}_I$, $\underline{u}_{II}$ and $\underline{\theta}_{III}$ are the N-dimensional vectors of displacements with respect to the principal reference axes (I, II, III). Moreover, k_I , k_{II} , k_{III} are numerical coefficients associated to the principal directions of the torsionally coupled single-storey system that are calculated, along with angle ω , by the diagonalization of the stiffness matrix coefficients of the torsionally coupled single-storey system, i.e. k_{ij} of Eq. (7.19). Finally, $\underline{K}_0$ corresponds to the stiffness matrix of the N-storey plane frame.

The fundamental periods of the structure along the principal axes I and II are the principal fundamental periods, T_I and T_{II} , respectively. These periods are uncoupled in the principal reference system (I, II, III) and can be calculated as the fundamental periods of vibration when the masses of all diaphragms are located on the vertical principal axis III. The calculation of T_I and T_{II} can alternatively be made by considering the structural reference axes (X, Y, Z) and by determining the fundamental period of vibration in two cases. Firstly, T_I is the fundamental period when the translational dofs that correspond to the direction $\omega+90^\circ$, i.e. orientation of axis II, as well as the torsional dofs of all diaphragms are restrained. Secondly, T_{II} is the fundamental period when the translational dofs that correspond to the direction ω , i.e. orientation of axis I, as well as the torsional dofs of all diaphragms are restrained. The uncoupled fundamental periods of the structure may also be determined for an arbitrary orientation α , from Eq. (7.5).

The procedure developed in Section 7.2.3 for single-storey buildings can be then fully implemented by replacing the principal stiffness matrix coefficients of Eq. (7.2) with the k_I , k_{II} , k_{III} coefficients defined in Eq. (7.21). Therefore, the ASI_{crit} can be explicitly determined in multi-storey buildings that have a real elastic axis. It is important to note that the elastic axis of such buildings, i.e. multi-storey buildings with isotropic properties, is a property of the structure (Makarios and Anastassiadis 1998). Therefore it doesn't depend on the vertical distribution of the lateral forces. As a consequence, the ASI_{crit} doesn't depend on such vertical distribution either. Also, the storey displacements and the

interstorey drifts of points located along the same vertical axis reach their maximum values for the same ASI_{crit} . An illustrative example addressing the calculation of ASI_{crit} for this type of structures is presented in Section 7.5.

7.3.2 Buildings without a real elastic axis

7.3.2.1 Determination of the critical angle of seismic incidence of the storey displacements

Since the elastic axis and the principal bending directions are good descriptors of the behaviour of a building, efforts have been made to extend and generalize these concepts for general multi-storey buildings (Makarios and Anastassiadis 1998; Marino and Rossi 2004; Athanatopoulou and Doudoumis 2008; Doudoumis and Athanatopoulou 2008). These studies determined an *optimum torsional axis* representing an approximation of the real elastic axis and an extension of the principal directions, herein termed *optimum principal directions*. The *optimum torsional axis* is defined as the vertical axis for which a preselected vertical distribution of lateral forces minimizes the sum of the squares of the rotations of all the diaphragms (the optimum torsion criterion according to Makarios and Anastassiadis (1998)). In addition, the *optimum principal directions* are defined as the two horizontal and orthogonal directions associated with the minimum and maximum translational stiffness of the building (Athanatopoulou and Doudoumis 2008). According to Doudoumis and Athanatopoulou (2008), a vertical distribution of lateral forces is initially assumed for the determination of the *optimum torsional axis*. Subsequently, the same distribution is applied along the *optimum torsional axis* and the orientation of the *optimum principal axis* I_{opt} is defined and denoted by angle α_{01} with respect to axis X (Athanatopoulou and Doudoumis 2008). The *optimum principal axis* II_{opt} is defined as the axis orthogonal to α_{01} and is denoted by angle α_{02} . The uncoupled fundamental periods that correspond to these directions are considered to be the *optimum principal periods* and are termed $T_{I_{opt}}$ and $T_{II_{opt}}$, respectively. These periods are calculated as follows: $T_{I_{opt}}$ is the fundamental period when the translational dofs that correspond to the direction α_{02} , i.e. the orientation of the *optimum principal axis* II_{opt} , as well as the torsional dofs of all diaphragms are restrained. Similarly, $T_{II_{opt}}$ is the fundamental period when the translational dofs that correspond to the direction α_{01} , i.e. the orientation of the *optimum principal axis* I_{opt} , as well as the torsional dofs of all diaphragms are restrained. Consequently, the *optimum uncoupled fundamental periods* for an arbitrary orientation α can be approximately expressed by:

$$T_{unc.opt.}(\alpha) = \frac{T_{Iopt} \cdot T_{IIopt}}{\sqrt{\cos(\alpha - \alpha_{01})^2 \cdot T_{IIopt}^2 + \sin(\alpha - \alpha_{01})^2 \cdot T_{Iopt}^2}} \quad (7.22)$$

Based on the strict definition of the *optimum torsional axis*, it can be deduced that analysing the structural behaviour of a general multi-storey building as a combination of an N-storey plane frame and a torsionally coupled system is not possible, since the translational dofs along the two orthogonal directions are always coupled with torsion. Hence, the application of the procedure described in Section 7.3.1 for isotropic multi-storey buildings can't be directly implemented. To overcome this limitation, an alternative procedure is proposed herein for the determination of the ASI_{crit} in general multi-storey buildings. According to this procedure, a 3 dof stiffness matrix is defined initially that corresponds to that of an individual storey of a multi-storey building. Subsequently, the procedure presented in Section 7.2 is applied in the current section for the storey displacements and extended in the following section for the interstorey drifts (ISD).

The manipulation of each storey separately is achieved by using the static condensation technique proposed by Guyan (1965). According to this technique, the definition of a stiffness matrix condensed to the dofs of one storey, i.e. two translational and one rotational dofs, is performed. In the condensation procedure, the dofs of the i^{th} storey are defined as the masters (m) while the dofs of the rest of the storeys are defined as the slaves (s). The static equilibrium of the structure is then expressed by:

$$\begin{bmatrix} \underline{K}_{mm} & \underline{K}_{ms} \\ \underline{K}_{sm} & \underline{K}_{ss} \end{bmatrix} \cdot \begin{Bmatrix} \underline{u}_m \\ \underline{u}_s \end{Bmatrix} = \begin{Bmatrix} \underline{F}_m \\ \underline{F}_s \end{Bmatrix} \quad (7.23)$$

where $[\underline{u}_m \ \underline{u}_s]^T$ is the displacement vector of the master and the slave dofs and $[\underline{F}_m \ \underline{F}_s]^T$ is the force vector acting on the master and on the slave dofs. The equilibrium of the condensed system is given by (Guyan 1965):

$$\underline{K}_R \cdot \underline{u}_m = \underline{F}_R \quad (7.24)$$

where $\underline{K}_R$ is the condensed stiffness matrix that corresponds to the master dofs and $\underline{F}_R$ is the equivalent force vector acting on the master dofs, respectively, expressed by (Guyan 1965):

$$\underline{K}_R = \underline{K}_{mm} - \underline{K}_{ms} \cdot \underline{K}_{ss}^{-1} \cdot \underline{K}_{sm} \quad (7.25)$$

$$\underline{F}_R = \underline{F}_m - \underline{K}_{ms} \cdot \underline{K}_{ss}^{-1} \cdot \underline{F}_s \quad (7.26)$$

Based on this condensation procedure, a methodology similar to the one presented in Section 7.2 for single-storey buildings can also be established for the determination of the ASI_{crit} of the displacement in multi-storey buildings. In this case, the K_{ij} coefficients of Eq. (7.1) are replaced by $\underline{K}_R$, obtained by Eq. (7.25). Moreover, the lateral unit force $\underline{F}_{"1"}$ of Eq. (7.6) is expressed by $\underline{F}_R$ from Eq. (7.26) after it is vertically distributed and it is noted $\underline{F}_{R"1"}$. The vertical distribution of the unit force is defined according to a standard prescribed distribution (i.e. a triangular or a uniform distribution) and needs to have the same pattern adopted for the definition of the *optimum torsional axis*.

After determining $\underline{K}_R$, the 3×3 stiffness matrix condensed for the i^{th} storey, its diagonalization is performed and a new reference system representing the *fictitious principal system* of the i^{th} storey is defined $C_S(I, II, III)$. Subsequently, the geometrical and mechanical parameters $K_I, K_{II}, K_{III}, C_S, x_A', y_A', x_{Cs}', y_{Cs}'$ and ω , referred hereon as the *fictitious parameters* of the i^{th} storey, are determined following the procedure presented in Section 7.2.1 for single-storey buildings. While defining $\underline{K}_R$ and the *fictitious parameters* is straightforward, defining $\underline{F}_R$, however, depends on the vertical distribution of the lateral forces when $\underline{K}_{ms}$ is not equal to zero, i.e. when the building doesn't have isotropic properties. Hence, for general multi-storey buildings, the overall procedure is conditional to the considered vertical distribution of the lateral forces.

As for single-storey buildings, the displacement of C_S (i.e. the *fictitious centre of stiffness* of the i^{th} storey) due to the seismic action applied along an arbitrary orientation α can be expressed as the product of the displacement due to a unit static force multiplied by the base shear. Both terms are expressed along the same orientation α , which is defined with respect to the initial reference system that was used to perform the static condensation. Following the procedure presented for single-storey buildings, $\underline{F}_{R"1"}$ is projected onto the *fictitious principal system* of the i^{th} storey according to Eq. (7.7) and using the angle ω instead of ω .

Similar to Eq. (7.8), the displacement of C_S due to a uniaxial seismic action is given by:

$$\begin{bmatrix} u_{I,\text{uniax}}^{C_s}(\alpha) \\ u_{II,\text{uniax}}^{C_s}(\alpha) \\ \theta_{III,\text{uniax}}(\alpha) \end{bmatrix} = F(\alpha) \cdot \begin{bmatrix} \frac{F_{I,R''1''}^{C_s}(\alpha)}{K_I} \\ \frac{F_{II,R''1''}^{C_s}(\alpha)}{K_{II}} \\ \frac{F_{I,R''1''}^{C_s}(\alpha) \cdot y_{Cs}' - F_{II,R''1''}^{C_s}(\alpha) \cdot x_{Cs}' + M_{III,R''1''}(\alpha)}{K_{III}} \end{bmatrix} = F(\alpha) \cdot \begin{bmatrix} u_{I,R''1''}^{C_s}(\alpha) \\ u_{II,R''1''}^{C_s}(\alpha) \\ \theta_{III,R''1''}(\alpha) \end{bmatrix} \quad (7.27)$$

where $F_{I,R''1''}^{C_s}(\alpha)$ and $F_{II,R''1''}^{C_s}(\alpha)$ are the projections of $\underline{F}_{R''1''}$ onto the *fictitious principal axes* I and II , respectively, and $M_{III,R''1''}(\alpha)$ is a torsional moment that is generated during the condensation of the force vector and is related to the effect of $\underline{F}_s$ when K_{ms} is not equal to zero (see Eq. (7.26)). The base shear action $F(\alpha)$ is given by Eq. (7.11) using Eq. (7.22) for the determination of the *optimum uncoupled fundamental periods* and by following the rules presented in the Appendix. The displacement vector of an arbitrary point A in the plan-view of the i^{th} storey $[u_{I,F}^A(\alpha) \ u_{II,F}^A(\alpha) \ \theta_{III,F}(\alpha)]^T$ can then be determined based on rigid body kinematics. The base shear and the displacements due to the orthogonal component of the seismic action are similarly defined and denoted as $F(\alpha+90^\circ)$ and $[u_{I,F}^A(\alpha+90^\circ) \ u_{II,F}^A(\alpha+90^\circ) \ \theta_{III,F}(\alpha+90^\circ)]^T$, respectively.

The resultant displacement (i.e. the *final demand*) of an arbitrary point A of the i^{th} storey is expressed by:

$$u^A(\alpha) = \sqrt{(u_I^A(\alpha))^2 + (u_{II}^A(\alpha))^2} = \sqrt{(F(\alpha) \cdot u_{I,R''1''}^A(\alpha) \pm \beta \cdot F(\alpha+90^\circ) \cdot u_{I,R''1''}^A(\alpha+90^\circ))^2 + (F(\alpha) \cdot u_{II,R''1''}^A(\alpha) \pm \beta \cdot F(\alpha+90^\circ) \cdot u_{II,R''1''}^A(\alpha+90^\circ))^2} \quad (7.28)$$

where the displacements $u_{I,R''1''}^A(\alpha)$ and $u_{II,R''1''}^A(\alpha)$ are the displacements of A due to the application of $\underline{F}_{R''1''}$ along direction α , and $u_{I,R''1''}^A(\alpha+90^\circ)$ and $u_{II,R''1''}^A(\alpha+90^\circ)$ are the displacements of A due to $\underline{F}_{R''1''}$ applied along the orthogonal direction, i.e. $\alpha+90^\circ$. Finally, the α_{crit} is determined by applying Eq. (7.16).

7.3.2.2 Determination of the critical angle of seismic incidence of the interstorey drifts

The procedure developed in the previous Section is extended herein for the determination of the ASI_{crit} of the ISD of two arbitrary points of two consecutive storeys that are vertically aligned. The ISD is one of the most common EDPs to express structural demand

at a global (system) level and has been extensively used by many researchers (e.g. see (Lagaros 2010a; Cantagallo et al. 2015)) and seismic standard provisions (e.g. see (CEN 2004; ASCE 2014)). In addition, cost-related recommendations have been established (e.g. FEMA P58 2012) that correlate the ISD ratio to expected losses under earthquake loading. Hence, determining the ASI that maximizes this parameter is of particular interest.

The determination of this ASI_{crit} requires Eqs. (7.23) – (7.26) to be applied for the two adjacent storeys for which the ISD is requested. The parameters obtained correspond to the *fictitious parameters* of the *Upper* and the *Lower* storeys. For both storeys and for both orthogonal directions, Eq. (7.27) is then used considering the *fictitious principal directions* of each storey in order to obtain the displacement vectors of the storeys. Subsequently, the displacements resulting from the application of the seismic action along the two orthogonal directions are combined using a directional combination rule, e.g. the percentage combination rule. The displacements obtained separately for the two storeys are measured in the *fictitious principal system* of each storey. In the general case, the *fictitious principal systems* of two consecutive storeys are not necessarily aligned. Since the determination of the ISD requires the combination of the displacements of both storeys, these displacements need to be aligned. A common reference system is therefore selected (here the initial reference system (X, Y, Z) is used as a reference) and the displacements of each storey are projected onto that system according to:

$$\begin{bmatrix} u_{X,F}^{A, i^{th} storey}(\alpha) \\ u_{Y,F}^{A, i^{th} storey}(\alpha) \end{bmatrix} = \begin{bmatrix} \cos(-\omega_{i^{th} storey}) & \sin(-\omega_{i^{th} storey}) \\ -\sin(-\omega_{i^{th} storey}) & \cos(-\omega_{i^{th} storey}) \end{bmatrix} \cdot \begin{bmatrix} u_{I,F}^{A, i^{th} storey}(\alpha) \\ u_{II,F}^{A, i^{th} storey}(\alpha) \end{bmatrix} \quad (7.29)$$

with ω_{Upper} and ω_{Lower} representing $\omega_{i^{th} storey}$ for the *Upper* and the *Lower* storeys, respectively. The final expression that leads to the ISD of an arbitrary point A, ISD^A , as a function of α , is then given by:

$$ISD^A(\alpha) = \sqrt{(u_X^{A, Upper}(\alpha) - u_X^{A, Lower}(\alpha))^2 + (u_Y^{A, Upper}(\alpha) - u_Y^{A, Lower}(\alpha))^2} / h_{storey} \quad (7.30)$$

where h_{storey} is the height of the storey and the subscripts *Upper* and *Lower* refer to the displacements of the *Upper* and *Lower* storeys, respectively. Since all parameters of Eq. (7.30) are functions of α , the α_{crit} may be determined by Eq. (7.16) using the $ISD^A(\alpha)$ function instead of $u^A(\alpha)$.

7.4 Summarised presentation of the proposed procedure

The proposed methodology establishes a straightforward procedure to determine the ASI_{crit} in the context of LFA. The ASI_{crit} is determined for two EDPs commonly used for seismic safety assessment according to current earthquake-related standards. These EDPs are the storey displacements and the ISDs. The data required to apply the procedure is:

- The building configuration (geometry and materials)
- The response spectrum defining the earthquake demand
- The vertical distribution of the base shear.

The latter affects the results in multi-storey buildings without isotropic properties, while it has no effect in isotropic buildings, as explained earlier. The procedure comprises three main steps:

1st step: Determination of the base shear as a function of α

The orientation of the principal axis I (for the case of single-storey buildings and multi-storey buildings with isotropic properties) or of the *optimum principal axis* I_{opt} (for the case of general multi-storey buildings) is initially determined. In the first case, the orientation is determined according to (Roussopoulos 1932) and expressed by the angle ω , while in the second case its determination is performed according to Athanatopoulou and Doudoumis (2008) and is expressed by the angle α_{01} . The principal periods, i.e. T_I and T_{II} , or the *optimum principal periods*, i.e. T_{Iopt} and T_{IIopt} , are then determined to establish Eqs. (7.5) or (7.22), respectively. Finally, Eq. (7.11) defines the base shear as a function of α by following the rules presented in the Appendix. The same procedure is performed for the orthogonal component of the seismic action, expressed by $\alpha+90^\circ$.

2nd step: Determination of the displacement due to constant lateral loading as a function of α .

The procedure described in this step determines the displacement of a storey with respect to the orientation α of a unit force. This step is performed once in case of buildings with a real elastic axis, while it is applied at each storey separately in case of general multi-storey buildings. For the latter case the static condensation technique is used (Guyan 1965) to reduce the number of dofs of the structure to three, i.e. the dofs of the storey under consideration (Eqs. (7.23)-(7.26)). The three-dof stiffness matrix is subsequently transformed from the initial coordinate system onto the principal reference system, for

buildings with a real elastic axis, or onto the *fictitious principal system*, for any other case, according to Roussopoulos (1932). Simultaneously, the main mechanical and geometrical parameters, K_I , K_{II} , K_{III} , ω , x_{Cs}' , y_{Cs}' , x_A' , y_A' , are obtained for the first category of buildings. For the second category of buildings, the *fictitious mechanical* and *geometrical* parameters K_I , K_{II} , K_{III} , ω , x_{Cs}' , y_{Cs}' , x_A' , y_A' of the i^{th} storey are obtained instead. The principal system, or the *fictitious principal system*, allow the displacement of the storey to be expressed only as a function of the angle α of the constant force. The latter is defined by Eq. (7.10) for buildings with isotropic properties and by the second term of the right part of Eq. (7.27) for buildings without isotropic properties. Although the angle α is defined with respect to axis X, it is noted that the forces introduced into Eq. (7.10) and Eq. (7.27) need to be aligned with the principal directions. The projection onto these axes is performed by Eq. (7.7). As a result the orientation of the displacements produced within this step is along the principal axes. The same procedure is applied for the determination of the displacement due to the orthogonal component of the seismic action, expressed by $\alpha+90^\circ$.

3rd step: Combination of steps 1 and 2.

Since linear superposition is valid for LFA, the outputs of steps 1 and 2 are combined in this step according to Eq. (7.12) and Eq. (7.27). The effects of the two orthogonal components of the seismic action are also combined using a combination rule provided by the standard under consideration, e.g. the β percentage rule or the SRSS applied in Eq. (7.13) and Eq. (7.14), respectively. Finally, the resultant displacement (*final demand*) is derived by combining the displacements along the two orthogonal axes, Eq. (7.15) and Eq. (7.28), and α_{crit} is determined by maximizing these expressions using Eq. (7.16). For the case of ISDs, an additional intermediate step is required before the formulation of the final expression. This step consists on the alignment of the displacements of the two consecutive storeys according to Eq. (7.29) before obtaining the *final demand* using Eq. (7.30) and α_{crit} using Eq. (7.16).

7.5 Examples of application

7.5.1 Initial considerations

The proposed methodology is applied and validated for three reinforced concrete structures with different configurations in plan and elevation. To cover a wider range of buildings, the selected structures are a single-storey building, a three-storey isotropic

building and a five-storey building without isotropic properties. The modulus of elasticity of the material of the three buildings is equal to 25 GPa and the moment of inertia of all structural elements is reduced to 50% according to standard provisions to account for the effect of cracking (CEN 2004; CEN 2005). All structures are considered fixed at the ground level. Further details regarding the configuration of each building are given in the following Sections.

The seismic action is represented by the Type 1 elastic response spectrum provided by EC8-1 (CEN 2004) for ground type B. The parameters describing the spectrum are $S = 1.2$, $T_B = 0.15$ sec, $T_C = 0.50$ sec, $T_D = 2.00$ sec, 5% viscous damping ($\eta = 1$) and a ground acceleration $a_g = 0.35g$. The corresponding elastic response spectrum is presented in Figure 7.3.

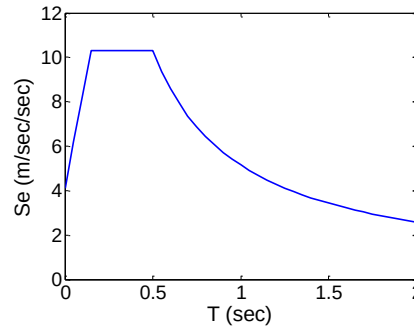


Figure 7.3 Elastic response spectrum defining the considered earthquake demand

The analytical procedure is applied to determine the ASI_{crit} of storey displacements and ISDs for preselected elements (columns and walls) and for the CM of all storeys of the three structures. The values of the ASI_{crit} are then validated by performing a parametric LFA involving all the non-redundant ASIs analysed in steps of 1° . The validation procedure involves the following steps. First, the uncoupled fundamental periods of each building along every ASI are determined. The uncoupled fundamental period along an arbitrary direction corresponds to the period of the first mode of vibration when the rotational dofs and the translational dofs of the orthogonal direction of all diaphragms are fixed. Secondly, the base shear corresponding to each uncoupled period is computed using the response spectrum of Figure 7.3. Each building is then subjected to LFA for all the ASI-dependent orthogonal couples of base shears. The 30% combination rule is used to combine the demand of each direction and the *final demands*, i.e. the resultant displacement and resultant ISD for every ASI, are then derived. The ASIs that maximize the EDPs are eventually derived from the parametric analysis and are compared to the

ones determined analytically. All structural analyses are performed using the finite element analysis software OpenSees (McKenna 2011), while the calculation of Eq. (7.16) is performed using a commercial software package (Maple 18 2002). The units in the next Sections are expressed in kN and m for force and length, respectively.

7.5.2 Single-storey building

The first case study comprises a single-storey building with the in-plan configuration shown in Figure 7.4. The height of the structure is 4 m and the mass is equal to 50 tons, concentrated at the CM. The cross section sizes and the local axes of the columns are shown in Figure 7.4 (a). The cross section of all beams is $0.25 \times 0.50 \text{ m}^2$. Although this structure does not correspond to a realistic building, it was chosen in order to represent a complex case study where the principal directions do not coincide with the main structural axes and the principal periods fall on different branches of the response spectrum.

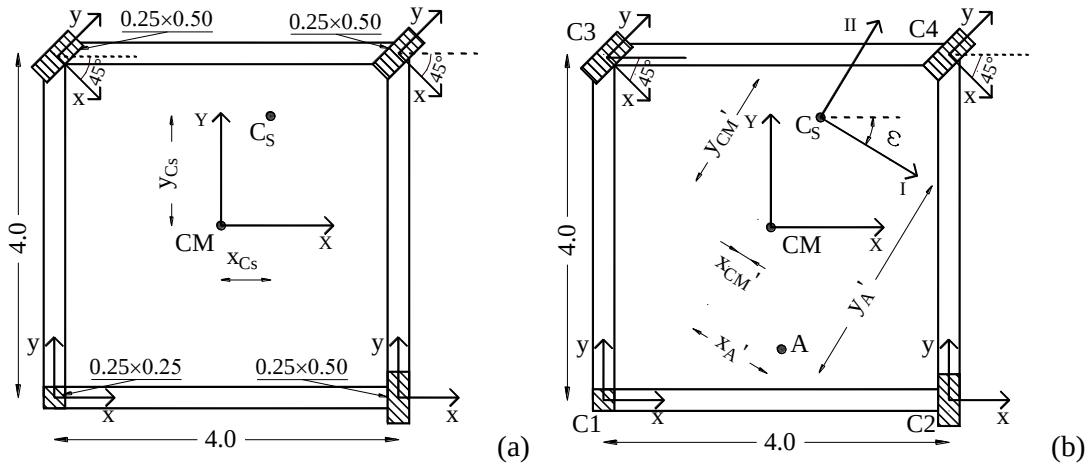


Figure 7.4 Plan view of the single-storey building with the geometric characteristics (a) and the principal system (b) (all dimensions are in m)

Following the procedure presented in Section 7.2 for single-storey structures, the coordinates of the elastic centre C_S $x_{Cs} = 0.436 \text{ m}$ and $y_{Cs} = 1.031 \text{ m}$ (shown in Figure 7.4 (a)) and the angle $\omega = -31.61^\circ$ (shown in Figure 7.4 (b)) are determined which then lead to the principal reference system of the building and to the following principal stiffness matrix:

$$K_{I,II,III} = \begin{bmatrix} 5793.13 & 0 & 0 \\ 0 & 12218.07 & 0 \\ 0 & 0 & 72142.79 \end{bmatrix} \quad (7.31)$$

The principal periods are $T_I = 0.584$ sec and $T_{II} = 0.402$ sec. It is observed that $T_C < T_I < T_D$ and $T_B < T_{II} < T_C$, thus the base shear evolution expressed by Eq. (7.11) involves 8 branches (see the Appendix). The limits of each branch are defined by the angle that corresponds to the corner period T_C which is calculated from Eq. (7.5) assuming $\omega = 0$ and equals to $\theta_{T_C} = 34.9^\circ$. Since $\theta_{T_C} < 45^\circ$, Eq. (7.11) follows the rules of Figure 7.11 (a) of the Appendix. The resultant displacement of an arbitrary point in plan-view (*final demand*) for every ASI is determined according to Eq. (7.15). The maximization of the expression according to Eq. (7.16) leads to the ASI_{crit} of the point under consideration. The critical angle is determined for all four columns of the structure as well as for the CM and the results are summarized in Table 7.1. The point under examination and its coordinates in the principal reference system are presented in the first column. The second column contains the $ASIs_{crit}$ determined by the analytical expression ($\alpha_{ana.}$) while the third column presents the $ASIs_{crit}$ derived from the parametric analysis ($\alpha_{par.}$). The comparison of the two angles is performed in the fourth column of the same table.

Table 7.1 The $ASIs_{crit}$ of the single-storey structure

Storey displacements			
Point (coord. (m))	$\alpha_{ana.}$	$\alpha_{par.}$	$\alpha_{ana.} - \alpha_{par.}$
CM (0.17,-1.11)	114.35°	114.00°	0.35°
C1 (-0.43,-3.83)	114.35°	114.00°	0.35°
C2 (2.97,-1.73)	118.29°	118.00°	0.29°
C3 (-2.48,-0.43)	114.35°	114.00°	0.35°
C4 (0.88, 1.68)	114.35°	114.00°	0.35°

The results show that the proposed methodology adequately predicted the value of the ASI_{crit} of the displacement of all columns and of the CM. The small differences, which are lower than 1° , are the result of selecting a 1° angle step. A higher discretization of the later would eliminate the error and lead to the exact angle. Based on the results, it is important to highlight the fact that, for the EDPs that were considered, the $ASIs_{crit}$ doesn't coincide with 0° , 90° or with the principal directions. The evolution of the resultant displacement of the CM, extracted from the parametric analysis, is presented in Figure 7.5 along with the angle 114.35° determined from the proposed methodology (indicated by the marker). The maximization of the response is evident at the angle $\alpha_{ana.}$ as well as the underestimation of the response when the angle equals to 0° and 90° .

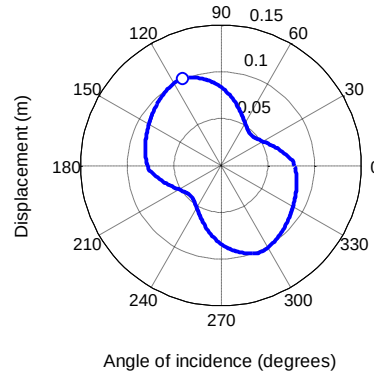


Figure 7.5 The resultant displacement of the CM of the single-storey building subjected to parametric LFA; the value of α_{ana} is defined by the marker

7.5.3 Isotropic three-storey building

The second case study is a three-storey building with isotropic properties. The plan view of a typical storey and of the typical planar frame are illustrated in Figure 7.6 (a) and (b), respectively. All beams and columns have constant cross sections, which are defined in Figure 7.6 (b). The mass of each storey is 142 tons and is concentrated at the CM.

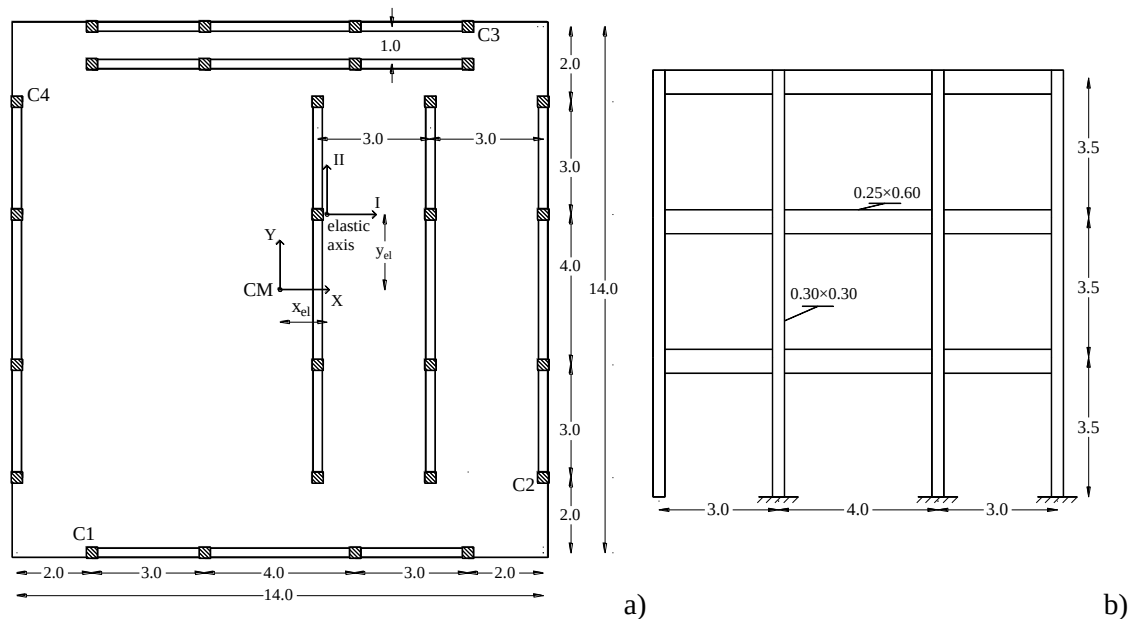


Figure 7.6 Plan view of a typical storey of the 3-storey isotropic building (a). View in elevation of a typical frame (b) (all dimensions are in m)

Since the building consists of vertical resisting elements with proportional stiffness matrices, it possesses a real elastic axis and principal bending directions. The lateral stiffness matrix with respect to the CM (X,Y,Z) reference system is:

$$\underline{K} = \begin{bmatrix} 3\underline{K}_0 & 0\underline{K}_0 & -6\underline{K}_0 \\ 0\underline{K}_0 & 4\underline{K}_0 & 5\underline{K}_0 \\ -6\underline{K}_0 & 5\underline{K}_0 & 249\underline{K}_0 \end{bmatrix} \quad (7.32)$$

where $\underline{K}_0$ is the 3×3 lateral stiffness matrix of the individual frame featured in Figure 7.6 (b):

$$\underline{K}_0 = \begin{bmatrix} 55664.8 & -32738.2 & 1769.6 \\ -32738.2 & 61642.2 & -30359 \\ 1769.6 & -30359 & 288668.1 \end{bmatrix} \quad (7.33)$$

The location of the elastic axis is determined, $x_{el} = 1.25$ m and $y_{el} = 2.00$ m, and shown in Figure 7.6 (a). The orientation of the principal reference system is also presented in the same figure. Since all frames are aligned in two orthogonal directions that coincide with axes X and Y, the principal axes I and II are parallel to them and $\omega = 0^\circ$. The principal stiffness matrix is subsequently defined by:

$$\underline{K}_{princ} = \begin{bmatrix} 3\underline{K}_0 & 0\underline{K}_0 & 0\underline{K}_0 \\ 0\underline{K}_0 & 4\underline{K}_0 & 0\underline{K}_0 \\ 0\underline{K}_0 & 0\underline{K}_0 & 230.75\underline{K}_0 \end{bmatrix} \quad (7.34)$$

The fundamental uncoupled periods along the principal directions are $T_I = 0.76$ sec and $T_{II} = 0.66$ sec. Since both T_I and T_{II} belong to the third branch of the spectrum, Eq. (7.11) has only one branch:

$$F(\alpha) = \frac{a_g \cdot S \cdot \eta \cdot 2.5 \cdot T_C \cdot \sqrt{\cos(\alpha)^2 \cdot T_{II}^2 + \sin(\alpha)^2 \cdot T_I^2}}{T_I \cdot T_{II}} \quad (7.35)$$

where a_g , S , η , T_C are given in Section 7.5.1. The base shear along the orthogonal direction is also expressed using Eq. (7.35) for an angle $\alpha + 90^\circ$. Consequently, Eq. (7.15) has only one branch as well.

Due to the isotropic properties of the building, all vertically aligned points attain their maximum displacements for the same ASI_{crit} . The same is also valid for the maximum ISDs. The critical angle is determined for Columns C1-C4 shown in Figure 7.6 (a) as well as for the CM and the results are summarized in Table 7.2. In the first column of the table, the point under consideration is presented as well as its coordinates in the principal reference system. The angle α_{ana} , determined from the proposed methodology and the

angle $\alpha_{\text{par.}}$ derived from the parametric analysis are presented in columns 2 and 3 of Table 7.2, respectively. Finally, the comparison of the two angles is performed in the fourth column of the table.

Table 7.2 The ASI_{crit} of the isotropic three-storey building

1 st 2 nd and 3 rd storey displacements and ISDs			
Point (coord (m))	$\alpha_{\text{ana.}}$	$\alpha_{\text{par.}}$	$\alpha_{\text{ana.}} - \alpha_{\text{par.}}$
CM (-1.25, 2.00)	143.05°	143.00°	0.05°
C1 (-6.25, -9.00)	133.88°	134.00°	-0.12°
C2 (5.75, -7.00)	151.67°	152.00°	-0.33°
C3 (3.75, 5.00)	6.82°	7.00°	-0.18°
C4 (-8.25, 3.00)	114.72°	115.00°	-0.28°

The parametric LFA was performed by assuming a triangular distribution of the base shear. Nonetheless, it is worth noting that, in isotropic buildings, the selection of the vertical distribution of the base shear only affects the magnitude of the displacements and not the ASI_{crit} . The results show that the methodology was able to predict adequately the ASI_{crit} in this case study as well, with small deviations that are also due to the size of the angle step. Such as for the single-storey building, further discretization of the angle step would eliminate the error. It is also observed that different columns achieve their maximum displacement under different $\text{ASIs}_{\text{crit}}$ and that none of them coincide with the principal directions of the structure. Finally, the evolution of the resultant displacements and the ISDs of the CM for all three storeys obtained from the parametric analysis is presented in Figure 7.7. The $\alpha_{\text{ana.}}$ is also presented and indicated by the marker. It is observed that the maximum displacement and the maximum ISD occur under the same ASI (i.e. 143°) for all three stories, as expected due to the isotropic properties of the building. To be able to represent the evolutions of the two demand parameters in Figure 7.7, the ISD evolutions shown are replaced by those of the differential displacement of two adjacent storeys (i.e. the ISD defined by Eq. (7.30) multiplied by the corresponding storey height). Therefore, the evolutions of the storey displacement and of the ISD are coincident in the first storey plot.

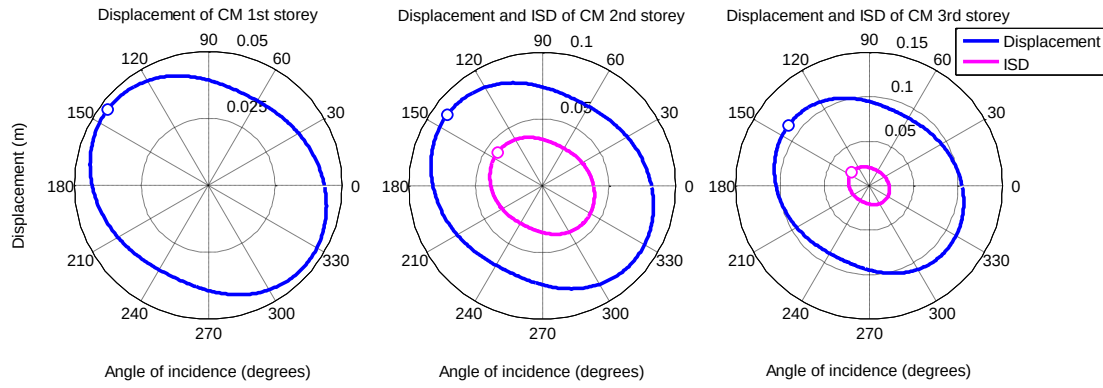


Figure 7.7 The resultant displacements and ISDs of the CM for each storey of the three-storey isotropic building subjected to parametric LFA. The α_{ana} are defined by the marker

7.5.4 General five-storey building

The last case study involves a five-storey building with the planar configuration featured in Figure 7.8. The building has a 4.0 m high first storey and four upper storeys 3.2 m high each. The cross sections of all vertical elements, presented in Figure 7.8, are constant throughout the height. The cross sections of all beams are $0.25 \times 0.60 \text{ m}^2$. The mass of each storey is equal to 268.6 tons, concentrated at the CM. The building does not have isotropic properties, thus it does not have a real elastic axis nor principal directions. For the application of the proposed methodology, an *optimum torsional axis* and the *optimum principal directions* need to be defined in advance. The determination of these properties depends on the vertical distribution of the lateral forces selected for the analysis. In the current example, only the triangular distribution is considered. The ASI_{crit} is determined for all four walls at their centroid, i.e. W1, W2, W3, W4, as well as for the CM (see Figure 7.8).

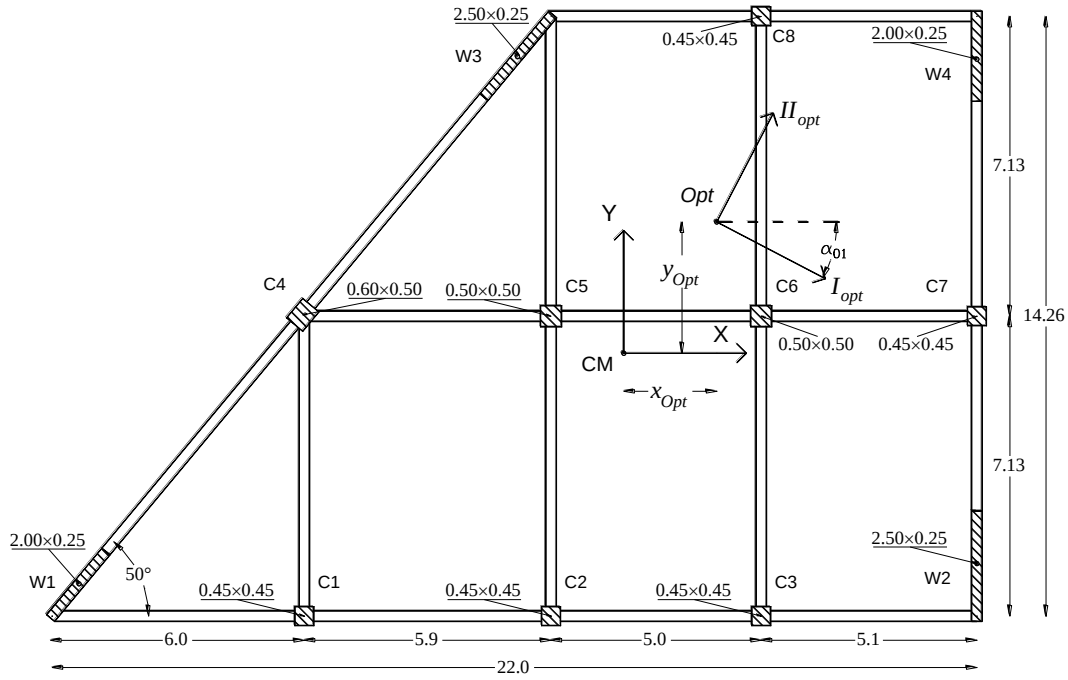


Figure 7.8 Plan view of a typical storey of the 5-storey building (all dimensions are in m)

Following the procedure presented in Section 7.3.2, the base shear given by Eq. (7.11) is initially determined as a function of α . Assuming a triangular distribution of lateral forces and following the procedure proposed by Doudoumis and Athanatopoulou (2008) and Athanatopoulou and Doudoumis (2008), the coordinates of the *optimum torsional axis* $x_{opt} = 2.21$ m and $y_{opt} = 3.12$ m are determined as well as the orientation of the *optimum principal directions* I_{opt} and II_{opt} , $\alpha_{01} = -27.54^\circ$ and $\alpha_{02} = \alpha_{01} + 90^\circ$ (shown in Figure 7.8). The *optimum principal periods* are then $T_{I_{opt}} = 0.73$ sec and $T_{II_{opt}} = 0.53$ sec. Assuming an ellipsoid evolution of the fundamental periods, Eq. (7.22), the base shear is obtained by Eq. (7.11) using the response spectrum of Figure 7.3. According to the Appendix, Eq. (7.11) only has one branch since both $T_{I_{opt}}$ and $T_{II_{opt}}$ belong to the third branch of the spectrum:

$$F(\alpha) = \frac{a_g \cdot S \cdot \eta \cdot 2.5 \cdot T_C \cdot \sqrt{\cos(\alpha - \alpha_{01})^2 \cdot T_{II_{opt}}^2 + \sin(\alpha - \alpha_{01})^2 \cdot T_{I_{opt}}^2}}{T_{I_{opt}} \cdot T_{II_{opt}}} \quad (7.36)$$

where a_g , S , η , T_C are given in Section 7.5.1. The base shear along the orthogonal direction, expressed by the angle $\alpha+90^\circ$, is also obtained by Eq. (7.36).

The second step of the methodology, which is performed for each storey separately, comprises the determination of the displacement due to a unit force as a function of α .

The parameters that correspond to the first storey are presented analytically for illustrative purposes. The stiffness matrix $\underline{K}_R$ condensed to the dofs of the first storey is derived from the 15×15 global stiffness matrix using Eq. (7.25) and its diagonalization is performed leading to the *fictitious principal matrix* of the first storey:

$$\underline{K}_{I,II,III} = \begin{bmatrix} 380937.80 & 0 & 0 \\ 0 & 1441428.42 & 0 \\ 0 & 0 & 58138858.62 \end{bmatrix} \quad (7.37)$$

The *fictitious parameters* of the first storey are determined ($x_{Cs} = 4.14$ m, $y_{Cs} = 8.50$ m and $\omega = -20.99^\circ$) as well as the coordinates of the walls (at their centroids) and the CM with respect to the *fictitious principal system*. These coordinates are presented in the first column of Table 7.3. A unit horizontal force expressed as a function of α is vertically distributed following the triangular pattern. The unit force that corresponds to the orthogonal direction, expressed by the angle $\alpha + 90^\circ$, is also defined. For both force vectors, Eq. (7.26) is applied for the derivation of the force vectors $\underline{F}_{R"1"}$ acting on the masters, i.e. the dofs of the first storey. Both force vectors are subsequently projected onto the *fictitious principal axes* using Eq. (7.7) and the displacement vector due to the unit force is obtained.

The third step of the methodology is the combination of the results obtained from the two preceding steps. Thus, Eq. (7.28) is applied for the selected vertical elements of the first storey and the $ASIs_{crit}$ of the first storey displacements are determined. The same procedure is applied for all five storeys and the resulting $ASIs_{crit}$ are presented in column 2 of Table 7.3. Similarly, the $ASIs_{crit}$ of the ISDs of the same points are determined by forming and maximizing Eq. (7.30) and are presented in column 6 of Table 7.3. The results obtained from the parametric analysis are also presented in the table, in column 3 with respect to storey displacements and in column 7 for the ISDs. Finally, columns 4 and 8 show the comparison between the analytical and the parametric results for the storey displacements and the ISDs, respectively.

It can be observed that the proposed methodology predicted the $ASIs_{crit}$ with small errors for both EDPs. In the majority of the cases the error is lower than 1° , which corresponds to an acceptable numerical error considering that the angle discretization of the parametric analysis is also 1° . In a few cases, however, the error is higher than 1° and increases up to 1.60° and 3.37° for storey displacements and ISDs, respectively.

Table 7.3 The ASI_{crit} of the five-storey building

1 st Storey displacements			
Point (coord (m))	$\alpha_{ana.}$	$\alpha_{par.}$	$\alpha_{ana.} - \alpha_{par.}$
CM (-0.82,-9.41)	122.93°	124.00°	-1.07°
W1 (-10.95,-19.18)	119.70°	120.00°	-0.30°
W2 (8.82,-11.07)	127.28°	128.00°	-0.72°
W3 (-5.70,-3.74)	118.92°	120.00°	-1.08°
W4 (4.52, 0.12)	131.04°	131.00°	0.04°
2 nd Storey displacements			
CM (-0.72,-6.63)	120.68°	121.00°	-0.32°
W1 (-10.54,-16.71)	116.99°	118.00°	-1.01°
W2 (8.96,-7.99)	126.53°	127.00°	-0.47°
W3 (-5.78,-1.11)	115.40°	117.00°	-1.60°
W4 (4.32, 3.06)	129.82°	130.00°	-0.18°
3 rd Storey displacements			
CM (-0.62,-4.93)	118.66°	120.00°	-1.34°
W1 (-10.08,-15.35)	114.88°	116.00°	-1.12°
W2 (9.10,-5.94)	125.60°	126.00°	-0.40°
W3 (-5.88, 0.41)	112.48°	114.00°	-1.52°
W4 (4.06, 4.94)	127.82°	128.00°	-0.18°
4 th Storey displacements			
CM (-0.53,-3.82)	116.71°	118.00°	-1.29°
W1 (-9.52,-14.64)	113.00°	114.00°	-1.00°
W2 (9.23,-4.40)	124.50°	125.00°	-0.50°
W3 (-6.02, 1.28)	109.83°	111.00°	-1.17°
W4 (3.72, 6.25)	125.38°	126.00°	-0.62°
5 th Storey displacements			
CM (-0.43,-2.92)	114.53°	116.00°	-1.47°
W1 (-8.73,-14.28)	111.08°	112.00°	-0.92°
W2 (9.35,-2.88)	123.16°	124.00°	-0.84°
W3 (-6.22, 1.83)	107.01°	108.00°	-0.99°
W4 (3.18, 7.40)	122.03°	123.00°	-0.97°

1 st - 2 nd ISD			
Point	$\alpha_{ana.}$	$\alpha_{par.}$	$\alpha_{ana.} - \alpha_{par.}$
CM	116.97°	118.00°	-1.03°
W1	113.15°	114.00°	-0.85°
W2	124.63°	125.00°	-0.37°
W3	110.17°	111.00°	-0.83°
W4	125.99°	127.00°	-1.01°
2 nd - 3 rd ISD			
CM	111.27°	113.00°	-1.73°
W1	108.20°	109.00°	-0.80°
W2	120.88°	121.00°	-0.12°
W3	102.95°	104.00°	-1.05°
W4	116.62°	119.00°	-2.38°
3 rd - 4 th ISD			
CM	102.66°	104.00°	-1.34°
W1	101.35°	102.00°	-0.65°
W2	113.59°	115.00°	-1.41°
W3	93.87°	95.00°	-1.13°
W4	98.63°	102.00°	-3.37°
4 th - 5 th ISD			
CM	83.19°	83.00°	0.19°
W1	84.50°	85.00°	-0.50°
W2	89.50°	90.00°	-0.50°
W3	78.03°	78.00°	0.03°
W4	76.36°	76.00°	0.36°

The discrepancies between the two angles (the real one determined from the parametric analysis and the one determined from the proposed analytical methodology) are attributed to two main factors. The first source of error is connected to the assumption that the evolution of the fundamental uncoupled periods for different angles follows an ellipse (see Eq. (7.22)). In general multi-storey buildings the fundamental periods are always coupled. The degree of coupling affects the validity of the assumed elliptical shape of the periods and, consequently, the value of the base shear which will then directly affect the accuracy of the results. The second source of error is connected to the fact that the *fictitious principal stiffness matrix* of a given storey does not hold the real properties of the principal stiffness matrix of single-storey buildings or isotropic multi-storey buildings. Instead, it is an approximation to the real principal properties and it was adopted in the proposed methodology to be able to determine an equivalent principal system (i.e. the *fictitious principal system*). Nevertheless, these assumptions are expected to yield reasonable results for buildings that are regular in elevation, i.e. buildings that meet the requirements for LFA by common seismic design/assessment standards. These requirements aim to identify buildings governed by translational fundamental periods (one in each main axis of the structure) and with limited influence of higher modes (CEN 2004). Although EC8-1 does not clearly define how to identify such type of buildings, the 35% and 50% limits on the increase and decrease, respectively, of mass and relative stiffness between adjacent storeys proposed by the Greek seismic design standard (EAK 2003) may be considered as a reference. The current case study has a stiffness increase of 25% between the first and the second storeys and has yielded satisfactory results. This five-storey building was also selected in order to be representative of a building typology in which the *fictitious centres of stiffness* of all storeys are located far from the *optimum torsional axis*. Furthermore, the *fictitious principal axes* also have different orientations when compared to those of the *optimum principal directions*. Both features indicate a structure with properties far from isotropic. Given these characteristics, it is observed that the referred ASI_{crit} differences are low and acceptable from an engineering point of view (less than 5%). Still, more buildings with different typologies need to be analysed to validate the use of the proposed methodology for general multi-storey buildings.

The results that were obtained also indicate that all the considered EDPs attain their maximum value for a different ASI_{crit} , as found in other previous studies. Unlike the case of isotropic buildings, columns that belong to the same vertical axis have different ASI_{crit}

both in terms of displacements and ISDs. In addition, the results also show that the ASI_{crit} of the ISDs do not coincide with the ASI_{crit} of any of the storeys involved, i.e. the *Lower* or *Upper*, following the notation of Section 7.3.2. These conclusions are illustrated in Figure 7.9 which shows the evolution of the displacements, Figure 7.9(a), and of the ISDs, Figure 7.9(b), of wall W4 for all storeys, obtained from the parametric analysis. It is observed that the ASI_{crit} of wall W4 changes along the height for both EDPs studied. These changes are more pronounced for the ISDs, particularly for the last storey. This change is adequately captured by the analytical methodology since the error that was obtained is low (see Table 7.3). In the same figure, the values of the $\alpha_{ana.}$ are also identified to illustrate the fact that the maximization of the response is always well captured by the analytical procedure. As in Figure 7.7, the ISD evolutions shown in Figure 7.9 are replaced by those of the differential displacement of two adjacent storeys.

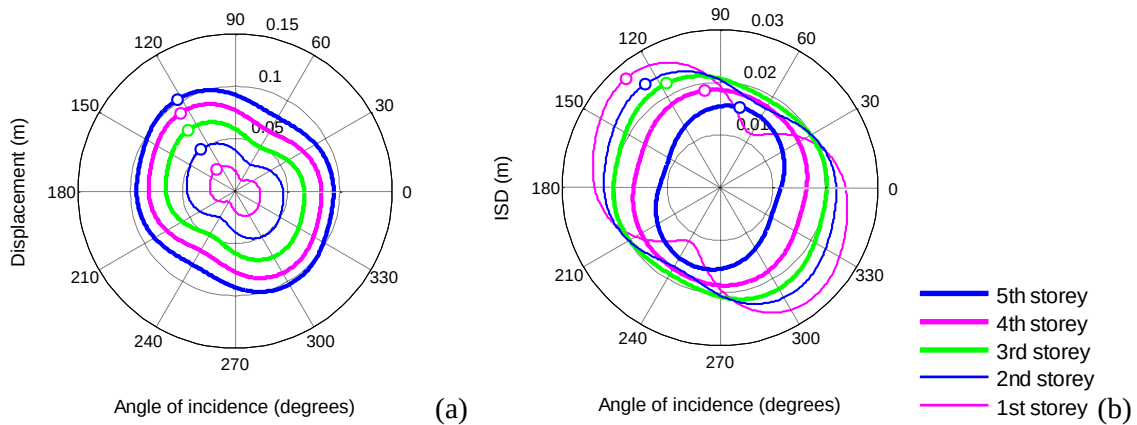


Figure 7.9 The evolution of the resultant displacements (a) and ISDs (b) of W4 for the five storeys of the 5-storey building subjected to parametric LFA. The $\alpha_{ana.}$ are identified by the marker

7.6 Conclusions

An analytical methodology was developed for the determination of the ASI_{crit} in the context of LFA. Two widely used global demand parameters were considered, namely the storey displacement and the ISD, and analytical expressions were derived to determine their ASI_{crit} . The developed methodology is based on fundamental concepts addressing the static behavior of linear elastic single-storey structures that were developed in the 1930's and are still applicable nowadays within standardised methods of analysis. The application of the proposed analytical methodology only requires information about the material and geometrical characteristics of the structure and the seismic action

represented by a response spectrum. For general multi-storey buildings, the vertical distribution of the lateral forces is also required and needs to be decided in advance.

The methodology was applied and validated for three case studies. With respect to the first two, i.e. a single-storey and a three-storey isotropic building, the exact value of the ASI_{crit} was determined using the proposed analytical methodology. The small reported errors were related with the resolution of the angle step adopted for the parametric analysis. For the last case study, a five-storey building without isotropic properties, the ASI_{crit} was also adequately predicted. The few, yet acceptable, errors that were found are attributed to two sources related to simplified assumptions that are considered by the methodology. Finally, it is noted that the analytical methodology may be easily incorporated into existing software for the earthquake analysis of structures for the straightforward determination of the ASI_{crit} and the corresponding *critical demand*.

7.7 Appendix

Derivation of the piecewise expression for the final demand:

The terms $F(\alpha)$ and $F(\alpha+90^\circ)$ included in the expression of the *final demand*, which is expressed by Eq. (7.15) (for buildings with a real elastic axis), Eq. (7.28) (for buildings without a real elastic axis) and Eq. (7.30) (for the ISD), depend on the corner periods T_B and T_C of the response spectrum and on the principal periods of the structure T_I and T_{II} . As a result, the referred equations may have different forms according to the following scenarios:

- $T_B \leq T_{II}$ and $T_I \leq T_C$ – In this case all $T_{unc}(\alpha)$ fall on the horizontal branch of the spectrum and the spectral acceleration S_e is constant for all angles α . Therefore, $F(\alpha)$ and $F(\alpha+90^\circ)$ obtained from Eq. (7.11) are also constant and equal to F_{const} for all angles α . Eqs. (7.15), (7.28) and (7.30) have one branch in which the base shear can be omitted in the maximization procedure since it is not a function of the angle α . For example, Eq. (7.15) has the following form:

$$u^A(\alpha) = F_{const}^2 \cdot \sqrt{\left(u_{I,1''}^A(\alpha) \pm \beta \cdot u_{I,1''}^A(\alpha+90^\circ)\right)^2 + \left(u_{II,1''}^A(\alpha) \pm \beta \cdot u_{II,1''}^A(\alpha+90^\circ)\right)^2} \quad (7.38)$$

- $0 < (T_{II}, T_I) \leq T_B$ or $T_C \leq (T_{II}, T_I) \leq T_D$ – In this case all $T_{unc}(\alpha)$ fall on the first or the third branch of the spectrum and S_e is a function of the angle α . Since both S_e

(α) and $S_e(\alpha+90^\circ)$ belong to the same branch of the spectrum, $F(\alpha)$ and $F(\alpha+90^\circ)$ are expressed by the same equation. Hence, Eqs. (7.15), (7.28) and (7.30) have one branch in which $F(\alpha)$ and $F(\alpha+90^\circ)$ are expressed as functions of the angle α . For example, the terms $F(\alpha)$ and $F(\alpha+90^\circ)$ of Eq. (7.13) which lead to the final displacement in Eq. (7.15) are given by Eq. (7.39) assuming $\omega = 0$.

$$F(\alpha) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{T_I \cdot T_{II}}, \quad F(\alpha+90^\circ) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{T_I \cdot T_{II}}$$

$$\sqrt{\cos(\alpha)^2 \cdot T_{II}^2 + \sin(\alpha)^2 \cdot T_I^2} \quad \sqrt{\cos(\alpha+90^\circ)^2 \cdot T_{II}^2 + \sin(\alpha+90^\circ)^2 \cdot T_I^2}$$

(7.39)

- All other combinations of T_{II} and T_I – To illustrate this scenario, the case in which $T_C \leq T_I \leq T_D$ and $T_B \leq T_{II} \leq T_C$ (see Figure 7.10 (a)) is presented herein and the same rationale can be followed for any other combination.

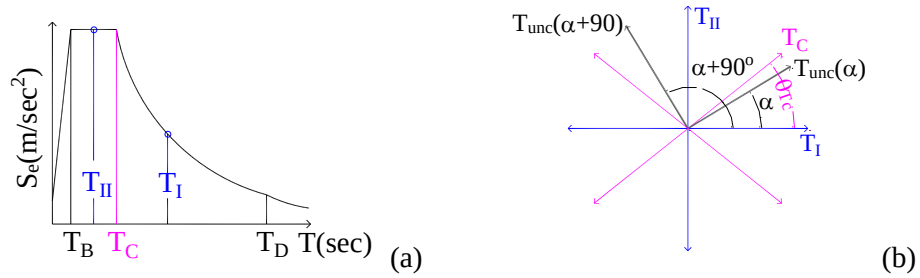


Figure 7.10 Position of T_I and T_{II} on the response spectrum (a) and relative position of the two orthogonal T_{unc} (b)

In this case, $T_{unc}(\alpha)$ falls on different branches of the spectrum depending on the angle α . As a result, S_e is expressed by different equations as a function of α and $F(\alpha)$ has multiple branches, where at least one of these branches is a function of α . The base shear of the perpendicular direction $F(\alpha+90^\circ)$ is expressed following the same rules. The relative position of $T_{unc}(\alpha)$ and $T_{unc}(\alpha+90^\circ)$ for a given angle α , shown in Figure 7.10 (b), affects the development of Eqs. (7.15), (7.28) and (7.30). It can be seen in Figure 7.10 (b) that the angle θ_{T_C} corresponding to the corner period T_C determines the angle range of the branches of the final expression. To illustrate the formulation of Eqs. (7.15), (7.28) and (7.30), the relative position of $T_{unc}(\alpha)$ and $T_{unc}(\alpha+90^\circ)$ is presented in Figure 7.11 for all ASIs and for all three cases with respect to the angle θ_{T_C} . The solid hatched areas represent the position of $T_{unc}(\alpha)$, while the dashed hatched areas represent the respective position of $T_{unc}(\alpha+90^\circ)$. As a result, Eqs. (7.15), (7.28) and (7.30) comprise eight branches with

specific angle range limits. The terms $F(\alpha)$ and $F(\alpha+90^\circ)$ of Eq. (7.15) that are used to form the piecewise equation are presented in Eq. (7.40) for the first case of Figure 7.11, i.e. $\theta_{Tc} < 45^\circ$ and $\omega = 0$.

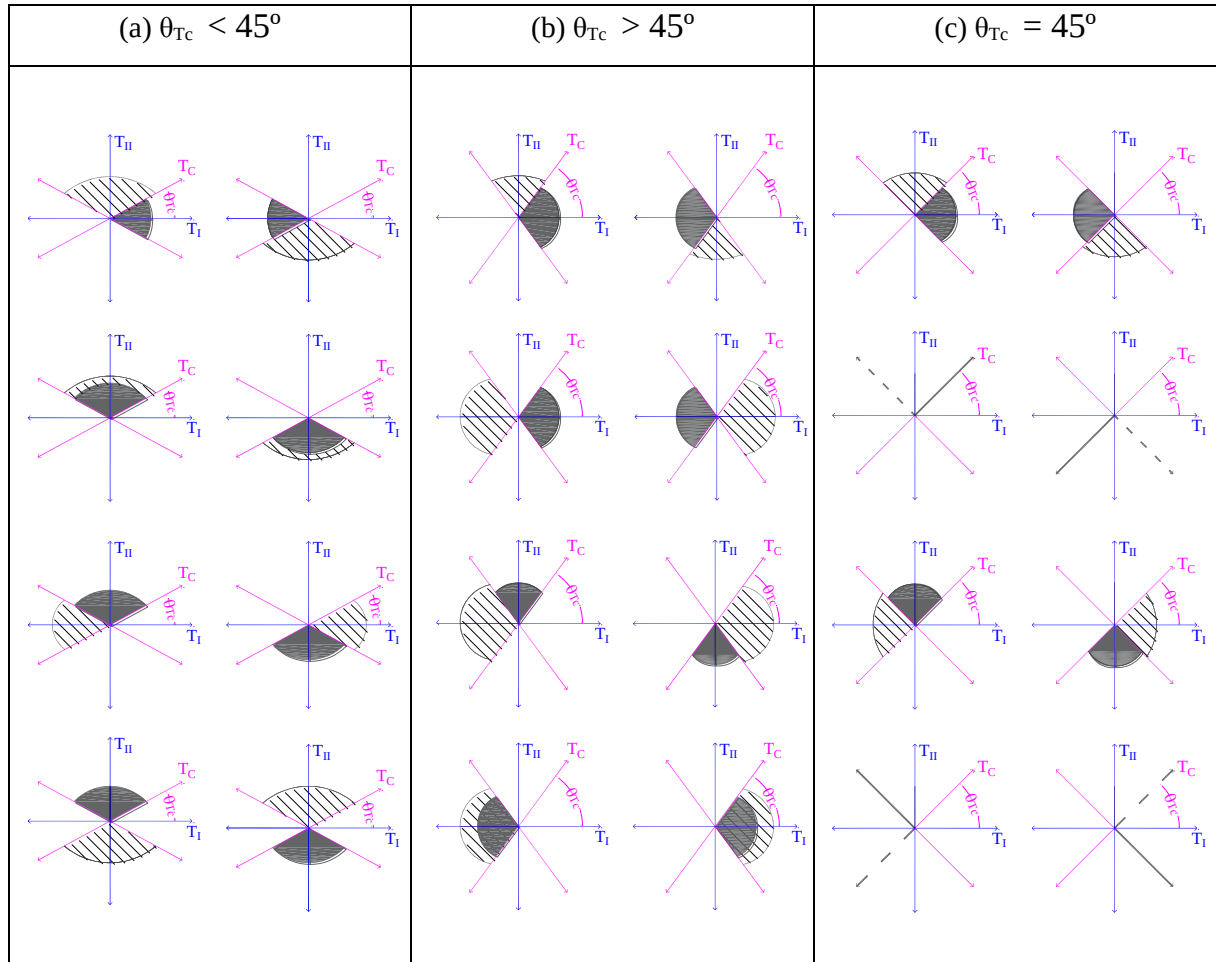


Figure 7.11 Determination of the branches of the piecewise equation

$$\left\{ \begin{array}{ll}
\begin{array}{l} 2\pi - \theta_{T_C} \leq \alpha \leq \theta_{T_C} \\ \text{and} \\ \theta_{T_C} \leq \alpha + 90^\circ \leq \pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{\frac{T_I \cdot T_{II}}{\sqrt{\cos(\alpha)^2 \cdot T_{II}^2 + \sin(\alpha)^2 \cdot T_I^2}}}, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5 \\
\\
\begin{array}{l} \theta_{T_C} \leq \alpha \leq \pi - \theta_{T_C} \\ \text{and} \\ \theta_{T_C} \leq \alpha + 90^\circ \leq \pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5 \\
\\
\begin{array}{l} \theta_{T_C} \leq \alpha \leq \pi - \theta_{T_C} \\ \text{and} \\ \pi - \theta_{T_C} \leq \alpha + 90^\circ \leq \pi + \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{\frac{T_I \cdot T_{II}}{\sqrt{\cos(\alpha + 90^\circ)^2 \cdot T_{II}^2 + \sin(\alpha + 90^\circ)^2 \cdot T_I^2}}} \\
\\
\begin{array}{l} \theta_{T_C} \leq \alpha \leq \pi - \theta_{T_C} \\ \text{and} \\ \pi + \theta_{T_C} \leq \alpha + 90^\circ \leq 2\pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5 \\
\\
\begin{array}{l} \pi - \theta_{T_C} \leq \alpha \leq \pi + \theta_{T_C} \\ \text{and} \\ \pi + \theta_{T_C} \leq \alpha + 90^\circ \leq 2\pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{\frac{T_I \cdot T_{II}}{\sqrt{\cos(\alpha)^2 \cdot T_{II}^2 + \sin(\alpha)^2 \cdot T_I^2}}}, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5 \\
\\
\begin{array}{l} \pi + \theta_{T_C} \leq \alpha \leq 2\pi - \theta_{T_C} \\ \text{and} \\ \pi + \theta_{T_C} \leq \alpha + 90^\circ \leq 2\pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5 \\
\\
\begin{array}{l} \pi + \theta_{T_C} \leq \alpha \leq 2\pi - \theta_{T_C} \\ \text{and} \\ 2\pi - \theta_{T_C} \leq \alpha + 90^\circ \leq \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = \frac{T_C \cdot a_g \cdot S \cdot \eta \cdot 2.5}{\frac{T_I \cdot T_{II}}{\sqrt{\cos(\alpha + 90^\circ)^2 \cdot T_{II}^2 + \sin(\alpha + 90^\circ)^2 \cdot T_I^2}}} \\
\\
\begin{array}{l} \pi + \theta_{T_C} \leq \alpha \leq 2\pi - \theta_{T_C} \\ \text{and} \\ \theta_{T_C} \leq \alpha + 90^\circ \leq \pi - \theta_{T_C} \end{array} & \Rightarrow F(\alpha) = a_g \cdot S \cdot \eta \cdot 2.5, \quad F(\alpha + 90^\circ) = a_g \cdot S \cdot \eta \cdot 2.5
\end{array} \right.$$

(7.40)

Chapter 8

8 Closure

8.1 Conclusions

The primary focus of the dissertation was the evaluation of the effect of the angle of seismic incidence (ASI) during the assessment of the seismic behaviour of reinforced concrete (RC) buildings. Considering the inherent differences between the different representations of the seismic action, two distinct approaches of examination of the aforementioned effect were established. Those are, the probabilistic approach, which involves a group of ground motions (GMs) and examines statistics of the output, and the deterministic approach, which focuses on a *critical* -most adverse- output and involves a response spectrum.

8.1.1 Probabilistic approach

With respect to the probabilistic approach, the current state-of-the-art for assessment PEER-PBEE framework was applied and the effect of the ASI was determined in all relevant stages. The application of the framework was performed using six RC buildings. The seismic hazard was identified using probabilistic seismic hazard analysis and was represented by a conditional spectrum (CS). Four CSs were developed for each building, each corresponding a different probability of exceedance of a spectral acceleration. Each CS was then used as a target spectrum for the selection of a group of 40 far-field GMs, resulting in 4 groups of 40 GMs for each building. The 4 groups of 40 GMs were further scaled to cover more intensities, but keeping the scale factor at low levels. Ultimately, 20 intensity levels were considered for each building and all 40 GMs of each intensity were

applied on the buildings along 12 ASIs, i.e. multi-stripe analysis (MSA) was performed using multiple ASIs.

The first stage of the PEER-PBEE framework that implicates the ASI is the demand stage. In this stage, all relevant engineering demand parameters (EDPs) are collected from structural analyses and are further processed in order to be used in the subsequent stages. Three EDPs were examined in the research and it was shown that, depending on the purpose of the analysis and on the EDP under consideration, the ASI could have either a negligible or a determinant effect. More specifically, it was shown that if only mean/median values of the EDPs are required (e.g. in the context of standard-based analysis procedures), using only 1 ASI could be sufficient as long as a large enough GM group size was also considered. In most of the cases, though, using 2 or 3 ASIs appeared to be a safer choice, while the required number of ASIs was further dependent on the level of admissible risk of missing a certain confidence interval. On the other hand, it was demonstrated that using more than 4 ASIs doesn't seem to provide relevant benefits. The appropriate number of ASIs was also found to depend on the GM group size: considering larger group sizes were seen to require a lower number of ASIs to fully account for the effect of the ASI.

When information about the variability of the EDP distribution was required as well, using only 1 ASI was seen to be inadequate regardless of the GM group size. In that case, using 2 to 4 ASIs was highly suggested, depending again on the GM group size. Unlike the estimation of the central tendencies, for the estimation of the variability, 2 ASIs seemed to be necessary even with large GM group sizes (i.e. 30 and 35), while for smaller GM group sizes (i.e. 20 and 25) up to 4 ASIs could be required. Estimating the variability of the peak floor acceleration (PFA) is, however, an exception to this rule since it appeared to be insensitive to the effect of the ASI when 30 or more GMs were used. Furthermore, in most cases where GM group sizes of 20 or 25 were used, the variability of the PFA could also be captured by considering only 2 ASIs. Finally, it is worth restating that, for all the statistics considered, the GM group size had the most determinant effect and the use of at least 20 GMs is suggested in all situations.

Still in the seismic demand stage, the type of the statistical model that best fits the EDP datasets when the ASI is accounted for was also examined. In particular, the unconditional use of the lognormal model was investigated, as well as the possibility of using alternative statistical models. A preliminary, mainly qualitative, study was performed in order to

compare the performance of alternative distribution models to fit the EDP data. Five models were employed additional to the lognormal model and their performance was assessed based on Q-Q plots and two measures of linear regression, the coefficient of determination (R^2) and the root-mean-square error (RMSE). Both measures showed that the Gumbel model is the most inappropriate to characterize the demand distribution of all EDPs and intensities. The normal and the Weibull distributions exhibited good fitting performance for linear and low inelastic structural response. For higher inelastic structural response, however, the fitting of these distributions was characterized by significant variability and, especially for the normal model, by much lower performance. The lognormal and the Gamma distributions showed adequate fitting performance for all cases with a variability that generally increased when increasing the nonlinearity of the structural behaviour. The performance of the Fréchet model was particularly interesting since it showed very low performance for most of the cases, but appeared to provide an adequate fit in some cases where all alternative models were inappropriate.

Based on the previous, mainly qualitative, results, the lognormal, the Gamma, the Weibull and the Fréchet models were further examined using appropriate goodness-of-fit tests. The lower rejection rates were obtained for the Gamma and the lognormal model, in agreement with the conclusions reached using the R^2 and the RMSE results. The higher rejection rates were observed for the Weibull model that was then considered to be the least appropriate one, and for the Fréchet model, especially for elastic structural response. The number of ASIs was seen to affect the rejection rates. In particular, the lognormal and the Gamma models exhibited higher rejection rates when a larger number of ASIs was involved in the sample. This was highlighted by the goodness-of-fit plots that showed a change in the shape of the empirical data with the increase in the number of ASIs. Still, no specific trend was able to be identified regarding the effect of the number of ASIs on the shape of the empirical data. Overall, it was concluded that the most appropriate model to fit the examined EDP data, coming from any combination of number of ASIs and number of ground motions, is the lognormal model. Regarding the Gamma model, although the rejection rates that were found are particularly low (similar or even lower than those found for the lognormal model), the lower reliability of the test found in the power analysis indicates that it should be used with caution. Nevertheless, the similarity of the alternative distributions for the examined parameter range is also an indication that alternative models could be used without significant consequences.

Moving on to the following stage of the PEER-PBEE framework, i.e. the damage stage, the effect of the ASI and of the GM group size were studied on the probability of failure as well as on the failure risk. In this stage, failure was associated to local or global collapse of the building, using specific collapse criteria. The influence of the number of ASIs on the risk variation was found to be low and it only appeared to have an effect when small GM group sizes were considered, i.e. smaller than 25. Despite the smaller effect the number of ASIs had on the variability of collapse risk estimates, using only 1 ASI introduces a bias in the median risk estimate. This bias was found to be up to 10% in some cases, leading to an over- or underestimation of the collapse risk. This effect was seen to be independent of the GM group size and could be particularly important for larger group sizes where the probability of obtaining the reference risk value could actually be zero. Nevertheless, for GM group sizes larger than 15, this bias could be adequately corrected by considering 2 ASIs only, i.e. 0° and 90° , while using more ASIs only offered minor additional contributions. The GM group size, on the other hand, was found to have a significant effect on the variability of the collapse risk, causing a reduction of the collapse risk variation when it increases. The uncertainty introduced in the collapse risk estimate when 15 GMs or less were considered was found to be significant and no improvement was observed by increasing the number of ASIs. It was therefore suggested to use a minimum of 20 GMs to determine collapse risk.

The results obtained for the buildings that were analysed indicate that using a group of 20 or more GMs in combination with 2 ASIs (0° and 90°) can provide an adequate (i.e. unbiased) estimate of the collapse risk, regardless of the regularity of the building's in-plan configuration. The proposed minimum number of GMs prevents the occurrence of large variations around the median due to the limited sample size, while the 2 ASIs remove the bias of the median estimate of collapse risk. An alternative approach was also explored involving only 1 ASI, randomly selecting 0° or 90° , to reduce the computational cost related to the number of dynamic analyses. The results showed that this procedure reduces the bias of the median, but it can be associated with a larger dispersion of the risk estimates.

In the final stage of the PEER-PBEE framework, the effect of the GM group size and of the ASI was analysed on the expected annual losses (EAL). The results obtained for the six buildings showed that the median value of EAL (EAL_{median}) was not significantly influenced by the GM group size or the ASI. The GM group size was found to have a

significant influence on the EAL variability, with the latter reducing with the increase of the former. The effect of the ASI on the variability, on the other hand, was shown to be much smaller and to induce only minor fluctuations on the variability of the total EAL and of the EAL of each individual component. As a result, the use of only 1 ASI is considered to be adequate for the estimation of the EAL. The importance of having a large enough number of observations for probabilistic analysis was also emphasised, since extreme outliers were observed when only 10 GMs were used applied along 1 ASI. In that case, outliers were seen to disappear by increasing the number of ASIs to at least 2 or, more efficiently, by increasing the number of GMs to at least 15. Overall, for the analysed buildings, the use of 15 GMs along 1 ASI was found to be adequate for the EAL estimation. Yet, depending on the required level of precision, using a larger GM group size could be advantageous since it was shown to reduce the EAL variability. Increasing the number of ASIs, however, did not provide additional benefits

Summarising, the following were concluded:

- The effect of the ASI decreases when progressing through the stages of the PEER-PBEE framework.
- More than 2 ASIs are necessary to estimate most of the EDP distributions.
- Using the lognormal model to characterise the EDP distributions is a valid assumption.
- Two ASIs are adequate to estimate the expected value of collapse risk.
- The use of only 1 ASI is adequate to estimate the EAL.
- The GM group size has a larger effect, when compared to that of the ASI, and the use of at least 20 GMs is suggested to reduce the variability of the results in all stages of the framework.

8.1.2 Deterministic approach

The second approach was developed in order to be applicable in existing standardised procedures. The most commonly applied analysis procedures are the response spectra analysis and the lateral force analysis (LFA), in which linear elastic material behaviour is assumed and the seismic action is represented by a response spectrum. Given that relevant solutions have already been provided for situations involving response spectrum analysis, the thesis focused on providing a solution for the case where LFA is involved. An analytical methodology was developed for the determination of the ASI_{crit} in the context of LFA. Two widely used global demand parameters were considered, namely the storey

displacement and the interstorey drift (ISD), and analytical expressions were derived to determine their ASI_{crit} . The developed methodology is based on fundamental concepts addressing the static behaviour of linear elastic single-storey structures that were developed in the 1930's and are still applicable nowadays within standardised methods of analysis. The application of the proposed analytical methodology only requires information about the material and geometrical characteristics of the structure and the seismic action represented by a response spectrum. For general multi-storey buildings, the vertical distribution of the lateral forces is also required and needs to be decided in advance.

The methodology was applied and validated for three case studies. With respect to the first two, i.e. a single-storey and a three-storey isotropic building, the exact value of the ASI_{crit} was determined using the proposed analytical methodology. The small reported errors were related with the resolution of the angle step adopted for the parametric analysis. For the last case study, a five-storey building without isotropic properties, the ASI_{crit} was also adequately predicted. The few, yet acceptable, errors that were found are attributed to two sources related to simplified assumptions that are considered by the methodology. Finally, it is noted that the analytical methodology may be easily incorporated into existing software for the earthquake analysis of structures for the straightforward determination of the ASI_{crit} and the corresponding critical demand.

8.1.3 Concluding remarks

The present dissertation dealt with the long lasting question of how to account for the effects of the ASI when assessing the seismic behaviour of RC buildings. With respect to the probabilistic approach, it was shown that, depending on the requested outcome of the analysis, the ASI may have a determinant effect or its influence can be trivial. Suggestions to account for these effects were made for each stage of the analysis individually, showing that the effect of the ASI reduces as we advance through the stages of the probabilistic framework. It was also shown that the size of the group of GMs involved in the analysis has a much more significant effect in the outcome of the analysis and thus relevant suggestion were made as well. As far as the deterministic/code oriented approach is concerned, an analytical approach was developed for the determination of the critical ASI when using LFA. The procedure is straightforward and determines the critical ASI for specific EDPs, commonly used for the assessment of existing buildings, using the building characteristics, as well as the shape of the RS.

8.2 Limitations and future work

The present dissertation provided insightful results on the effect of the ASI in probabilistic analyses, as well as a solution for the determination of the critical ASI in deterministic analyses. However, in the pursuit of these goals some assumptions and simplifications were adopted that were necessary for the completion of the research. Even though these assumptions did not compromise the quality or the purpose of the present work, they comprise limitations whose effect should be studied in the future.

As far as the probabilistic approach is concerned, although buildings with different heights and configurations in plan were examined in order to cover a wide variety of buildings, the conclusions reached are bounded to the selected building characteristics, such as the typology, the material, etc. All examined buildings represent low- to mid-rise RC residential buildings, with frame structural systems, while buildings with different materials (e.g. structural steel, composite), structural systems (e.g. wall systems, coupled wall-frame systems), number of storeys, levels of irregularity (in-plan and in-elevation), and buildings with different uses (e.g. offices) were not covered. Changing any of the previously mentioned characteristics has the potential to lead to different conclusions in specific stages or in a series of stages of the PEER-PBEE framework.

Furthermore, the EDPs considered herein mainly constitute parameters related to global structural demand, while EDPs expressing local structural demand were not examined. More particularly, the studied EDPs were the maximum ISD, the maximum roof drift (RD) and the maximum PFA among all storeys, while the effect of the ASI on local EDPs, such as column or beam chord-rotation, sectional forces (axial force, moments, shear force), etc., was not analysed.

Moreover, one of the main assumptions of the present study was that it only considered far-field GMs, whereas near-field GMs with prominent directionality effects were disregarded. In near-field scenarios, the arbitrary orientation of the bi-directional GMs ceases to be valid and the randomness of the ASI can no longer be justified. As a result, in case near-field effects are taken into account, the selection of the ASI should be based in different premises with the potential to have different effects on structural demand.

As far as the GM selection procedure is concerned, it was based on the use of a CS using strict matching criteria. The latter were found to have an effect on the record-to-record variability. Since the effect of the ASI was inevitably connected to the effect of the GM

group, altering the latter can have an effect on the effect of the ASI, as well. As such, the GM selection based on different criteria, such as a standard based procedure using a uniform hazard spectrum and more lenient spectral matching provisions, can be further examined to determine the effect of the ASI.

The probabilistic model was based on Multiple Stripe Analysis (EDP-based procedure) using a different CS, and thus a different group of GMs, for different probabilities of exceedance. Alternative available, widespread methods of defining the probabilistic model, such as Incremental Dynamic Analysis or cloud analysis (IM-based procedures), can be also used with a potential influence on the effect of the ASI.

Regarding the considered uncertainties, the present research focused on uncertainties related to the seismic action, while additional uncertainties were mainly disregarded, except that of the capacity model uncertainty when defining the collapse risk. Additional uncertainties, such as uncertainties related to structural modelling, to the capacity models, as well as uncertainties related to the loss models, can be further taken into account in the future.

Last but not least, with respect to the probabilistic approach, only direct economic losses were considered, while casualties, activity interruptions and downtime were not taken into account into the definition of the loss model. The main reason was due to the lack of existing models to capture the referred losses. Future research on the effect of the ASI these parameter should also be accounted for, given reliable models are available.

Regarding the deterministic approach, although solutions have been provided for the LFA, unresolved and mainly conceptual issues still remain related to the use of the critical demand in practice rather than to its determination. The critical demand can be undoubtedly used in cases where only one EDP is sufficient to express structural demand. Such cases involve, for example, the assessment of structural performance using the roof displacement of the building or the interstorey drift of a given storey. The determination of the critical demand in the aforementioned cases, which is associated to the critical ASI, leads to the most adverse condition. On the contrary, this situation is not applicable in a scenario where more than one EDP is required to express structural demand. A typical example of such cases may involve the assessment of the behaviour of a column in bi-axial flexure with axial force. Given that each EDP reaches its critical value for a different critical ASI, the question that arises is should the critical demand of all parameters be used or simultaneous demands of a critical one? Although some researchers have already

asked the question and tried to tackle it, no definite answer has been provided and the issue remains unsolved.

Appendix A

Appendix A Uncertainty quantification of fragility and risk estimates due to seismic input variability and capacity model uncertainty

Based on:

Skoulidou, D., & Romão, X. (2019). Uncertainty quantification of fragility and risk estimates due to seismic input variability and capacity model uncertainty. *Engineering Structures*, 195, 425-437.

Abstract

When evaluating the probabilistic seismic performance of a structure in the context of Performance-Based Earthquake Engineering (PBEE) applications, only estimates of the true response are obtained due to the finite size of the group of records selected to represent the seismic scenario. The proposed study examines the variability of these estimates, in terms of fragility parameters and failure rates, by implementing a procedure that creates seismic scenario-consistent groups of records based on regrouping criteria applied to a larger set of records. The results are then compared to those obtained from a statistical approach based on the bootstrap resampling procedure, whose validity is disputed due to its incompatibility with common applications involving probabilistic seismic performance assessment. Given that bootstrap resampling assumes that each bootstrap sample is random, this condition is incompatible with requirements involving the need to match specific spectral statistics of the group of records with a target spectrum. Due to the increased computational cost of the procedure based on regrouping criteria,

the effect of this incompatibility is analysed by examining the agreement between the two procedures for several case studies and the conditions under which bootstrap resampling leads to a reduced level of error. Further insights about the variability of the estimates are obtained by analysing scenarios with deterministic and probabilistic thresholds of the limit state capacity. Among other aspects, the results show that, for both capacity thresholds, bootstrap resampling can provide acceptable results as long as a sufficient number of GMs and number of stripes are used.

A.1 Introduction

Practical guidelines provided by the Performance-Based Earthquake Engineering framework for the probabilistic assessment of the seismic performance of a building (FEMA P-58 2012) draw attention to various sources of uncertainty. Common sources are those associated with the seismic input, usually expressed by the seismic hazard uncertainty (e.g. see (Abrahamson and Bommer 2005)) and the record-to-record variability (e.g. see (Haselton et al. 2009)), the structural modelling, such as the variability of geometric properties and material constitutive laws (e.g. see (FEMA P-58 2012)), and limit state definitions, like the capacity modelling errors (e.g. see (fib 2003)). These uncertainties can be accounted for during the structural analysis of the numerical model or in a subsequent post-processing stage of the analysis results. Depending on the stage of the analysis in which the different uncertainties are accounted for, their combination can be performed using Monte Carlo simulation-based techniques or alternative simplified methods, such as first-order second-moment techniques (e.g. see (Gokkaya et al. 2016; Jalayer et al. 2010; Vamvatsikos and Fragiadakis 2010; Celarec and Dolšek 2013; Miano et al. 2017)). Nevertheless, it is commonly agreed that accounting for uncertainties changes the seismic demand, the fragility function parameters (e.g. see Gardoni et al. 2002), the associated seismic risk (e.g. see (Liel et al. 2009; Iervolino 2017)), as well as the expected losses (e.g. see (Bradley & Lee 2010)).

The present research analyses the variability of the fragility function parameter estimates and of the associated seismic risk estimates due to uncertainty of the input seismic action when nonlinear dynamic analysis is performed. In nonlinear dynamic analysis, the size of the considered group of ground motion records is selected in order to capture record-to-record variability and, at the same time, to accommodate availability and computational time-related restrictions. However, due to the finite size of the group of

records, only an estimate of the seismic demand is obtained and it can be argued that a different group of records, representative of the same seismic scenario, would lead to a different estimate of the same demand (Baker and Cornel 2008). Existing procedures commonly used to account for the uncertainty associated to the use of a finite size group of records, termed group-to-group⁷ variability hereon, rely on analytical procedures, such as the Delta method, or on numerical simulations, such as the *bootstrap resampling* (Iervolino 2017; Baker and Cornel 2008; Bradley 2010). The applicability of each procedure depends on the methodology applied to derive the fragility functions, the desired precision as well as time- and computation-related restrictions. Given its widespread use (e.g. see (Tothong and Luco 2007; Rajeev et al. 2008; Silva et al. 2013; Eads et al. 2013)), as well as the possibility to combine multiple sources of epistemic uncertainty (Bradley 2010), the *bootstrap resampling* is found to be the most commonly used procedure to assess group-to-group variability.

One of the main conditions of *bootstrap resampling* is that bootstrap samples are random samples (Efron and Tibshirani 1994). However, this condition is not always compatible with a situation involving the resampling of groups of records (each group of records being a sample) where the properties of each group need to match those of a certain seismic scenario. Enforcing a match between statistics representative of the sample (i.e. the group of records) and parameters of the selected seismic scenario conditions the sample, since it is by combining those specific records that the sample is valid (where valid means that it matches the parameters of the seismic scenario). Therefore, by conditioning the way it is defined, a sample cannot be considered to be random. To take this issue into account, an alternative procedure is proposed that involves creating new groups of records based on a process that regroups selected earthquake records and maintains the compatibility with the seismic scenario for all new groups.

The variability of the estimates of the fragility function parameters and of the associated seismic risk is determined according to the proposed procedure, i.e. the *regrouping* procedure, and is then compared to the variability obtained using the *bootstrap resampling* procedure. Even though *bootstrap resampling* is inconsistent (see Section A.3.1 for a more detailed discussion), since the *regrouping* procedure is more

⁷ The term group-to-group variability is used herein to differentiate from the record-to-record variability, which refers to the variability of the seismic input due to the different properties of the records within the same group.

computationally demanding, the ability of *bootstrap resampling* to provide variability estimates that are adequate is analysed by comparing them to those obtained by the *regrouping* procedure. The comparisons are performed using structural response data obtained from the nonlinear dynamic analysis of three structures for increasing levels of earthquake loading. Furthermore, fragility parameter estimates and risk estimates are analysed for two scenarios of the limit state capacity: one that considers deterministic capacity and one that considers probabilistic capacity, the latter leading to counterintuitive observations regarding the variability of the estimates.

A.2 Modelling the fragility function and the rate of failure

The probability of failure $P(f)$ of the structure given the seismic intensity measure (IM), also termed the fragility function, is often conveniently represented by a lognormal cumulative distribution function (CDF):

$$P[f|IM = im] = \Phi\left(\frac{\log(im / \theta)}{\beta}\right) \quad (A.1)$$

where Φ is the standard normal CDF with parameters $\log(\theta)$ and β . Fragility functions are usually derived based on numerical analysis results obtained from an inelastic structural model subjected to a group of hazard-consistent records applied with various intensities. Currently available methodologies for deriving numerical simulation-based fragility functions can be grouped into IM-based and EDP-based methodologies, according to Baker (2015). A compilation of fragility curve generation procedures based on nonlinear analysis can also be found in Bakalis and Vamvatsikos (2018), including additional information about other features such as the consideration of non-deterministic capacity limit states. For the purpose of the present study, an EDP-based approach (Baker 2015) will be applied, using analytically compatible numerical analysis results from multiple-stripe analyses (MSA) (Jalayer and Cornel 2009). Briefly, MSA involves groups of records, each selected to be hazard-compatible at a specific IM level (i.e. each stripe level), applied to the structure whose response, conditional to the predefined intensity, is defined by Engineering Demand Parameters (EDPs). The generated group of EDPs is also termed a stripe of EDPs and multiple intensity levels are considered to generate multiple stripes of EDPs. The EDPs of each stripe are compared to their corresponding capacities and, for each comparison, a failure/no failure state of the structure is assigned. The term

failure is considered herein as reflecting the inability of the structure to comply with predetermined criteria and can be associated to any predefined limit state or performance measure. Furthermore, for the purpose of the current study, each limit state is conditioned by one EDP. To derive the function of Eq. (A.1) according to the EDP-based approach, the total number of analyses at each IM level z_{IM} is partitioned into the number of analyses that led to failure of the structure c_{IM} and the complementary number of analyses that didn't cause failure. Subsequently, the ratios:

$$p_{IM} = \frac{c_{IM}}{z_{IM}} \quad (A.2)$$

are determined for each IM level, indicating the probability of failure of the structure at a given IM level. By considering all failure and no failure observations to be independent, the binomial distribution can be used to express the probability of having c_{IM} failures in z_{IM} records at each IM level and the fragility function parameters of Eq. (A.1) can then be determined using the maximum likelihood estimation procedure suggested in (Baker 2015).

The fragility function can then be numerically integrated with the derivative of the seismic hazard curve dH_{IM} to yield the mean failure rate of the structure (simply termed failure rate hereon):

$$\lambda_f = \int_0^\infty P[f|IM] |dH_{IM}| \quad (A.3)$$

Finally, assuming that both hazard and fragility are memoryless, failure can be considered to be a Poisson process and the probability of failure, i.e. the risk of failure, can be derived by:

$$P_f = 1 - e^{-\lambda_f} \quad (A.4)$$

which, in case of small failure rates, can be considered equal to the failure rate without significant error.

A.3 Methodology to analyse the performance of the procedures assessing the variability of fragility and failure rate estimates

The following Sections describe the different type of scenarios and methodological approaches considered in the case studies of Section A.4 to compare the performance of the *regrouping* and *bootstrap resampling* procedures in assessing the variability of fragility parameter estimates and failure rate estimates.

A.3.1 Analysing the variability of fragility parameter estimates due to group-to-group variability

The fragility function parameters determined using a particular group of records are estimates $\{\hat{\theta}, \hat{\beta}\}$ of the true unknown parameters $\{\theta, \beta\}$ due to the relatively small (finite) number of records used to represent the seismic input (Iervolino 2017; Baker and Cornell 2008). It is therefore expected that a different group of records will lead to a different set of $\{\hat{\theta}, \hat{\beta}\}$. The variability of the fragility parameters obtained from the EDP-based approach due to group-to-group variability can thus be determined using a large number of groups of records obtained using the same selection criteria. A procedure simulating this rationale is applied herein, according to which a number N of records is initially selected and used to perform MSA of the structural model. Next, multiple groups of records with a smaller size n (with $n < N$) are created by regrouping n records of the initial N and enforcing the same selection criteria. This procedure, termed the *regrouping* procedure hereafter, aims to create m new groups of n records consistent with the original seismic scenario. The EDPs corresponding to each new group are then post-processed according to Section 2 and m fragility functions are created, allowing for the statistical analysis of their variability.

As an alternative to using a large number of groups, i.e. the *regrouping* procedure described before, *bootstrap resampling* is often applied to examine the effect of the group-to-group variability on the fragility parameters, as described in (Iervolino 2017). This procedure, referred to as the *bootstrap* procedure hereon, considers a group of records of size n (i.e. the parent group) associated to a certain IM level and resamples it by selecting n records with replacement from this group (some of the records will appear more than once and others will not be present in the resampled group). The n failure/no-failure observations that correspond to the demand associated to this resampled group of records represents a demand pseudo-sample of size n for the selected IM level. Repeating

this resampling procedure for all the IM levels (using the same parent group) generates the demand pseudo-samples needed to obtain the ratios defined by Eq. (A.2) for all IM levels and determine a new fragility function following the procedure described in Section A.2. This procedure is then repeated m times (using the same parent group) to obtain m new fragility functions.

The *bootstrap* procedure generates consistent results as long as the two following assumptions are met: the data sample that is resampled (i.e. the parent sample) is representative of the population and is a random sample of the population. Under these conditions, the *bootstrap* procedure assumes that randomly resampling from the parent sample is equivalent to sampling from the population and that each bootstrap sample is a possible sample of the population (Efron and Tibshirani 1994). In the context of the present study, where the parent sample is a group of ground motion records that need to fulfil a set of earthquake scenario compatibility criteria (see the spectral matching criteria defined in Section A.4.2), the parent sample cannot be considered to have been randomly sampled from the population. By enforcing the referred earthquake scenario compatibility criteria, the parent sample is a conditioned sample, where conditioned means that certain criteria need to be met for the sample to be accepted as valid (see (Macedo and Castro 2017) for details on the optimization procedure that leads to the conditioned sample of records). Therefore, each resampled group of records randomly generated by bootstrapping the parent group of records are unlikely to fulfil the earthquake scenario compatibility criteria that led to the selection of the parent group. A simple example of this situation is the case where a group of n records is selected in order to have average spectral values over a certain period range that are within a $\pm 10\%$ bound with respect to a certain target response spectrum. In this case, it can be understood that many random bootstrap combinations of those n records may not comply with the referred $\pm 10\%$ bound. As other selection criteria are added to tighten the compatibility between the group of records and the target earthquake scenario (see Section A.4.2 and the discussion in (Macedo and Castro 2017), it is expected that most, if not all, of the bootstrap samples will not comply with the earthquake scenario compatibility criteria that are enforced.

In light of this, the value of the group-to-group variability obtained from the pseudo-samples generated by the *bootstrap* procedure is expected to be inconsistent with the value that would be obtained using a large number of groups, all selected to match the same earthquake scenario compatibility criteria, or using the *regrouping* procedure, that

also enforces the same selection criteria. In order to analyse the significance of this inconsistency, the *regrouping* and the *bootstrap* procedures are applied to three case studies that also involve different numbers of group sizes and IM levels. Furthermore, since the *regrouping* procedure is more computationally demanding, the ability to still obtain adequate variability estimates with *bootstrap* is also analysed. Therefore, the performance of the procedures in determining the variability of fragility function parameter estimates and failure rate estimates is compared and the variability is quantified using statistics of the estimates (i.e. mean, standard deviation and coefficient of variation).

A.3.2 Analysing the variability of fragility parameter estimates due to group-to-group variability and uncertainty of the limit state capacity

The sets of fragility parameter estimates and associated statistics obtained according to Section 3.1 are determined by considering that the EDPs selected to establish the failure conditions have deterministic capacities (Cap_{det}). Additional sets of fragility parameter estimates and of the corresponding statistics were also determined by considering uncertainty in the capacity model of the EDPs. Existing research has shown the non-negligible influence of that source of uncertainty on both parameters of the lognormal fragility function, i.e. the shape and scale (Liel et al. 2009; Lupoi et al. 2006). The considered probabilistic capacity model (Cap_{prob}) is defined according to the following format:

$$Cap_{prob} = m_{Cap} \cdot \varepsilon_{Cap} \quad (A.5)$$

where m_{Cap} is the median value of the capacity model given as a function of deterministic geometry and material properties and ε_{Cap} is the error term accounting for scatter and bias of the capacity model. In the following, ε_{Cap} is assumed to be a lognormal random variable (Cornell et al. 2002) with unit median, i.e. no bias, and dispersion β_ε . Variability in capacity is combined first with record-to-record variability and then with group-to-group variability as follows: for each record/analysis of a given group, the obtained EDPs are compared with a sample of k_{cap} equiprobable capacity values determined by Eq. (A.5) to define a series of failure/no-failure observations of size k_{cap} . The EDP-based procedure described in Section A.2 is then applied, leading to one fragility function for each group of records (the algorithm that is used can be found in (Bakalis and Vamvatsikos 2018)). These new fragility parameter estimates involve uncertainty due to both the record-to-

record variability and the uncertain capacity threshold. Finally, group-to-group variability is determined by applying both the *regrouping* and the *bootstrap* procedures as defined in Section A.3.1 (i.e. by repeating this process for the n groups in the former, and by bootstrapping the sample of failure/no-failure observations of size $n \times k_{cap}$ in the latter). The performance of the two procedures is then compared as defined in Section A.3.1, and the variability statistics are further compared with those obtained when considering a deterministic limit state capacity.

A.3.3 Analysing the variability of failure rate estimates

When a fragility function with parameter estimates $\{\hat{\theta}, \hat{\beta}\}$ is integrated with the seismic hazard according to Eq. (A.3), an estimate $\hat{\lambda}_f$ of the true unknown failure rate λ_f is obtained (Iervolino 2017). The variability associated to $\hat{\lambda}_f$ stems from the variability of the fragility function described in Sections A.3.1 and A.3.2, which is related to group-to-group variability and to how limit state capacity is defined (i.e. if it is deterministic or probabilistic), but it is also influenced by the seismic hazard curve, namely by the part of the curve contributing the most to the failure rate. By considering the multiple sets of fragility functions created in Sections A.3.1 and A.3.2, the corresponding sets of $\hat{\lambda}_f$ estimates are then obtained using Eq. (A.3). The variability of $\hat{\lambda}_f$ is then quantified for the several cases analysing statistics similar to those considered for the results of Sections A.3.1 and A.3.2.

A.4 Description of the selected case studies

A.4.1 Structural modelling

The selected case studies are three reinforced concrete buildings with infilled frame systems and with three, four and five storeys. All buildings have the plan view and the reinforcement detailing presented in Figure A.1. The structures are regular in-plan and in-elevation with a storey height equal to 3.0 m. The concrete strength and the yield strength of the reinforcement are equal to 25 MPa and 500 MPa, respectively. The buildings are located in Lisbon, Portugal and were designed for gravity loads only. Specifically, a permanent load of 4 kN/m² is considered to be uniformly distributed on all floors, additional to the slab self-weight (all slabs have a 0.12 m thickness). Additionally, the quasi-permanent component of a uniform live load of 3 kN/m² is also assigned to all

floors, except to the top floor where this load is 1 kN/m^2 . Staircases are modelled only as permanent and quasi-permanent loads, which are transferred to the supporting beams and are applied uniformly. The corresponding loads are 7.75 kN/m and 8.60 kN/m for the permanent and the quasi-permanent loads, respectively. Masonry infills are present in all peripheral frames and represent a permanent uniform load of 7 kN/m .

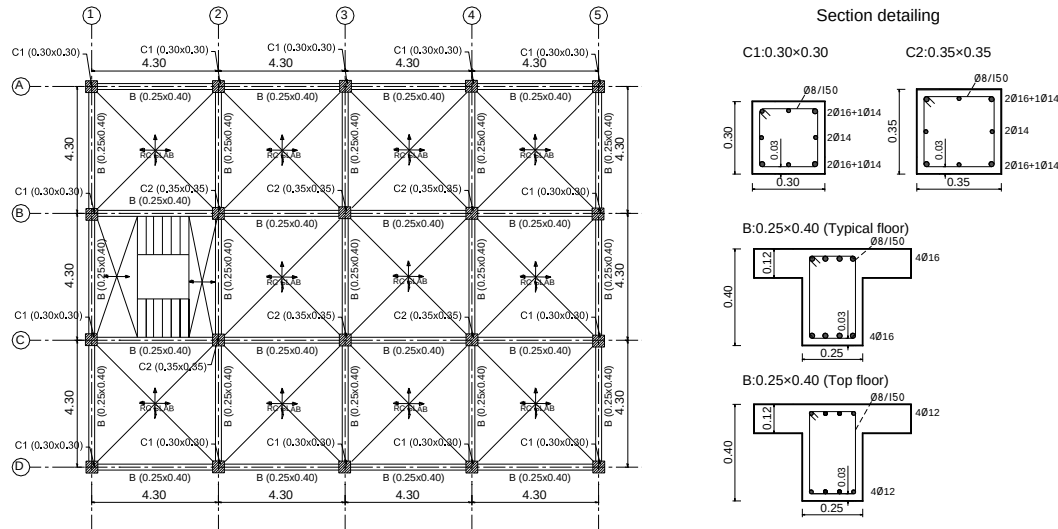


Figure A.1. Plan view of a typical storey of the three buildings and design details of the end sections of the elements (all dimensions are in m)

Three-dimensional models of the buildings are simulated in the OpenSees (OS) computer software (McKenna 2011) considering mean values of all material and geometrical properties. A lumped plasticity approach is adopted to simulate the inelastic behaviour of all structural elements. Phenomenological hysteresis laws are assigned in rotational springs located on both ends of all columns and beams to simulate inelastic flexural behaviour. Two independent springs are assigned to each end of the columns, one for each orthogonal direction, while one spring is assigned to each end of the beams modelling the in-plane flexural behaviour. Due to the nature of the selected modelling approach, biaxial moment interaction or axial force moment interaction are not considered when modelling the behaviour of columns. The hysteretic flexural behaviour is defined by a moment-rotation law that is simulated using the *hysteretic* material provided by OS. The properties of the backbone curve of all columns are determined considering the axial force due to vertical loads of the seismic combination ($G+0.3Q$), where G and $0.3Q$ stand for all permanent and quasi-permanent loads, respectively. The yielding moment (M_y) and the yielding rotation capacity (θ_y) are determined according to

(Panagiotakos and Fardis 2001). The capping (θ_c) and the ultimate (θ_u) rotation capacities are computed according to (Haselton et al. 2008) along with the corresponding capping (M_c) and residual (M_r) moments, where the latter is considered to be 20% of M_c (see the backbone curve in Figure A.2(a)). Stiffness, strength and unloading stiffness degradation are considered in the hysteresis curves as shown in Figure A.2. The unloading stiffness degradation is implemented in OS via the beta factor of the *hysteretic* material. A beta factor of 0.75 is used for the columns and of 0.85 for the beams in order to introduce higher degradation for the latter (Dolšek and Fajfar 2005). The *hysteretic* material parameters related to pinching and to damage due to ductility and dissipated energy are all set equal to zero. The springs are connected in-series with a linear elastic interior element. A stiffness modification factor equal to 10 is applied according to (Zareian and Medina 2010) to account for the effect of the series connection on the total stiffness of the element. Beam-column node failure is not considered and rigid nodes are modelled in all element cross-sections. Even though the possibility of shear failure due to the weak detailing of the columns in flexure can be accounted for when post-processing the results of the analyses, the cases that are examined herein only deal with ductile demand which is defined by the maximum interstorey drift ratio of the two directions (ISD_{max}).

Masonry infills are considered in all peripheral frames and are modelled with a single strut active only in compression. Two diagonal struts (strut 1 and strut 2 as shown in Figure A.2(b)) are used to simulate one infill and are connected to the beam-column joints. The equivalent area of each strut is established based on the maximum lateral force of the infill (Žarnić and Gostič 1997), transformed to the direction of the diagonal, and on the masonry compressive stress f_m . Maximum strength is assumed to be reached at an interstorey drift of 0.2% (Dolšek and Fajfar 2008). The lateral displacement of each infill is transformed into the diagonal displacement for the subsequent definition of the strain of the strut. The parameters obtained, i.e. the maximum stress and strain, are used to define the masonry material with zero tensile strength simulated by the *Concrete01* constitutive model (Figure A.2(b)) (Noh et al. 2017). The stress f_m is equal to 3.10 MPa and all infills have a thickness of 0.15m. Additionally, a residual stress equal to 10% of the maximum stress is considered for numerical stability, which is reached at an interstorey drift five times the interstorey drift at maximum strength.

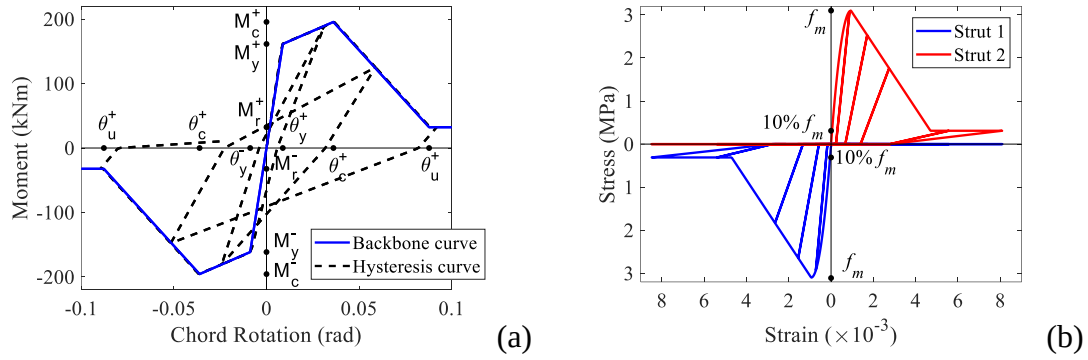


Figure A.2. Moment-rotation backbone curve and hysteresis loop simulation for a column (a) and stress – strain relationship of the diagonal struts (in compression), measured in the diagonal direction (b)

Geometric nonlinearities are incorporated in the form of P- Δ effects. Furthermore, soil structure interaction phenomena are not modelled and the building is considered fixed at the ground level. The first and the second fundamental periods of vibration of the buildings, with and without the masonry infills, are presented in Table A.1. Finally, the average of the first two periods of the infilled structures and the first two periods of the bare structures T^* is determined (also shown in Table A.1) to be used for the ground motion record selection. It is noted that involving the period of vibration of the bare structure when defining T^* is conceptually similar to accounting for the period elongation of the structure after yielding and failure of the infills, after which the behaviour of the structure will tend to that of the bare structure.

Table A.1 First and second mode periods of vibration of the buildings

Period\Building	3-storey	4-storey	5-storey
$[T_1, T_2]_{\text{infilled}}$	[0.31s, 0.25s]	[0.41s, 0.31s]	[0.52s, 0.39s]
$[T_1, T_2]_{\text{bare}}$	[0.73s, 0.72s]	[0.96s, 0.93s]	[1.18s, 1.15s]
T^*	0.5s	0.66s	0.82s

A.4.2 Ground motion selection and analysis

The ground motion record selection is performed using the recently developed SelEQ software (Macedo and Castro 2017) using a Conditional Spectrum (CS) in terms of the 5% damped spectral acceleration (Sa) (Baker 2010) as the target spectrum. After identifying the sources contributing to the hazard at the site (Lisbon), the probabilistic

seismic hazard analysis of the site was performed within the SelEQ software using the open source software OpenQuake (Pagani et al. 2014) and the seismic hazard model developed by the SHARE project (Woessner et al. 2015). The annual seismic hazard curve H_{IM} was determined at T^* for each building at the benchmark site and is shown in Figure A.3(a). For each source and four probabilities of exceedance, i.e. 30%, 10%, 5% and 2% in 50 years, the disaggregation of the hazard was performed at T^* and the contribution of each magnitude-distance (M-R) bins for the hazard was then computed. For each probability of exceedance, the results were aggregated and the contribution of each M-R pair and ground motion prediction equation to the exceedance of the spectral acceleration of interest was computed to determine a CS that considers the contributions of all individual M-R pairs using the methodology proposed in (Lin et al. 2013a; 2013b). A preliminary record selection was performed using the NGA-WEST2 database (Ancheta et al. 2014) based on seismological and strong motion criteria. These criteria are: magnitude larger than 5.5, closest distance from source to site larger than 10 km, soil shear wave velocity, in upper 30m of soil, greater than 200 m/s, a limit of five records from a single seismic event, a lowest useable frequency lower than 0.5 Hz, faulting scenarios compatible with the site (Lisbon), only free-field and far-field records. Subsequently, the final selection and scaling of a group of 40 bi-directional records (for each probability of exceedance) was carried out by ensuring compatibility between the target spectrum, i.e. the CS, and the average of the spectral accelerations of the 40 records, where the spectral acceleration of each record is defined by the geometric mean of the spectral accelerations of the two horizontal components of the record. SelEq ensures this compatibility by minimizing the difference between statistics of the target spectrum (i.e. the mean and standard deviation of the logarithms of the spectral accelerations) and the same statistics of the group, within a period range of $0.2T^*$ and $1.5T^*$, as described in (Macedo and Castro 2017). An additional criterion was included in the process which involves minimizing also the skewness of the logarithms of the spectral accelerations to a value close to zero for the same period range. This criterion is based on the fact that the spectral accelerations are expected to follow a multivariate lognormal distribution (Jayaram and Baker 2008) and, therefore, the logarithms of the spectral accelerations are expected to follow a multivariate normal distribution. The scale factors (SF) of individual records that were involved during this record selection process range from 0.263 to 3.995. To illustrate the results of the ground motion selection process, Figure A.3(b) shows the geometric means of the selected 40 records along with the CS for the 30% in 50 years

probability of exceedance for the 3-storey building. All the 40 records of a group defined for a certain probability of exceedance were then scaled using the SF shown in Table A.2 to define *IM* levels whose spectral accelerations $Sa(T^*)$ are also shown. Each group of records is only scaled to match a few *IM* levels in order to have the four groups of records spanning a total of 20 *IM* levels. Values of SF equal to 1 correspond to the initial selection of a given group of 40 records that match the corresponding CS. The overall maximum and minimum SFs that are applied to any individual record, combining the scaling needed during the record selection process and the scaling to match the selected *IM* levels, are 7.8445 and 0.1078.

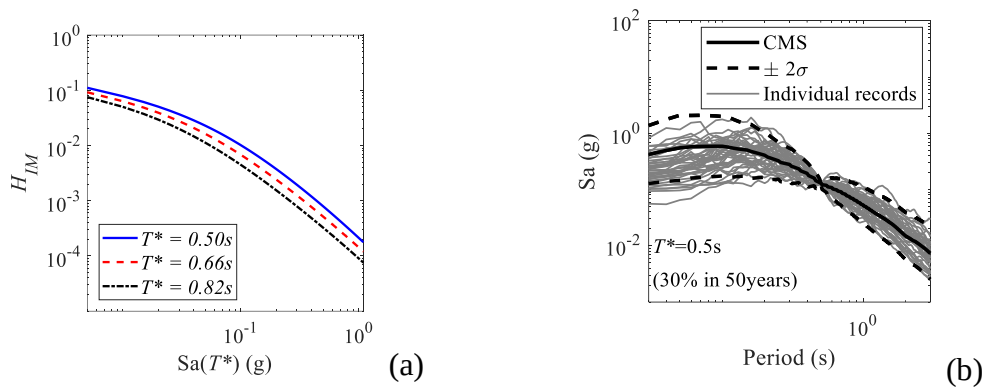


Figure A.3. The seismic hazard curves for the three buildings (a) and the CS with the geometric mean spectra of the 40 records for the 30% in 50 years probability of exceedance for the 3-storey building (b).

MSA was then performed following a procedure similar to the one proposed in (Christovasilis et al. 2014) using the four sets of 40 records scaled according to Table A.2 spanning the 20 *IM* levels. For the *regrouping* procedure, each group of 40 records is then regrouped into groups of 10, 20 and 30 records implementing selection criteria consistent with the criteria used for the initial selection to ensure compatibility between the new groups of 10, 20 and 30 records and the original group of 40 records. The regrouping procedure starts by defining the total number of combinations of m items (in this case 40) taken k (in this case, 10, 20 or 30) at a time, which is given by the binomial coefficient:

$$\binom{m}{k} = \frac{m!}{(m-k)!k!} \quad (A.6)$$

For example, for the group of 20 records, the binomial coefficient is equal to 1.3×10^{11} . However, not all combinations are valid since admissible groups of records need to represent a seismic scenario consistent with that of the reference group of size 40.

Table A.2 Scale factors and corresponding spectral accelerations used in the MSA procedure.

Probability of exceedance	IM	Scale factor	Sa($T^*=0.50$) g	Scale factor	Sa($T^*=0.66$) g	Scale factor	Sa($T^*=0.82$) g
30% in 50 years	(1)	0.38	0.05	0.41	0.04	0.40	0.03
	(2)	0.77	0.10	0.72	0.07	0.67	0.05
	(3)	1.00	0.13	1.00	0.10	1.00	0.07
	(4)	1.54	0.20	1.45	0.14	1.48	0.11
10% in 50 years	(5)	0.86	0.25	0.82	0.18	0.82	0.14
	(6)	1.00	0.29	1.00	0.22	1.00	0.17
	(7)	1.21	0.35	1.19	0.26	1.23	0.21
5% in 50 years	(8)	0.91	0.40	0.89	0.30	0.90	0.24
	(9)	1.00	0.44	1.00	0.34	1.00	0.27
	(10)	1.20	0.53	1.22	0.41	1.21	0.32
	(11)	1.36	0.60	1.39	0.47	1.39	0.37
2% in 50 years	(12)	0.92	0.67	0.91	0.52	0.91	0.42
	(13)	1.00	0.73	1.00	0.57	1.00	0.46
	(14)	1.08	0.79	1.10	0.63	1.09	0.50
	(15)	1.16	0.85	1.20	0.69	1.17	0.54
	(16)	1.30	0.95	1.39	0.80	1.37	0.63
	(17)	1.44	1.05	1.57	0.90	1.54	0.71
	(18)	1.58	1.15	1.71	0.98	1.70	0.78
	(19)	1.71	1.25	1.83	1.05	1.83	0.84
	(20)	1.85	1.35	1.97	1.13	1.96	0.90

Consistency is assumed to be verified when a ground motion group of size k meets three conditions consistent with the criteria that were used for the initial selection as described previously. These conditions are applied between statistics of the spectral values of each new group and the target spectrum, where the target spectrum now corresponds to the spectrum of the 40 records. As for the selection of the initial group of 40 records, the selected statistics are the mean, the standard deviation and the skewness of the logarithms of the spectral accelerations. Regarding the first two statistics, a variation of 10% is allowed between the statistics of the reference group of size 40 and the statistics of the

new group with a lower size. With respect to the skewness, admissibility is checked by ensuring the skewness of the new group with a lower size cannot be rejected according to the standard error test for skewness (Tabachnick and Fidell 1996), considering a significance level of 5% and the reference skewness of the group of size 40. Following this procedure, a total of 100 groups were created for each of the new group sizes (10, 20 and 30). These 100 groups can then be used to analyse the uncertainty associated with group-to-group variability.

A.5 Comparing the performance of the procedures for assessing the variability of fragility and failure rate estimates

A.5.1 Variability of the estimates of the fragility parameters

A.5.1.1 Deterministic limit state scenario

The MSA yielded 20 stripes of ISD_{max} and, to determine the ratios defined by Eq. (A.2), failure is considered to occur when ISD_{max} is higher than a limit value ISD_{cap} or when there is numerical failure of the model due to nonconvergence. For the deterministic limit state scenario, ISD_{cap} is set to 2.5%. One hundred fragility curves are constructed according to the *regrouping* procedure for the three buildings using all 20 stripes of demand. Figure A.4 shows the 100 fragility functions of the 3-storey building determined for the 100 groups of 20 records obtained by the *regrouping* procedure when all 20 stripes of demands are considered. One group of 20 records, termed parent group i , with parameters $\{\hat{\theta}, \hat{\beta}\}$ shown in Table A.3, is randomly selected from the 100 groups to carry out the *bootstrap* procedure. Two bootstrap sample sizes of 100 and 500 are considered to analyse the sensitivity of the bootstrap estimates and the corresponding results are identified by the number in parentheses in Table A.3. The fragility functions corresponding to the *bootstrap i (100)* case are shown in Figure A.4(a) along with the fragility function of the parent group i , which is represented by the thick black line. To emphasize some key points of the two procedures, Figure A.4(b) shows results similar to those of Figure A.4(a) but considering a different parent group j . As can be seen, the domain covered by the bootstrap fragility functions is not always the same and depends on the parent group. Statistics of the parameters of the fragility functions, i.e. their mean μ and variability values, the latter in terms of the standard deviation σ and of the coefficient of variation CV , are presented in Table A.3 for all three buildings studied and

for both procedures. Even though the discussion about the variability of the parameters is mostly focused on the values of σ , values of CV are also provided for all the results presented in this Section and in the following ones in order to better understand the magnitude of that variability.

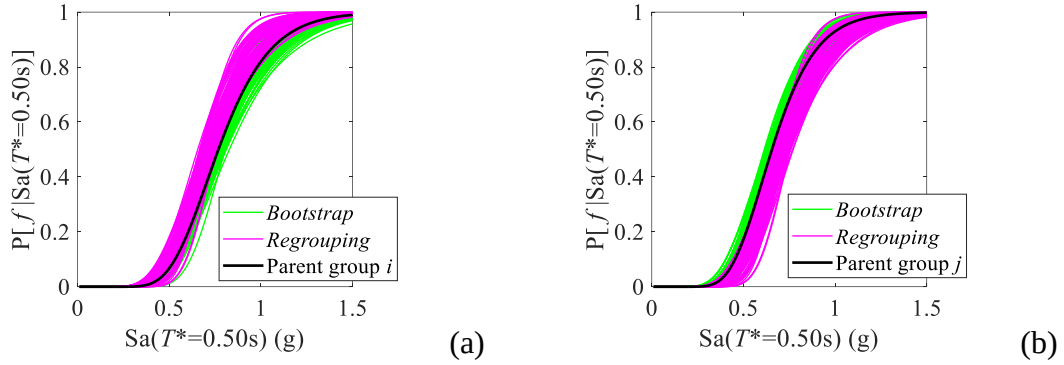


Figure A.4. Comparison of the 100 fragility functions of the 3-storey building obtained from the *regrouping* and the *bootstrap* (100) procedures using 20 stripes. Parent groups i and j are used for the *bootstrap* resampling in (a) and (b), respectively.

By comparing the statistics obtained from *bootstrap* (100) and *bootstrap* (500) presented in Table A.3, it appears that using a bootstrap sample size of 100 is adequate to estimate the selected statistics for all the considered case studies. The mean values, $\mu_{\hat{\theta}}$ and $\mu_{\hat{\beta}}$, are nearly identical and the differences in the σ are small (less than 15%). Therefore, subsequent results are only based on the *bootstrap* (100) case. Furthermore, when comparing the performance of the *regrouping* and *bootstrap* procedures based on the visual analysis of Figure A.4 and on the statistics of Table A.3, an overall good match can be observed between the corresponding statistics of the variability of the fragility function parameters, $\sigma_{\hat{\theta}}$ and $\sigma_{\hat{\beta}}$. It is noted, however, that the parent group that is selected for the *bootstrap* affects the fragility functions that are subsequently generated. With respect to the mean values of the parameters, which differ for the different parent groups, they can be either higher or lower than the corresponding parameters obtained with the *regrouping* procedure. Although this biased mean cannot be corrected since it only depends on the original data being bootstrapped, i.e. the parent group, it is not a relevant aspect of the procedure since bootstrapping is normally used to estimate the variability of the estimates and not their mean values. The variability values obtained by the *bootstrap* procedure, on the other hand, do not seem to vary substantially for the alternative parent groups, for both parameters examined. Furthermore, they are in good agreement with

those obtained by the *regrouping* procedure for both parameters and for all case studies. Nevertheless, *bootstrap* seems to lead to marginally lower variability for most of the cases, in particular for parameter β .

Table A.3 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 20 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.72	0.026	3.58	-	0.28	0.040	14.38
	<i>Bootstrap i (100)</i>	0.77	0.77	0.023	2.99	0.29	0.29	0.031	10.86
	<i>Bootstrap i (500)</i>	0.77	0.77	0.022	2.82	0.29	0.28	0.028	9.88
	<i>Bootstrap j (100)</i>	0.66	0.66	0.019	2.93	0.29	0.28	0.029	10.23
	<i>Bootstrap j (500)</i>	0.66	0.66	0.020	3.00	0.29	0.28	0.027	9.57
4-Storey	<i>Regrouping</i>	-	0.59	0.020	3.39	-	0.26	0.034	13.04
	<i>Bootstrap i (100)</i>	0.58	0.58	0.015	2.58	0.26	0.25	0.028	11.01
	<i>Bootstrap i (500)</i>	0.58	0.58	0.017	2.84	0.26	0.25	0.030	11.81
	<i>Bootstrap j (100)</i>	0.61	0.62	0.018	2.95	0.28	0.27	0.026	9.49
	<i>Bootstrap j (500)</i>	0.61	0.62	0.018	2.99	0.28	0.27	0.027	9.76
5-storey	<i>Regrouping</i>	-	0.47	0.015	3.12	-	0.22	0.020	9.31
	<i>Bootstrap i (100)</i>	0.47	0.47	0.014	2.89	0.24	0.23	0.027	11.77
	<i>Bootstrap i (500)</i>	0.47	0.47	0.012	2.55	0.24	0.23	0.026	11.14
	<i>Bootstrap j (100)</i>	0.49	0.49	0.012	2.50	0.22	0.22	0.026	11.87
	<i>Bootstrap j (500)</i>	0.49	0.49	0.012	2.54	0.22	0.22	0.023	10.66

To complement the previously presented results with cases involving different numbers of records, similar analyses were performed using groups of 30 and 10 records. Groups of 30 and 10 records are selected for being representative of larger and of smaller group sizes, respectively. Fragility function parameter statistics similar to those of Table A.3 are presented in Table A.4 and Table A.5, for the groups of size 30 and 10, respectively. As expected (e.g. see (Eads et al. 2013)), the results obtained for smaller groups of records using the *regrouping* and the *bootstrap* procedures exhibit a larger variability of both fragility estimates, while results obtained for the larger groups are the opposite. Despite

the reduction in the variability of the estimates that is observed when using 30 records (Table A.4), the trend of the results is similar to the one observed when using 20 records. As such, when using groups of 30 records, the variabilities obtained from the *regrouping* and from the *bootstrap* procedures are also very similar for both fragility parameters. On the other hand, using a small number of records, such as 10 (Table A.5), can lead to large differences between the results of the two procedures. As can be seen from Table A.5, these differences are present for both fragility parameters and their relative variations depend on the structure. It can thus be said that using an adequate number of records is crucial to ensure that *bootstrap* variability estimates can still be considered valid, despite the previously discussed inconsistency between the *regrouping* and the *bootstrap* procedures.

Table A.4 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 30 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.71	0.016	2.22%	-	0.28	0.022	7.95%
	<i>Bootstrap i (100)</i>	0.71	0.71	0.016	2.18%	0.28	0.28	0.023	8.07%
4-Storey	<i>Regrouping</i>	-	0.59	0.012	2.04%	-	0.27	0.020	7.28%
	<i>Bootstrap i (100)</i>	0.57	0.57	0.014	2.42%	0.24	0.24	0.025	10.34%
5-storey	<i>Regrouping</i>	-	0.47	0.008	1.78%	-	0.22	0.018	7.91%
	<i>Bootstrap i (100)</i>	0.47	0.47	0.009	1.88%	0.22	0.21	0.018	8.37%

Table A.5 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 10 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.70	0.048	6.78%	-	0.27	0.070	25.59%
	<i>Bootstrap i (100)</i>	0.74	0.74	0.031	4.23%	0.27	0.27	0.032	11.72%
4-Storey	<i>Regrouping</i>	-	0.59	0.036	6.13%	-	0.25	0.063	24.72%
	<i>Bootstrap i (100)</i>	0.55	0.55	0.018	3.31%	0.17	0.16	0.030	18.83%
5-storey	<i>Regrouping</i>	-	0.46	0.024	5.25%	-	0.22	0.031	13.97%
	<i>Bootstrap i (100)</i>	0.53	0.53	0.019	3.66%	0.24	0.24	0.038	16.07%

A further assessment of the performance of the two procedures is carried out by replicating this fragility curve analysis using a lower number of *IM* levels (i.e. fewer stripes). Although using a higher number of *IM* levels is generally suggested to reduce the fitting error, it is not uncommon to find situations where a much lower number of *IM* levels is used. Five *IM* levels covering uniformly the range of response of the structures are selected for the purpose of the present research. In particular, the lowest *IM* corresponds to the lowest *IM* value that contributes to the lower tail of the fragility curve of a given building, the highest *IM* corresponds to the maximum *IM* value considered for each building and contributes to the upper tail of the fragility curve. The remaining three *IM* levels are selected in order to cover the range of the fragility curves. Hence, referring to Table A.2, stripes [7, 11, 15, 18, 20] are selected for the 3-storey building, stripes [8, 11, 15, 18, 20] are selected for the 4-storey building and stripes [8, 14, 16, 18, 20] are selected for the 5-storey building and new fragility curves are constructed using both the *regrouping* and the *bootstrap* procedures. The fragility curves developed for the three buildings using 20 records are presented in Figure A.5, while the respective statistics of the fragility parameter estimates are shown in Table A.6. Given the good agreement of the results obtained using 100 and 500 bootstrap samples for this case as well, only the results obtained for the bootstrap sample size of 100 are presented and discussed.

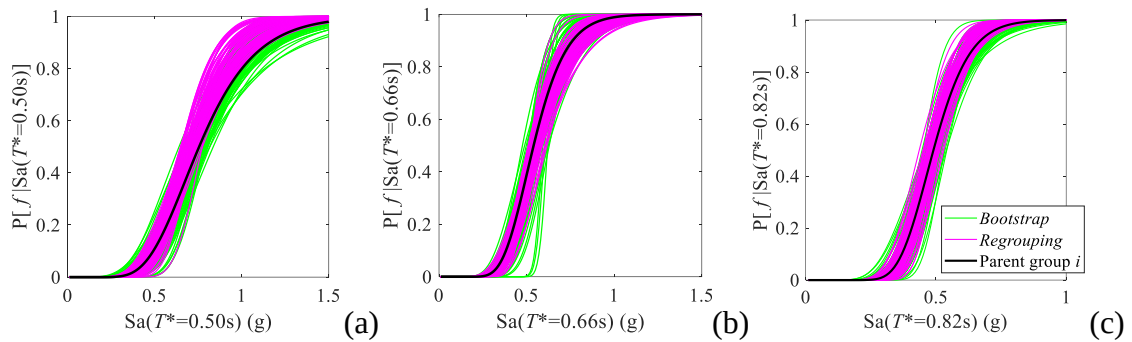


Figure A.5. Comparison of the 100 fragility functions of the 3-storey (a), 4-storey (b) and 5-storey (c) buildings obtained from the *bootstrap* and the *regrouping* procedures using 20 records and 5 stripes.

The reduction of the number of stripes appears to have little effect on the mean estimates of the parameters for both the *regrouping* and the *bootstrap* procedures, as shown by the good agreement of the mean values of Table A.3 and Table A.6. Nevertheless, such as for the observations derived for 20 stripes, the mean values of $\hat{\theta}$ and $\hat{\beta}$ obtained for the *bootstrap* procedure depend on the selection of the parent group. With respect to the

variability of the parameter estimates. Unlike the previous case, using only 5 stripes leads to a much higher variability of the parameter estimates. This effect can be initially attributed to the higher epistemic error that is unavoidably introduced by having a lower number of points to fit the fragility function (i.e. fewer stripes). However, this increase of the variability doesn't affect equally the two procedures, i.e. the *regrouping* and the *bootstrap*. While the standard deviation σ of both $\hat{\theta}$ and $\hat{\beta}$ is marginally higher for the *regrouping* procedure (they increase by 20%, on average), almost all values of σ increase to more than twice their initial value for the *bootstrap* procedure. These results show that the inconsistency between the *regrouping* samples and the *bootstrap* pseudo-samples (see Section A.3.1) has a larger impact in the variability of the fragility parameter estimates when a lower number of stripes is considered. As such, when focusing on the comparative assessment of the *regrouping* and the *bootstrap* procedure for the case where only 5 stripes are used, the *regrouping* procedure is seen to lead systematically to a lower variation of the parameters. This effect is graphically illustrated in Figure A.5 and can be quantitatively observed in Table A.6. It can be seen that the standard deviation of both $\hat{\theta}$ and $\hat{\beta}$ is larger for the *bootstrap* procedure, leading to an overestimation of $\sigma_{\hat{\beta}}$ up to 50% and of $\sigma_{\hat{\theta}}$ up to 35%.

Table A.6 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 20 records 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}} (\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}} (\%)$
3-Storey	<i>Regrouping</i>	-	0.71	0.029	4.02	-	0.29	0.047	16.06
	<i>Bootstrap i (100)</i>	0.76	0.74	0.039	5.18	0.34	0.31	0.060	19.37
4-Storey	<i>Regrouping</i>	-	0.56	0.022	4.01	-	0.28	0.038	13.58
	<i>Bootstrap i (100)</i>	0.54	0.55	0.030	5.53	0.27	0.25	0.057	23.11
5-storey	<i>Regrouping</i>	-	0.49	0.018	3.72	-	0.20	0.031	15.12
	<i>Bootstrap i (100)</i>	0.49	0.49	0.020	4.07	0.22	0.21	0.043	20.41

Conclusions similar to those obtained for the groups of 20 records can also be seen for the case involving groups of 30 records, as shown in Table A.7. Although the variability of the parameter estimates is inherently lower when compared to that of the 20 records and 5 stripes, using 30 records and 5 stripes with the *regrouping* procedure yields

insignificant differences in the *CV* of both parameters when compared to the case involving 30 records and 20 stripes. On the contrary, a similar comparison for the results obtained for the *bootstrap* procedure shows that the *CV* of both parameters more than doubles when using only 5 stripes. When using groups of 10 records and 5 stripes (results shown in Table A.8), on the other hand, larger differences are found in the variability of the parameters with respect to the case involving 20 stripes. For the *regrouping* procedure variations can be up to 80%, but much larger values are found for the results obtained from the *bootstrap* procedure where variations can go up to 300%. In the overall, it can be concluded that the *bootstrap* procedure can provide adequate estimates of the variability of parameters, i.e. comparable with those obtained from the *regrouping* procedure, as long as an adequate number of records and number of stripes are employed. If these conditions are not met, the variability estimates obtained from *bootstrap* can either under- or overestimate the true variability of the parameters.

Table A.7 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 30 records and 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.71	0.019	2.69%	-	0.29	0.027	9.34%
	<i>Bootstrap i (100)</i>	0.70	0.70	0.031	4.36%	0.30	0.29	0.048	16.51%
4-Storey	<i>Regrouping</i>	-	0.56	0.013	2.36%	-	0.29	0.021	7.39%
	<i>Bootstrap i (100)</i>	0.54	0.54	0.024	4.46%	0.27	0.26	0.043	16.42%
5-storey	<i>Regrouping</i>	-	0.49	0.012	2.51%	-	0.21	0.018	8.87%
	<i>Bootstrap i (100)</i>	0.48	0.48	0.018	3.81%	0.22	0.21	0.042	19.78%

Table A.8 Statistics of the fragility function parameters due to group-to-group variability for the three buildings using 10 records and 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.70	0.058	8.32%	-	0.27	0.093	34.09%
	<i>Bootstrap i (100)</i>	0.76	0.77	0.051	6.65%	0.29	0.28	0.070	25.07%
4-Storey	<i>Regrouping</i>	-	0.56	0.049	8.74%	-	0.26	0.089	34.55%
	<i>Bootstrap i (100)</i>	0.51	0.53	0.062	11.79%	0.17	0.11	0.078	72.30%
5-storey	<i>Regrouping</i>	-	0.48	0.032	6.70%	-	0.19	0.046	24.04%
	<i>Bootstrap i (100)</i>	0.57	0.56	0.026	4.71%	0.17	0.16	0.063	40.56%

A.5.1.2 Probabilistic limit state scenario

The evolution of the fragility function parameter estimates is examined next when the effect of the uncertainty of the capacity model is also taken into account according to Eq. (A.5), where m_{Cap} is equal to the deterministic capacity of 2.5% and ε_{Cap} is a lognormal random variable with unit median and β_ε equal to 0.5. Although only the lognormal model is adopted in the present research to express the uncertainty related to the capacity model error (Cornell et al. 2002), similar results are expected for different statistical models. Monte Carlo (MC) analysis is performed using a sample of 10 capacity values (Bakalis and Vamvatsikos 2018) that are generated using stratified sampling (Iman and Conover 1982) and the fragility functions are determined using the *regrouping* and the *bootstrap* (parent group i considering 100 *bootstrap* samples) procedures. The fragility functions obtained for the 3-storey building are presented in Fig. 6, along with the corresponding fragility functions that do not account for uncertainty in capacity for comparison purposes. The parameter estimate statistics for all buildings are presented in Table A.9 and Table A.10, corresponding to the results obtained from groups of 20 records when 20 and 5 stripes are considered, respectively.

As expected, introducing variability in the limit state capacity results in larger $\hat{\beta}$ values for all fragility curves, representing the record-to-record variability and the uncertain capacity. This effect can be observed in Figure A.6 by the lower steepness of all the new fragility functions and in Table A.9 and Table A.10 by the larger $\mu_{\hat{\beta}}$ values, when compared to those of Table A.3 and Table A.6, respectively. Depending on the building,

the increase of $\mu_{\hat{\beta}}$ can be from 50% to more than 100% and occurs in a similar way in the results of both the *regrouping* and the *bootstrap* procedures. The variability in capacity, on the other hand, has a much smaller effect on $\mu_{\hat{\theta}}$, which remains almost unchanged for both procedures and for all three buildings analysed.

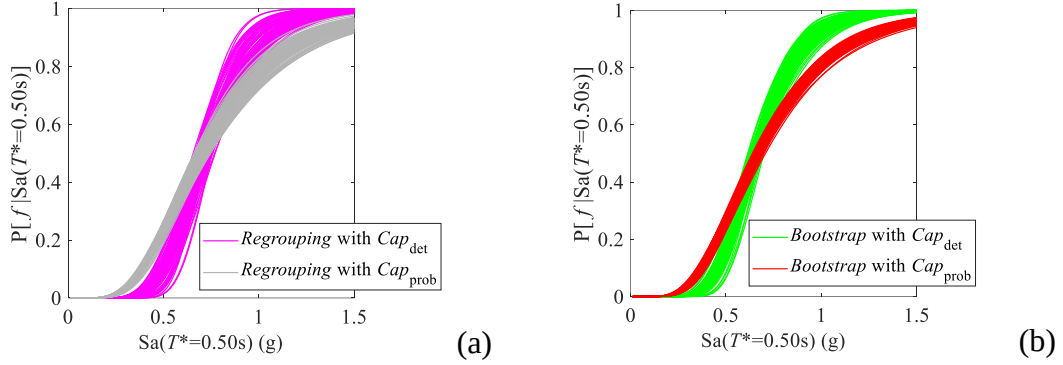


Figure A.6. Fragility functions of the 3-storey building obtained by the *regrouping* (a) and by the *bootstrap* (b) procedure with deterministic and probabilistic capacity using 20 stripes.

Regarding the comparison of the statistics of the parameter estimates when 20 or 5 stripes of intensities are used, the observations are similar to those of Section A.5.1.1, i.e. to the case involving a deterministic limit state. As such, when a larger number of stripes is used (Table A.9), the *regrouping* and *bootstrap* procedures provide similar results both in terms of mean values and in terms of variability, although the *regrouping* procedure generally yields a larger variability. When a smaller number of stripes is used (Table A.10), the mean values of the estimates predicted by both procedures are similar, but the variability estimates obtained by *bootstrap* can be as large as twice the values obtained when using 20 stripes. Regarding the variability of the parameters obtained with the *regrouping* procedure, the increase that is observed is much smaller. As seen in the results presented in the previous Section, the inconsistency issue between the *regrouping* samples and the *bootstrap* pseudo-samples has a much larger effect when a lower number of stripes is used.

Figure A.6 highlights an additional feature of the results obtained by introducing the uncertainty in capacity: a reduction of the variability of both $\hat{\theta}$ and $\hat{\beta}$ with respect to the case where this uncertainty is not considered. This effect can be seen for both procedures by the thinner range of variation of the new fragility functions in Figure A.6 and, quantitatively, by the smaller standard deviation values of $\hat{\theta}$ and $\hat{\beta}$ in Table A.9 and

Table A.10 when compared to those of Table A.3 and Table A.6, respectively. Even though this result appears to be counterintuitive, it is a direct outcome of the selected procedure to develop the fragility functions according to the EDP-based approach when uncertainty in capacity is involved using the MC approach.

Table A.9 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 20 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.71	0.020	2.86	-	0.47	0.025	5.40
	<i>Bootstrap i (100)</i>	0.75	0.75	0.015	2.05	0.48	0.48	0.019	3.99
4-Storey	<i>Regrouping</i>	-	0.58	0.016	2.77	-	0.46	0.025	5.37
	<i>Bootstrap i (100)</i>	0.55	0.56	0.008	1.46	0.41	0.41	0.017	4.21
5-storey	<i>Regrouping</i>	-	0.47	0.013	2.87	-	0.52	0.021	4.07
	<i>Bootstrap i (100)</i>	0.46	0.46	0.008	1.78	0.52	0.52	0.017	3.21

Table A.10 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 20 records and 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.71	0.021	2.90	-	0.47	0.030	6.36
	<i>Bootstrap i (100)</i>	0.74	0.75	0.028	3.69	0.49	0.49	0.039	8.06
4-Storey	<i>Regrouping</i>	-	0.57	0.017	2.98	-	0.47	0.029	6.21
	<i>Bootstrap i (100)</i>	0.55	0.55	0.015	2.68	0.41	0.41	0.031	7.49
5-storey	<i>Regrouping</i>	-	0.47	0.015	3.28	-	0.54	0.023	4.27
	<i>Bootstrap i (100)</i>	0.45	0.45	0.012	2.68	0.54	0.55	0.035	6.47

Results obtained for the group sizes of 30 and 10 are presented in Table A.11 to Table A.14 for both cases in terms of number of stripes. The trends regarding the reduced variability observed for the groups of size 20 when the uncertain capacity limit state is used instead of the deterministic capacity limit state, are also found for these two other group sizes. As such, variability estimates of Table A.11 and Table A.13 are lower when

compared to those of Table A.4 and Table A.5, respectively, and variability estimates of Table A.12 and Table A.14 are also lower when compared to those of Table A.7 and Table A.8, respectively. Furthermore, differences between the results obtained for a given group size using 20 and 5 IMs are smaller when capacity uncertainty is involved. As such, when the *regrouping* procedure is employed, groups of 10 records using 5 stripes can also be seen to lead to low variability estimates when compared to the case involving 20 stripes (Table A.13 and Table A.14). The *bootstrap* procedure, on the other hand, leads to a larger variability of the fragility parameters when fewer stripes are used, rendering it incompatible with *regrouping* regardless of the number of records that is used. Ultimately, the conclusions found for the analyses involving a deterministic capacity limit state can be seen to be applicable to analyses that consider uncertainty in the capacity limit state. Nonetheless, differences between the results obtained with the *regrouping* and the *bootstrap* procedures are much smaller in the former analyses. In the following Section, additional findings are presented to further analyse the effects of propagating these results when computing the failure rate.

Table A.11 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 30 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.70	0.012	1.69%	-	0.46	0.016	3.46%
	<i>Bootstrap</i> i (100)	0.69	0.69	0.010	1.51%	0.45	0.45	0.015	3.28%
4-Storey	<i>Regrouping</i>	-	0.58	0.010	1.65%	-	0.47	0.016	3.47%
	<i>Bootstrap</i> i (100)	0.56	0.56	0.008	1.47%	0.45	0.45	0.014	3.03%
5-storey	<i>Regrouping</i>	-	0.47	0.008	1.63%	-	0.52	0.014	2.78%
	<i>Bootstrap</i> i (100)	0.47	0.47	0.006	1.21%	0.50	0.50	0.014	2.89%

Table A.12 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 30 records and 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.70	0.013	1.79%	-	0.46	0.018	3.98%
	<i>Bootstrap i (100)</i>	0.69	0.69	0.019	2.81%	0.46	0.46	0.030	6.49%
4-Storey	<i>Regrouping</i>	-	0.58	0.010	1.76%	-	0.48	0.019	3.92%
	<i>Bootstrap i (100)</i>	0.56	0.56	0.014	2.47%	0.47	0.47	0.029	6.23%
5-storey	<i>Regrouping</i>	-	0.46	0.009	2.02%	-	0.54	0.016	2.96%
	<i>Bootstrap i (100)</i>	0.46	0.46	0.009	1.93%	0.53	0.53	0.027	5.06%

Table A.13 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 10 records and 20 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.69	0.036	5.20%	-	0.46	0.051	11.01%
	<i>Bootstrap i (100)</i>	0.71	0.70	0.020	2.88%	0.47	0.47	0.023	4.94%
4-Storey	<i>Regrouping</i>	-	0.58	0.028	4.82%	-	0.47	0.045	9.68%
	<i>Bootstrap i (100)</i>	0.56	0.56	0.013	2.32%	0.46	0.46	0.021	4.57%
5-storey	<i>Regrouping</i>	-	0.47	0.026	5.50%	-	0.52	0.044	8.50%
	<i>Bootstrap i (100)</i>	0.53	0.53	0.013	2.41%	0.60	0.60	0.029	4.92%

Table A.14 Statistics of the fragility function parameters due to group-to-group variability for the three buildings and probabilistic capacity using 10 records and 5 stripes.

Building	Procedure	$\hat{\theta}$	$\mu_{\hat{\theta}}$	$\sigma_{\hat{\theta}}$	$CV_{\hat{\theta}}(\%)$	$\hat{\beta}$	$\mu_{\hat{\beta}}$	$\sigma_{\hat{\beta}}$	$CV_{\hat{\beta}}(\%)$
3-Storey	<i>Regrouping</i>	-	0.69	0.038	5.46%	-	0.46	0.057	12.48%
	<i>Bootstrap i (100)</i>	0.70	0.71	0.035	5.02%	0.46	0.46	0.047	10.23%
4-Storey	<i>Regrouping</i>	-	0.58	0.031	5.45%	-	0.48	0.056	11.69%
	<i>Bootstrap i (100)</i>	0.56	0.57	0.023	4.03%	0.48	0.47	0.044	9.24%
5-storey	<i>Regrouping</i>	-	0.46	0.029	6.33%	-	0.54	0.046	8.45%
	<i>Bootstrap i (100)</i>	0.54	0.54	0.018	3.41%	0.58	0.58	0.043	7.45%

A.5.2 Variability of the estimate of the failure rate

A.5.2.1 Deterministic limit state scenario

Failure rates are determined from Eq. (A.3) using the fragility functions obtained in Section A.5.1.1 and the seismic hazard curves that were presented in Section A.5.1.1. The statistics of the $\hat{\lambda}_f$ values obtained for the sets of fragility functions determined in Section A.5.1.1 are presented in Table A.15 and Table A.16 for the cases involving groups of 20 records and 20 and 5 stripes, respectively. It is noted that the first column of values in Table A.15 and Table A.16 corresponds to the values of $\hat{\lambda}_f$ obtained with the fragility function of the parent group i .

Results indicate that computing $\hat{\lambda}_f$ for the fragility functions derived from the *regrouping* or the *bootstrap* procedure yields similar mean values of $\hat{\lambda}_f$, irrespective of the number of stripes. The variability of $\hat{\lambda}_f$ expressed by $\sigma_{\hat{\lambda}_f}$, however, can be marginally larger when using the *regrouping*-based fragility functions and 20 stripes. On the contrary, when only 5 stripes are used, the value of $\sigma_{\hat{\lambda}_f}$ for the *bootstrap* procedure can be up to 50% larger than the value obtained for the *regrouping* procedure. Therefore, although the relationship between the fragility curves and the failure rates is inherently nonlinear (accounting also for the shape of the seismic hazard curve), the variability of the fragility function parameters propagates to the variability of the failure rate. It is also noted that variability values obtained from fragility curves involving 5 stripes and the *bootstrap*

procedure are systematically larger than the corresponding values derived from fragility curves involving 20 stripes and, in some cases, they exhibit an increase of more than 100%. Therefore, the issues related with the inconsistency between the *regrouping* samples and the *bootstrap* pseudo-samples that are highlighted in the results of the previous sections propagate in a similar way to the failure rate estimates. Results obtained considering different sizes of group records follow similar trends and are omitted herein for the sake of brevity.

Table A.15. Statistics of $\hat{\lambda}_f$ due to group-to-group variability for the three buildings using 20 records and 20 stripes

Building	Procedure	$\hat{\lambda}_f (\times 10^{-4})$	$\mu_{\hat{\lambda}_f} (\times 10^{-4})$	$\sigma_{\hat{\lambda}_f} (\times 10^{-5})$	$CV_{\hat{\lambda}_f} (\%)$
3-Storey	<i>Regrouping</i>	-	4.46	3.54	7.94
	<i>Bootstrap i (100)</i>	3.88	5.30	3.70	6.98
4-Storey	<i>Regrouping</i>	-	4.12	2.31	5.61
	<i>Bootstrap i (100)</i>	4.18	4.16	2.23	5.36
5-storey	<i>Regrouping</i>	-	3.82	2.26	5.92
	<i>Bootstrap i (100)</i>	3.81	3.82	2.29	5.98

Table A.16. Statistics of $\hat{\lambda}_f$ due to group-to-group variability for the three buildings using 20 records and 5 stripes

Building	Procedure	$\hat{\lambda}_f (\times 10^{-4})$	$\mu_{\hat{\lambda}_f} (\times 10^{-4})$	$\sigma_{\hat{\lambda}_f} (\times 10^{-5})$	$CV_{\hat{\lambda}_f} (\%)$
3-Storey	<i>Regrouping</i>	-	4.57	3.94	8.61
	<i>Bootstrap i (100)</i>	4.23	4.19	6.02	14.35
4-Storey	<i>Regrouping</i>	-	4.68	3.46	7.40
	<i>Bootstrap i (100)</i>	4.87	4.73	5.27	11.13
5-storey	<i>Regrouping</i>	-	3.55	2.75	7.73
	<i>Bootstrap i (100)</i>	3.53	3.49	3.19	9.13

A.5.2.2 Probabilistic limit state scenario

The failure rates presented in this Section are determined from Eq. (A.3) using the seismic hazard curve presented in Section 4.1.2 and the fragility functions obtained in Section A.5.1.2, which involve record-to-record variability and uncertainty of the limit state capacity. The statistics of $\hat{\lambda}_f$ are shown in Table A.17 and Table A.18 for the cases involving groups of 20 records and for 20 and 5 stripes, respectively. The results indicate that including the variability of the capacity leads to higher mean values of $\hat{\lambda}_f$ but to a lower variability of $\hat{\lambda}_f$. The reduction in the variability of $\hat{\lambda}_f$ reflects the reduction in the variability of the fragility function parameters discussed in Section A.5.1.2. This observation is valid for the results obtained by the *bootstrap* or the *regrouping* procedures and is consistent for all buildings considered. Finally, as for the results in the previous Section, when using a lower number stripes, the variability of $\hat{\lambda}_f$ obtained from the *regrouping* procedure is seen to increase between 10% to 30%, while that obtained by the *bootstrap* procedure is much larger, increasing between 60% to 100%. The effects of the inconsistency between the *regrouping* samples and the *bootstrap* pseudo-samples are therefore similar to those found in the results of the previous Section, further supporting the proposed conditions for using the *bootstrap* procedure.

Table A.17. Statistics of $\hat{\lambda}_f$ due to group-to-group variability and probabilistic capacity for the three buildings using 20 stripes

Building	Procedure	$\hat{\lambda}_f (\times 10^{-4})$	$\mu_{\hat{\lambda}_f} (\times 10^{-4})$	$(\times 10^{-5})$	$CV_{\hat{\lambda}_f} (\%)$
3-Storey	<i>Regrouping</i>	-	5.75	2.66	4.62
	<i>Bootstrap i (100)</i>	5.26	6.30	2.83	4.49
4-Storey	<i>Regrouping</i>	-	5.34	1.87	3.49
	<i>Bootstrap i (100)</i>	5.35	5.35	1.68	3.14
5-storey	<i>Regrouping</i>	-	5.29	2.12	4.01
	<i>Bootstrap i (100)</i>	5.45	5.45	1.87	3.43

Table A.18. Statistics of $\hat{\lambda}_f$ due to group-to-group variability and probabilistic capacity for the three buildings using 5 stripes

Building	Procedure	$\hat{\lambda}_f (\times 10^{-4})$	$\mu_{\hat{\lambda}_f} (\times 10^{-4})$	$\sigma_{\hat{\lambda}_f} (\times 10^{-5})$	$CV_{\hat{\lambda}_f} (\%)$
3-Storey	<i>Regrouping</i>	-	5.68	3.24	5.70
	<i>Bootstrap i (100)</i>	5.36	5.37	4.72	8.79
4-Storey	<i>Regrouping</i>	-	5.46	2.12	3.88
	<i>Bootstrap i (100)</i>	5.45	5.44	2.65	4.87
5-storey	<i>Regrouping</i>	-	5.49	2.77	5.06
	<i>Bootstrap i (100)</i>	5.78	5.77	3.64	6.31

A.6 Conclusions

The uncertainty in estimating the probabilistic seismic performance of structures associated to the use of a certain finite-size group of ground motion records, i.e. the group-to-group variability, was examined for several situations. The *regrouping* and the *bootstrap* procedures were applied to quantify this variability, even though the validity of the latter was disputed due to its incompatibility with cases where the probabilistic seismic performance assessment enforces specific criteria for the ground motion selection. Although *bootstrap* is widely applied for determining uncertainty estimates, the randomness condition of the bootstrap pseudo-samples is not compatible with the situation of selecting several group of records compatible with a certain seismic scenario. The *regrouping* procedure proposed in this study, on the other hand, does not have this incompatibility since it considers additional constraints accounting for the required spectral matching conditions for each new ground motion group sample. Due to the higher computational cost of the latter though, the performance of the two procedures in determining the variability of fragility function parameter estimates and seismic risk estimates was analysed in order to establish the conditions under which the *bootstrap* procedure is expected to lead to small errors.

The findings show that the *regrouping* and the *bootstrap* procedures provide comparable results for both variables examined (i.e. the fragility function parameters and the failure rates) when a relatively high number of stripes was used to define the probabilistic

demand model (i.e. 20 *IMs*) combined with an adequate number of records (i.e. more than 10). When using a lower number of stripes, i.e. 5, the variability of the parameter estimates determined from the *regrouping* procedure did not change substantially from the case involving 20 *IMs*, thus showing the robustness of the procedure. On the other hand, the variability quantified using the *bootstrap* procedure was systematically much larger than in the case involving 20 *IMs*. In some cases, the variability estimates of the examined parameters more than doubled.

The variability of the fragility function parameters was further examined when additional uncertainty was involved by introducing a probabilistic capacity model. In this case, the results showed a reduction of the variability of all parameters, contradicting the potential expectation that an increase in the variability of the input would result in an increase in the variability of the output. This reduction in variability was consistently observed in the results obtained from the *regrouping* and the *bootstrap* procedures, for all group sizes, and also for the two scenarios involving a different number of stripes. Still, when using a low number of stripes, the *bootstrap* procedure led to a significant increase of the variability estimates, while those obtained from the *regrouping* procedure showed much smaller differences with respect to the case involving 20 stripes. This variability further propagated to the failure rate estimates, leading to results with similar trends.

In the overall, although the *regrouping* procedure requires additional computational effort, it provides robust variability estimates for the fragility function parameters and the failure rates, independently of the number of stripes that is used, as long as an adequate number of records is used. On the contrary, the variability estimates obtained from the *bootstrap* procedure are highly dependent on the number of selected stripes. Bootstrap results were considered to be adequate when obtained using 20 stripes as well as a group of at least 20 records. In addition, the relative performance of both procedures was not seen to be affected by the selected capacity modelling approach. Nevertheless, more analyses are required in order to validate these findings for cases including additional sources uncertainties, such as modelling uncertainties. Finally, all conclusions obtained are site-specific, and their validity for different sites characterized by different seismic hazard therefore requires further verification.

Appendix B

Appendix B Tables of ground motions

Table B.1 Ground motions for $T^*=0.27s$ and 30% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chalfant Valley-03	1986	5.65	Zack Brothers Ranch	15.24	16.04	271.4	0.935
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	2.557
Imperial Valley-07	1979	5.01	El Centro Array #1	24.36	26.15	237.3	2.466
Whittier Narrows-01	1987	5.99	Rancho Cucamonga - FF	47.13	49.33	390.2	2.809
Chi-Chi, Taiwan-05	1999	6.2	TCU145	77.94	78.58	215	1.088
Chi-Chi, Taiwan-02	1999	5.9	HWA032	48.22	48.88	473.9	3.507
Chi-Chi, Taiwan-05	1999	6.2	TCU053	59.74	60.57	454.6	1.251
Friuli, Italy-01	1976	6.5	Feltre	108.56	108.68	659.6	3.616
N. Palm Springs	1986	6.06	Cranston Forest Station	35.88	37.53	370.8	1.03
Chi-Chi, Taiwan-05	1999	6.2	CHY094	77.35	77.99	221.9	1.598
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.409
San Fernando	1971	6.61	Lake Hughes #9	23.1	26.51	670.8	1.284
Mammoth Lakes-06	1980	5.94	Fish & Game (FIS)	12.02	18.45	338.5	0.488
Chi-Chi, Taiwan-06	1999	6.3	TCU122	41.24	44.23	475.5	0.76
Chi-Chi, Taiwan-05	1999	6.2	TCU060	63.98	64.76	272.6	2.232

Table B.1 Ground motions for $T^*=0.27s$ and 30% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
N. Palm Springs	1986	6.06	Hemet Fire Station	45.4	46.71	338.5	1.503
Chi-Chi, Taiwan-02	1999	5.9	TCU073	10.3	13.04	272.6	1.162
Northridge-01	1994	6.69	Monte Nido Fire Station	19.19	25.97	659.6	0.522
Denali, Alaska	2002	7.9	Carlo (temp)	67.7	68.39	963.9	1.347
N. Palm Springs	1986	6.06	Anza - Red Mountain	46.17	47.46	684.9	1.171
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	1.022
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	1.42
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	1.696
Imperial Valley-06	1979	6.53	SAHOP Casa Flores	12.43	15.93	338.6	0.287
Imperial Valley-07	1979	5.01	Bonds Corner	12.67	15.83	223	1.966
N. Palm Springs	1986	6.06	Silent Valley - Poppet Flat	27.7	29.8	684.9	1.129
Chi-Chi, Taiwan-05	1999	6.2	TCU048	64.31	65.09	551.2	1.055
San Fernando	1971	6.61	Lake Hughes #4	24.19	27.46	821.7	0.989
N. Palm Springs	1986	6.06	Winchester Bergman Ran	59.94	60.94	684.9	3.28
Chi-Chi, Taiwan-05	1999	6.2	TCU141	62.76	63.56	215	0.681
Victoria, Mexico	1980	6.33	Cerro Prieto	33.73	35.48	659.6	0.316
Chi-Chi, Taiwan-05	1999	6.2	CHY033	93.08	93.62	197.6	1.487
Chi-Chi, Taiwan-05	1999	6.2	HWA030	37.65	38.96	272.6	1.699
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	1.101
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	1.143
Chi-Chi, Taiwan-05	1999	6.2	CHY082	80.27	80.89	193.7	1.455
San Fernando	1971	6.61	Wrightwood - 6074 Park Dr	72.17	73.33	486	1.343
Chi-Chi, Taiwan-05	1999	6.2	CHY111	86.84	87.42	276.3	0.759
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.462
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	0.691

Table B.2 Ground motions for $T^*=0.27$ s and 10% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	1.866
Chi-Chi, Taiwan-06	1999	6.3	CHY087	73.65	75.37	505.2	2.118
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.369
Northridge-01	1994	6.69	LA - Baldwin Hills	28.2	33.18	297.1	0.73
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	0.901
Chi-Chi, Taiwan-05	1999	6.2	TCU053	59.74	60.57	454.6	2.681
Chi-Chi, Taiwan-05	1999	6.2	CHY082	80.27	80.89	193.7	3.117
Chi-Chi, Taiwan-05	1999	6.2	CHY027	85.09	85.68	210	3.089
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	1.941
Whittier Narrows-01	1987	5.99	LA - Hollywood Stor FF	24.21	28.27	316.5	2.213
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	3.044
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	3.634
Chi-Chi, Taiwan-05	1999	6.2	TCU082	55.66	56.55	472.8	3.43
Superstition Hills-02	1987	6.54	Plaster City	25.98	27.49	345.4	1.248
Mammoth Lakes-04	1980	5.7	Long Valley Dam (Upr L Abut)	14.19	15.04	345.4	1.196
Chalfant Valley-02	1986	6.19	Long Valley Dam (L Abut)	23.76	25.78	345.4	2.428
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.876
Mammoth Lakes-06	1980	5.94	Fish & Game (FIS)	12.02	18.45	338.5	1.046
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	2.079
Chi-Chi, Taiwan-05	1999	6.2	CHY094	77.35	77.99	221.9	3.424
San Fernando	1971	6.61	Santa Felita Dam (Outlet)	31.55	34.13	389	2.638
San Fernando	1971	6.61	Lake Hughes #4	24.19	27.46	821.7	2.12
Superstition Hills-02	1987	6.54	Calipatria Fire Station	31.62	32.88	205.8	1.109
Northridge-06	1994	5.28	Santa Monica City Hall	24.55	27.82	336.2	3.475
Northern Calif-05	1967	5.6	Ferndale City Hall	29.73	0	219.3	1.906
San Fernando	1971	6.61	Lake Hughes #9	23.1	26.51	670.8	2.752

Table B.2 Ground motions for $T^*=0.27s$ and 10% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	0.758
Gulf of California	2001	5.7	Bonds Corner	83.55	84.15	223	3.528
Chi-Chi, Taiwan-05	1999	6.2	TCU145	77.94	78.58	215	2.332
Chi-Chi, Taiwan-02	1999	5.9	TCU089	16.13	18	553.4	3.63
Victoria, Mexico	1980	6.33	Cerro Prieto	33.73	35.48	659.6	0.678
Loma Prieta	1989	6.93	WAHO	12.56	21.52	376.1	0.44
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	2.448
N. Palm Springs	1986	6.06	Hemet Fire Station	45.4	46.71	338.5	3.22
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.99
N. Palm Springs	1986	6.06	Cranston Forest Station	35.88	37.53	370.8	2.206
Chi-Chi, Taiwan-05	1999	6.2	TCU141	62.76	63.56	215	1.458
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	2.359
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	1.441
Mammoth Lakes-06	1980	5.94	Convict Creek	12.03	18.46	338.5	1.291

Table B.3 Ground motions for $T^*=0.27s$ and 5% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-03	1999	6.2	CHY041	48.77	49.39	492.3	2.041
Hector Mine	1999	7.13	Big Bear Lake - Fire Station	68.47	70.06	338.5	1.337
Whittier Narrows-01	1987	5.99	Norwalk - Imp Hwy, S Grnd	14.98	20.92	270.2	2.291
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	2.517
Coalinga-07	1983	5.21	Sulphur Baths (temp)	12.02	14.66	617.4	2.051
Chi-Chi, Taiwan-05	1999	6.2	TCU138	50.86	51.84	652.9	1.91
Chi-Chi, Taiwan	1999	7.62	ILA050	113.48	113.77	473.9	3.21
Morgan Hill	1984	6.19	Gilroy Array #6	36.34	37.32	663.3	0.777

Table B.3 Ground motions for $T^*=0.27$ s and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Hector Mine	1999	7.13	Joshua Tree N.M. - Keys View	75.38	76.82	684.9	2.721
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	2.577
Whittier Narrows-01	1987	5.99	LA - 116th St School	21.26	25.79	301	1.076
Chi-Chi, Taiwan-05	1999	6.2	CHY046	73.01	73.69	442.2	1.971
Hector Mine	1999	7.13	Baker Fire Station	77.04	78.45	271.4	2.214
N. Palm Springs	1986	6.06	Sunnymead	50.13	51.32	271.4	1.992
Whittier Narrows-01	1987	5.99	Hacienda Heights - Colima	14.34	20.46	337	1.496
Loma Prieta	1989	6.93	Fremont - Mission San Jose	54.54	57.27	367.6	2.503
Northridge-01	1994	6.69	Simi Valley - Katherine Rd	12.18	21.32	557.4	0.508
Northridge-01	1994	6.69	Lake Hughes #1	53.34	56.13	425.3	2.492
Coalinga-05	1983	5.77	Coalinga-14th & Elm (Old CHP)	11.21	13.43	338.5	0.767
Hector Mine	1999	7.13	Fun Valley	75.77	77.2	345.4	3.299
Northridge-01	1994	6.69	Monte Nido Fire Station	19.19	25.97	659.6	1.615
San Fernando	1971	6.61	Palmdale Fire Station	31.61	34.18	452.9	1.928
Chi-Chi, Taiwan	1999	7.62	ILA043	127	127.25	473.9	3.389
Northridge-01	1994	6.69	N Hollywood - Coldwater Can	13.12	21.87	446	0.836
Fruili, Italy-03	1976	5.5	Forgaria Cornino	19.97	20.85	412.4	2.982
Chi-Chi, Taiwan-06	1999	6.3	TCU105	61.53	63.58	575.5	3.698
Northwest Calif-01	1938	5.5	Ferndale City Hall	54.88	0	219.3	2.53
Whittier Narrows-01	1987	5.99	LA - Hollywood Stor FF	24.21	28.27	316.5	3.197
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	1.062
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	1.066
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	1.545
Mammoth Lakes-06	1980	5.94	Convict Creek	12.03	18.46	338.5	1.865
Chi-Chi, Taiwan-05	1999	6.2	TCU145	77.94	78.58	215	3.369
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	2.138
Whittier Narrows-02	1987	5.27	LA - 116th St School	20.52	24.45	301	2.346

Table B.3 Ground motions for $T^*=0.27$ s and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-01	1994	6.69	Burbank - Howard Rd.	23.18	29.05	821.7	1.413
Northridge-01	1994	6.69	Lake Hughes #12A	40.65	44.26	602.1	1.231
San Fernando	1971	6.61	Pearblossom Pump	45.41	47.23	529.1	3.775
Chi-Chi, Taiwan-05	1999	6.2	CHY087	74.26	74.93	505.2	2.834
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	3.536

Table B.4 Ground motions for $T^*=0.27$ s and 2% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-05	1999	6.2	TCU061	65.02	65.78	272.6	2.363
Taiwan SMART1(40)	1986	6.32	SMART1 E01	65.48	67.36	274.5	3.206
Northridge-01	1994	6.69	Monte Nido Fire Station	19.19	25.97	659.6	2.559
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	2.415
Chi-Chi, Taiwan	1999	7.62	CHY028	32.67	33.63	542.6	0.494
Northridge-01	1994	6.69	LA - Century City CC North	20.22	26.74	278	1.837
Northridge-01	1994	6.69	Simi Valley - Katherine Rd	12.18	21.32	557.4	0.805
Northridge-01	1994	6.69	Baldwin Park - N Holly	54.68	57.41	308.6	3.735
Whittier Narrows-01	1987	5.99	Pasadena - CIT Mudd Lab	10.71	18.11	370.8	2.575
Northridge-01	1994	6.69	LA - Cypress Ave	33.25	37.58	446	2.077
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	2.062
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	2.266
Mammoth Lakes-06	1980	5.94	Fish & Game (FIS)	12.02	18.45	338.5	2.395
Northridge-01	1994	6.69	Pacific Palisades - Sunset	18.22	25.26	446	1.121
Loma Prieta	1989	6.93	Gilroy Array #2	29.77	34.52	270.8	1.25
Whittier Narrows-01	1987	5.99	West Covina - S Orange Ave	12	18.9	308.6	3.813

Table B.4 Ground motions for $T^*=0.27$ s and 2% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-05	1999	6.2	TCU138	50.86	51.84	652.9	3.026
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	3.989
Victoria, Mexico	1980	6.33	Cerro Prieto	33.73	35.48	659.6	1.552
Chi-Chi, Taiwan-05	1999	6.2	TCU141	62.76	63.56	215	3.338
Loma Prieta	1989	6.93	WAHO	12.56	21.52	376.1	1.007
Imperial Valley-06	1979	6.53	El Centro Array #8	28.09	29.8	206.1	1.226
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	2.004
Chalfant Valley-02	1986	6.19	Zack Brothers Ranch	14.33	17.47	271.4	1.04
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	3.387
Chi-Chi, Taiwan	1999	7.62	TCU129	14.16	16.27	664.4	0.772
Chalfant Valley-02	1986	6.19	Benton	31.25	32.81	271.4	3.317
Mammoth Lakes-06	1980	5.94	Convict Creek	12.03	18.46	338.5	2.956
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.844
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	0.944
Chi-Chi, Taiwan	1999	7.62	HWA032	62.24	62.75	473.9	3.362
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.48
Northridge-01	1994	6.69	LA - Saturn St	25.52	30.95	308.7	1.295
Whittier Narrows-01	1987	5.99	Inglewood - Union Oil	24.32	28.37	316	2.403
Chi-Chi, Taiwan	1999	7.62	CHY010	51.22	51.85	473.9	1.481
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	2.193
Chi-Chi, Taiwan-03	1999	6.2	TCU129	18.5	20.07	664.4	1.386
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	3.297
Imperial Valley-06	1979	6.53	SAHOP Casa Flores	12.43	15.93	338.6	1.406
Imperial Valley-06	1979	6.53	El Centro Array #1	35.18	36.56	237.3	3.164

Table B.5 Ground motions for $T^*=0.38$ s and 30% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	1.358
N. Palm Springs	1986	6.06	Anza - Red Mountain	46.17	47.46	684.9	1.377

Table B.5 Ground motions for $T^*=0.38s$ and 30% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epical distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-02	1999	5.9	CHY028	53.61	54.2	542.6	2.97
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	2.27
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	1.156
Whittier Narrows-01	1987	5.99	Downey - Birchdale	15.29	21.14	245.1	0.289
Taiwan SMART1(33)	1985	5.8	SMART1 001	45.1	45.22	274.5	2.628
Chi-Chi, Taiwan-05	1999	6.2	TCU101	61.08	61.9	272.6	2.382
Coalinga-04	1983	5.18	Palmer Ave	10.76	14.03	376.1	1.735
Chi-Chi, Taiwan-05	1999	6.2	CHY102	78.79	79.42	553.4	2.911
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	1.009
Chi-Chi, Taiwan-05	1999	6.2	TCU048	64.31	65.09	551.2	1.873
Whittier Narrows-01	1987	5.99	Rancho Cucamonga - FF	47.13	49.33	390.2	3.346
Big Bear-01	1992	6.46	Sage - Fire Station	70.72	71.91	622.9	3.932
Chi-Chi, Taiwan-06	1999	6.3	TCU072	25.09	29.76	468.1	0.6
Chi-Chi, Taiwan-06	1999	6.3	CHY102	80.39	81.97	553.4	3.451
Big Bear-01	1992	6.46	Desert Shores	111.55	112.31	345.4	2.36
CA/Baja Border Area	2002	5.31	El Centro Array #11	54.56	55.01	196.3	1.875
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.417
Chi-Chi, Taiwan-05	1999	6.2	CHY027	85.09	85.68	210	1.826
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	2.037
Chi-Chi, Taiwan-05	1999	6.2	TCU138	50.86	51.84	652.9	1.066
Chalfant Valley-02	1986	6.19	McGee Creek - Surface	32.49	34	359.2	2.566
Chi-Chi, Taiwan-05	1999	6.2	TCU050	61.37	62.18	272.6	1.296
Chi-Chi, Taiwan-05	1999	6.2	CHY111	86.84	87.42	276.3	1.006
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	1.107
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.402
Northridge-06	1994	5.28	Santa Monica City Hall	24.55	27.82	336.2	2.214

Table B.5 Ground motions for $T^*=0.38$ s and 30% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	1.594
Little Skull Mtn,NV	1992	5.65	Station #2-NTS Control Pt. 1	30.17	32.47	659.6	1.33
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	1.098
Chi-Chi, Taiwan-05	1999	6.2	TCU056	60.48	61.3	272.6	1.115
Northridge-06	1994	5.28	LA - Hollywood Stor FF	20.1	23.99	316.5	1.142
Northern Calif-05	1967	5.6	Ferndale City Hall	29.73	0	219.3	0.745
Morgan Hill	1984	6.19	Gilroy - Gavilan Coll.	38.73	39.65	729.7	2.01
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	1.233
Whittier Narrows-01	1987	5.99	LA - Baldwin Hills	26.21	30	297.1	0.463
N. Palm Springs	1986	6.06	Silent Valley - Poppet Flat	27.7	29.8	684.9	1.457
Whittier Narrows-01	1987	5.99	Big Tujunga, Angeles Nat F	29.49	32.9	446	1.367
N. Palm Springs	1986	6.06	Hemet Fire Station	45.4	46.71	338.5	1.481

Table B.6 Ground motions for $T^*=0.38$ s and 10% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	2.612
Chi-Chi, Taiwan-06	1999	6.3	TCU075	36.04	39.43	573	2.189
N. Palm Springs	1986	6.06	Hemet Fire Station	45.4	46.71	338.5	3.136
Mt. Lewis	1986	5.6	Halls Valley	15.86	18.32	281.6	1.303
N. Palm Springs	1986	6.06	Indio - Coachella Canal	52.66	53.79	345.4	3.377
Chi-Chi, Taiwan-05	1999	6.2	CHY087	74.26	74.93	505.2	2.165
Whittier Narrows-01	1987	5.99	Pasadena - CIT Indust. Rel	10.71	18.11	370.8	1.506
N. Palm Springs	1986	6.06	Anza - Red Mountain	46.17	47.46	684.9	2.915
Chi-Chi, Taiwan-05	1999	6.2	TCU104	68.63	69.36	473.9	3.889
Chalfant Valley-03	1986	5.65	Zack Brothers Ranch	15.24	16.04	271.4	2.723

Table B.6 Ground motions for $T^*=0.38$ s and 10% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Loma Prieta	1989	6.93	Coyote Lake Dam (SW Abut)	30.78	35.4	597.1	0.605
Chi-Chi, Taiwan-05	1999	6.2	CHY027	85.09	85.68	210	3.866
Chi-Chi, Taiwan	1999	7.62	NST	88.84	89.2	375.3	0.844
Chi-Chi, Taiwan-05	1999	6.2	TCU050	61.37	62.18	272.6	2.745
Chi-Chi, Taiwan-05	1999	6.2	TCU145	77.94	78.58	215	2.563
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	2.449
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	1.154
Chi-Chi, Taiwan-05	1999	6.2	TCU048	64.31	65.09	551.2	3.967
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.883
Superstition Hills-02	1987	6.54	Brawley Airport	29.91	31.24	208.7	1.703
N. Palm Springs	1986	6.06	Silent Valley - Poppet Flat	27.7	29.8	684.9	3.086
Loma Prieta	1989	6.93	APEEL 2E Hayward Muir Sch	70.81	72.93	271.1	1.135
Northridge-01	1994	6.69	LA - Baldwin Hills	28.2	33.18	297.1	0.853
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.851
CA/Baja Border Area	2002	5.31	El Centro Array #11	54.56	55.01	196.3	3.97
Little Skull Mtn,NV	1992	5.65	Station #2-NTS Control Pt. 1	30.17	32.47	659.6	2.816
Chi-Chi, Taiwan-05	1999	6.2	CHY033	93.08	93.62	197.6	2.361
N. Palm Springs	1986	6.06	San Jacinto - Valley Cemetary	41.82	43.24	338.5	2.589
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	2.138
Chi-Chi, Taiwan-02	1999	5.9	TCU073	10.3	13.04	272.6	3.377
Chi-Chi, Taiwan-04	1999	6.2	TTN051	49.99	53.13	553.4	2.43
Chi-Chi, Taiwan-05	1999	6.2	TCU106	61.73	62.53	473.9	2.552
Whittier Narrows-01	1987	5.99	Big Tujunga, Angeles Nat F	29.49	32.9	446	2.895
Loma Prieta	1989	6.93	WAHO	12.56	21.52	376.1	0.363
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	2.877

Table B.6 Ground motions for $T^*=0.38s$ and 10% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Livermore-01	1980	5.8	Del Valle Dam (Toe)	26.79	29.35	338.5	1.637
Chi-Chi, Taiwan-05	1999	6.2	TCU138	50.86	51.84	652.9	2.258
Chi-Chi, Taiwan-05	1999	6.2	CHY111	86.84	87.42	276.3	2.131
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	2.325
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	1.463

Table B.7 Ground motions for $T^*=0.38s$ and 5% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	1.277
Northwest China-04	1997	5.8	Xiker	39.73	45.61	274.5	2.045
Chalfant Valley-02	1986	6.19	Benton	31.25	32.81	271.4	1.755
Coalinga-05	1983	5.77	Pleasant Valley P.P. - yard	16.17	17.79	257.4	1.216
Livermore-02	1980	5.42	Livermore - Morgan Terr Park	10.33	17.8	712.8	2.019
Northridge-06	1994	5.28	LA - Hollywood Stor FF	20.1	23.99	316.5	3.629
Mammoth Lakes-02	1980	5.69	Long Valley Dam (Upr L Abut)	20.07	24.47	345.4	3.207
Chi-Chi, Taiwan-05	1999	6.2	TCU129	40.98	42.19	664.4	1.636
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	3.26
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	3.673
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	3.516
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.324
Chi-Chi, Taiwan-03	1999	6.2	TCU129	18.5	20.07	664.4	0.816
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	2.004
Chi-Chi, Taiwan-04	1999	6.2	TTN051	49.99	53.13	553.4	3.645
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	1.911

Table B.7 Ground motions for $T^*=0.38s$ and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	3.257
Sierra Madre	1991	5.61	LA - City Terrace	27.77	30.25	365.2	3.301
Chi-Chi, Taiwan-06	1999	6.3	TCU079	12.26	20.16	364	0.458
Morgan Hill	1984	6.19	Gilroy Array #2	38.1	39.04	270.8	2.387
Superstition Hills-02	1987	6.54	Plaster City	25.98	27.49	345.4	1.099
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	1.016
Whittier Narrows-01	1987	5.99	LA - S Grand Ave	18.91	23.89	308.6	2.638
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	1.731
Northwest China-02	1997	5.93	Jiashi	23.13	38.68	274.5	2.117
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	3.487
Cape Mendocino	1992	7.01	Shelter Cove Airport	36.28	37.5	513.7	3.794
Northridge-01	1994	6.69	Mt Wilson - CIT Seis Sta	45.77	49	821.7	2.831
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	2.194
Mammoth Lakes-06	1980	5.94	Fish & Game (FIS)	12.02	18.45	338.5	1.139
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	3.918
Sitka, Alaska	1972	7.68	Sitka Observatory	42.85	45.4	659.6	3.508
Hollister-02	1961	5.5	Hollister City Hall	18.92	0	198.8	3.018
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	2.108
Coalinga-07	1983	5.21	Sulphur Baths (temp)	12.02	14.66	617.4	1.877
Victoria, Mexico	1980	6.33	Cerro Prieto	33.73	35.48	659.6	0.665
Point Mugu	1973	5.65	Port Hueneme	18.1	19.79	297.9	2.609
Kobe, Japan	1995	6.9	OKA	100.15	101.74	609	2.941
Kern County	1952	7.36	Taft Lincoln School	43.49	46.21	385.4	1.468
CA/Baja Border Area	2002	5.31	El Centro Array #7	59.3	59.71	210.5	3.953

Table B.8 Ground motions for $T^*=0.38$ s and 2% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	3.229
Northridge-01	1994	6.69	Moorpark - Fire Sta	31.45	35.99	405.2	1.193
Whittier Narrows-01	1987	5.99	La Habra - Briarcliff	17.41	22.72	361.2	2.421
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.949
Imperial Valley-06	1979	6.53	El Centro Array #4	27.13	28.9	208.9	1.185
Landers	1992	7.28	North Palm Springs	32.26	33.01	345.4	3.159
Taiwan SMART1(40)	1986	6.32	SMART1 M01	69.14	70.92	274.5	3.085
Hector Mine	1999	7.13	Hector	26.53	30.38	684.9	1.499
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	3.515
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	2.133
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.702
Imperial Valley-06	1979	6.53	El Centro Array #8	28.09	29.8	206.1	1.123
N. Palm Springs	1986	6.06	North Palm Springs	10.57	15.26	345.4	0.879
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	3.638
Victoria, Mexico	1980	6.33	Chihuahua	36.67	38.29	274.5	2.806
Chi-Chi, Taiwan	1999	7.62	TCU068	47.86	48.52	487.3	0.822
N. Palm Springs	1986	6.06	Cabazon	18.17	21.24	345.4	3.11
Morgan Hill	1984	6.19	Gilroy Array #6	36.34	37.32	663.3	3.025
Coalinga-05	1983	5.77	Pleasant Valley P.P. - yard	16.17	17.79	257.4	1.959
Chi-Chi, Taiwan	1999	7.62	HWA051	74.12	74.55	272.6	2.479
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.69
Chi-Chi, Taiwan	1999	7.62	HWA050	79.25	79.66	272.6	3.877
Chi-Chi, Taiwan	1999	7.62	TCU129	14.16	16.27	664.4	0.856
Loma Prieta	1989	6.93	LGPC	18.46	25.42	477.7	0.743
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	2.788
Northridge-01	1994	6.69	Lake Hughes #12A	40.65	44.26	602.1	3.691
Whittier Narrows-01	1987	5.99	Pasadena - Old House Rd	13.21	19.69	455.4	3.161
Chi-Chi, Taiwan-03	1999	6.2	TCU129	18.5	20.07	664.4	1.315
Northridge-01	1994	6.69	LA - Wadsworth VA Hospital South	19.55	26.24	413.8	1.893

Table B.8 Ground motions for $T^*=0.38$ s and 2% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-01	1994	6.69	LA - UCLA Grounds	18.62	25.55	398.4	1.763
Morgan Hill	1984	6.19	Gilroy Array #3	38.2	39.13	349.9	3.592
Chi-Chi, Taiwan	1999	7.62	NST	88.84	89.2	375.3	2.04
Northridge-01	1994	6.69	LA - Wadsworth VA Hospital North	19.55	26.24	392.2	2.568
Irpinia, Italy-01	1980	6.9	Calitri	15.04	17.79	600	2.465
Chi-Chi, Taiwan-06	1999	6.3	CHY024	42.99	45.87	427.7	3.468
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	2.057
Loma Prieta	1989	6.93	Coyote Lake Dam (Downst)	30.89	35.49	295	1.551
Whittier Narrows-01	1987	5.99	Pasadena - CIT Indust. Rel	10.71	18.11	370.8	3.639
Manjil, Iran	1990	7.37	Abbar	40.43	43.47	724	0.804
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	1.3

Table B.9 Ground motions for $T^*=0.50$ s and 30% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-06	1994	5.28	Santa Monica City Hall	24.55	27.82	336.2	1.77
San Francisco	1957	5.28	Golden Gate Park	11.13	13.7	874	2.029
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	1.952
Kozani, Greece-02	1995	5.1	Chromio Anapsiktirio	12.93	15.59	338.6	3.583
Chalfant Valley-03	1986	5.65	Bishop - LADWP South St	25.9	26.38	271.4	1.336
Imperial Valley-07	1979	5.01	Delta	51.21	52.08	274.5	2.86
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	2.151
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	0.766
Georgia, USSR	1991	6.2	Oni	50.94	51.3	274.5	1.683
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	0.836
Northridge-01	1994	6.69	Mt Wilson - CIT Seis Sta	45.77	49	821.7	1.11
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.485
N. Palm Springs	1986	6.06	Winchester Bergman Ran	59.94	60.94	684.9	3.935

Table B.9 Ground motions for $T^*=0.50$ s and 30% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Trinidad offshore	1983	5.7	Rio Dell Overpass, W Ground	71.24	72.23	311.8	1.178
San Fernando	1971	6.61	Lake Hughes #9	23.1	26.51	670.8	1.817
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	2.109
Cape Mendocino	1992	7.01	Shelter Cove Airport	36.28	37.5	513.7	1.722
Chi-Chi, Taiwan-04	1999	6.2	KAU054	37.5	41.59	473.9	2.205
Morgan Hill	1984	6.19	Gilroy - Gavilan Coll.	38.73	39.65	729.7	2.256
Hector Mine	1999	7.13	Wrightwood - Nielson Ranch	121.68	122.57	345.4	1.037
Chi-Chi, Taiwan	1999	7.62	ILA067	86.38	86.75	553.4	0.485
Oroville-01	1975	5.89	Oroville Seismograph Station	12.58	13.73	622.9	2.583
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	0.608
Northern Calif-05	1967	5.6	Ferndale City Hall	29.73	0	219.3	0.648
Whittier Narrows-01	1987	5.99	Mt Wilson - CIT Seis Sta	19.56	24.41	821.7	2.268
Chi-Chi, Taiwan-02	1999	5.9	CHY028	53.61	54.2	542.6	3.502
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	0.857
Chi-Chi, Taiwan-05	1999	6.2	TCU050	61.37	62.18	272.6	1.308
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	1.458
Sierra Madre	1991	5.61	LA - City Terrace	27.77	30.25	365.2	1.177
Borah Peak, ID-01	1983	6.88	CPP-610	89.48	90.9	424.8	2.161
Coalinga-05	1983	5.77	Coalinga-14th & Elm (Old CHP)	11.21	13.43	338.5	0.32
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	0.297
Chalfant Valley-02	1986	6.19	McGee Creek - Surface	32.49	34	359.2	2.724
CA/Baja Border Area	2002	5.31	El Centro Array #11	54.56	55.01	196.3	1.943
Little Skull Mtn,NV	1992	5.65	Station #2-NTS Control Pt. 1	30.17	32.47	659.6	2.182
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	0.918
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	2.666

Table B.9 Ground motions for $T^*=0.50$ s and 30% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Parkfield	1966	6.19	Temblor pre-1969	40.26	41.48	527.9	0.332
San Juan Bautista	1998	5.17	Hollister Diff. Array	15.62	18.09	215.5	3.81

Table B.10 Ground motions for $T^*=0.50$ s and 10% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-02	1999	5.9	TCU073	10.3	13.04	272.6	3.055
Northridge-01	1994	6.69	Mt Wilson - CIT Seis Sta	45.77	49	821.7	2.475
Chi-Chi, Taiwan-05	1999	6.2	TCU056	60.48	61.3	272.6	3.367
Friuli, Italy-02	1976	5.91	Forgaria Cornino	17.96	18.34	412.4	1.138
Sierra Madre	1991	5.61	LA - City Terrace	27.77	30.25	365.2	2.625
Denali, Alaska	2002	7.9	Carlo (temp)	67.7	68.39	963.9	1.595
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.058
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	1.865
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	1.355
Mammoth Lakes-03	1980	5.91	Long Valley Dam (L Abut)	11.51	19.71	345.4	3.005
Cape Mendocino	1992	7.01	Shelter Cove Airport	36.28	37.5	513.7	3.842
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.25
Kocaeli, Turkey	1999	7.51	Goynuk	77.63	79.27	424.8	1.434
Coalinga-04	1983	5.18	Palmer Ave	10.76	14.03	376.1	3.246
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	1.709
Trinidad offshore	1983	5.7	Rio Dell Overpass, E Ground	71.24	72.23	311.8	1.891
Hector Mine	1999	7.13	Frink	144.83	145.58	345.4	1.469
Whittier Narrows-01	1987	5.99	Big Tujunga, Angeles Nat F	29.49	32.9	446	2.725
Trinidad	1980	7.2	Rio Dell Overpass, W Ground	76.75	78.22	311.8	1.58
Northwest China-04	1997	5.8	Jiashi	24.99	33.56	274.5	1.195
CA/Baja Border Area	2002	5.31	El Centro Array #7	59.3	59.71	210.5	3.404

Table B.10 Ground motions for $T^*=0.50$ s and 10% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Taiwan SMART1(33)	1985	5.8	SMART1 O07	44.73	44.85	274.5	3.745
Almiros, Greece	1980	5.2	Almiros	14.76	16.34	338.6	3.141
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	2.049
Northridge-06	1994	5.28	Santa Monica City Hall	24.55	27.82	336.2	3.95
Chalfant Valley-04	1986	5.44	Zack Brothers Ranch	10.22	13.75	271.4	2.391
Mammoth Lakes-01	1980	6.06	Long Valley Dam (Upr L Abut)	12.65	15.52	345.4	0.508
Morgan Hill	1984	6.19	Gilroy Array #4	37.25	38.2	221.8	0.538
Upland	1990	5.63	Pomona - 4th & Locust FF	10.82	11.71	229.8	1.544
Tabas, Iran	1978	7.35	Ferdows	117.66	117.8	274.5	1.776
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	2.637
Chalfant Valley-03	1986	5.65	Bishop - LADWP South St	25.9	26.38	271.4	2.979
Chi-Chi, Taiwan-06	1999	6.3	TCU129	33.15	36.81	664.4	1.347
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	1.081
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	1.471
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	1.146
Coalinga-05	1983	5.77	Sulphur Baths (temp)	13.4	15.3	617.4	2.817
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	0.543
San Fernando	1971	6.61	Lake Hughes #12	20.04	23.89	602.1	1.392
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	0.988

Table B.11 Ground motions for $T^*=0.50$ s and 5% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Superstition Hills-01	1987	6.22	Wildlife Liquef. Array	24.79	26.73	207.5	2.256
Morgan Hill	1984	6.19	Gilroy Array #6	36.34	37.32	663.3	1.422
Hector Mine	1999	7.13	Leona Valley - Fire Station #1	185.09	185.68	345.4	3.703

Table B.11 Ground motions for $T^*=0.50$ s and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-05	1999	6.2	TCU141	62.76	63.56	215	3.299
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	0.823
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.391
Northwest China-04	1997	5.8	Jiashi	24.99	33.56	274.5	1.813
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	2.593
Trinidad offshore	1983	5.7	Rio Dell Overpass, W Ground	71.24	72.23	311.8	3.987
Coalinga-05	1983	5.77	Pleasant Valley P.P. - yard	16.17	17.79	257.4	0.878
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	2.65
N. Palm Springs	1986	6.06	Cabazon	18.17	21.24	345.4	2.227
Coalinga-01	1983	6.36	Slack Canyon	33.52	33.83	684.9	1.897
Northridge-01	1994	6.69	Mt Wilson - CIT Seis Sta	45.77	49	821.7	3.756
Chi-Chi, Taiwan-04	1999	6.2	CHY035	25.67	31.35	473.9	2.039
Chalfant Valley-04	1986	5.44	Zack Brothers Ranch	10.22	13.75	271.4	3.628
San Fernando	1971	6.61	Santa Felita Dam (Outlet)	31.55	34.13	389	3.036
Chi-Chi, Taiwan-06	1999	6.3	TCU138	42.57	45.48	652.9	3.241
Kocaeli, Turkey	1999	7.51	Arcelik	53.68	56.02	523	2.268
Trinidad	1980	7.2	Rio Dell Overpass - FF	76.75	78.22	311.8	3.995
Imperial Valley-06	1979	6.53	Victoria	43.9	45.02	274.5	3.467
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	1.64
Whittier Narrows-01	1987	5.99	Hacienda Heights - Colima	14.34	20.46	337	3.036
Parkfield	1966	6.19	Temblor pre-1969	40.26	41.48	527.9	1.124
Denali, Alaska	2002	7.9	TAPS Pump Station #11	189.65	189.9	376.1	3.626
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	1.739
Duzce, Turkey	1999	7.14	Lamont 1059	24.26	28.01	424.8	2.714
Big Bear-01	1992	6.46	Joshua Tree	48.33	50.05	379.3	3.577
Taiwan SMART1(5)	1981	5.9	SMART1 M07	29.54	31.56	274.5	1.933
Chalfant Valley-02	1986	6.19	Long Valley Dam (L Abut)	23.76	25.78	345.4	3.412
Hollister-04	1986	5.45	Hollister Diff Array #3	14.76	17.14	215.5	2.421

Table B.11 Ground motions for $T^*=0.50$ s and 5% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.475
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	2.968
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.896
Kobe, Japan	1995	6.9	Shin-Osaka	45.97	49.33	256	0.71
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	3.108
Superstition Hills-02	1987	6.54	Calipatria Fire Station	31.62	32.88	205.8	2.36
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	2.83
Sierra Madre	1991	5.61	LA - Obregon Park	29.61	31.95	349.4	2.736
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.606

Table B.12 Ground motions for $T^*=0.50$ s and 2% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	2.885
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	2.664
Imperial Valley-06	1979	6.53	Brawley Airport	43.15	44.29	208.7	2.726
Morgan Hill	1984	6.19	Gilroy Array #6	36.34	37.32	663.3	2.359
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	3.146
Northridge-01	1994	6.69	Lake Hughes #12A	40.65	44.26	602.1	3.664
Hector Mine	1999	7.13	Frink	144.83	145.58	345.4	3.698
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.649
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	2.488
N. Palm Springs	1986	6.06	Cabazon	18.17	21.24	345.4	3.695
Chi-Chi, Taiwan-06	1999	6.3	TCU129	33.15	36.81	664.4	3.392
Tabas, Iran	1978	7.35	Dayhook	20.63	21.41	659.6	1.229
Superstition Hills-01	1987	6.22	Wildlife Liquef. Array	24.79	26.73	207.5	3.742
Manjil, Iran	1990	7.37	Abbar	40.43	43.47	724	0.935
Coalinga-01	1983	6.36	Parkfield - Gold Hill 3W	49.47	49.68	438.3	3.955
Whittier Narrows-01	1987	5.99	Pasadena - CIT Indust. Rel	10.71	18.11	370.8	3.756
Chi-Chi, Taiwan-04	1999	6.2	CHY035	25.67	31.35	473.9	3.382

Table B.12 Ground motions for $T^*=0.50$ s and 2% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Hollister-01	1961	5.6	Hollister City Hall	20.61	0	198.8	2.321
Taiwan SMART1(45)	1986	7.3	SMART1 O04	75.67	77.14	274.5	1.806
Superstition Hills-02	1987	6.54	Calipatria Fire Station	31.62	32.88	205.8	3.916
San Fernando	1971	6.61	Lake Hughes #12	20.04	23.89	602.1	3.504
Northern Calif-03	1954	6.5	Ferndale City Hall	30.79	0	219.3	2.306
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	2.106
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	2.594
Northwest China-03	1997	6.1	Jiashi	19.11	27.66	274.5	1.476
Coalinga-03	1983	5.38	Burnett Construction	15.56	15.75	352.2	3.744
Chi-Chi, Taiwan-03	1999	6.2	TCU065	32.05	32.99	305.9	2.297
Chalfant Valley-02	1986	6.19	Benton	31.25	32.81	271.4	2.283
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	2.722
Irpinia, Italy-02	1980	6.2	Rionero In Vulture	29.83	30.64	530	3.799
Imperial Valley-02	1940	6.95	El Centro Array #9	12.99	15.69	213.4	1.212
Northwest China-04	1997	5.8	Jiashi	24.99	33.56	274.5	3.008
Kocaeli, Turkey	1999	7.51	Arcelik	53.68	56.02	523	3.763
Upland	1990	5.63	Pomona - 4th & Locust FF	10.82	11.71	229.8	3.887
Coalinga-05	1983	5.77	Pleasant Valley P.P. - FF	16.17	17.79	257.4	1.924
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	1.366
Big Bear-01	1992	6.46	San Bernardino - E & Hospitality	45.51	47.33	271.4	3.272
Chi-Chi, Taiwan-05	1999	6.2	TCU129	40.98	42.19	664.4	3.379
Kobe, Japan	1995	6.9	Kakogawa	24.2	30.1	312	1.33
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.788

Table B.13 Ground motions for $T^*=0.66$ s and 30% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-06	1994	5.28	LA - Baldwin Hills	26.87	29.89	297.1	2.175
Friuli, Italy-02	1976	5.91	Forgaria Cornino	17.96	18.34	412.4	0.964
Landers	1992	7.28	Twentynine Palms	44.1	44.65	684.9	2.213
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	2.399
Little Skull Mtn,NV	1992	5.65	Station #2-NTS Control Pt. 1	30.17	32.47	659.6	2.484
CA/Baja Border Area	2002	5.31	El Centro Array #11	54.56	55.01	196.3	2.028
Northridge-01	1994	6.69	Santa Susana Ground	14.66	22.83	715.1	0.343
Chi-Chi, Taiwan-06	1999	6.3	TCU129	33.15	36.81	664.4	0.655
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	0.519
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.69
Caldiran, Turkey	1976	7.21	Maku	55.65	56.54	274.5	2.552
Hector Mine	1999	7.13	Hector	26.53	30.38	684.9	0.263
Borah Peak, ID-01	1983	6.88	CPP-610	89.48	90.9	424.8	3.759
Whittier Narrows-01	1987	5.99	LA - Saturn St	25.27	29.18	308.7	1.792
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	0.954
Chalfant Valley-02	1986	6.19	Bishop - Paradise Lodge	15.42	18.37	345.4	1.636
Friuli, Italy-01	1976	6.5	Barcis	55.62	55.86	424.8	3.775
Coalinga-05	1983	5.77	Sulphur Baths (temp)	13.4	15.3	617.4	1.49
Livermore-01	1980	5.8	Del Valle Dam (Toe)	26.79	29.35	338.5	0.741
Mammoth Lakes-06	1980	5.94	Bishop - Paradise Lodge	19.97	24.39	345.4	1.59
Superstition Hills-02	1987	6.54	Brawley Airport	29.91	31.24	208.7	0.728
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	1.874
Whittier Narrows-02	1987	5.27	Mt Wilson - CIT Seis Sta	18.74	22.98	821.7	3.139
Chi-Chi, Taiwan-05	1999	6.2	TCU106	61.73	62.53	473.9	1.158
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	0.675
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	0.749

Table B.13 Ground motions for $T^*=0.66$ s and 30% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	1.319
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	2.614
N. Palm Springs	1986	6.06	Silent Valley - Poppet Flat	27.7	29.8	684.9	2.91
Chi-Chi, Taiwan	1999	7.62	HWA057	87.4	87.77	553.4	1.284
Cape Mendocino	1992	7.01	Shelter Cove Airport	36.28	37.5	513.7	2.059
Imperial Valley-07	1979	5.01	Delta	51.21	52.08	274.5	2.401
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	1.097
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	2.204
Chi-Chi, Taiwan-02	1999	5.9	CHY028	53.61	54.2	542.6	3.579
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	1.614
Morgan Hill	1984	6.19	Gilroy - Gavilan Coll.	38.73	39.65	729.7	2.966
Chi-Chi, Taiwan-04	1999	6.2	KAU054	37.5	41.59	473.9	2.232
Imperial Valley-05	1955	5.4	El Centro Array #9	14.78	0	213.4	1.698
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	1.895

Table B.14 Ground motions for $T^*=0.66$ s and 10% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-05	1999	6.2	HWA032	35.57	36.95	473.9	3.841
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	2.124
Imperial Valley-05	1955	5.4	El Centro Array #9	14.78	0	213.4	3.736
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01 (Continued)	35.45	256.8	1.324
Mammoth Lakes-06	1980	5.94	Benton	46.49	48.55	271.4	1.472
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.399
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	2.099
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	1.486

Table B.14 Ground motions for $T^*=0.66$ s and 10% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Coalinga-07	1983	5.21	Sulphur Baths (temp)	12.02	14.66	617.4	1.795
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	1.648
Northridge-01	1994	6.69	Glendale - Las Palmas	29.72	34.49	446	1.23
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	0.549
Coalinga-05	1983	5.77	Sulphur Baths (temp)	13.4	15.3	617.4	3.278
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	1.056
CA/Baja Border Area	2002	5.31	El Centro Array #10	55.23	55.67	202.9	3.814
Taiwan SMART1(5)	1981	5.9	SMART1 M07	29.54	31.56	274.5	1.792
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.638
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	2.287
Hector Mine	1999	7.13	Valyermo Forest Fire Station	146.66	147.41	345.4	2.156
Chi-Chi, Taiwan-02	1999	5.9	TCU079	16.24	18.11	364	1.644
Whittier Narrows-01	1987	5.99	LA - Saturn St	25.27	29.18	308.7	3.942
Imperial Valley-06	1979	6.53	Victoria	43.9	45.02	274.5	2.273
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	2.414
Duzce, Turkey	1999	7.14	Lamont 375	24.05	27.83	424.8	0.946
N. Palm Springs	1986	6.06	Palm Springs Airport	21.14	23.83	207.5	1.061
Chi-Chi, Taiwan-06	1999	6.3	TCU075	36.04	39.43	573	2.354
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	1.13
Mammoth Lakes-03	1980	5.91	Long Valley Dam (L Abut)	11.51	19.71	345.4	2.908
Imperial Valley-07	1979	5.01	El Centro Array #5	10.09	13.86	205.6	1.51
Chalfant Valley-01	1986	5.77	Zack Brothers Ranch	10.54	12.49	271.4	0.72
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.239
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	1.088
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	2.902
Taiwan SMART1(33)	1985	5.8	SMART1 C00	44.93	45.05	274.5	3.966

Table B.14 Ground motions for $T^*=0.66$ s and 10% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Almiros, Greece	1980	5.2	Almiros	14.76	16.34	338.6	2.367
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	3.552
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	1.518
Friuli, Italy-02	1976	5.91	San Rocco	17.53	17.91	659.6	3.536
Chi-Chi, Taiwan-04	1999	6.2	CHY046	38.91	42.87	442.2	1.341
Morgan Hill	1984	6.19	Gilroy Array #7	38.19	39.12	333.9	2.182

Table B.15 Ground motions for $T^*=0.66$ s and 5% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	1.682
Friuli, Italy-02	1976	5.91	Forgaria Cornino	17.96	18.34	412.4	3.277
Duzce, Turkey	1999	7.14	Lamont 1059	24.26	28.01	424.8	3.892
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	2.346
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.37
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.617
Northwest China-04	1997	5.8	Jiashi	24.99	33.56	274.5	1.519
Taiwan SMART1(5)	1981	5.9	SMART1 M01	31.06	32.99	274.5	3.749
Superstition Hills-02	1987	6.54	Salton Sea Wildlife Refuge	26.48	27.97	191.1	1.765
N. Palm Springs	1986	6.06	North Palm Springs	10.57	15.26	345.4	0.561
Chi-Chi, Taiwan	1999	7.62	HWA005	64.06	64.56	272.6	1.663
Chi-Chi, Taiwan-05	1999	6.2	TCU138	50.86	51.84	652.9	2.901
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	3.282
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	1.633
Imperial Valley-06	1979	6.53	Victoria	43.9	45.02	274.5	3.513
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	2.296
Kobe, Japan	1995	6.9	Kakogawa	24.2	30.1	312	0.978
Northridge-01	1994	6.69	Lake Hughes #12A	40.65	44.26	602.1	3.508
Northwest China-02	1997	5.93	Jiashi	23.13	38.68	274.5	1.798

Table B.15 Ground motions for $T^*=0.66s$ and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Whittier Narrows-01	1987	5.99	West Covina - S Orange Ave	12	18.9	308.6	3.383
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	3.244
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	2.046
Chi-Chi, Taiwan-02	1999	5.9	TCU079	16.24	18.11	364	2.54
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	1.763
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	3.535
Chalfant Valley-02	1986	6.19	Lake Crowley - Shehorn Res.	26.59	28.41	338.5	3.751
Hector Mine	1999	7.13	Amboy	47.97	50.2	271.4	1.344
San Fernando	1971	6.61	Lake Hughes #12	20.04	23.89	602.1	2.92
Coalinga-01	1983	6.36	Parkfield - Gold Hill 3W	49.47	49.68	438.3	2.319
Gulf of Aqaba	1995	7.2	Eilat	93.35	94.18	354.9	2.194
Big Bear-01	1992	6.46	Big Bear Lake - Civic Center	10.15	16.49	338.5	1.116
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	0.684
Coalinga-03	1983	5.38	Burnett Construction	15.56	15.75	352.2	3.472
Chi-Chi, Taiwan-06	1999	6.3	TCU056	50.73	53.2	272.6	3.99
Imperial Valley-08	1979	5.62	Westmorland Fire Sta	11.04	11.53	193.7	2.547
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.915
Morgan Hill	1984	6.19	Hollister Diff Array #1	52.82	53.5	215.5	1.947
Coyote Lake	1979	5.74	San Juan Bautista, 24 Polk St	23.24	24.58	370.8	1.999
Coalinga-05	1983	5.77	Palmer Ave	11.09	13.33	376.1	1.299
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	3.73

Table B.16 Ground motions for $T^*=0.66$ s and 2% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Superstition Hills-02	1987	6.54	Calipatria Fire Station	31.62	32.88	205.8	3.129
Chi-Chi, Taiwan-06	1999	6.3	TCU129	33.15	36.81	664.4	3.731
Tabas, Iran	1978	7.35	Dayhook	20.63	21.41	659.6	1.416
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	1.034
San Fernando	1971	6.61	LA - Hollywood Stor FF	39.49	41.57	316.5	2.711
Northridge-01	1994	6.69	Sylmar - Converter Sta East	13.6	22.16	370.5	0.645
Chi-Chi, Taiwan-04	1999	6.2	CHY030	35.44	39.75	204.7	3.491
Kocaeli, Turkey	1999	7.51	Atakoy	99.69	100.97	274.5	3.982
Chi-Chi, Taiwan-05	1999	6.2	TCU129	40.98	42.19	664.4	3.773
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.654
Kobe, Japan	1995	6.9	Kakogawa	24.2	30.1	312	1.64
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	1.423
Livermore-01	1980	5.8	San Ramon - Eastman Kodak	17.13	20.92	271.4	2.255
Spitak, Armenia	1988	6.77	Gukasian	36.19	36.68	274.5	2.199
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	2.929
Northern Calif-03	1954	6.5	Ferndale City Hall	30.79	0	219.3	1.534
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	3.849
Hector Mine	1999	7.13	Amboy	47.97	50.2	271.4	2.253
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	3.932
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	1.146
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	2.956
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	3.43
Westmorland	1981	5.9	Brawley Airport	15.71	15.87	208.7	3.696
Big Bear-01	1992	6.46	Big Bear Lake - Civic Center	10.15	16.49	338.5	1.871
Whittier Narrows-01	1987	5.99	Pasadena - Old House Rd	13.21	19.69	455.4	3.899
Manjil, Iran	1990	7.37	Abbar	40.43	43.47	724	0.929
Coalinga-05	1983	5.77	Pleasant Valley P.P. - yard	16.17	17.79	257.4	1.587
Coalinga-01	1983	6.36	Parkfield - Gold Hill 3W	49.47	49.68	438.3	3.888

Table B.16 Ground motions for $T^*=0.66$ s and 2% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Irpinia, Italy-01	1980	6.9	Bagnoli Irpinio	22.65	24.56	1000	1.921
N. Palm Springs	1986	6.06	Cabazon	18.17	21.24	345.4	3.85
Chalfant Valley-01	1986	5.77	Zack Brothers Ranch	10.54	12.49	271.4	1.865
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.62
Chalfant Valley-02	1986	6.19	Benton	31.25	32.81	271.4	2.062
Victoria, Mexico	1980	6.33	Chihuahua	36.67	38.29	274.5	1.759
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	2.737
Northwest China-02	1997	5.93	Jiashi	23.13	38.68	274.5	3.015
Morgan Hill	1984	6.19	Hollister Diff Array #1	52.82	53.5	215.5	3.264
Imperial Valley-06	1979	6.53	El Centro Array #8	28.09	29.8	206.1	0.961
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	2.297
Superstition Hills-01	1987	6.22	Wildlife Liquef. Array	24.79	26.73	207.5	2.939

Table B.17 Ground motions for $T^*=0.82$ s and 30% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Nahanni, Canada	1985	6.76	Site 3	22.36	23.75	659.6	2.201
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	0.496
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	0.663
Friuli, Italy-01	1976	6.5	Barcis	55.62	55.86	424.8	2.945
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.292
Northwest China-01	1997	5.9	Xiker	51.75	56.67	274.5	2.293
Chi-Chi, Taiwan-04	1999	6.2	CHY062	65.32	67.75	602.6	3.732
Chi-Chi, Taiwan-06	1999	6.3	CHY036	61.5	63.55	233.1	0.407
Chalfant Valley-02	1986	6.19	Lake Crowley - Shehorn Res.	26.59	28.41	338.5	1.475
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	0.928
Northridge-01	1994	6.69	Lake Hughes #9	44.77	48.07	670.8	1.375
Gulf of California	2001	5.7	Bonds Corner	83.55	84.15	223	3.469
Northridge-06	1994	5.28	LA - Hollywood Stor FF	20.1	23.99	316.5	2.693

Table B.17 Ground motions for $T^*=0.82$ s and 30% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Whittier Narrows-02	1987	5.27	Mt Wilson - CIT Seis Sta	18.74	22.98	821.7	3.347
Trinidad offshore	1983	5.7	Rio Dell Overpass, E Ground	71.24	72.23	311.8	1.385
Morgan Hill	1984	6.19	Gilroy - Gavilan Coll.	38.73	39.65	729.7	2.629
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	1.568
Landers	1992	7.28	Twentynine Palms	44.1	44.65	684.9	2.27
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	0.61
Livermore-01	1980	5.8	APEEL 3E Hayward CSUH	30.82	33.08	517.1	2.433
Cape Mendocino	1992	7.01	Shelter Cove Airport	36.28	37.5	513.7	2.197
Chalfant Valley-03	1986	5.65	Bishop - Paradise Lodge	11.92	12.93	345.4	3.665
Friuli, Italy-02	1976	5.91	Forgaria Cornino	17.96	18.34	412.4	1.255
Taiwan SMART1(5)	1981	5.9	SMART1 I12	30.5	32.46	274.5	0.703
CA/Baja Border Area	2002	5.31	El Centro Array #11	54.56	55.01	196.3	1.665
N. Palm Springs	1986	6.06	Puerta La Cruz	75.51	76.31	370.8	3.915
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	1.374
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	1.558
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	0.598
Chi-Chi, Taiwan-02	1999	5.9	TCU079	16.24	18.11	364	0.988
Little Skull Mtn,NV	1992	5.65	Station #2-NTS Control Pt. 1	30.17	32.47	659.6	1.988
Coalinga-05	1983	5.77	Pleasant Valley P.P. - yard	16.17	17.79	257.4	0.42
Caldiran, Turkey	1976	7.21	Maku	55.65	56.54	274.5	2.242
Whittier Narrows-01	1987	5.99	Hacienda Heights - Colima	14.34	20.46	337	1.183
Big Bear-01	1992	6.46	Snow Creek	38.28	40.42	345.4	2.837
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	1.61
Chi-Chi, Taiwan-05	1999	6.2	CHY082	80.27	80.89	193.7	2.051
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	0.298
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	1.018

Table B.17 Ground motions for $T^*=0.82$ s and 30% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Anza (Horse Canyon)-01	1980	5.19	Anza - Pinyon Flat	12.68	18.59	724.9	3.073

Table B.18 Ground motions for $T^*=0.82$ s and 10% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	0.723
Parkfield	1966	6.19	Cholame - Shandon Array #8	34.01	35.45	256.8	0.989
Almiros, Greece	1980	5.2	Almiros	14.76	16.34	338.6	1.861
Chalfant Valley-01	1986	5.77	Bishop - Paradise Lodge	17.22	18.47	345.4	3.911
Taiwan SMART1(5)	1981	5.9	SMART1 O01	31.99	33.86	274.5	1.91
Whittier Narrows-01	1987	5.99	Hacienda Heights - Colima	14.34	20.46	337	2.873
Northwest China-04	1997	5.8	Xiker	39.73	45.61	274.5	2.379
Coalinga-04	1983	5.18	Coalinga-14th & Elm (Old CHP)	11.9	14.92	338.5	3.784
Duzce, Turkey	1999	7.14	Lamont 375	24.05	27.83	424.8	0.725
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	2.253
Chalfant Valley-04	1986	5.44	Zack Brothers Ranch	10.22	13.75	271.4	2.589
San Fernando	1971	6.61	Santa Anita Dam	45.86	47.67	684.9	3.337
Friuli, Italy-02	1976	5.91	Forgaria Cornino	17.96	18.34	412.4	3.048
Trinidad offshore	1983	5.7	Rio Dell Overpass, E Ground	71.24	72.23	311.8	3.364
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	0.71
Coalinga-07	1983	5.21	Sulphur Baths (temp)	12.02	14.66	617.4	1.821
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	1.306
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	0.75
Taiwan SMART1(33)	1985	5.8	SMART1 I01	44.94	45.06	274.5	3.784
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	1.934
Denali, Alaska	2002	7.9	Carlo (temp)	67.7	68.39	963.9	1.837
Chi-Chi, Taiwan-05	1999	6.2	CHY024	48.6	49.62	427.7	1.961

Table B.18 Ground motions for $T^*=0.82$ s and 10% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	1.482
Loma Prieta	1989	6.93	Bear Valley #7, Pinnacles	88.19	89.91	597.1	3.482
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	1.293
Trinidad	1980	7.2	Rio Dell Overpass, E Ground	76.75	78.22	311.8	0.905
Coalinga-05	1983	5.77	Coalinga-14th & Elm (Old CHP)	11.21	13.43	338.5	1.138
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	2.471
Upland	1990	5.63	Rancho Cucamonga - FF	12.19	12.99	390.2	2.77
Chi-Chi, Taiwan-02	1999	5.9	TCU079	16.24	18.11	364	2.4
Northridge-06	1994	5.28	Burbank - N Buena Vista	15.01	19.91	271.4	3.639
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	1.453
Imperial Valley-06	1979	6.53	Superstition Mtn Camera	59.54	60.37	362.4	3.808
Landers	1992	7.28	Mission Creek Fault	32.86	33.6	345.4	1.383
Northridge-01	1994	6.69	Monte Nido Fire Station	19.19	25.97	659.6	1.946
Imperial Valley-05	1955	5.4	El Centro Array #9	14.78	0	213.4	3.602
N. Palm Springs	1986	6.06	Sunnymead	50.13	51.32	271.4	3.518
Big Bear-01	1992	6.46	Wrightwood - Swarthout	78.15	79.23	338.5	3.158
CA/Baja Border Area	2002	5.31	El Centro Array #10	55.23	55.67	202.9	3.312
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	1.609

Table B.19 Ground motions for $T^*=0.82$ s and 5% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	1.911
Denali, Alaska	2002	7.9	Carlo (temp)	67.7	68.39	963.9	2.917
Kozani, Greece-01	1995	6.4	Kozani	18.27	22.22	659.6	3.579

Table B.19 Ground motions for $T^*=0.82s$ and 5% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-01	1994	6.69	Glendale - Las Palmas	29.72	34.49	446	2.56
Chi-Chi, Taiwan-06	1999	6.3	TCU122	41.24	44.23	475.5	1.495
Whittier Narrows-02	1987	5.27	Altadena - Eaton Canyon	13.04	18.62	370.8	3.072
Irpinia, Italy-01	1980	6.9	Bagnoli Irpinio	22.65	24.56	1000	1.02
Imperial Valley-06	1979	6.53	Compuertas	22.43	24.55	274.5	2.417
Chi-Chi, Taiwan-02	1999	5.9	TCU079	16.24	18.11	364	3.812
Whittier Narrows-01	1987	5.99	La Habra - Briarcliff	17.41	22.72	361.2	2.884
Parkfield	1966	6.19	Temblor pre-1969	40.26	41.48	527.9	2.327
Mammoth Lakes-01	1980	6.06	Mammoth Lakes H. S.	10.91	14.14	370.8	2.556
Chi-Chi, Taiwan-04	1999	6.2	CHY028	22.19	28.57	542.6	2.801
Hector Mine	1999	7.13	Amboy	47.97	50.2	271.4	1.03
Mammoth Lakes-06	1980	5.94	Convict Creek	12.03	18.46	338.5	1.562
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.127
Chi-Chi, Taiwan-05	1999	6.2	CHY024	48.6	49.62	427.7	3.114
San Fernando	1971	6.61	Lake Hughes #12	20.04	23.89	602.1	2.413
Chi-Chi, Taiwan-03	1999	6.2	TCU071	20.51	21.95	624.9	2.354
Northridge-06	1994	5.28	Sun Valley - Sunland	10.04	16.49	271.4	3.891
Hollister-04	1986	5.45	Hollister Diff Array #3	14.76	17.14	215.5	2.479
Manjil, Iran	1990	7.37	Qazvin	84.03	85.54	274.5	1.852
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	2.054
Westmorland	1981	5.9	Niland Fire Station	18.45	18.59	207.5	3.925
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.441
Morgan Hill	1984	6.19	Gilroy Array #3	38.2	39.13	349.9	1.809
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	1.148
Superstition Hills-02	1987	6.54	Plaster City	25.98	27.49	345.4	1.675
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	1.191
Irpinia, Italy-02	1980	6.2	Rionero In Vulture	29.83	30.64	530	1.563
Tabas, Iran	1978	7.35	Dayhook	20.63	21.41	659.6	0.778

Table B.19 Ground motions for $T^*=0.82$ s and 5% probability of exceedance in 50 years (Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
N. Palm Springs	1986	6.06	Indio - Coachella Canal	52.66	53.79	345.4	2.934
Taiwan SMART1(5)	1981	5.9	SMART1 O01	31.99	33.86	274.5	3.034
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	0.614
Coalinga-05	1983	5.77	Coalinga-14th & Elm (Old CHP)	11.21	13.43	338.5	1.807
Duzce, Turkey	1999	7.14	Lamont 375	24.05	27.83	424.8	1.152
Northwest China-02	1997	5.93	Jiashi	23.13	38.68	274.5	1.997
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	2.075
Coalinga-01	1983	6.36	Parkfield - Stone Corral 3E	44.66	44.89	376.1	2.938
Superstition Hills-01	1987	6.22	Wildlife Liquef. Array	24.79	26.73	207.5	1.854

Table B.20 Ground motions for $T^*=0.82$ s and 2% probability of exceedance in 50 years

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Tabas, Iran	1978	7.35	Dayhook	20.63	21.41	659.6	1.326
Mammoth Lakes-01	1980	6.06	Long Valley Dam (Upr L Abut)	12.65	15.52	345.4	1.654
Landers	1992	7.28	Lucerne	44.02	44.58	684.9	1.046
Big Bear-01	1992	6.46	Big Bear Lake - Civic Center	10.15	16.49	338.5	1.953
Superstition Hills-01	1987	6.22	Wildlife Liquef. Array	24.79	26.73	207.5	3.158
Superstition Hills-02	1987	6.54	Calipatria Fire Station	31.62	32.88	205.8	2.989
Coyote Lake	1979	5.74	Gilroy Array #2	10.94	13.55	270.8	2.028
Irpinia, Italy-01	1980	6.9	Brienza	46.16	47.13	500	3.931
Northridge-04	1994	5.93	Moorpark - Fire Sta	17.33	19.92	405.2	2.557
N. Palm Springs	1986	6.06	Cabazon	18.17	21.24	345.4	3.822
Mammoth Lakes-03	1980	5.91	Long Valley Dam (Upr L Abut)	11.51	19.71	345.4	3.499
Hector Mine	1999	7.13	Hector	26.53	30.38	684.9	1.396
Livermore-01	1980	5.8	Del Valle Dam (Toe)	26.79	29.35	338.5	3.292
Chi-Chi, Taiwan-05	1999	6.2	TCU129	40.98	42.19	664.4	3.694

Table B.20 Ground motions for $T^*=0.82$ s and 2% probability of exceedance in 50 years
(Continued)

Earthquake name	Year	Magnitude	Station name	Epicentral distance (km)	Hypocentral distance (km)	Vs30 (m/s)	SF
Northridge-01	1994	6.69	Topanga - Fire Sta	14.19	22.53	376.1	3.316
Westmorland	1981	5.9	Parachute Test Site	20.47	20.6	348.7	1.161
Coalinga-01	1983	6.36	Parkfield - Gold Hill 3W	49.47	49.68	438.3	3.69
Parkfield	1966	6.19	Cholame - Shandon Array #5	32.56	34.06	289.6	2.247
Coalinga-05	1983	5.77	Coalinga-14th & Elm (Old CHP)	11.21	13.43	338.5	3.079
Northwest China-04	1997	5.8	Jiashi	24.99	33.56	274.5	3.378
Victoria, Mexico	1980	6.33	Chihuahua	36.67	38.29	274.5	2.127
Chi-Chi, Taiwan-04	1999	6.2	CHY101	27.97	33.27	258.9	2.901
Morgan Hill	1984	6.19	Gilroy Array #3	38.2	39.13	349.9	3.082
Loma Prieta	1989	6.93	UCSC	16.51	24.05	714	3.256
Duzce, Turkey	1999	7.14	Lamont 375	24.05	27.83	424.8	1.963
Gazli, USSR	1976	6.8	Karakyr	12.82	22.26	659.6	0.605
Chi-Chi, Taiwan	1999	7.62	TCU088	57.63	58.18	553.4	1.921
Chi-Chi, Taiwan-06	1999	6.3	TCU129	33.15	36.81	664.4	2.985
San Fernando	1971	6.61	LA - Hollywood Stor FF	39.49	41.57	316.5	2.143
Mammoth Lakes-06	1980	5.94	Convict Creek	12.03	18.46	338.5	2.661
Irpinia, Italy-02	1980	6.2	Calitri	11.97	13.87	600	1.29
Manjil, Iran	1990	7.37	Qazvin	84.03	85.54	274.5	3.155
New Zealand-02	1987	6.6	Matahina Dam	24.23	25.06	424.8	1.955
Double Springs	1994	5.9	Woodfords	16.89	17.62	345.4	3.984
Chi-Chi, Taiwan-03	1999	6.2	TCU065	32.05	32.99	305.9	1.185
Sierra Madre	1991	5.61	Cogswell Dam - Right Abutment	18.77	22.28	684.9	3.534
Kocaeli, Turkey	1999	7.51	Atakoy	99.69	100.97	274.5	2.424
Cape Mendocino	1992	7.01	Cape Mendocino	10.36	14.06	513.7	0.752
Imperial Valley-06	1979	6.53	El Centro Array #11	29.44	31.08	196.3	1.363
Hollister-04	1986	5.45	Hollister Diff Array #1	14.76	17.14	215.5	3.121

Appendix C

Appendix C Polar plots of ground motions

The polar plots of the 4 groups of 40 GMs selected for each buildings, each corresponding to a different probability of exceedance, are shown in the following figures. All GMs are aligned with their as-recorded directions. The dashed lines indicate the ASIs used for the analysis, while the black circle corresponds to the anchoring level of $S_a(T^*)$. It can be observed that the majority of the GMs do not present significant directionality effects, whereas only a few cases (no more than 2 in a given group) can be seen to be polarized. Nevertheless, the $S_a(T^*)$ appears to be well distributed for all the ASIs used to perform the analysis, supporting assumptions regarding far-field effects and randomness with respect to the ASI.

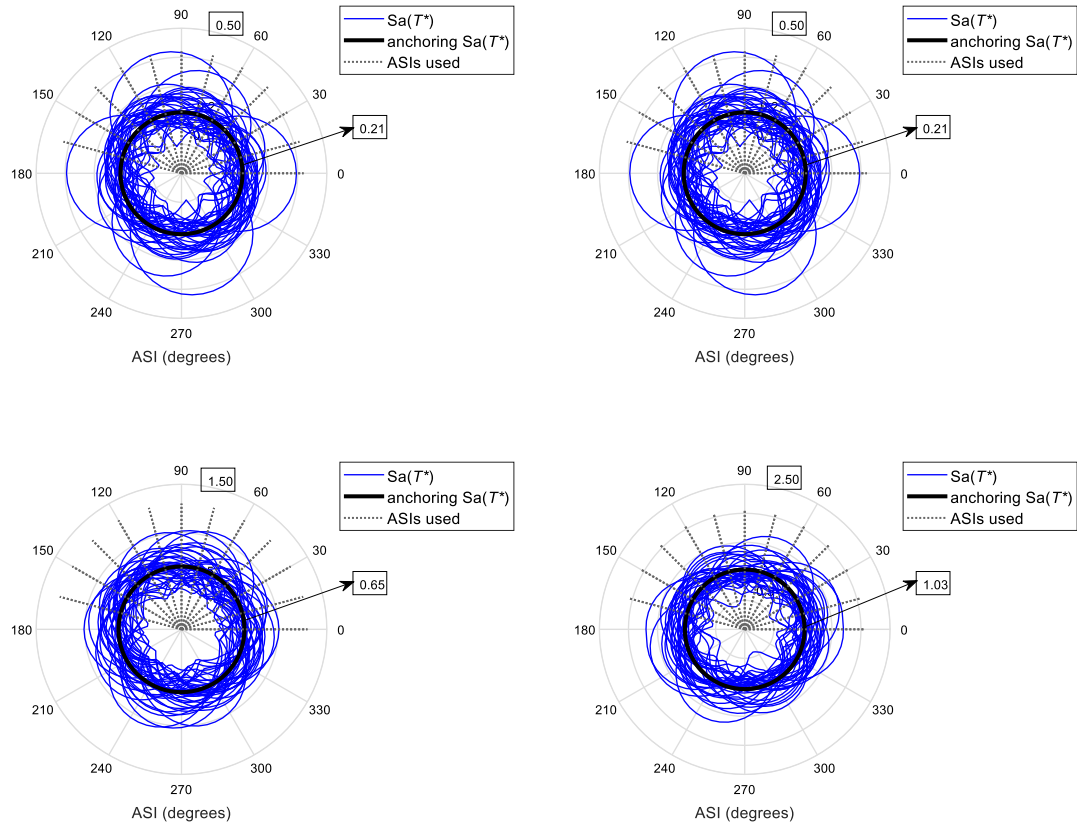


Figure C.1. Polar plots for the groups of 40 GMs selected for $T^*=0.27s$ and for the 30%, 10%, 5% and 2% in 50 years probability of exceedance. Spectral accelerations are in g and values in boxes correspond to intensities along the radial lines.

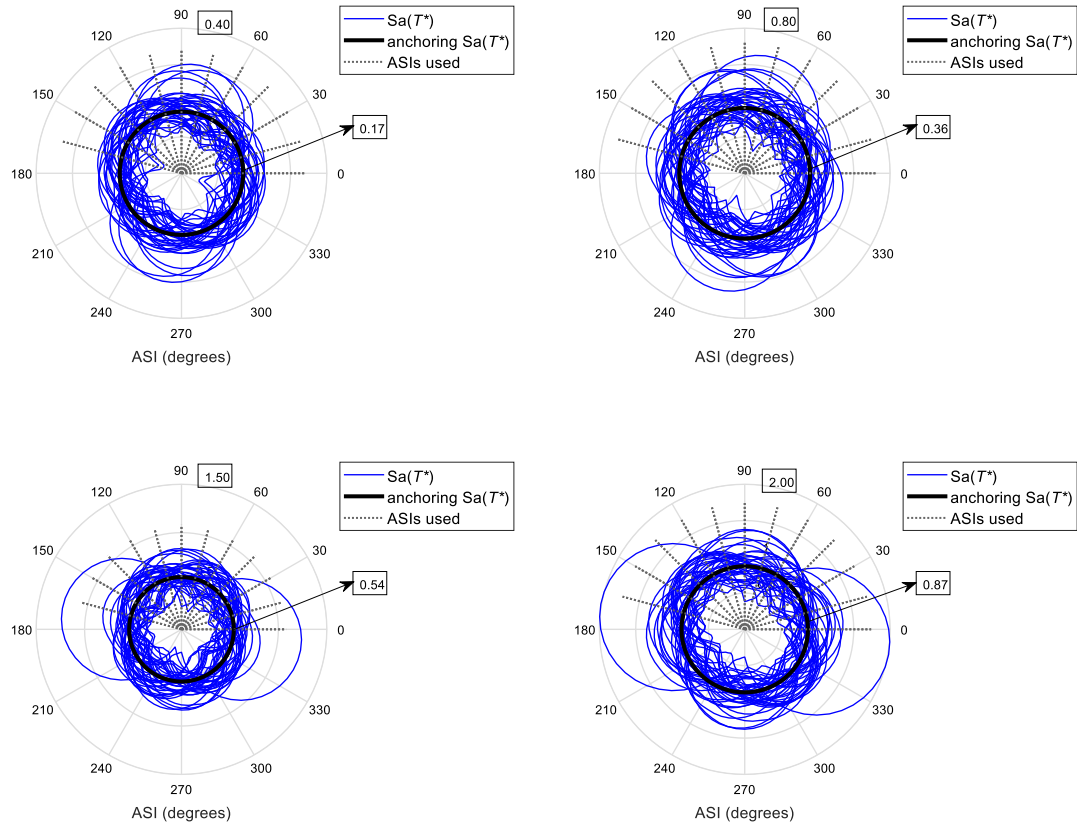


Figure C.2. Polar plots for the groups of 40 GMs selected for $T^*=0.38$ s and for the 30%, 10%, 5% and 2% in 50 years probability of exceedance. Spectral accelerations are in g and values in boxes correspond to intensities along the radial lines.

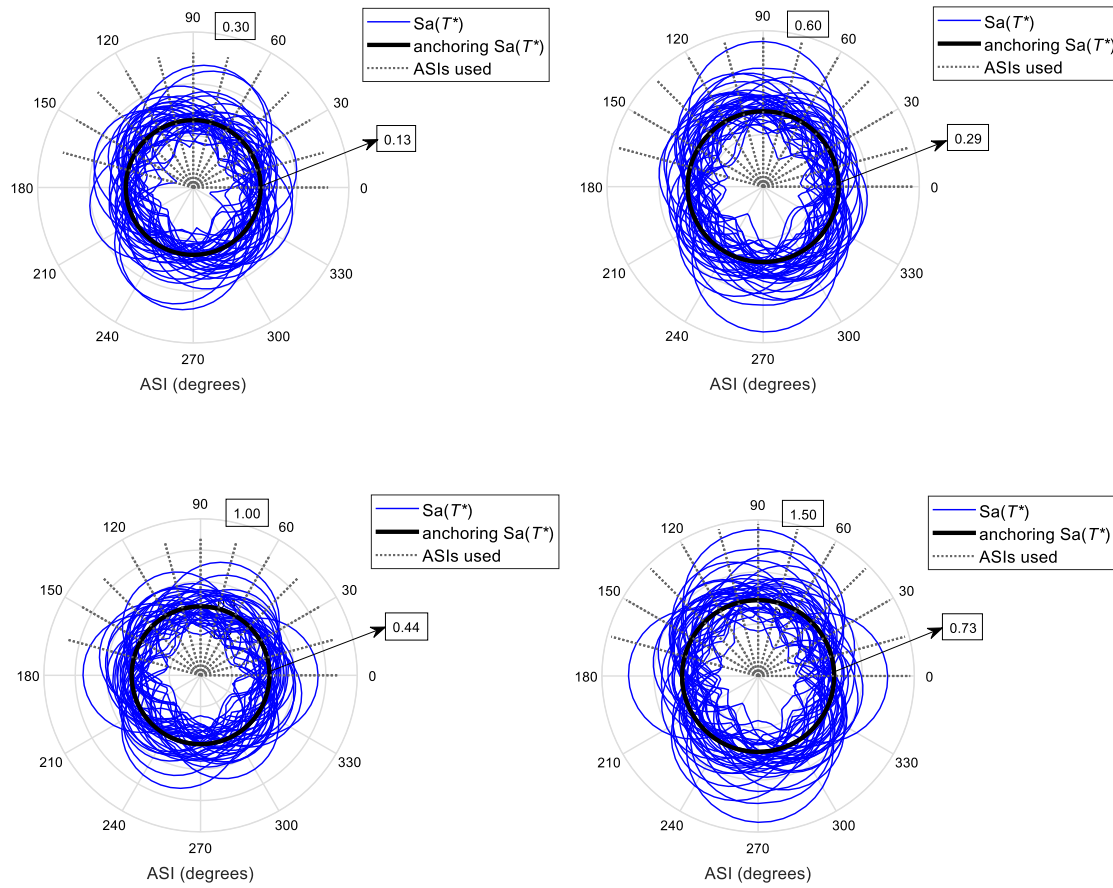


Figure C.3. Polar plots for the groups of 40 GMs selected for $T^*=0.5\text{s}$ and for the 30%, 10%, 5% and 2% in 50 years probability of exceedance. Spectral accelerations are in g and values in boxes correspond to intensities along the radial lines.

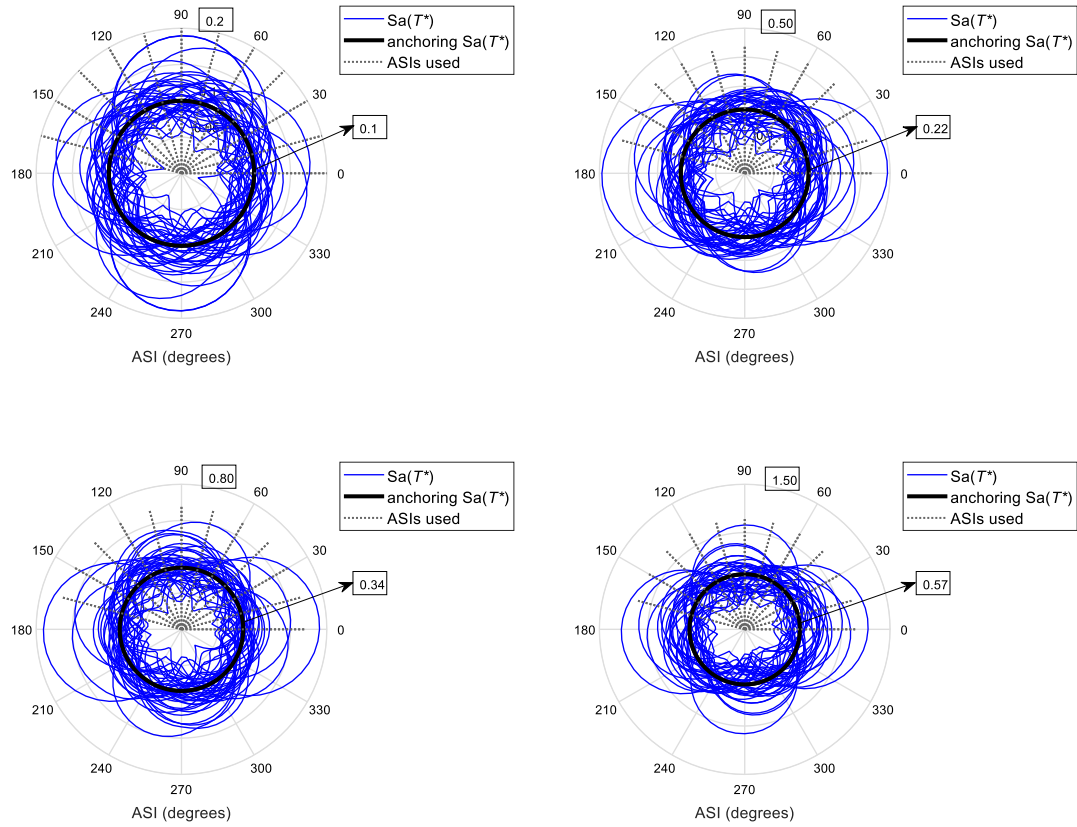


Figure C.4. Polar plots for the groups of 40 GMs selected for $T^* = 0.66$ s and for the 30%, 10%, 5% and 2% in 50 years probability of exceedance. Spectral accelerations are in g and values in boxes correspond to intensities along the radial lines.

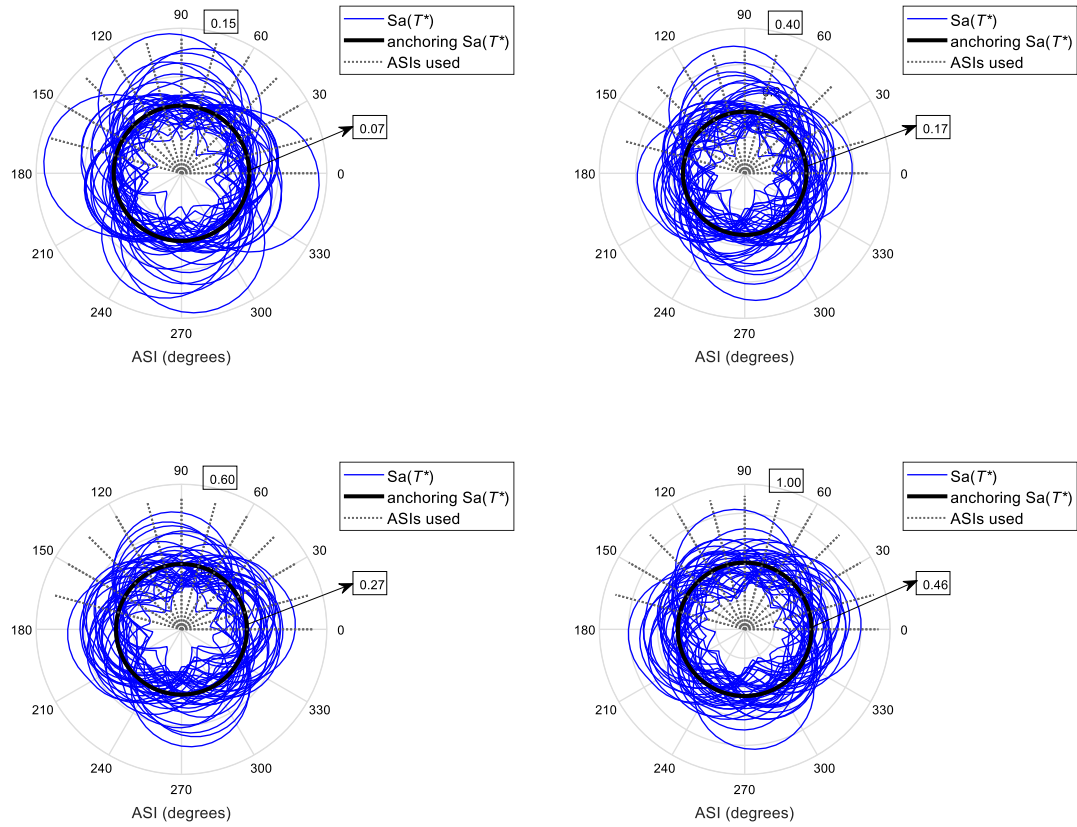


Figure C.5. Polar plots for the groups of 40 GMs selected for $T^*=0.82$ s and for the 30%, 10%, 5% and 2% in 50 years probability of exceedance. Spectral accelerations are in g and values in boxes correspond to intensities along the radial lines

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