GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERING

THE 1755 LISBON EARTHQUAKE: REVISITED

LUIZ A. MENDES-VICTOR
CARLOS SOUSA OLIVEIRA
JOÃO AZEVEDO
ANTÓNIO RIBEIRO
EDITORS

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250th Anniversary of the 1755 Lisbon Earthquake

Editorial Note

The 1755 earthquake and tsunami were influential not only in Portugal but in all European and North African countries, where its devastating effects were felt. The entire world was deeply impressed and the discussion of its causes generated a large amount of scientific and metaphysical speculation. It inspired philosophers, poets and writers. The socio-economic consequences of the event were great and affected the future organization and development of Portugal. The eventuality of a similar occurrence urges society and the scientific community to reflect on its lessons.

250 years after the 1755 earthquake, the opportunity to put together scientists, engineers, historians, philosophers, urban planers, architects, economists and policy makers, provides an integrated view on our global perception of natural disasters and how must Society deal with them.

In order to invocate this event an international conference - "250th Anniversary of the Lisbon Earthquake International Conference" – was organized in Lisbon from 1 to 4 November 2005, devoted to the following topics:

1) Socio-economic impact on communities exposed to earthquakes and tsunamis;
2) Urban Planning facing natural Hazards; information and warning;
3) Propagation and local effects on the seismic destruction;
4) How to build earthquake resistant buildings under the environmental constrains;
5) New approaches to the seismogenesis of the 1755 earthquake;
6) Global response to large earthquakes.

A large number of contributions was received covering the above mentioned topics, organized in a Volume of Proceedings, a number of which with high scientific quality and social interest deserving a more wide diffusion through a well known publisher, impacting an extensive audience.

In this publication we are very pleased to offer the opportunity to interested readers and users to dispose of a collection of high quality papers dealing with the different aspects of geosciences, engineering and humanities related to this kind of catastrophic event.

Authors were invited to submit selected improved versions of the original Proceeding papers, namely the invited lecturers, and the convenors of the Conference topics.

This Book should be considered as a quality reference in Institutional Specialized Libraries and Bookstores. It is a tribute from the academic society to acknowledge the contribution of Prof. Bruce Bolt to mitigate earthquake impacts. Prof. Bruce Bolt was an enthusiastic supporter of the Organising Committee to present the state-of-the-art and the importance of the scientific and technical background already achieved in this field of human kind. We are very proud to include his name as an expression of gratitude for his contribution to the advancement of science and public awareness.

The editors would like to express to all the contributors to the "250th Anniversary of the Lisbon Earthquake International Conference" which, in one way or another, have supported the idea of this evocation and made the Conference and the corresponding Proceedings a success. The high scientific level presentations in the Conference were the seeds of this Book.

We would like to highlight all supporters, institutions and entities involved in the "International Conference" as well as the devoted work performed by Dr. Alexandra Carvalho for the arrangements and contacts established with the authors of this Book.

Lisbon, January 2008

L.A. Mendes-Victor
Carlos Sousa Oliveira
João Azevedo
António Ribeiro

Part I
Introduction

Introduction

L.A. Mendes-Victor and Carlos S. Oliveira

The success of the "250th ANNIVERSARY of the LISBON EARTHQUAKE" motivates the challenge to produce a collection of papers, which make a "revisitation" of the 1755 Lisbon earthquake.

In order to achieve the objectives of wider outreach of all scientific and technical communities interested in this matter, the Editors were able to provide a selection of papers reflecting the most expressive contributions to assure those objectives, adopting similar distribution of the Conference topics, requesting from the convenors the designation and approval of the ideas.

We realize the importance of the work developed by Prof. Bruce Bolt to the efforts to join the communities of Seismology and Earth Engineering and to embrace the policy of seismic mitigation, being a strong personality launching this idea. We want to give our recognition to Prof. Bruce Bolt, an enthusiastic supporter of the International Conference whose surprise did not allow him to be present. His colleagues at the University of Berkeley wrote a short attribution to Prof. Bruce Bolt.

The historical framework constitutes a very expressive topic reported by a scientific Group of Personalities providing review of the most important aspects of the historical interpretations of the sources, coeval reports, impact and consequences of the event.

The social-economical impact of large event such as the 1755 Lisbon earthquake was analysed from different points of view but expressing modern concepts and assessment evaluations, involving the mitigation of seismic risks.

Urban planning facing natural hazards were another topic selected for presentation by several experts with illustrations in different European environments.

Propagation and local efforts was brought into discussion being new models to explain instrumental and macroscopic observations.

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The modern earthquake engineering concepts were treated in another topic with relevance to analytical and experimental modelling and comporting with the reality of damage observatory in past events.

A topic of still great controversy is the seismogenesis of the 1755 earthquake. New approach to the problem is presented representing the most updated development achieved in the last decade.

A final topic, global Response to large earthquake was a challenge for different experts to present their own views in different geodynamic environments, including several regions and countries with high potential seismic and tsunami risk.

We have to emphasize the excellent of all presentations providing some important guidelines as far as earthquake mitigation, prevention and management of risk are concerned.

Part II
Historical Framework

The Lisbon Earthquake of 1755 in Spanish Contemporary Authors

Agustín Udías and Alfonso López Arroyo

1 Introduction

The Lisbon earthquake of 1 November 1755 was felt over the whole Iberian Peninsula causing heavy damage by the shaking and subsequent tsunami, specially, in the nearby Spanish cities of Huelva, Cadiz and Seville (Martínez Solares 2001; Martínez Solares and López Arroyo 2004). This extraordinary event produced an abundant literature published in Spain, especially in Seville. Many were short popular accounts about how the earthquake was felt in some determined localities or religious considerations about the event, most of them anonymous. These were generally short works of a few pages of a popular character with exaggerated narratives of damages or curious occurrences during the earthquake. Some were of religious character asking or giving thanks for the deliverance from the effects of the earthquake; a few of them were written in verse. Other publications were extended treatises on the physical, philosophical and religious aspects of the event, written by ecclesiastics, philosophers and scientists. A list of the publications we have identified and examined is given in Appendix 1 (publications with author) and in Appendix 2 (anonymous works).

Most authors writing about the earthquake that we will examine in some detail handled two questions. The first was whether this was a natural event or a supernatural one, that is, one directly attributed to God. The second was about the natural causes of this earthquake and, in general, about the origin of earthquakes. In this second question traditional and modern ideas were presented and debated. Regarding the characteristics of this particular earthquake it was discussed, especially, how it was possible that the earthquake, which caused the main destruction in Lisbon, was felt at the same time in regions separated by long distances through the Iberian Peninsula and as far as central Europe and how it generated such large waves in the ocean.

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2 Natural or Supernatural Event

The occurrence of the Lisbon earthquake generated in Europe an intense debate about what has been called “eighteenth century earthquake-theology” (Kendrick 1955). In the center of this debate was the opinion, generally asserted by many of the clergy, that the earthquake was a deliberate punishment by God of sinful people. A constant theme in sermons, tracts and moralizing poetry, throughout Europe was that God in His anger had destroyed Lisbon because of the sins of its inhabitants. In Portugal the debate took a special strong character with the figure of the Jesuit Gabriel Malagrida on one side and Sebastian José de Carvalho e Mello, Marquis of Pombal on the other. Malagrida with an extreme position insisted in his sermons that the earthquake had been caused by the wrath of God for the sins of the people of Lisbon. Pombal, who took a pragmatic attitude to organize the care of the victims and the reconstruction of the city, regretted the sermons of the clergy that in his opinion motivated certain passivity in the people. Pombal ordered to put Malagrida in prison and four years later his cruel execution by the Portuguese Inquisition.

In France the earthquake came to question the generally sensed optimism, which held that the world was a good place in which everything that happened was viewed to be “for the best” (Kendrick 1955). François Marie Voltaire, in his poem, *Poème sur la désastre de Lisbonne* and his novel *Candide,* wrote the most furious and hard attacks on this optimistic view. On the other side authors like Jean Jacques Rousseau defended the optimist position, and rejected Voltaire gloomy picture of man’s unhappy fate on earth. In Germany Immanuel Kant, adhering to the optimistic theodicy of Gottfried Wilhem Leibniz, who held that this is “the best of the worlds”, published three short papers on the Lisbon earthquake in 1756. He was in fact more interested in the scientific aspects of the phenomenon, but touched also on the subject of earthquakes in relation to God’s government of the world. The optimist position was heavily wounded by Voltaire’s sharp attacks in *Candide*, which finally carried the day in the enlightened Europe.

In Spain the debate was centered about the supernatural or natural character of the earthquake. It is generally accepted that before the Enlightenment common knowledge assigned the cause of earthquakes to God’s punishment of sins, but this is an oversimplification. This was true in popular or religious accounts, but not in philosophical treatises, where Aristotelian ideas were held about the natural causes of earthquakes. Commentaries by Spanish authors of the 16th to 18th century on Aristotle’s *Meteorologica*, where the problem about the origin of earthquakes is treated, do not mention God’s intervention in these phenomena. On the other hand in the popular and religious writings the situation was different. For example, an earthquake, which caused heavy damage in the city of Malaga in 1680, was interpreted as God’s punishment for sins with no dissenting voices. In the earthquake of 1755 opinions were on both sides arguing in favor and against considering the earthquake as God’s punishment.

The references of the published works and manuscripts of the authors who are mentioned in the following paragraphs are given in Appendix 1.

The debate began a few days after the occurrence of the earthquake, with many popular publications many of them anonymous and sermons in the churches, in which the supernatural character was presented (Fig. 1). Some asked for the help of heavenly patrons in this situation or thanked for their protection, among them of the Virgin Mary, St. Francis of Borgia, St. Philip of Neri, St. Justa and St. Rufina. Many of the titles of the anonymous publications given in Appendix 2 correspond to this kind of texts and most of them were published in Seville. At popular level and by many of the clergy it was taken for granted that the earthquake was God's punishment for the sins, and public religious services were organized in the subsequent days for this reason (Aguilar Piñal 1973). The two sermons of Francisco Olazaval y Olayzola, Canon of the cathedral of Seville, preached on 27 of April of 1755 and 28 February 1756, were an example of this type of literature. Olazaval insisted in the many sins of the city of Seville, as the cause of this punishment, which the mercy of God had not permitted to be even greater. Agustín Sanchez, a Trinitarian theologian and preacher, insisted in a note included in Nifo's work, "God uses the creatures to infuse fear in sinners and to move them to repentance". The most firm defender of the supernatural character was Miguel de San José, Bishop of Guadix and Baza (Granada), who published a short letter in which he refuted the opinions of those who defended that this was a natural event, specially José de Cevallos, and went as far as to affirm that: "only to deny or doubt that earthquakes and other disasters are usually the effect of the wrath of God, can be considered as an error in the faith".

José de Cevallos (1726–1776), a theologian from Seville and later Rector of the University of Seville presented the contrary opinion, defending the purely natural character of the earthquake. He expressed his position in his introductory note (*Censura*) included in Feijoo's work published in 1756. He used arguments from the Scriptures and the Church fathers against considering earthquakes as signs of God's wrath and concluded firmly "the earthquake has been entirely natural, caused by natural and proportioned second causes". He recommended preachers not to be carried by devotion in their sermons and be guided by wisdom and discretion. Juan Luis Roche, a physician born in Catalonia and established in Seville, defended the same opinion, adding that there is no relation between the sins committed and the occurrence of earthquakes (Fig. 2). Rhetorically he asked: "Are Lisbon and Seville worse than other cities?" For him this type of considerations was only "pious opinions of theologians".

The natural character of the earthquake was also defended and discussed in several lectures held at the *Real Academia de Buenas Letras*, a learned society of Seville where enlightened ideas were discussed. Roche held the first lecture about the earthquake ten days after its occurrence (*Sobre el terremoto del 1 de Noviembre,* 12 November 1755). Jerónimo Audixe de la Fuente (*Formación y efectos de los terremotos,* 27 March 1756) and Francisco de Céspedes Espinosa

EL DESENGANO

A LA PRESUMPTUOSA IGNORANCIA,
Que intenta perſuadir efecto de los Elementos
los eſtragos del Terremoto , diſtrayendo la
compuncion de los Timoratos.

CANTO TRAGICO.

POR
D.FRANCISCO IGNACIO DE LA CRUZ.

CON PERMISSO EN MADRID.

En la Imprenta de los Herederos de Don Aguſtin
de Gordejuela , Calle del Carmen.

Fig. 1 Poem by Francisco de la Cruz presenting the earthquake as God's punishment for sins

CARTA

SEXTA DE DON JUAN LUIS ROCHE al Señor Don Franciſco de Buendia, y Ponce, Presbytero Theologo, Socio Medico de Numero, primer Secretario que fuè, y actual Conciliario primero de la Real Sociedad de Ciencias de Sevilla, Academico Numerario de la Real de Buenas Letras, Honorario de la Real Academia Portopolitana, &c.

1. MUY SEñOR MIO: RECIBI LA de V.md. de 29. del que acaba, y aunque yo no recibieſſe otro premio de las mayores tarèas Literarias, que los encomios, con que V.md. en ella me honra, quedaria muy ſatiſfecha mi fatiga. Si yo no conociera à V.md. ò V.md. no fueſſe quien es, bien podria ſoſpechar deſmedidos hyperboles, pero ſiendo V.md. el que ſaben todos, ſolamente puedo creer un exceſſo de bondad, y de afecto. Y aunque podia por lo miſmo deſvanecerme, le confieſſo à V.md. que ſolo me aquieta el animo de aquella ſuſpenſion, que ocaſionan los partos del ingenio, haſta vèr, como ſon baptizados de las gentes: porque un voto tal, aunque fueſſe ſolo, peſa muchos quintales.

El

Fig. 2 Letter by Juan Luis Roche defending the natural origin of the earthquake

(*Relación histórica del terremoto de 1755,* 17 September 1756) made presentations on the following year. These lectures discussed the occurrence of the earthquake from a purely secular perspective outside any religious consideration. Although members of the clergy took part in these sessions, the Academia stood away from formal theological discussions (Sánchez Blanco 1999).

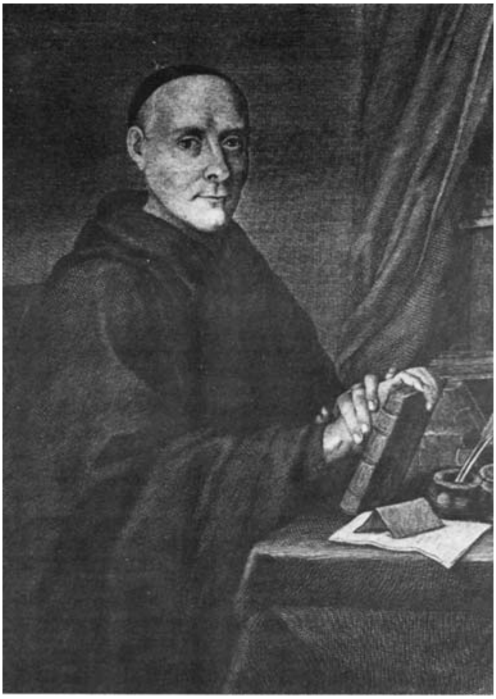


Fig. 3 Benito Jerónimo Feijoo y Montenegro (1676–1764)

Both Cevallos and Roche supported their opinions with the authority of Benito Feijoo y Montenegro (1676–1764), a Benedictine professor of theology at the University of Oviedo, author of *Teatro crítico universal* and *Cartas eruditas,* two very influential works in the introduction of modern scientific ideas in Spain (Fig. 3). Feijoo defended the natural character of the earthquake, but already an old man, did not enter the controversy. He wrote that man should fear more sudden deaths than earthquakes, since they are more common. Another defender of the natural character was Antonio Jacobo del Barco y Gasca (1716–1783), an ecclesiastic and historian of Huelva, whose main work was dedicated to the history and agriculture of the region. Barco in his writing said that he intended to study "as a philosopher" the causes, duration, extension and effects of the earthquake. Defending the natural character he added that natural does not mean "casual", and that this type of occurrence must be used as an occasion for men to turn to God. Isidoro Ortiz Gallardo de Villarroel, Professor of Mathematics of the University of Salamanca, explained the natural causes and did not want to enter into the theological question of whether it was a warning from the divine Providence.

Some authors held the mixed position that the earthquake was a natural event, but God has used it to punish or warn sinners. Miguel Cabrera, of the Order of Minims, a theologian of Seville, defended that the earthquake was "natural in its causes, in its being and in its consequences", but, however, a special Providence could have ordered it to happen at a particular place and time. Francisco de Buendía y Ponce (1721–1800), an ecclesiastic from Seville, poet and physician of the Archbishop of Seville, author of works on history and medicine, held the same opinion. He stated that earthquakes, although of natural causes, could be sometimes a "punishment by the Divine Hand". Francisco Martinez Moles, professor at the University of Alcalá de Henares who argued that earthquakes could be signs of divine wrath took a similar position. He wrote that "if this natural phenomenon was caused naturally, can be investigated". However, he continued, there are motives to say that Providence have ordered the earthquake as punishment for sins.

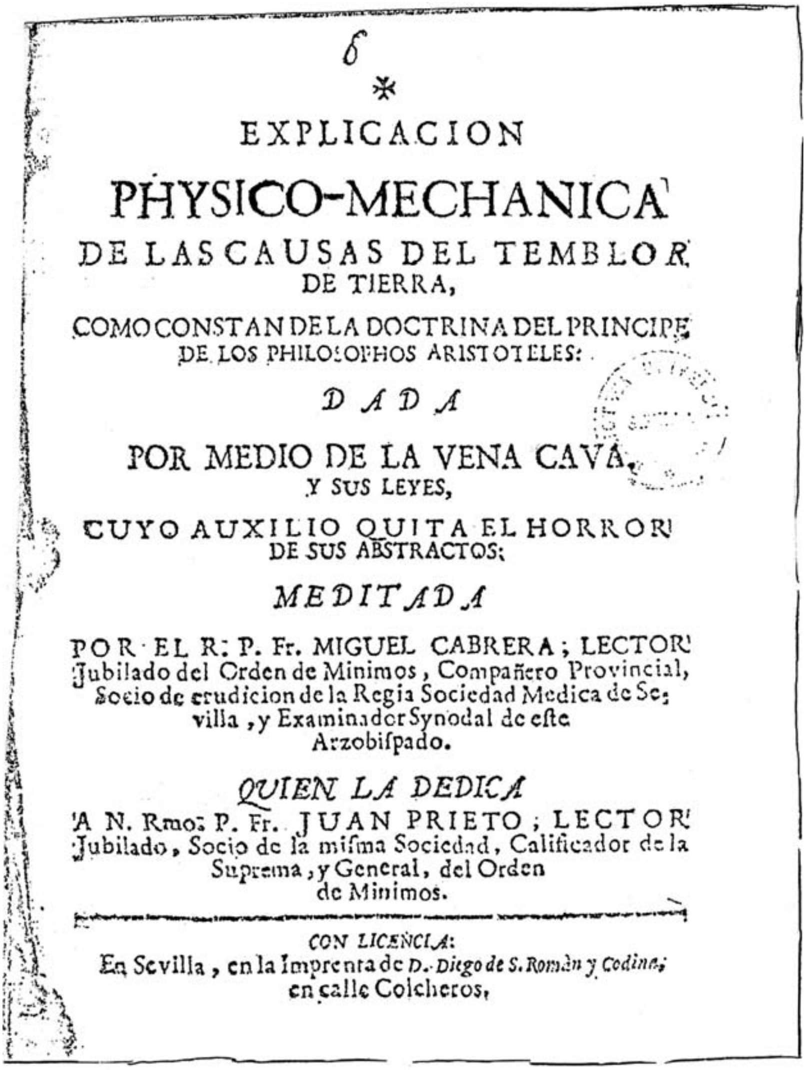
Francisco Mariano Nifo y Cagigal (1719–1803), founder of the first newspaper in Madrid, held a similar view. After explaining the natural causes of earthquakes, he added the consideration of what can be considered their moral causes and effects, as God can use these phenomena as warnings to sinners for their repentance. Juan de Zúñiga in a letter to Feijoo explained the natural causes of earthquakes and how God uses natural causes to show its displeasure of man sins. Pedro Trebnal, a member of the learned societies of Seville, after giving the details of this debate in his manuscript, gave a twist to the problem and, rejecting the supernatural character, defended that although it was a natural event it was not entirely so, but it had a preternatural character, that is, some evil spirit may have produced the earthquake.

In conclusion, in Spain there were defenders of both opinions about the natural or supernatural character of the earthquake. Authors holding the modern ideas of the enlightenment (*ilustrados*) defended that the earthquake was a natural event and one should not see in it a punishment from God, while traditional clergymen considered the earthquake as a punishment or warning of God to sinners. Even as late as 1784 a Dominican friar, Alvarado, showing his rejection of modern ideas, wrote that: "we prefer to be mistaken with St. Basil and St. Augustine than to be correct with Descartes and Newton" (Aguilar Piñal 1973). An intermediate position was also presented in which the earthquake was thought to be a natural phenomenon, but God's providence used it to warn sinners to repentance. There was not, however, any reference to the debate in Europe about the optimistic or pessimistic views of the world. Spanish authors never mentioned Voltaire, Leibniz, Kant or any other of the participant in this debate.

3 The Natural Causes of Earthquakes

The authors who held that the earthquake was a natural phenomenon took this occasion to explain the general causes of earthquakes. In their explanation we can see to what extent they knew about the modern scientific ideas being developed at that time in Europe. In the end of the 17th century and beginning of the 18th century new theories about the origin of earthquakes were proposed which substituted the traditional views based on the doctrine proposed by Aristotle. According to him the causes of earthquakes are the dried exhalations (spirits or winds) contained in cavities inside the earth which trying to escape shake the earth. The criticism to Aristotelian ideas on other subjects by the proponent of modern science extended also to the production of earthquakes. Martin Lister in England in 1648 and Nicolas Lemery in France about 1700 proposed that large explosions of inflammable material formed by a combination of sulfur, carbon, iron pyrites and other products accumulated in the earth interior produced earthquakes (Taylor 1975). These authors compared earthquakes with the explosions in mines. This theory became soon very popular and can be found in Newton's *Optics* (1718) and Buffon's *Histoire naturelle* (1749–1788).

In the writings of Spanish authors we find a variety of theories proposed, from the traditional Aristotelian doctrine to the ideas introduced by modern authors. Cabrera presented the most traditional point of view and defended the Aristotelian doctrine against the attacks of modern authors, especially Descartes, whom he called "the chief of this new sect," and their followers (called in Spain *eruditos* or *novatores*) and extended his criticisms of modern authors to Newton's gravitational theory (Fig. 4). He introduced organicistic ideas in which the earth is compared with a living organism and departed from the strict Aristotelian explanation. In these ideas he showed the influence of the *Mundus Subterraneus* (1664) by Atanasius Kircher, Jesuit professor of mathematics at the Collegio Romano, whose ideas were at that time very popular in Spain (Glick 1971; Capel 1980; Sierra 1986). Kircher had proposed the existence in the interior of the earth of three systems of conducts through which fire, water and air circulate, named by him *pyrophylacia*, *hydrophylacia* and *aerophylacia*. He proposed that the first are related to the volcanoes and connected them with a fire in the center of the earth. For Kircher the cause of earthquakes is the underground fire of the *pyrophylacia*, which heats the air in the cavities of the earth expelling it with shaking forces. Cabrera proposed a somewhat different system of conducts consisting in a large cavity in the interior of the earth, following its axis with many ramifications, in which water and winds circulate. He called this cavity *vena cava,* in analogy with the main vein in the human body. In fact Cabrera thought that this cavity and its ramifications function in the earth as the veins in animals and men. He supposed that all phenomena in the earth (winds, fountains, earthquakes, etc.) could be explained in terms of this system, in analogy with the function of veins in living organisms. Thus he stated that earthquakes could be considered as "sicknesses of the earth". In the Lisbon earthquake, the shaking propagated through the ramifications of the "*vena cava*" explained how it was felt as far away as Germany. He stated also that the influence of the Sun, Moon and stars affects the occurrence of earthquakes in the same way as it affects living organisms. He refuted nominally the opinions of Nifo and López de Amezua and their criticism of Aristotle, and Feijoo's electrical theory.



6

EXPLICACION
PHYSICO-MECHANICA
DE LAS CAUSAS DEL TEMBLOR
DE TIERRA,
COMO CONSTAN DE LA DOCTRINA DEL PRINCIPE
DE LOS PHILOSOPHOS ARISTOTELES:
DADA
POR MEDIO DE LA VENA CAVA,
Y SUS LEYES,
CUYO AUXILIO QUITA EL HORROR
DE SUS ABSTRACTOS:
MEDITADA
POR EL R. P. Fr. MIGUEL CABRERA; LECTOR
Jubilado del Orden de Minimos, Compañero Provincial,
Socio de erudicion de la Regia Sociedad Medica de Se-
villa, y Examinador Synodal de este
Arzobispado.
QUIEN LA DEDICA
A N. Rmo. P. Fr. JUAN PRIETO, LECTOR
Jubilado, Socio de la misma Sociedad, Calificador de la
Suprema, y General, del Orden
de Minimos.
CON LICENCIA:
En Sevilla, en la Imprenta de D. Diego de S. Román y Codina,
en calle Colcheros.

Fig. 4 Miguel Cabrera's explanation of the origin of earthquakes

Nifo followed with certain criticisms the traditional Aristotelian doctrine, and explained that the cause of earthquakes is dry exhalations produced by winds, that penetrate through cavities inside the earth. For this reason, earthquakes are more frequent in spring and fall, when winds are stronger. In a like manner, Trebnal followed in part the Aristotelian doctrine. He showed his knowledge of the modern theories, making references to the French Antoine Pluche and the ideas presented by Roche and Feijoo, but found insufficient their explanations. He proposed that the main cause of earthquakes is the mechanism of the elastic force of condensed air trapped in cavities inside the earth, without the need of the action of fire. Trebnal proposed that the force of the sea introduces the air in these cavities and when the air is sufficiently condensed it releases its elastic energy shaking the ground. He quoted modern authors like Robert Boyle and Nicolas Lemery in this context.

Some authors adhered to the more modern theory of the explosive nature of earthquakes. Barco who wrote in Huelva, where the earthquake was very strongly felt, compared the origin of the earthquake with the explosion in a mine. For him the accumulation of inflammable materials like sulfur, nitrate and bitumen inside the earth caused these explosions by the contact with rarified air and fire. Barco examined the problem about where the shaking began, and located "focus" of the earthquake under the ocean nearer to the African coast than to that of Portugal. He assigned the origin of the tsunami to the motion of the ocean bottom and explained the occurrence of aftershocks as produced by the inflammable materials, which had not been exploded in the first shocks. Roche, following modern theories and quoting Lemery, also assigned the cause of earthquakes to the explosion of the mixture of inflammable materials accumulated at certain places inside the earth. Ortiz Gallardo combined the explosive nature with Kircher's theory of the existence inside the earth of conducts of fire, water and air. The fire in these conducts, which gets in contact with the accumulated inflammable material, is the true cause of its explosion. He stated that in the 1755 earthquake, the frequent rains and snow in the winter of the previous year, the moderate summer and, again, many rains in the fall, produced an accumulation of a mixture of water with inflammable materials, which favored its explosion.

Feijoo presented the most modern ideas about the origin of earthquakes in his five letters (Glendining 1966; Ordaz 1983). After refuting in the third letter the common ideas held at the time for the cause of earthquakes, especially the collapse of underground cavities and the explosion of inflammable material inside the earth, he presented in his fourth letter his new theory in which electric charges produced earthquakes. He stated that, in the same way that lightning and thunder are produced in the atmosphere by the electricity of the clouds, earthquakes are caused by the electricity accumulated inside the earth by the action of vitreous materials. William Stuckley in England in 1750 and Giovanni Battista Beccaria in Italy in 1753 had proposed already these ideas about the electrical nature of earthquakes (Taylor 1975). Feijoo didn't mention these authors and claimed for himself the originality of this theory. For him the electrical theory explained well, in the case of the Lisbon earthquake, that it was felt at the same time in so distant places up to central Europe, since electricity propagates at very high velocity. However, he didn't rule out completely the explosive nature, as electricity could have also caused the explosion of concentrations of inflammable materials accumulated in some places inside the earth.

4 The Origin of the Tsunami

The large tsunami produced by the Lisbon earthquake affected in Spain especially the coastal towns of Huelva and Cadiz (Baptista et al. 1998; Martínez Solares and López Arroyo 2004). As a matter of fact, according the last two authors, the greatest part of the estimated 2000 victims, produced by the earthquake in Spain, was due to the tsunami, which was known as the "*maremoto de Cadiz*". Some of the authors, we have mentioned before, tried also to explain the cause of this phenomenon. Barco, writing from Huelva were waves reached several meters, assigned the motion of the sea to the fact that the earthquake had its origin under the bottom of the sea. He speculated that the earthquake raised the bottom of the sea several times causing the water to flow into the coast. Roche described with detail the large waves, which flooded the town of Puerto de Santa María, but he didn't give an explanation of this effect. He took it to be a normal effect of an earthquake and even mentioned that after feeling the shock he expected the arrival of the ocean waves.

Trebnal put explicitly the question if the tsunami had a different cause as the earthquake. He denied that the cause of the motion of the sea were the tides by the influence of the Moon and assigned it to the same cause as that of the shock, that is, to the air through the earth cavities which caused, in this case, changes in the level of the sea bottom and pushing the water waves toward the coast.

5 Conclusions

The Lisbon earthquake of 1755, which was strongly felt in many Spanish cities, especially, Huelva, Cadiz and Seville, generated a large number of writings and a debate about the natural or supernatural character of this event, that is, whether it was God's punishment of sinners. Authors took positions at both sides of the controversy; Cevallos, Barco, Roche, and Feijoo defended the natural origin of the earthquake and denied that there was any connection between the earthquake and the sinful behaviour of the citizens of Lisbon or of other cities. On this occasion authors tried also to explain the different theories about the origin of earthquakes. The theories proposed ranged from the strict Aristotelian doctrine, organiscistic ideas and the more modern explosive and electric theories. Spanish authors showed that they were familiar with modern ideas, though some of them still kept to traditional viewpoints.

Appendix 1: Publications About the Lisbon Earthquake of 1 November 1755 by Contemporary Spanish Authors

Audixe de la Fuente, Jerónimo, Discurso meteorológico sobre el origen de los terremotos. (Ms. Archivo Academia Sevillana de Buenas Letras, II, Disertaciones, folios 20–41), 1755.

Barco, Antonio Jacobo del. Cartas del Doctor . . .., Catedrático de Philosophia y Viario de la villa de Huelva, a Don N. satisfaciendo algunas preguntas curiosas sobre el terremoto de

primeros de Noviembre de 1755. In Juan Enrique Graef (ed), Discursos Mercuriales, XIV (21-4-1756): 565-606.
Barreda, Iñigo de. Causa del terremoto. Sermón histórico moral. Burgos, 1756
Benegasi y Luxán, Joseph. Descripción del terremoto, según se experimentó en la villa de Herencia el día 1 de Noviembre de este año de 1755, à las diez del día, compuesto a impulsos del desengaño, para mayor escarmiento por ... en endechas endecasílabas. Madrid: Joseph de Orga, 1755.
Buendía y Ponce, Francisco de, Censura de Don.... (Incluido en Feijóo).
Cabrera, Miguel. Explicación physico-mechánica de las causas del temblor de tierra, como constan de la doctrina del príncipe de los filósofos, Aristóteles, dada por medio de la vena cava y sus leyes, cuyo auxilio quita el horror de sus abstractos. Sevilla: Diego de S. Román y Codina, 1756.
Cabrera, Miguel. Copia de carta en que se manifiesta que la electricidad ya natural ya maquinaria no puede servir de fundamento para explicar la divergencia de los terremotos como persuade en su quarta carta el Ilmo y Rmo. Mro. Fr. Benito Feijoo, escribiola a un correspondiente de la Ciudad y Gran Puerto de Santa María, con las respuestas a las dudas de un prólogo que forma D. Luis Roche contra el systema de la vena cava, su autor.... Sevilla: Joseph Padrino, 1756.
Cevallos, José de. Censura de Don ... (Incluido en Feijóo).
Cevallos, José de. Respuesta a la carta del Ilmo. y Rmo. Señor D. Fr. Miguel de San Josef Obispo de Guadix y Baza sobre escritos acerca del terremoto por el Doctor Joseph de Cevallos. Sevilla: Joseph Navarro y Armijo, 1757.
Chacón, Ignacio. Las gloriosas santas tutelares de Sevilla, Juta y Rufina, triunfantes de la impureza y de los vicios sus confederados, en el terremoto experimentado el sabado, día primero de Noviembre año 1755. Oración panegírico-moral ... Sevilla: Doctor Gerónimo de Castilla, 1756.
Cruz, Francisco Ignacio de la. El desengaño a la presuntuosa ignorancia que intenta persuadir efecto de los elementos los estragos del terremoto del primero de Noviembre de 1755, distraen la compunción de los timoratos. Madrid: Herederos de Agustín Gordejuela, 1755; Sevilla: Imprenta Mayor, 1756.
Espinosa de los Monteros, Damián. Respuesta que dio a una carta del Doctor D. Joseph Zevallos en assumpto de varios escritos impresos sobre el terremoto el Ilm. y Rmo Señor D. Fray Miguel de San Joseph, Obispo de Guadix y Baza . Granada, 1756.
Feijóo y Montenegro, Benito Gerónimo. Nuevo systhema sobre la causa physica de los terremotos, explicado por los phenomenos eléctricos y adaptado al que padeció España en primero de Noviembre del año antecedente de 1755. (Contiene además de las 5 cartas del autor: Dedicatoria de Juan Luis Roche, Censura de Joseph Cevallos, Censura de Francisco de Buendía, Aprobación de Manuel Antonio de Origuela, Prólogo apologético y Carta Sexta de Juan Luis Roche) Puerto de Santa María: Casa Real de las Cadenas, 1756.
Feijoo y Montenegro, Benito Gerónimo. Copia de carta del P. Mro Fr. Benito Feyjoo a un caballero de Sevilla en que apunta noticias sobre terremotos con ocasión del que se experimentó en 1 de Noviembre de 1755. Sevilla: Joseph Navarro y Armijo, 1756.
Feijóo y Montenegro, Benito Gerónimo. Respuesta y dictamen del Rmo. Feijoo (Incluido en Zuñiga)
Ferrer, Leonardo. Contracartas a las philosóphicas publicadas por los que se nombran D. Fernando López de Amezua y D. Thomas Moreno sobre el terremoto del día 1 de Noviembre de el año 1755. Madrid, 1755.
Gonzales, Francisco Xavier. Reflexiones sobre la respuesta a la carta del Yllmo. Doctor Fray Miguel de San Joseph: juicio reflejo sobre la verdadera causa del terremoto. Sevilla: Francisco Sanchez Recientes, 1757.
Hoyos, Agustín. Puntual descripción del formidable terremoto que se experimentó el día primero de Noviembre del año 1755. Romance de arte mayor, parifrástica versión de la antecedente elegía hecha por D. ... sevillano. Sevilla: Joseph Navarro Armijo, 1756.

López de Amezua, Fernando. Carta philosóphica sobre el terremoto que se sintió en Madrid y toda la península el día primero de Noviembre de 1755. Sevilla: José Navarro y Armijo, 1755; Madrid, 1755.

Llano y Zapata, José Eusebio. Respuesta dada al Rey nuestro Señor D. Fernando Sexto, sobre una pregunta que S. M. hizo a un mathemático y experimentado en las tierras de Lima sobre el terremoto acaecido en el día primero de Noviembre de 1755. Sevilla: Viuda de D. Diego López de Haro, 1756.

Martínez Moles, Francisco. Dissertación physica: origen y formación del terremoto padecido el día 1 de Noviembre de 1755: las causas que lo produjeron y las a que todos los producen. Presagios que antecedentemente anuncian este temible meteoro y explicación de todas las cuestiones que sobre tan extraño suceso pueden hacerse. Madrid: Juan de San Martín, 1755; Sevilla: José Navarro y Armijo, 1755.

Moreno, Thomás. Copia de carta escrita por un profesor salmantino a un amigo suyo de esta Corte en que le descubre la verdadera causa physica y natural del terremoto experimentado en esta península de España el día primero de Noviembre de este año de 1755. Madrid: Antonio Marín, 1755.

Nifo y Cagigal, Francisco Mariano. Explicación physica y moral de las causas, señales, diferencias y efectos de los terremotos, con una relación muy exacta de los más formidables y ruinosos que ha padecido la Tierra desde el principio del Mundo hasta el que se ha experimentado en España y Portugal el día primero de Noviembre de este año de 1755. Madrid: Herederos de A. Gordejuela, 1755.

Olazaval y Olayzola, Francisco José de. Motivos del terremoto experimentado el sábado día 1 de Noviembre de 1755, con respecto a la ira de Dios en la ciudad de Sevilla y remedios para su templanza ofrecidos el martes veintisiete de Abril en el parroquial del Señor San Julian a el nobilísimo Ayuntamiento de dicha ciudad en la fiesta de acción de gracias. Sevilla: Imp. del Dr. D. Gerónimo de Castilla, 1755.

layzola, Francisco José de. Motivos que fomentaron la ira de Dios explicada en el espantoso terremoto del sábado día primero de Noviembre año de 1755. En la Santa Patriarchal Iglesia de Sevilla y remedios para mitigarla: ofrecidos el sabado 28 de febrero de 1756 en la colocación del Santísimo Sacramento y María Santísima de la sede a su ilustrísimo cabildo y Nobilísima Ciudad, día en que se rezaba el oficio de la Concepción Inmaculada y renovaron el voto de defenderla estas dos comunidades. Sevilla: Imprenta Mayor de la Ciudad, 1756.

Ortiz Gallardo de Villarroel, Isidoro. Lecciones entretenidas y curiosas, phýsico astrológicas metheorológicas sobre la generación, causas y señales de los terremotos y especialmente de las causas y señales y varios efectos del sucedido en España en el día 1 de Noviembre del año pasado de 1755. Sevilla: Viuda de Diego de Haro, 1755; Salamanca: Antonio Joseph Villargordo, 1756.

Reyes, Francisco. Carta sobre los phenómenos que aparecieron en la atmósfera Hispalense después del terremoto de 1755. Sevilla: Viuda de Diego López de Haro, 1756.

Roche, Juan Luis. Relación y observaciones phýsico mathemáticas y Morales sobre el general terremoto y la irrupción del mar del día primero de Noviembre de este año de 1755, que comprehendió a la ciudad y gran Puerto de Santa María y a toda la costa y tierra firme del Reyno de Andalucía. Puerto de Santa María: Casa Real de las Cadenas, 1756.

Roche, Juan Luis. Prólogo apologético de .... A las cartas del Ilmo. Y Reverendísimo Sr. Don Fr. Benito Gerónimo Feyjoo, con una explicacoón nueva del phenómeno celeste que se observó en esta ciudad del gran Puerto de Sta María el día 10 del presente mes de Mayo de 1756 (Incluido en Feijoo)

Roche, Juan Luis. Carta sexta de Don ....al Señor Don Francisco de Buendía y Ponce, presbítero Theólogo, Socio médico de Número...de la Real Sociedad de Ciencias de Sevilla (Incluido en Feijoo).

Rodriguez de Arellano, Joseph. Carta quinta en respuesta de otra erudita (histórica-moral) que sobre el mismo asunto de terremotos le escribió al Ilmo. y Rmo. Sr. Don Fr. Benito Gerónimo Feyjoo, el Señor Don . . ...Canónigo de la Santa Iglesia de Toledo (Incluido en Feijoo)
Rodríguez González Osorio, Pablo. Despertador y recuerdo de dormidos para que abran los ojos del alma, al gran golpe que padeció esta M.N. y M.L. Ciudad con el terremoto acaecido en ella y en otras muchas partes de España, África, Europa, etc. a primeros de Noviembre de 1755. Sevilla: Viuda de D. Diego López de Haro, 1755.
San José, Miguel. Respuesta que dio a una carta del doctor Jose de Zevallos en assumpto de varios escritos impresos sobre el terremoto. Granada, 1756.
Trebnal, Pedro. "Tratado phisico histórico en que hecha una completa relación del funesto terremoto sobrevenido a España y Africa en 1 de Noviembre de 1755, se procura indagar las causas de los terremotos en general y particularmente la del nuestro paragonando con otros muchos notables". (Manuscript, Biblioteca Real Academia de la Historia, Madrid, 9/2766).
Ulloa, Antonio de. An account of the earthquakes at Cadiz, Nov. 1st 1755. Philosophical transactions of the Royal Society, Londres, 69, 427, 1755.
Valle, Thomás del (Obispo de Cádiz y Algeciras) "Después de la terrible, espantosa y a nuestros ojos jamás vista tormenta del Temblor de Tierra…" (Carta a los fieles de Cádiz) 1755.
Zúñiga, Juan de. El terremoto y su uso, Dictamen del Rmo. P. Mro. Fr. Benito Feijoo, explorado por el Lic. . . .. (Con Respuesta y Dictamen del Rmo. Feijoo). Toledo: Francisco Martín, 1756.

Appendix 2: Spanish Anonymous Publications About the Lisbon Earthquake of 1 November 1755

Breve compendio de las innumerables lamentables ruinas y lastimosos estragos que a la violencia y conjuración de todos cuatro elementos experimentó la gran Ciudad y Corte de Lisboa, el día primero de Noviembre de este año de 1755. Barcelona: Teresa Piferrer, 1755. Sevilla: Joseph Padrino, 1755.
Completa relación del asombrosos terremoto que ha padecido la ciudad de Sevilla en el día de Todos los Santos, primero de Noviembre de 1755 a las 10 de la mañana, estragos que causó en templos, casas y personas con todo lo que se ha executado en ayunos, procesiones y practicado hasta el día 8 de dicho mes, con lo acaecido en la villa y Corte de Madrid de muertes y estragos en dicho día. Sevilla: Joseph Navarro y Armijo.
Copia de una carta que escribió D. N. N. a un amigo suyo, dándole cuenta del terremoto y retirada del mar acaecidos en Cádiz, el sábado primero de Noviembre de 1755. Sevilla: Joseph Padrino, 1755.
Copia de una carta que escribió desde la Ciudad de Cádiz un comerciante a otro de esta, en que noticia de las ruinas y desgracias que ocasionó el terremoto del día de 1 de Noviembre de este año 1755 en las ciudades, villas y lugares y puertos de la Corte de África sujetas al dominio del Mule y Andalá, Emperador de Marruecos, con lo demás que verá el curioso lector. Cadiz, 1755.
Coplas en alabanza de la Virgen del Patrocinio, por su intercesión para que cesase el terremoto de 1755. Sevilla: Viuda de Diego López de Haro, 1755.
Carta del Dean de la Catedral de Córdoba relacionando el terremoto acaecido el día 1 de Noviembre de 1755. (Manuscript, Biblioteca Nacional, Madrid, R/34612).
Descripción funesta del terremoto que se experimentó el día primero de Noviembre de este presente año de mil setecientos cincuenta y cinco. Madrid: Imprenta de la Calle de la Paz, 1755.

Descripción verídica y nota a la letra de el nunca experimentado día y lacrymosa confusión que padeció el Santo Monasterio de S. Gerónimo de Sevilla, en el fiero, espantoso terremoto que sucedió en el 1 de Noviembre de este presente año de 1755. Sevilla: Joseph Navarro y Armijo, 1755.

Distribución de Iglesias y predicadores para la misión general y reformación a una nueva vida, que se ha de empezar el domingo 30 de Noviembre por nueve tardes consecutivas que concluirá el día 8 de Diciembre, día destinado para ganar el jubileo de la Misión con la comunión general en el día de la Concepción Purísima, libertadora por su Santo Patrocinio del estrago amenazado del terremoto del día de Todos los Santos, primero de Noviembre en esta ciudad de Sevilla. Sevilla: Joseph Navarro y Armijo, 1757.

Distribución de Iglesias y predicadores para la misión desde el 30 de Noviembre a 8 de Diciembre con motivo del terremoto; y breve noticia de los temblores de tierra que ha habido en esta ciudad de Sevilla. Sevilla: Viuda de D. Diego López de Haro, 1755.

Extensa y completa relación de todo lo acaecido de estragos y muerte en el Reyno de Berbería, en el pasado terremoto como asimismo la voracidad del fuego, que por 40 horas padeció la gran corte de Constantinopla la noche del 27 de septiembre de este año de 55 que por cartas de los padres misioneros de aquellas provincias, escritas a los religiosos de esta ciudad se ha participado como asimismo otras de la plaza de Gibraltar como en ella se verá. Sevilla: Joseph Navarro y Armijo, 1756.

Habiendo esperimentado la Ciudad de Sevilla grandes estragos en sus casas y templos entre ellos el Mayor y su Giralda con el gran Terremoto acaecido a las diez de la mañana en 1 de Noviembre de este año de 1755, prorrumpió un afecto Sevillano suyo en estas mal concertadas rimas. Sevilla: Joseph Navarro, 1755.

Leve rasgo y suscinta descripción de los lastimosos efectos que en esta ciudad de Sevilla causó el espantoso terremoto que acaeció el día primero de Noviembre de este año de 1755. Madrid: Imprenta de la calle de la Paz, 1755. Sevilla: Joseph Navarro y Armijo, 1755.

Memoria fúnebre y descripción trágica de los inauditos y formidables estragos que ocasionó en toda la Española península el violentísimo temblor de tierra experimentado en ella la mañana del día 1 de Noviembre del año de 1755, deducida y extractada de diferentes noticiosas cartas que se han recibido en esta Imperial y Coronada villa de Madrid. Madrid (s.a.).

Memoria ... y descripción de los estragos causados en la Península...por el temblor de tierra de 1755. Extractada de varias cartas recibidas de Madrid. Sevilla: Joseph Padrino, 1755.

Noticia breve de el terremoto y salida del mar que se experimentó en esta ciudad de Cádiz el día de Todos los Santos, primero de noviembre de 1755. Cádiz: Imprenta Real de Marina, 1755.

Noticias de lo acaecido en la Ciudad de Lisboa, Corte del Reyno de Portugal y otras de dicho Reyno, en el día 1 de Noviembre de 1755 a causa del horroroso terremoto y una relación individual de los lugares que se ha tragado el mar, el número de personas que han muerto, y una descripción de lo sucedido en Cádiz en el mismo día.

Segunda parte en que se siguen los lamentables estragos y ruinas y muerte de que las repetidas cartas de Lisboa informan y el estado en que se halla aquella dolorida patria y otras muchas de su jurisdición y las acertadas disposiciones así espirituales como temporales que se están practicando. Sevilla: Joseph Padrino (s.a.).

Noticia de lo acaecido en el reyno de Portugal de resultas del terremoto, experimentado el día primero de noviembre de este presente año de 1755. Sevilla: Joseph Navarro y Armijo, 1755.

Noticia individual que da la Academia de la Historia del terremoto del 1º de Noviembre de 1755 por orden del Rey Nuestro Señor. 1755 (Manuscript, Real Academia de la Historia, Madrid).

Nueva relación de lo acaecido en la ciudad de Lisboa, Corte del Fidelísimo Rey de Portugal, el día primero de noviembre de este año en la conjuración de todos los quatro elementos, que le acometieron en el terremoto, Aire, Fuego y Agua, temblor que padeció a las diez de la mañana; desgracias ocasionadas, con pérdida de inmumerables personas y entre ellas el Eminentísimo Señor Conde de Perelada, Embaxador de su Magestad Cathólica en aquella Corte. Año 1755.

Segunda parte: Realación en que se sigue y da cuenta de las lamentables desgracias que han resultado en la ciudad de Lisboa. Sevilla: Joseph Navarro y Armijo, 1755.

Nueva relación y curioso romance en que se declara el mas lastimoso suceso que ha sucedido en la ciudad de Cádiz, donde se cuenta por extenso el gran estruendo y tragedias que han ocasionado el Temblor de Tierra y Tormenta de Mar con muchas muertes repentinas como lo verá el curioso lector: sucedió en este presente año de 1755, el día 1 de Noviembre. Sevilla: Viuda de D. López de Haro, 1755.

Segunda parte en que se declaran los estragos y ruinas ocasionadas por el temblor de Tierra que ha acaecido en la Ciudad de Cádiz el día de Todos los Santos y priemro de Noviembre de este presente año de 1755. Sevilla: Viuda de D. López de Haro, 1755.

Tercera parte en que prosiguen las ruinas que han sucedido en la Ciudad de Cádiz y en la de Xerez y en la Villa de Conil y asimismo las rogativas con que pidieron al Cielo han procurado mitigar la justa ira de Dios. Sevilla: Joseph Padrino, 1755.

Nuevo y curioso romance del estrago causado el día de Todos los santos en la villa de Huelva, declárase como reventó la mar y el río pereciendo más de dos mil personas arruinados los Templos y las Casas, y asistiendo los pocos vecinos que han quedado, en chozas, dando noticia como cayendo en el Convento de la Victoria parte de su templo y Altar Mayor, entre las ruinas se encontró el Sagrario todo rompido, menos el Sagrado Copón que lo cubría milagrosamente un medio ladrillo, con otras particularidades. Año de 1755. Sevilla: Joseph Navarro y Armijo, 1755.

Patrocinio admirable del Glorioso Patriarca y perfectísimo modelo del Estado Eclesiatico San Phelipe Neri, segundo thaumaturgo y especial avogado en tiempos de terremotos. Sácalo a luz pública la devosión de sus hijos, para excitar al pueblo Sevillano acudan a su Patrocinio en semejantes calamidades. Sevilla: Los Recientes, 1755.

Penitente reconocimiento de un pecador a la inmensa piedad de su Dios, usando de su alta misericordia, no confundió en desolación total a la ciudad de Sevilla en el formidable terremoto del día primero de Noviembre de 1755.... Romance endecasílabo. Sevilla: Joseph Padrino, 1756.

Prevención espiritual para los temblores de tierra dispuesta por un Devoto este presente año de 1755. El Ilmo. Señor D. Onésimo de Salamanca y Zaldivar, mi Señor, Arzobispo de Granada, del Consejo de su Mag. concede ochenta día de Indulgencia a todas las personas de ambos sexos , que devotamente dixeren el Acto de Contrición, que contiene este impreso, con el diálogo entre el Doctor y el Idiota y al mismo tiempo rogaren a Dios nuestro Señor por la paz y la concordia entre los príncipes christianos, extirpación de las heregías y aumento de nuestra Santa Madre Iglesia que hago fe. Sevilla: Viuda de Diego López de Haro, 1755. Madrid: Francisco Xavier García, 1755.

Prevención espiritual para los temblores de tierra y otros accidentes repentinos, que con ocasión del terremoto del año 1701 se imprimió en Granada y en este año de 1755 se ha vuelto a reimprimir y el ilustrísimo Señor Arzobispo actual de aquella ciudad ha concedido ochenta día de Indulgencia a todas las personas, que devotamente dixeren el Acto de Contrición, que contiene este impreso, con el diálogo entre el Doctor y el Idiota y al mismo tiempo rogaren a Dios nuestro Señor por la paz y la concordia entre los príncipes christianos, extirpación de las heregías y aumento de nuestra Santa Madre Iglesia. Granada, 1755.

Prevención espiritual para los temblores dispuesta por un Devoto este presente año de 1755. Diálogo entre un Doctor y un Idiota. Sevilla: Viuda de D. Diego López de Haro, 1755.

Prodigios obrados por el gran Patriarca San Phelipe Neri en tiempos de terremoto, recogidos de diferentes relaciones auténticas para excitar a los fieles a acudir al patrocinio del Santo en semejantes calamidades. Madrid: Herederos de la Viuda de Juan García Infanzón, 1755.

Profecía política verificada en lo que está sucediendo a los portugueses por su ciega afición a los ingleses. Hecha luego después del terremoto del año de mil setecientos cincuenta y cinco. Madrid: Imprenta de la Gaceta, 1762; Sevilla: Joseph Padrino, 1762.

Puntual relación en que se da cuenta del terrible terremoto que en esta ciudad de Córdoba se experimentó el día primero de noviembre de este presente año de 1755. Refiérese su duración espantosa; la consternación de todos; el estrago hecho en los edificios arruinados y maltratados; el raro prodigio de no haver perecido alguno entre confusión y ruinas tantas, debido a la singular protección del Santo Archangel Raphael sobre esta ciudad y le exhorta a todos a dar a Dios y a su Santo Archangel las debidas gracias, especialmente con una estable verdadera mudanza de costumbres. Sevilla: Joseph Padrino, 1755.

Segunda parte del nunca visto conflicto que ha experimentado la gran ciudad de Córdoba, en el terremoto del día primero de Noviembre del corriente de 1755, el que arruinó muchos de sus edificios y principalmente la torre de la Catedral... Estragos causados en la villas de Hornachuelos, Peña Flor, Plama y Posadas...Sevilla: Viuda de D. Diego López de Haro, 1755.

Relación de lo acaecido en la ciudad de Granada el día 1 de Noviembre de 1755 con el terremoto que principió entre 9 y 10 de la mañana y duró 10 minutos. Sevilla: Joseph Navarro y Armijo, 1755.

Relación de los patronatos que tiene S. Francisco de Borja en varios Reynos y ciudades de la cristiandad contra los terremotos y beneficios que con dichos patronaros recibieron sus habitadores, sacada de varios autores. Madrid: Viuda de Manuel Fernandez, 1755.

Relación escrita por el Padre Guardián del Real Convento de Mequinez y Vice Prefecto Apostólico de las Santas misiones que en las partes de Berbería conserva la Religiosa Provincia de San Diego de RR. PP. Franciscanos Descalzos, al Padre procurador de ellas con motivo del Terremoto acaecido en Ceuta, Tetuán, Larache, Mamorra, Tánger y Marruecos en los días 1 y 18 de noviembre de este año de 1755. 1755.

Relación fúnebre en que se declara las lamentables desgracias sucedidas en la villa de Trigueros, a causa del terremoto que experimentó el día de Todos los Santos 1 de noviembre, se declara las grandes ruinas que ocasionó de casas y templos,...con otras curiosas noticias que verá el Curioso. Sevilla: Joseph Navarro y Armijo, 1755.

Relación verídica del horroroso terremoto que acaeció en la muy noble y muy real ciudad de Sevilla, el día primero de noviembre de 1755. Refiérese el grandísimo estrago que ha executado, arruinando todos los templos y edificios en 10 minutos que duró. Con lo demás que verá el curioso lector.

Segunda parte en que se prosiguen los estragos ocasionados en la ciudad de Sevilla en el día de todos los Santos de 1755. Sevilla: Viuda de D. Diego López de Haro, 1755.

Relación verídica del terremoto y agitación del mar, acaecido en la ciudad de Ayamonte el primero del mes de Noviembre de este presente año de 1755. Ayamonte 2 de Noviembre 1755. Sevilla: Imprenta de la calle de la Paz, 1755.

Trágica relación y verdadero y lastimoso romance en que se declara y expecifica el impensado y formidable temblor de tierra que se experimentó entre diez y once de la mañana del primero de Noviembre del presente año de 1755 en la Imperial y coronada villa de Madrid. Refiérese la conmoción general que hizo en todos los templos, casas y edificios, los grandes estragos...que ocasionó, junto con el terror, susto de los vecinos y varias noticiosas individualidades. Sevilla: Viuda de Diego de López de Haro, (s.a.).

Verdadera relación que declara la gran tormenta de aire, agua, relámpagos, truenos, rayos y centellas que huvo en la gran plaza de Orán y terremoto que duró seis minutos; notíciase en ella las muertes, ruinas y desgracias que huvo debido a los referidos acontecimientos el día primero de Noviembre de este año de 1755. Sevilla: Viuda de D. Diego de López de Haro, 1755.

Verídica relación en que se declara el estupendo prodigio que a vista del innumerable Pueblo de esta Ciudad de Sanlúcar de Barrameda ha obrado Señora Santa Rita de Casia en la misma hora que padeció dicha Ciudad la fatalidad del terremoto: Dase cuenta como

viendo ya perdidos los moradores, se libertaron de improviso atribuyéndolo a la protección de la Abogada de Imposibles la que salió de su Convento en hombros de quatro religiosos y poniéndola a la vista del mar éste se retiró instantáneamente. Sucedió a uno de noviembre de 1755. Sevilla: Joseph Padrino, 1755.

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The Lisbon Earthquake of November 1st, 1755: An Historical Overview of its Approach

Maria do Rosário Themudo Barata

Let me begin by stressing the fairness and usefulness of interdisciplinary cooperation for the study of seismicity in Portugal. Also, under the same heading, let me pay tribute to Professor Doctor Luis Mendes Victor coordinator of the geophysical sciences area of the Science College of the Lisbon University, who has been promoting interdisciplinary studies of seismic phenomenon for over twenty-five years. Also, he is responsible for creating multi-disciplinary teams that have been contributed to investigations and publications and also have afforded elements for discussion. It was also Professor Mendes Victor who invited me to follow the project that – as well as other projects – was the basis for a study published in 1988 and 1989 by the Nuclear Protection Agency, a member of the Commission for the Portuguese Seismic Catalogue.

This Commission was created on December 30, 1983 by a joint order of the Secretary of Industry and Energy and the Secretary of Social Equipment and the goal was to "prepare a unified seismic catalogue and to update it regularly". All this work would also be relevant to the Portuguese Weather and Geophysical Institute , to the Portuguese Civil Engineering Laboratory, to the electric power and water companies, to the Civil Protection, to the Nuclear Protection and Security Agency and to all those involved in regional and urban planning. The cooperation of people with degree in History was necessary basically for the heuristic and critique of the document sources pertaining to the periods in which there was no instrumental data record.

This was not entirely new. There was a summary report of the investigation carried out by Professor Doctor José Mattoso along with a team of graduates in History from the College of Social and Human Sciences of the New University of Lisbon. It was in this college that Professor Doctor Ana Maria Pereira Ferreira took up this investigation for a period including the 16th century and the whole team published a work about the Lisbon earthquake of 1531, while another investigator, Dr. Lucília Runa, analysed the data after 1755.

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The investigation of the sources dating back to the 17th and 18th century, which I myself coordinated, as a member of the Language College of the Lisbon University, it was the result of the intensive teamwork of a team of five History graduates of the same College, whose names I would like to highlight, as they should have much of the credit for this work.

We worked for over two years in an atmosphere of careful scientific investigation and warm friendship. They are Maria Luisa Braga, Mafalda Noronha Wagner, Berta Guerra, José Felix Alves e Joana Neto.

Each one of us conducted our studies and investigations in specific areas according to each one's professional areas. Now, twenty years passed, some of us run with this subject again in an attempt to exchange points of view, now that time has allowed us to cross data and deepen the meaning and interpretations of some issues. On a personal basis, and attempting to explain the birth of the questionnaire for 1756, the results of experience gathered in other areas converge towards the understanding of previously suspended issues for which History is – unsurprisingly – the focal point for different types of learning.

More and more the groundwork for several scientific specialities shows the importance of history and unity of knowledge, which is the result of an integrated process with great symbolism and varied meanings. In a space and time where knowledge constantly develops, it is natural that human and scientific knowledge can mingle in a single seamless housing, organized in a logical and aesthetical manner. This is a sign of the illuminist order of the 17th century, what we can call the first illuminism, or the first illuminist generation. The schematic of knowledge show then reflects the traditional progression of constitution and development of each specific field and is expressed in an organic and clear fashion with its own style, a style geared towards the search of general understanding. This progress and development was meant to act as a request to the powers that be to include the elite in the knowledge to be given generally to all social groups. In Portuguese society the debate was very lively, as it was stated that the ruling monarchy was interested in enriching the tradition.

Taking this into consideration, as well as all the history produced in Portugal and in Europe, I would like to point out some familiar points and to delve deeper into uncharted lands.

The knowledge that dates from the Portuguese 18th century shows that it is a dynamic time, of creation and not decadence. We may ask ourselves if it is the end of the ancient regime or its fulfilment, but today we don't believe there was any gap within enlightened Europe. Historical time has been handled by historiography about Portugal (I recall the final works of Kenneth Maxwell and Arno Wehling to quote only two figureheads of English and Brazilian historiography that sum up Portuguese historiography on the topic) to analyse and interpret all the forms and vitality of a regime and of a society that – regardless of all the difficulties – were socially, institutionally and ideologically equipped to deal with the challenges, adversities and catastrophe common to those day. The latter had, according to Jorge Borges de Macedo, greater consequences in the structuring of a State and power with trans-continental responsibilities, as was the case of Portugal.

In the 18th century, the history of the "Portuguese Empire" – especially the development of Brazil – is a dynamic period, which goes beyond the type of vision, which has now been cast aside – of the three times: Joaninos, Josefinos and Marianos. Setting up a court (Relação) in Rio de Janeiro in 1751, making this city the capital of Brazil in 1763, the Companies of Grão-Pará e Maranhão, Pernambuco e Paraiba were all initiatives that followed a major reform of the Secretaries of State of João the 5th, international negotiations on the boundaries and overseas influence areas, following the negotiations of the treaties of Utrecht-Rastadt. Madrid and Paris, while in the Kingdom they organized the Companhia Geral da Agricultura das Vinhas do Alto Douro, the Fábrica do Rato, the Junta of Commerce was created as well as the Noblemen College and the University of Coimbra was reformed.

This society – as well as others – was hit by catastrophe. Voltaire's illuminist critique wanted to stress the problems of conciliating the Philosophical Optimism with the earthquake on November 1st 1755. However, said critique does not exempt one from having response capabilities to the catastrophe, which did not undercut the value of the Portuguese 18th century, rather it showed the clear interest in studying it and analysing all its possibilities.

And the answer could only come from one man, even if only the Minister who was fully supported by the King José 1st.

João Duarte Fonseca recently drew attention to the fact that organizing the public response to a calamity was not solely the responsibility of Carvalho e Melo: heads of the religious and civil society – amongst which were Patriarch Monsenhor Sampaio, D. João, Duque of Lafões and his brother, the Marquis of Alegrete, president of the City Hall's Senate, who coordinated the support given to the population – immediately undertook measures. Many more could have joined them. History will forget about them and attribute all the credit to the Minister, but the fact is that it was he who took the political responsibility and who led all the actions.

The name of Sebastião José de Carvalho e Melo, future Duque of Oeiras and Marquis de Pombal is attached to the historical fact of November 1st 1755. This minister of José the 1st is a historical character that can be regarded by himself as the epitome of an era, with all the contradictory interpretations expressed in his biography, because of his undeniable climb up the rungs of the ladder of power, due to the way he organized and implemented the Portuguese response to the catastrophe. After this response twenty years were to follow, where he controlled the political power, until the King, José, his staunchest supporter, died and the so-called "viradeira" ("turn-about") replaced the social groups, the influential people, without ever questioning the political status quo and all the options that had been thus far taken.

The Marquis is a perfect depiction of the complexities of the Portuguese 18th century, however, more often than not, those who praise the political actions of this statesman are sceptic of the general capabilities of the Portuguese people in this century, and present the Marquis as an isolated – or almost isolated – occurrence, as a singularity, the exception to the rule of unavoidable decadence, an unfettered individuality without consequences in his failed attempt to modernize Portugal. This is by far an exaggeration and clearly unrealistic. Maxwell already said it as he mentioned the "estrangeirado" (one influenced by the culture and of other "more civilized" countries, thus being looked down on by the Portuguese society) of the 18th century, a "by-product of the current atmosphere in Portugal", where the discussions on governance, economy, diplomacy, conservation and use of overseas territories was played out to its greatest extent. Many of the reforms undertaken in the second half of the 18th century would be applauded by all those which lauded the enlightened despotism as the greatest accomplishment of the ideals of the monarchy. The Treasury, the creation of the great companies, the modernization of the army, the reform of the school system, the manumission of the State in all its areas, the curtailing of the power of the Inquisition and the creation of the royal Censorship, the abolition of the distinction of new and old Christians, and the corresponding accreditation for civil service, the restructuring of the colonial administration, all these are aspects that can be considered a paradigm of Illuminism.

But let us not forget that Pombal mobilized traditional resources, institutions and social groups that were well known to him to initiate his reforms or innovations of the regime and to search for domestic, as well as foreign, collaborators and supporters.

Is this a paradox of the Illuminist era? I believe not. In this respect Carvalho e Melo followed the examples of the Austrian and English society, so well known to him. The chamber system in the Austria of Leopold I, Mary Therese and Joseph II follows the guidelines of Enlightened Despotism with essential reforms, cultural and technical strengthening of the elite, giving new gear to the armed forces, participating in the international political scene, in search of alliances, defining ancillary influence areas. England, now with the Hanover dynasty – of German descent – represented by George I in power, left the Prime-Minister Walpole in charge of conciliating the royal power with parliamentary interests and colonial administration.

But in face of power, the behaviour of social groups is drastically reduced to two fractions: those who support it unconditionally and those who are bitter enemies.

The Minister of King José 1st defined the priorities, dictated the sacrifices to be imposed upon the Portuguese society and generally made use of power. In his interpretation of Sebastião José de Carvalho e Melo, Kenneth Maxwell considers him to be an economic nationalist, Iberian and illuminated, which may well be the most accurate way of defining Pombal and all his actions are logical when it comes to defending the interests of the Portuguese. This helps to understand the reasons for his actions, but not the way he acted nor the relentless destruction of his opponents.

This extremism has in a way contributed towards diminishing the cultural, social and other types of bonds which connected the Marquis to the various sections of Portuguese society. Today we know that that outlook is not adjusted to his familiar, professional and cultural background, to the support he sought and found in the metropolitan and overseas civil service, in the military forces, in the economic and intellectual groups and members of the clergy and religious orders. However, it would be his use of power that would highlight the separation

As for the response to the Lisbon earthquake, the response was given by the Portuguese institutional resources, which maintained order in the normal course of public life, in planning, by making it possible to rebuild the city and then by effectively building it and all else connected to the seat of power of the Portuguese institutions and Monarchy. When developing these topics, the political historiography is followed by the historiography of art, which draws attention to the scientific and technical skills of Portuguese engineering, to the architectural and urban solutions. But often, or to a great number of people, the image of reconstruction is the iconographic *motif* and few probes into the real significance of the reconstruction of Lisbon for the Portuguese society and for the evaluation of that very society later on. Even if it was represented by the Prime-minister, the response could not come from a single individual.

And we thus come to a crucial point.

All through the work of caring for the living and rebuilding the city, head of the monarchy, royal house and central institutions, home to the church hierarchy and various religious foundations, to the major noble houses, to the main offices of the big trading companies, cultural institutions etc, a major seismologic questionnaire is drafted, composed of thirteen questions, and even today seismologists are surprised by how adjusted and modern it is. Asking all Portuguese bishops to forward this questionnaire to all parishes and stressing that the replies were to be returned as fast as possible, His Majesty, the King, thus initiated the recording of an observation that was to be as wide and detailed as possible. This historic and scientific initiative was prepared by the country's cultural and scientific resources and was known as the "Marquis de Pombal survey", as it was the Minister, in his capacity of Secretary of State who signed off on the document. It can be construed as a sign of the close relation between Carvalho e Melo and the King and the Portuguese cultural a scientific institutions, with their work programmes, with the most relevant scholars of the church and civil society in general. The historical perspective was to Pombal a heuristic way of obtaining all information he needed, to ask for an interpretation, to base his options on. A basis for all modern scientific work, the observation was carried out and recorded and handed to Carvalho e Melo by agents of human sciences, the church, men of science and of literature who, rather than have their backs turned to one another, were integrated in institutions such as the Royal Academy of Portuguese History, founded by King João V of which the Marquis was a member.

I believe it is safe to say that this was a process that can easily be understood taking into consideration what we know of the potential of Portuguese culture, of the cultural institutions, of how international it was, the teaching reform, the debates in the press, etc. all these traits help understand the survey ordered in 1755 by King Ferdinand VI of Spain, which can be compared tot he Portuguese one. Making a reference to the work of Martinez Solares, I think, however, that these are two separate processes, each with a genesis all its own.

As for the Portuguese survey, I suggest one look at the goal, members and work of the Royal Academy of Portuguese History.

It is undisputable that the scientific study of the Lisbon earthquake of 1755 was made possible by launching the aforementioned seismologic survey, whose relevance and adequacy is still today grounds for admiration. At a time when there were no instrumental records, in spite of the secular attempts both by Western and Chinese culture, and before the Newtonian scientific paradigms as models for new guidelines in physical and mathematical sciences were defined as integrative parameters for general culture (as seen in the English editions of the 18th century editions of the new encyclopaedias and in the famous *Encyclopedie Francaise*) this survey was still a surprise.

This is the main issue. As for the next issue (how to explain the source, the way of transmission and the effectiveness of choice – regardless of how much was lost from the answers of Lisbon and the Algarve) one may well accept an explanation as to how the Royal Academy of Portuguese History to implement the so-called "System of History", designed under the same spirit as the decree by King João V to found the Academy on December 8th 1720. In this decree the King stated that "I have decided that this Academy is to be established, to record the Ecclesiastic History of these kingdoms and that after that all is part of their history and conquests; and because the necessary news are not to be found solely in printed and manuscript books, rather also stored in the Archives, I shall issue royal letters requesting access to all Catalogues of said Archives and Registries be facilitated to people who will draft Statutes, so that I, as Protector of the Academy, may examine and approve them, so they may be carried out. And, as I have chosen many people – for their qualities, scientific and otherwise – to compose this body, others will be nominated until said body has the numbers deemed necessary to fulfil the tasks as I so define them: it is my order that this Decree be read in the first Conference, to be held on the day of Our Lady of the Conception, Patron to these Kingdoms, and it is to be so recorded."

Joaquim Veríssimo Serrão, renown Portuguese historian, chairman of the Portuguese Academy of History (a 20th century continuation of the Academy of King João), transcribes the royal decree in his work dedicated to Portuguese historiography. He does so in the first pages about the 17th century and all the admiration shown in that text is seen by any and all who might read the historical records of the Academy, the minutes of the sessions and the discussions, due diligences towards archives and libraries, observation tours, acknowledgement and investigation by scholars, all controversy and printed works they stimulated.

The full list of scholars and correspondents bears proof to the broadness of the creation of the Academy, the intellectual relevance of some nobility, the freshness of their cultural standpoints (let us recall that all Academies in Europe corresponded. The Count of Ericeira proposed a direct link to the Royal Society), the actuality of the discussions and the clarity of the information, all the political and legal matters, the civil and canon law as well as economy-related matters.

As for members of the clergy, both secular and regular members of all orders and congregations were represented, with their own peculiar sensibilities and traditions. There were also a smaller number of professional and technical elements of what is today termed the exact and natural sciences. The emphasis on historical sciences should not lead us ascribe a less than scientific content neither to many discussions nor to the intellectual qualifications of its members.

While there were fewer representatives of the so-called exact sciences were less than those with "canonical" and artistic (amongst the latter were people schooled in philosophy – with all its areas, as well as mathematics) backgrounds, we can say that many of them had key roles in the works of the Academy. At any rate, the work of the Academy in the 18th century was hindered by the distancing of King John V, who had taken ill. Let us not forget than on November 1st 1755, the house of the Duque of Bragança, where the Academy convened, collapsed, and its library was lost forever.

Their work did not begin immediately, but must have had an impact on the actions of Carvalho e Melo and in the drafting of the inquiry. While the work of the Royal Academy of Portuguese History slowly decayed, some of its members continued their work by debating some of their topics. The Royal Academy of Sciences was founded by Queen Maria I on December 24th, 1779 and devotes part of its attention to History, and welcomes the last academic of the previous Academy, Gonçalo Xavier Alcáçova Carneiro. Their orientation was different, but one cannot truly speak of a rupture.

But let us return to the Royal Academy of Portuguese History.

This is not the time to write a history of the Royal Academy of Portuguese History, however, one must recall some of the elements which led to the famous "Marquis de Pombal Inquiry". This Portuguese inquiry closely follows the one held in Spain, especially with the Spanish Academy of History studied by historian Rodriguez de la Torre, mentioned in Martinez Solares's work on the Lisbon earthquake of 1755. It is therefore possible to compare both processes.

Firstly, let us consider the reason behind the creation of the Academy, which was to re-write the ecclesiastical and secular history of the kingdoms and their conquests in a integral way, which is displayed in the crest of the Academy, which boasts *Restituet omnia*, which means "reconstitute all" that had anything to do with history, viewed as the integrative hub of all knowledge deemed necessary. The progression was to hinge on the goal, the quality and the diversity of the cultural and professional background of the scholars, on the logical, scientific and systematic development of the discussions. There was no epistemological limitation to scientific development. The hub, the integrative centre was a human science, but like European culture had maintained since the classical age – an inheritance developed by the Middle Ages and highlighted by the Renaissance – the integrative concept of the knowledge that should have no breaks, one underlying all encyclopaedic knowledge, accepting the contribution of new sciences such as etymology, philology, geography, natural sciences, astronomy as stellar physics. That was the time to develop observation, classification, the mathematical reading of the real world. And the science of the sublunar world, the geography and those which could be considered the first geological questions that arise and follow the progress of the establishing of astronomy as a science. This direction was quite clear, also due to everyday needs in the 17th and 18th centuries. Let us recall the expansion of overseas and intercontinental travels in this era of mercantilism, the money-exchange operations, the insurance, the development of war machines, especially the artillery, the fortifications, all aspects which required a careful study of mathematics and science with extreme care and attention.

These demands followed the development of other academies in Europe. Academies of arts, languages and sciences, as well as the development of other trades and professions that grew in number and renown, which were created by people of secular or religious background, oftentimes with the sponsorship of the sovereign powers. The development of the Academies in Portugal is also related to a visit by a papal ambassador in 1715. Of this time we know the interest show by several people such as Cardinal da Cunha, Doctor João Mota, priest to the royal chapel, João Tavares, a Jesuit, Dominican Friar José da Purificação, Oratorian João Antunes and many more. The more prominent names amongst the nobles were the Count of Ericeira and Count of Vilar Maior. The former was also a mentor of an academic area where one dealt with matters of literature and science. Let us not forget the Academy of Enlightened which will continue to exist after the creation of the Royal Academy of History from the home of José de Carvalho e Melo. John V's initiative was the first step down a road taken by Portuguese society, thus receiving many global directions and a wider scope of action, including in many cases projects that already existed.

The shield seen on the 1st tome of the "*Collecçam dos Documentos, Estatutos, e Memorias da Academia Real da História Portuguesa*" (*Collection of Documents, Statutes and Memories of the Royal Academy of Portuguese History*), published for the first time in 1721, wants history to rise – "Historia Resurges" – as it quotes the motto "Restituet omnia". In the very beginning of 1721, on the 3rd February, begins the *System of the Ecclesiastic and Secular History of Portugal to be written by the Academy of History*, drafted by the count of Ericeira and by Priest D. Manuel Caetano de Sousa, in a text signed also by the Marquis of Abrantes, Marquis of Alegrete and Count of Vilar Maior. This system was approved and set up as a rule of the Academy. Maria José Mexia Bigotte Chorão has produced the Syllabus of the Academy, and the questionnaires known to us from the 18th century. The first questionnaire is directed to the archbishops and bishops, heads of cathedrals, religious orders, city halls and villages and heads of various regions.

I will try to make a critique of conditions and to show the consequences.

When viewing the Royal position, let us mention the high number of sessions attended by His Majesty, often accompanied by the Queen and the princes, excepting some exceptions duly indicated, such as the session on April 1st 1721, which the King could not attend, as he was attending to matters pertaining to Brazil, as the fleet was to set sail from Lisbon on the following day. As João V paid so much attention to the Academy, a number of initiatives were established, such as various orders that the Academy should receive all Inventories and copies of Documents of all registries and archives in the Kingdom (I. fl. 26), all the texts for drafting the history were to be sent in an envelope marked "From His Majesty's service to the Count of Villarmayor, Western Lisbon" (fl. 27v).

The Academy requested "some literate people to read ancient papers" (fol. 28v), and also "curious people with ancient memories or Manuscript books of History and ancient times". As for Archaeological materials, "all the signs of Romans, Goths or Moors were to be moved and all letters still legible are to be copied, as they are, even if parts are missing or partially destroyed" (fl. 31).

In a description of March 4th 1721 (fl. 45), one could read that Priest Manuel de Campos of the Company of Jesus, "one of the scholars in charge of analysing the Geographic points", asked all scholars that, in the course of their work, were to find names of cities, villages, places, territories, rivers, bridges, mountains and promontories that were in any way out of the ordinary, convey said names to him.

The work of the scholars did not exempt them from visiting the sites they studied, as can be seen by the fact that the Count of Monsanto, who had to write about the episcopacy of Portalegre, after examining the chorography of the diocese, "decided to go to Portalegre take a closer look at everything relevant (fl. 80). It would then be the turn of Doctor Manuel Pereira da Silva Leal to give indications of ancient and modern geography to "whom it may concern" (fl. 84).

One more issue before moving on to "whom it concerned". An important point of the Royal Decree dated August 13 1721 whereby João V ordered ancient monuments to be maintained to illustrate and testify to the truth of History. This Decree was the creation of the official preservation of our heritage. It was approved by the Royal Appeals Court Judge on August 14 and signed by His Majesty and the Secretary of State, Diogo Mendonça Corte Real. Soon there were reflexions by the Academy on the "*Decreto y ley de Conservação dos Monumentos antigos*" (Decree on preservation on Ancient Monuments), fl. 172 and following, as mentioned.

Clearly, all these elements are part of a broader context than that which we can analyse in this work. Let us stress the King-s attitude towards historical, cultural and scientific matters and the nomination of the Secretary of State as the official who was to follow said topics. Later, the same entities will be mentioned in the inquiry on the catastrophe of 1755.

We will now try to briefly sum up the essence of the activities of the Royal Academy of Portuguese History.

Before all let us stress that History was the meeting point of various types of knowledge, not separating human sciences from other sciences, and the language was valued not only by its aesthetical quality, but also by its clarity and how adjusted it was to the topic under appraisal. It was an aesthetical architecture with a rational side, and knowledge was considered to be a building and all its elements were to be analysed considering the final product. This aesthetic of order followed the classical tradition and – according to some scholars – was influenced by Cartesian ideas.

The text of the Academy says "for the building to be constructed according to the rules of art by all who work on it, firstly we need a schematic of the building, and to make Portuguese history, as it is written by all its authors, a balanced body in all its components, firstly we need to have a broad picture, establishing which observations are essential to reach the goal for which the whole system was created".

This is how systematization appears connected to the writing of History.

The first recommendation of this text is that the *Memórias da História Portuguesa (*Memories of Portuguese History) be written "in pure Portuguese, in a clear fashion", not going into detail, because "historical Art allows these writings to be free". It also said that the works would not be structured like annals, but the chronological order of events should be maintained.

Two main demands were immediately introduced regarding Geography and Chronology "the twain eyes of History" and all writings should respect these principles, and show no division "in calculations, times and locations". The Geography should be uniform "in the divisions of ancient Lusitania and Astronomical observations and distances". As for Chronology, the system used was that of Priest Dionísio Petavio, a Jesuit, which marked the coronation of D. Afonso Henriques in 1139, recalled that it had been João I who in 1422 had ordered the years to be counted starting from the date of birth of Christ (rather than from Cesar's era, which had a hiatus of 38 years). Each of the tomes of the memories was to include a "Chronological Chart drafted by the Academy, where one sees the years of Christ, Pontiffs, and Kings, the memories of all triumphs, divided into matters". As History wanted to be seen as a science forced us to cast a scientific eye onto other branches of knowledge and the initiative stemmed from the Academy of History.

This was followed by a definition of the rules for the writing of history, the separation of sequential and situational history, the critique to past authors, demanding the "analysis of their writings as well as of their times" and as for the documents, "the demonstration, or conjectures of their legal value". The text further drew attention to cases where the "allegations" or references were deemed necessary to clarify the matters at hand and anything else the authors thought ought to be brought to the attention of the Academy Censors for a discussion.

The presentation of the System was followed by the distribution of the work by the scholars of the Academy. All matters Geographical were assigned to

Manuel de Azevedo Fortes, Master-Engineer to the Kingdom and a Brigadier in the royal army and to Priest Manuel de Campos, the aforementioned Jesuit.

The fact that a military man was in charge of this area meant a meeting of mathematical and deductive geography with empirical and deductive geography. At the time being an engineer meant, with few exceptions, that one was a military man, and this not only in Portugal, but all over Europe, until very recently. One did not merely look for the descriptions and observation records. It was not a traditional chorography, it was a translation of the knowledge obtained by scientific and technical means with guarantees of universal validity. Moreover, as is the case with the intercommunication of areas of knowledge, the technical means guaranteed by the underlying scientific theory might be applicable on a scale unseen until then.

It is my belief that the simple progression via natural sciences would not in itself explain the advances in physics and mathematics that were demanded by the study of seismology, neither would they explain the reconstruction urge that followed. Not to say that this was the main geographical and geological conception around the Marquis de Pombal and that it is enough to explain the inquiry of 1756, but I put it to you that this modern vision, shared by Galileo and Newton, was already a contender, but shoulder to shoulder with other more traditional readings. It did not cancel out other interpretations, rather, it made it possible to create a theoretic interpretation to support the used techniques. This was a stimulus to the observation and helped create a scientific questionnaire.

As for the possible interpretations, let us briefly recall these other positions so that we may fully appreciate the process of rationalizing space and time shown in the Academy's *System*.

In his "Discours de la Méthode" Descartes summed up all his thoughts on the four elements – Gas, Water, Fire and Earth –, the result of the relationship between philosophy and the natural world yielded chronometry and topometry and, in conceiving the system of the world, the works of Kepler, Descartes, Huygens and Newton were of great importance. Gilbert, author of *De Magnete* and *De mundo nostro sublunaris philosophia nova* (published posthumously), was a friend of Edward Wright, a keen scholar of the determination of longitudes at sea, of Richard Hakluyt, geographer who promoted the famous collection of travel stories, of Henri Briggs, geometry professor and inventor of a logarithm system and of William Harvey, who proposed the theory of blood-flow.

As for the earth sciences, the 17th century had produced the unitary concept of geography and theory and practice had come closer to solve cartography problems. Scientific and technical concerns were on a par as far as border marking, stabilising regular oceanic travels, defining land and sea itineraries, forts. Adding to the concrete problems of the 17th and 18th centuries, there were now problems with the dates and locations of Biblical History, a mark for the more famous civilizations.

As for the study of the inner earth, the earth was seen as an organism in the works of the Jesuit Athanasius Kircher dated 1665, a crucial scholar in the process of creation of geology as a science, as stated by Jan.I.Kozak, Victor S. Moreira and David Oldroyd amongst others in the recent work *Iconography of the 1755 Lisbon earthquake,* Praha, 2005.

The type of questions raised were reflected by a wide and non-specialist audience and had to do with the explanation of rivers, their source, their course, their destination, the explanation of sources and the idea of capillarity, the knowledge of the interior of mountains, the constitution of rocks. All these topics were enticing, even if their approach often was based on the exploration of dreams, return to legends or even primal forms of speleology or observation through mining techniques.

Generally speaking this was the stance of European culture: as a witness to possible diversity, let us bring to mind the writings of Miguel Cervantes y Saavedra on the origins of the Guadiana river as seen by D. Quixote, after descending into the famous cave of Montesinos, and later the works of Pierre Perrault *De l'origine des fontaines,* inspired by Torricelli, Mariotte and Huygens.

It is hardly surprising to see that shortly after the collection of what was intended to be a General History of Portugal and of its conquests, the Portuguese culture of the 18th century had a questionnaire on three topics, Places, Mountains and Rivers. With the goal of revising the existing corography, the proposal for new questionnaires belonged to someone who was to be invited to the Royal Academy of History.

In the *Dicionario Geografico* so effectively executed by Priest Luis Cardoso, the edition of the 1st part was published in 1747 (the second part would follow the Lisbon earthquake of 1755) we can see that when speaking of the place called Agrello in the province of Beira, county of Coimbra, that their parish priest, upon hearing of a lagoon located in that mountain range, knowing that that was a location traditionally inhabited by moors, had a pump constructed to drain the lagoon. After draining enough of the lagoon, some volunteers went into the depths and accessed a platform whose steps led to a portal where two figures bearing arms discouraged them from venturing any farther.

Fable? These testimonies are contemporary with the questionnaire. In France, in the 18th century, there was a topographic chart of the land based on a geodesic triangulation, the French Encyclopaedia divides cosmology into Geology, Uranology, Aerology and Hydrology, Buffon presents his theory of the Earth in 1749.

In Portugal, after the earthquake, early in 1756, the church authorities receive a seismological questionnaire sent by the King and signed by the Secretary of State. This can be see as a sequel to the Academy questionnaire dated 1721 and to that of Priest Luis Cardoso, in the 30s.

There is, however, another piece of data that is important and which has thus far not been raised: the work of the man in charge of drawing a geographical chart of Portugal and who followed a Method to train the corps of engineers and all collaborators as soon as 1721. An infantry Brigadier in His Majesty's Army, Chief-Engineer to the Kingdom, Knight of the Order of Christ, a lead Scholar in the Royal Academy of History, his image and actions relate to the renewal of military theory and practice under D. João V, where one must recall Luís Serrão Pimentel, Francisco Ferreira da Cunha, D. João de Mascarenhas, Manuel Antonio de Matos: and let us not forget that after the arrival of Schomberg and the way he impacted the military organization, the king had issued new military ranks in 1707.

Manuel de Azevedo Fortes is part of the same group as Manuel da Maia and will mention him when quoting an anonymous work on modern fortifications (written by Manuel da Maia by translating an eponymous work by Pfeffinger). Manuel da Maia, just like Eugénio dos Santos and Carlos Mardel, renown collaborators of Carvalho e Melo in the reconstruction of Lisbon, are also military men.

Manuel da Maia published *O Governador das Praças* in 1708. Azevedo Fortes later wrote *O Engenheiro Portuguez* and *Evidência Apologética*, which compiles the principles and practices deemed adjusted and modern. His work will greatly influence authors like Bento Gomes Coelho e Tomas Telles da Silva.

Manuel Azevedo Fortes first addresses the Royal Academy on May 27, 1721. Following the division of work, he would handle modern Geography. As per the project, the geographical charts were to be made so as to mark villages and locations which had disappeared. There was a reference to the Teixeira chart, as it was considered to be the more correct one, but which was "at times so incorrect that it seemed necessary to make one befitting the truth of History. To that effect one employed the most capable engineers in all provinces and drafted a method for drawing up the maps in all clarity, to be given to the Secretary and after being scrutinized by the censors, is to be printed".

In October of that same year, Azevedo Fortes would again report on his work, which he insisted was valid. The geographical charts were a part of the History that they drafted and could be great credit to the kingdom "the only one in Europe without full charts and is Europe's foremost country in Geographical and Seafaring matters. I can say that this kingdom invented the first Geographical instruments. The astrolabe was invented by Master Rodrigo and Master Joseph and Martim de Bohemia, disciple of the great João de Monte-Régio. The famous Pedro Nunes, renown Portuguese mathematician invented the astronomical ring, preferred by all over the astrolabe. Through these rough instruments we set out to discover the four corners of the Earth and covered ourselves in everlasting glory". Azevedo Fortes mentioned the discoveries stimulated by Infante D. Henrique, the drafting of the first sea charts with equal degrees and parallel meridians and concluded that "the science of Geography was so evolved and made so much easier by all instruments invented, that the Portuguese engineers were expected to draw precise charts of the kingdom. They are to be started and will not be finished as promptly as promised, as it will take more than four months of relentless work". The report ended in the usual fashion, asking the opinion of the Head and the Censors (fl. 202).

In fact, in 1721 began the drafting of the *Tratado do modo o mais fácil, e o mais exacto de fazer as cartas geográficas, assim da Terra como do Mar, e tirar as plantas das praças, cidades, edifícios com instrumentos e sem instrumentos, para servir de instruçam a fábrica das cartas geográficas da hostória Eclesiástica e Secular de Portugal* (Treaty on the simplest and most accurate way to draw the Geographical charts of Earth and Sea as well as the plans of the squares, cities and buildings with and without instruments to serve as models for the drafting of Geographical Charts of the Secular and Ecclesiastic History of Portugal). This work was published in 1722 after being approved by the Count of Ericeira and the Count of Monsanto, an approval and six licences from the Holy Office.

In this work Azevedo Fortes talks about key moments in the development of Geography in European culture, and quotes Ptolemy, Mercator, Hortelius. Du Val, de Sanson, Telemont, Canteli, Coroneli, Dufes and Del'Isle. He mentions the crucial importance of direct knowledge of the terrain, and establishes parallels with Painting and writes 9 chapters, including one about problem solving. The first chapter was about "Petipe and all things necessary to draft port or maritime charts". This is exceedingly interesting scientific-based work with immediate applications in the fields of geography, navigation and defence, which broached many practical aspects, such as, for instance, "measuring, without resorting to instruments, the distance between two sides on enemy grounds, or over any inaccessible distance" or "instrument-free measurement of the width of a river" or advice, like the one directed at any engineer who, in enemy territory, wishes to do reconnaissance work. The advice is to disguise himself as a merchant, passenger or pilgrim, but the best would be to pretend to be a wandering salesman and to that effect the languages and measuring systems should be known to him. He should have coded script to translate the measurements, i.e.: "instead of plainly writing that the length of the right-side wall of a fortress was 214 feet, left-side wall 268, right flank 130 feet, etc, he should write as if registering a sale, on the 5th of this month, 214 rolls of cotton thread, on 7.8 and 9, 268 rolls of various cloths; from the 12th tot he 16th, 130 rolls of cloth" (fl. 416). If that engineer could carry no instruments, he could still have a sundial, whose compass he could use to carry out some operations. But there were more suggestions – the measuring of regular steps (so-called "free" paces), the use of sticks of different lengths, ways to use the compass when drawing up maps, a method used in countries with great deserts, "how to draw a road on a map, using the same angles that would be registered using those instruments, many engineers have used the compass to draw geographical charts" (fl.79).

The book *O Engenheiro Portuguez* (The Portuguese Engineer) was published in 1728 and is divided into two parts or treaties, as stated in the title: Tome one "with practical geometry applied on paper and on the ground: the use of instruments essential to engineers: how to draw and read military charts; and an attachment with trigonometry". Inside we could see that the treaty was divided into three books, "the first one dealing with longimetry, or measuring distances; the second one was about planimetry, measuring the surfaces; the third was about stereometry, or the measuring of bodies". The treaty further included a part about Fortresses, spanning over 8 books, where – as well as other things – they explained methods "by the three more famous authors, Chevalier António de Ville, Count Pagan, and Mariscal of France, Monsieur de Vauban, and a fortress by an anonymous author, to be used as a guide in this work" which could poss-ibly be a reference to the aforementioned work of Manuel da Maia.

This technical and theoretical work was illustrated by means of seven paintings or images.

This work too was approved, but went through a much more demanding process, similar to the one used in Spain for akin processes mentioned by Martinez Solares. The first few pages include the opinion by Brigadier João Massi, of mathematician, jesuite João Baptista Carbonne, two officials from the Holy Office and a further five signatures dated June 20, 1727; it was followed by a regular opinion of the censorship and the censorship from the Royal House signed by Luis Francisco Pimentel, Cosmographer-General to the kingdom, followed by another opinion followed by a new order from the Holy Office dated July 29 with three signatures, a page with 4 validations and a further 4 signatures!

As mentioned by Jorge Borges de Macedo, the theory and practice of the more advanced and more traditional sectors were often handled together in a effort by the authors to show how useful these works were. Here, Azevedo Fortes showed the usefulness of the notions of geometry for measuring fields, in dividing into lots, for solving heritage-related disputes, to repair military and commercial ports as well as walls and fortresses. Knowing that his work was to be used mainly by the military, the author addressed the civilian public, stone-masons, and carpenters, and described the proper use of a ruler and compasses, of the "petipe", which he defined as being "one line divided into a certain number of equal parts, to mean fathoms, palms or inches and lines" (fl. 204).

He described the use of the compasses, the set square, the board, the half-circle, the compass and showed how to calculate the calibre of the parts, how to figure the weight ratios of six metals (gold, lead, silver, copper, iron and tin), and showed their "signs" in the metal-board (fl.362).

His examples were followed by practical problems, like in his previous work, i.e. "using a compass to find the orientation of a wall", "to chart a wooded area and a stream in all its meanders", to draw paths, hills, ravines, stone quarries, vineyards, orchards, olive groves, trees, ploughed soil, gardens, meadows, rivers, creeks, swamps.

But moving ahead, in our previous work dated 1988–1989 we mentioned that seismic activity in mainland Portugal and on the Adjacent Islands was constant in modern times.

However, such activity was only recorded when, because of the magnitude of the earthquake of November 1st 1755, all parish priests received from their church superiors the questionnaire that was meant to record the consequences of the most recent and catastrophic earthquake and also to try and recall similar prior events.

Portuguese culture was no stranger to questionnaires like the one drafted by the Royal Academy of Portuguese History in 1721. Of the 12 questions on church history and their equivalent set on civil history, the search for manuscripts and unknown books, the record of memories, traditions, miracles, the results of visits made by clergymen the request to register "noteworthy things happening in the parish", with exactness, detail, and specifics "rare successes in the progress of each administration, with not military of political bearing", the questionnaire on public works, ecclesiastic and civil, the questions on the territories conquered by the Portuguese in Africa, America and Asia ("Laws and Customs of their people, news on animals, plants and minerals and description of the shores and inland elements" etc) showed proper observation and recording techniques.

A few years later, as he became a scholar of the Academy of History after the death of the Marquis of Alegrete, priest Luis Cardoso had already written a few of his books. After the questionnaire he published the *Dicionário Geográfico* (Geographic Dictionary) with his personal mark in the arranging of three separate questionnaires on places (29 questions), Mountain Ranges (13) and Rivers (21). He had various suggestions: did the locations have ancient privileges (...) or any other noteworthy elements (...) not present in the recent questionnaire; as for the mountain ranges he wanted to know the width and length, the names of rivers, villages, locations, fountains, ore mines, rocky beds or deposits of other relevant materials as for the rivers he wanted their name, their course, characteristics of the waters, relevant elements, had they ever yielded gold, etc, etc.

As for the work of Manuel de Azevedo Fortes, one may say that since 1721 Portuguese culture had shown the ability to have a mathematical reading of geography and to translate and project and accurate image of said reading. As an earth science, Geography is related to Astronomy and now leads to Geology without neglecting current events.

In the press of those days, with its existence documented in Portuguese archives and libraries, there were notes on the earthquake on Island Terceira dating back to 1612, a comment on the earthquake of Algiers, 1673, the *Notícia Verdadeira* on the earthquake of Rome, 1703 from the offices of António Pedroso Galrão that same year, the works of José Freire Monterroso Mascarenhas, the *Prodigiosas Appariçoens e successos espantosos vistos no presente anno de 1716,* works on the Palermo earthquake of 1726, of Naples in 1732, a comment on the Constantinople earthquake that same year, the work *Tratado de la Naturaleza, Origen y Causas de los Cometas con la Historia de Todos los que se tiene noticia haverse vista y de los efectos, que se les han atribuido (...) y con el Methodo de Observar Astronomicamente sus lugares aparentes, y hallar los verdaderos en el Cielo (...)* by jesuite Priest Joseph Cassani, published in Madrid in 1737, exploring the astronomic relations with the earthly events it reported. There were also the Portuguese-language works *Prognostico Novo do Cometa e mais impressoens meteorologicas do anno de 1737 ate o presente de 1742,* by Priest Vitorino Joseph, wich also addressed meteors, a record of the earthquake on the Madeira Island in several records on the earthquakes in North Africa, from the office of Miguel Manescal da Costa in 1751, another on the Tunis Earthquake in 1752, published the following year by the Alvarense office.

This is a list of the works published before the Lisbon earthquake of 1755, with various approaches. Some were published by the same Office that published the Collected Documents of the Royal Academy of Portuguese History at that time. The same documental sources yielded yet another work, written in Spanish with the revealing title of: *Pregunta, que ha un Geographo a un Artifice Architecto, sobre si los edificios de ladrillos son mas permanentes, que los fabricados de Piedras, y si las barras, y pernos de hierros son perjudiciales en las Piedras, ó favorables en las fabricas de ladrillos,* Seville, Joseph Padrino en la Calle Genova, s/d. All we need to add would be "in case of earthquake".

Even if we do not know the name of the author(s) of the "Pombal questionnaire", we can understand its genesis and its consistency with the cultural, technical and scientific debate going on in the Portuguese society. After comparing several records, traditional culture, standard observation, scientific perspective, moral exhortation, Portuguese society and culture were aware of seismic phenomenon, of the historical and geographical questionnaires which interlinked with other branches of the tree of knowledge and had institutionalized political action in the broader sense of the word. One did not, however, have a systematic record of all seismic occurrences. After the catastrophe of November 1, 1755, there were two reactions by the Portuguese society, they rebuilt and they drafted a scientific questionnaire, observing and theorizing about the phenomena had become extremely urgent and could not be postponed. Essential steps were being taken, stages overcome.

Paradoxes and ancillary items in the search for a global scientific perspective where human sciences made an essential contribution and can be a lesson even today. Sacrificing scientific questioning and exchanging it by immediate and occasional profitability and by preferring certain areas of knowledge may compromise the future of other scientific areas as well as the ones one is trying to promote. In the case of the Lisbon earthquake of 1755, its scientific knowledge in the area of geophysical and geological sciences was made possible because of the extreme exactitude of the record of this earthquake and of all similar occurrences known to the subjects of the questionnaire all over the kingdom. Earthquakes were handled like historical facts, with their own place, time and duration, their priors, circumstances in which they occurred and consequences. The related documentation may be considered a monument of Portuguese culture, one that challenges and defies inter-disciplinarity even today. This bears witness to such work.

As well as the works mentioned above, we would like to name a few more:

Further Readings

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The Great Earthquakes of Lisbon 1755 and Aceh 2004 Shook the World. Seismologists' Societal Responsibility

Karl Fuchs

1 Introduction

This year on 1st of November is the 250th anniversary of the Great 1755 Lisbon earthquake. It is the greatest earthquake in Europe with magnitude $M \approx 8^3/_4$ in historical time. It shook Europe physically and generated a shockwave within the society of the 18th century. It gave birth to seismology and disaster management. In the 21st century modern seismology together with civil engineers senses the societal shockwave which reaches the global, much more vulnerable society. The Aceh earthquake with magnitude $M \approx 9.0$ and its tsunami were similar in strength, extent and impact to the Lisbon 1755 event. We experience increasingly that we are dealing with a societal shockwave originating from a Low Probability Extreme Event (LPEE) on December 26th, 2004.

To understand the shock to society 250 years ago we remember briefly the 18th century. The time was characterized by the end of religious wars which harassed the previous century. There was an enormous desire that everything develops peacefully, not only trade and commerce but also philosophy and the starting natural sciences contributed to a stable world.

The world became calculable. Two eminent universal scientists stand for the spirit of the 18th century. Isaac Newton (1642–1727) and his laws allowed to predict the course of the planets and the fall of an apple. Gottfried Wilhelm Leibniz (1646–1716), (Fig. 1) the inventor of the differential calculus together with Newton, was trying to expand the notion of optimization from mathematical functions and physics into the metaphysics.

He asked the basic questions of philosophy: *Why is there something rather than nothing?* And: *Why is it as it is*? He answered it in the spirit of optimism prevailing in the 18th century with his Essay *Theodicée on the Goodness of God, the Freedom of Men, and the Origin of Evil* (Leibniz 1710) and he proved in the end that this must be the *best of all possible worlds*. This was a widespread

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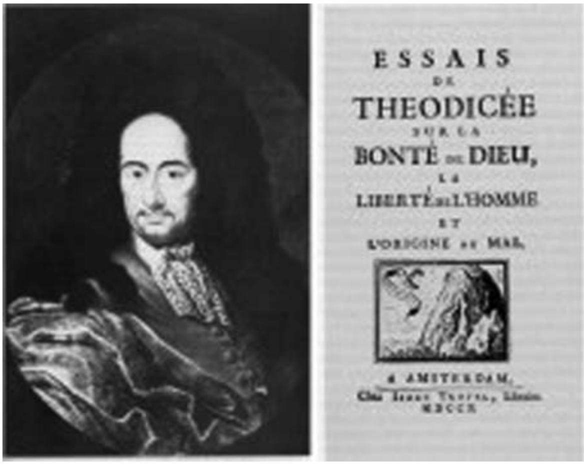


Fig. 1 Gottfried Wilhelm Leibniz (1646–1716) Philosopher, mathematician and theologist defended God in his "Theodicée" and proved that "this is the best of all possible worlds" (adapted from Leibniz, 1710)

feeling in Europe at the beginning of the 18th century. It was expressed concisely by the British poet Alexander Pope (1688–1744): *What is, is good* (Pope 1733/34).

In 1755, Lisbon was one of the most beautiful cities in Europe. The city retained some of its Moorish influence during the Middle Ages and Renaissance. It was famous for its wealth. It was full of palaces and churches and, because of commercial activity, it was one of the best known cities in the world since traders, particularly English and German, did most of the business in town. But Lisbon was also known as a city of the inquisition and was characterized as being the centre of superstition and idolatry (Dynes 2003).

Why should a seismologist be interested in the cultural environment in which "his" seismic waves are propagating? Why is he interested in the intellectual debate of the 18th century and in a comparison of the societal impact after the Aceh/Sumatra 2004 with the Lisbon earthquake 1755? Responsibility towards society was at the very roots of the science of seismology originating after the Lisbon earthquake 1755. Today the impact on society is a growing concern of seismologists involved in cooperation with civil engineers in disaster management, e.g. in the Mega-cities project of ILP (Wenzel and Bendimerad 2003) or in major research projects.

2 The Great 1755 Lisbon Earthquake

The earthquake occurred on November 1st, 1755 at 9:40 local time, 30 minutes later the Tsunami arrived in town! Both events came completely unexpected not only in time and space but especially also in magnitude (Fig. 2).

Fig. 2 Panoramic view of Lisbon before (*top*) and after the earthquake (*bottom*) (adapted from Kozak)

Today, 250 years after the event, we know a lot more about earthquakes, their distribution in space and in time. The earthquakes are located in narrow girdles or belts. As we know today, they occur mostly at the borders of about twelve plates which form the Earth's surface (Fig. 3).

These plates are in relative continuous motion to each other. Lisbon 1755 and Aceh 2004 earthquakes are located in the plate collision zones which are the sources of the strongest earthquakes on our planet.

The magnitude of the 1755 earthquake was recently reconstructed from the observations of the extent of damage collected immediately after the event by

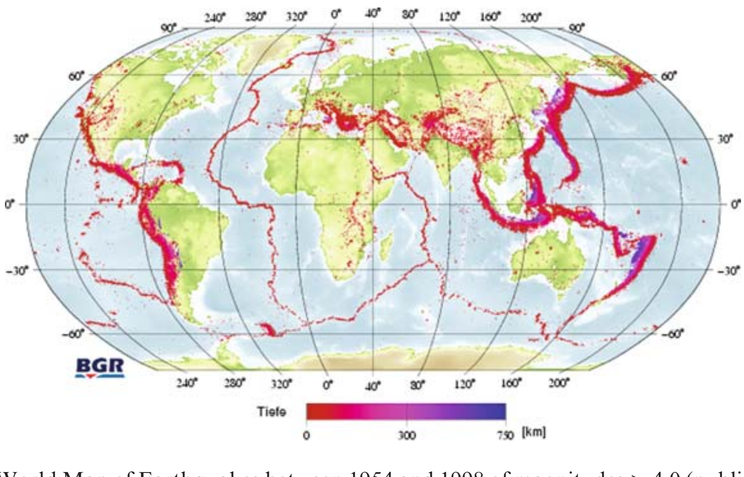


Fig. 3 World Map of Earthquakes between 1954 and 1998 of magnitudes > 4.0 (published by Bundesanstalt für Geowissenschaften und Rohstoffe/BGR, Hannover/Germany, modified and with permission by BGR). The earthquakes occur in narrow belts marking the boundaries of plates which form the surface of the Earth

order of Marquês de Pombal (1699–1782, called, Sebastião José de Carvalho e Mello) by questionnaires sent out throughout the mainland of Portugal and the Atlantic islands.

The answers are still archived in the Tower of Tombo, the national historical archive. The map of lines of equal damage (Fig. 4), according to the Modified Mercalli Scale (Richter 1958) reconstructed by Johnston (1996), formed the basis to estimate the magnitude as M $\approx 8^3/_4$ and to find a possible model for the focal processes with effects matching the observations from the 18th century.

There are a number of new approaches to improve the location and the mechanism of the source of the Great Lisbon earthquake, e.g. (Gutscher 2004), Baptista et al. 2003). The European plate is colliding with the approaching African plate. A thrust movement on a plane of about 16,000 km^2 broke with a relative movement of 12 m. This elevated the seafloor and generated the tsunami which reached Lisbon with a height of about 7 m in the trumpet like mouth of the Tejo. It flooded Lisbon about half an hour after the destruction by the earthquake. The tsunami spread throughout the Atlantic, it reached London harbour; the tsunami is reported from Norway, as well as from the African coast, it reached Madeira with a height of 4 m, the Canary Islands, the Azores crossed the Atlantic to the Caribbean Sea and East coast of North America with run-ups of 3–5 m on Barbados and Martinique (e.g. Baptista et al. 2003).

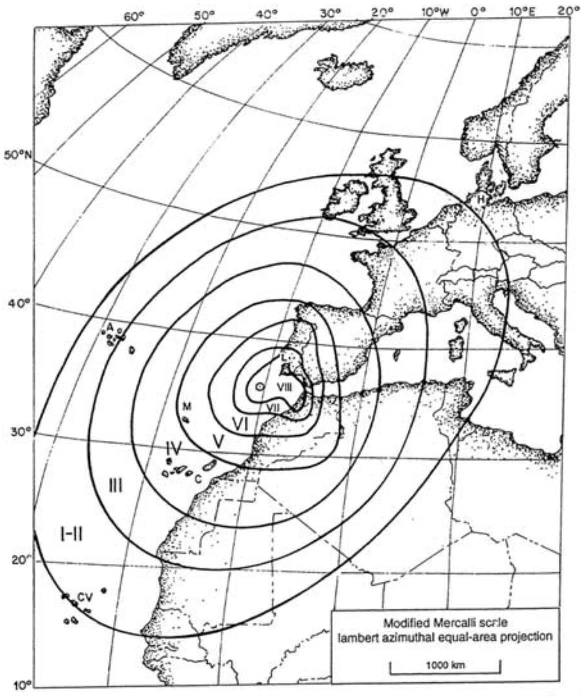


Fig. 4 Generalized isoseismal map of 1755 Lisbon earthquake. (adapted from Johnston (1996). Important isoseismal control points are labelled: A, Azores; C, Canary Island; CV, Cap Verde Islands, H, Hamburg; L, Lisbon; M, Madeira

2.1 Europe is Shocked

The earthquake was felt physically through large parts of Europe not only by the tsunami but also by long-period surface waves. In Paris the church bells started to ring and in England, northern Finland and Scotland seiches were observed and reported (Richter 1958). This happened long before the news spread out slowly by the media of those days and left Europe in a state of mental shock.

The Instant of Time: The amount of destruction and the number of fatalities were certainly parts of the shock inflicting messages. But the day and the hour of the earthquake contributed strongly to the heated and long lasting debates of philosophers, theologians, politicians and artists (poets, painters, composers). I dare to say, if the earthquake had happened 1 day before or after, it would also have been a catastrophe with a similar amount of loss, but it would today probably only be registered in the long lists of strong earthquakes in the text books of seismology. It would not have transformed Europe in a state of shock as observed. Why was this so?

Today the significance of the day and the hour of the 1755 earthquake are easily forgotten. The 1st of November was and is All Saints Day, the highest catholic festival; it was also judgment day of the inquisition. At 9 h 40 min, the moment of the earthquake, the churches were crowded to their capacities. 30 of the 40 churches collapsed by the earthquake killing the faithful, many of the survivors fleeing to the harbour were drowned by the incoming tsunami and the fire did the rest. 30,000 to 60,000 were estimated as dead victims. Lisbon was plagued by more than 500 aftershocks during the next 9 months.

The question immediately arose: how could God have allowed such a catastrophe to happen with his faithful believers? The question was discussed at Lisbon and throughout Europe by philosophers, theologians, politicians and artists.

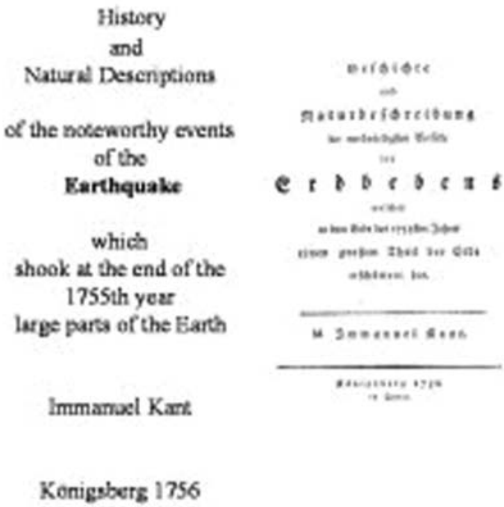
Philosophy: The Lisbon earthquake shattered the notion of the best of all possible worlds (G.W. Leibniz, A. Pope) drastically. Two famous philosophers Voltaire and Kant (Fig. 5) were struggling about earthquakes as natural phenomenon in contrast to the will of providence with both philosophical optimism and theological fundamentalism.

The news reached Voltaire (i.e. François-Marie Arouet; 1694–1778) about a fortnight after the earthquake by stage coach in Geneva. His almost immediate response was his attack on Leibniz's theorem of "This is the best of all possible worlds" vividly expressed in his *Poème sur le désastre de Lisbonne* (Voltaire 1956) and widely distributed throughout Europe. A few years later (Voltaire 1759) in Candide, Voltaire's hero in Lisbon, asked the question: *Is this the best of all possible worlds*? And he answered: *How would then the others look like*?

Immanuel Kant (1724–1804) published in 1756 his early essay . . . *The Earthquake which shook at the end of the 1755th large parts of the Earth* (Kant 1756) (Fig. 6).

Fig. 5 Voltaire and Kant are taken as representing the position of enlightenment towards earthquakes as natural phenomenon

In less than a year after the earthquake he gave a remarkable account of the physical observations in and around Lisbon and throughout Europe. He drew conclusions about the possible origin of the earthquake which show his close connections to other famous scientists like Bouguer (leader of the French expedition in Peru to determine the flatness of the Earth).


History
and
Natural Descriptions

of the noteworthy events
of the
Earthquake

which
shook at the end of the
1755th year
large parts of the Earth

Immanuel Kant

Königsberg 1756

Fig. 6 Title page of (Kant 1756). In less than a year after the Lisbon earthquake Kant wrote the essay. (Translated title page)

Kant was convinced that an earthquake is a natural phenomenon and not a moral subject. He wrote: If humans are building on inflammable material, over a short time the whole splendour of their edifices will be falling down by shaking. However, is this reason to blame providence for it (Kant 1756, p. 79)? Kant (as well as Rousseau) discussed earthquakes for the first time in a framework of disaster management, e.g. Kant continued: The inhabitants of Peru are living in houses which are walled only to little height, the rest consists of reed. Kant (ibid p. 86) finished his essay with a note to his king Friedrich II while the 7 years war was already impending – in short – Sire, we cannot yet prevent an earthquake, but you can prevent a war.

Trade and Commerce: Because of the role of Lisbon as an international commerce centre of that time, the earthquake was soon recognized as a catastrophe at several places of the large trade houses and their partners in London, Amsterdam, Hamburg and Venice which had invested considerable sums in Lisbon. In Amsterdam the news from Lisbon was received mostly as "miserable accident". "Therefore our stock exchange is today in extreme consternation". In London one took pain for fast support to remedy the most urgent need and after a short hesitation one did not protest the bills of exchange any longer. One accepted them to continue business and to minimize the damage of commerce. Thus in 1755 international financial aid measures succeeded to avoid a global collapse of commerce. The king asked both houses of Parliament for support which was granted without delay. Parliament grants 100,000 Pound Sterling and the king adds 50,000 Pound (Günther 2005). In contrast, in 2004 the earthquake and tsunami never really affected the stock exchange seriously. The insurances hardly registered the Aceh earthquake and the tsunami in the Indian Ocean because the affected countries took negligible insurance.

Theology: In the debate between earthquakes as natural cause and as moral subject, theologians throughout Europe, both catholic and protestant were inclined to take the Lisbon earthquake as "God's hand". In Lisbon during the disaster management the government through Marquês de Pombal got sturdily into conflict with the Jesuits adhering to the earthquakes as a moral theme. As an example from the protestant part of Germany, at Hildesheim the council of the town decided under the impression of the earthquake to introduce special hours for praying. The clergyman and preacher to the council Johann Carl Koken (1756) gave a sermon on Ps. 104, 32 "The Lord's voice in the earthquake". The effect on the faithful was so strong that he decided to publish it. Thereby he tried to combine the factual interest on the origin of the earthquakes with the reminder "for a true improvement of life and the necessary prudence in performance". The author intended to rewrite a second improved version *Seismotheology or recognition of God in the earthquake*. Here he wanted to combine the natural conditions with a proof of God's existence.

The 1755 Lisbon earthquake is also considered by some as the birth of modern seismology. It was the first disaster to arise coordinated state emergency response and forward looking comprehensive effort for reconstruction including mitigation efforts to reduce future disaster consequences (Dynes 2003).

2.2 *Birth of Modern Seismology*

Marquês de Pombal (Fig. 7) was given responsibility for the emergency management and the reconstruction of the capital of Portugal by order of the King José Manuel I. of Portugal (1714–1777) after the Lisbon earthquake (Kendrick 1956).

Fig. 7 Monument of Marquês de Pombal at the end of Avenida da Liberdade. He can be regarded as a founder of seismology (photo by the author)

The disaster management in 1755 had many elements which are still used today. One important contribution was the systematic collection of quantitative information on the distribution and the amount of shaking produced by the earthquake.

Questionnaires were sent out to priests, convents and officials with the following questions: How long did the earthquake last? How many shocks were felt? What kind of damage was caused? Did animals behave strangely? What happened in wells and water holes? Such questions are also included in today's questionnaires send out after earthquakes by state surveys.

Even today in the age of digital networks of seismometers the questioning of the population is an important and useful part of information on the source of the earthquake, its location, its intensity and approximate magnitude. Shake maps (Wald et al. 1999) are produced by computer simulations almost in real time shortly after the earthquake are the modern form of the 1755 collection of damage information for disaster management.

After the immediate concern for the burial of the 30-60.000 dead bodies, the care for the wounded and for food and taxation, security measures, Marquês de Pombal had to face the clash between the conflicting notions for a large earthquake as a natural phenomenon or as a moral theme. Malagrida, a leading Jesuit and successful preacher, went in his sermons against the notion of natural causes of earthquakes (Livermore 1976). His message to the people was humility and repentance to God; in the 9 months long series of strong aftershocks he insisted on this not to be the time for reconstruction and rebuilding because *the Lord is still shaking the Earth*. These sermons were a massive impediment to the reconstruction measures at Lisbon. In the end Marquês de Pombal won the struggle for power. Malagrida went to trial was convicted and ended on the wheel. That led to the termination of the rule of the inquisition in Portugal (Kendrick 1956). The Marquês could continue with the farsighted rebuilding of Lisbon with admirable broad avenidas (Avenida da Libertade) possibly even before the construction of the Paris Boulevards (L.A. Mendes Victor, personal comm. 2004).

In summary, seismology in 1755 started as earthquake disaster management with a responsibility towards society. Earthquakes were begun to be understood as natural phenomena. The notion of earthquakes as a moral subject had to be overcome even against strong resistance. The call for humility towards God was an impediment for disaster management.

3 The Shockwave reaches the 21st Century

It took more than a century until seismologists discovered seismic waves also as the most effective tool to illuminate and to study the structure and physical properties of the shallow and deep interior of the earth. Artificial earthquakes were used to explore the deposits of oil and gas on which our civilization depends so much. Every disaster produced by large earthquakes reminds seismologists with growing concern that the seismic waves do not only illuminate the Earth's interior but that they hit the human civilization erected on this planet. Seismologists applied their skills also to the reduction of disasters. This let to the establishment of the International Decade for Natural Disaster Reduction (IDNDR) proposed by Frank Press (1984).[1]

3.1 Modern Earthquake Disaster Management

Seismologists become aware that the society and its structure of civilization have changed drastically within the past 250 years. It has become more complex and more vulnerable to the impacts of seismic waves. The main reason is the increase in population density and higher technical

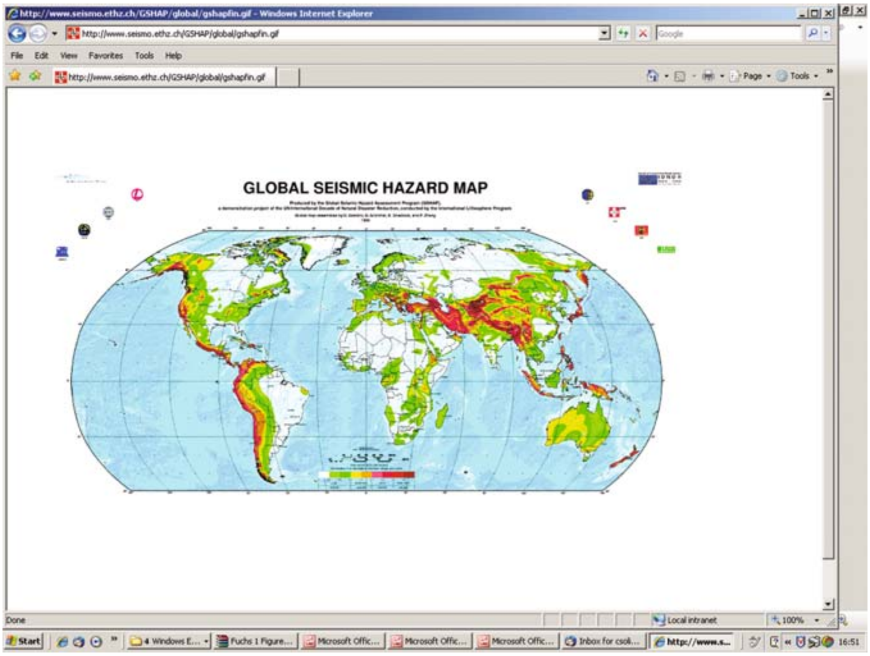


Fig. 8 GSHAP – World Map of Seismic Hazard. It displays intensity that can be expected with a return period of 475 years. Global Seismic Hazard Map forms base of risk mitigation (adapted from Giardini et al. 1999)

[1] UN-Resolution 42/169 to establish IDNDR: www.un.org/documents/ga/res/42/a42r169.htm

sophistication of buildings and the complexity of the infrastructure. This development is still accelerating.

Modern management of earthquake disaster is based on a close alliance between seismologists and civil engineers. There is much more information about earthquakes, their origin and their locations as well as more advanced tools for hazard estimation at hand.

Since the precise prediction of earthquake time, magnitude and location is presently beyond realization, earth scientists are concentrating on the forecasting of the maximum possible hazard (PGA = Peak Ground Acceleration) at a given place on the earth by earthquakes within limited windows of time and space. The recurrence rate of earthquakes in seismic active regions is estimated from the magnitude-frequency relations (Gutenberg and Richter 1944) of earthquakes. In international cooperation seismologists have constructed a world map of global seismic hazard (Fig. 8) within the GSHAP-program (Giardini et al. 1999) of the International Lithosphere Program (ILP). An example from the global GSHAP map is the regional hazard map for the D-A-CH countries in Europe (Fig. 9).

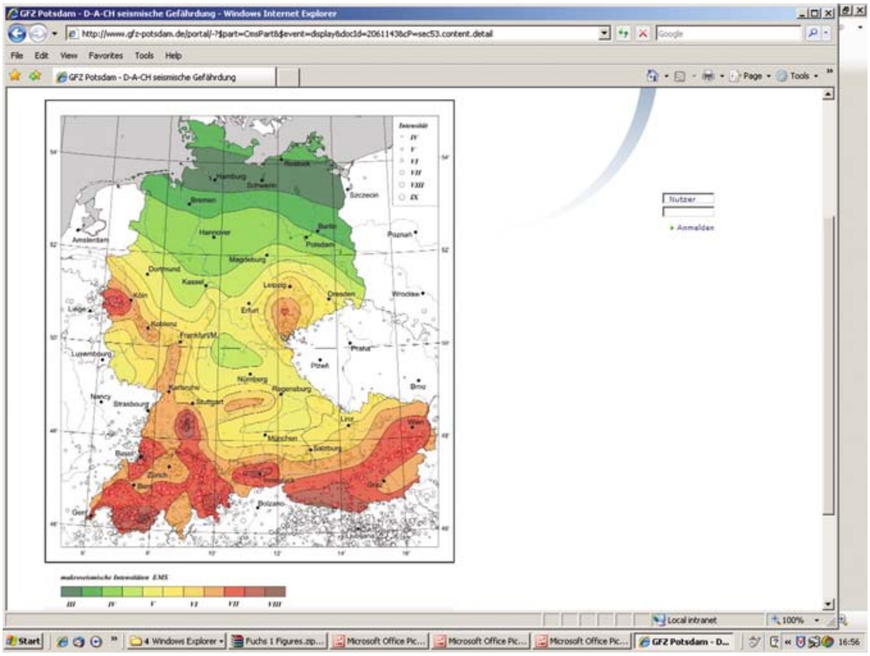


Fig. 9 Earthquake Hazard Map for the D-A-CH States (Germany, Austria, Switzerland). The earthquake hazard is displayed as intensity values with a non-trespassing probability of 90% within 50 years. (Grünthal et al. 1998; with kind permission of the authors)

The real contest for the disaster management, however, is not the estimation of hazard but the risk for loss of life and property and its possible reduction:

$$\text{Risk} = \text{Hazard} \otimes \text{Vulnerability} \otimes \text{Inventory}, \tag{1}$$

where $\otimes$ stands for convolution.

In contrast to hazard maps which give an assessment of the physical damage potential, risk maps asses the possibility of a loss (fatalities and economic) resulting from the exposure to a hazard. Risk maps are obtained from hazard maps by convolution of probabilistic hazard estimates with the vulnerabilities of the objects in the inventory and the "value" of the objects of the Damage Inventory.

This is shown here for the case of Germany (Fig. 10) as a first draft adapted from CEDIM (2004). Although the hazard of maximum possible ground acceleration at a certain location is given by nature and cannot be reduced, the goal is to reduce vulnerability of the "Inventory" in general by preparedness.

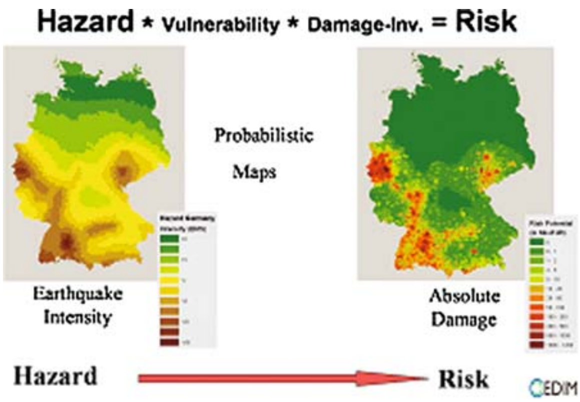


Fig. 10 From hazard to risk. (*Left*) Probabilistic seismic hazard map of Germany (compare Fig. 9). (*Right*) Seismic risk map of Germany (adapted from CEDIM (2004))

The birth of seismology was closely connected with service to society and struggle for a rational societal response and preparation for a large natural disaster of low probability.

3.2 Aceh/Sumatra Earthquake and Tsunami on the 26.12.2004

On December 26th 2004 most seismologists were exposed almost in real time to the media coverage of the tsunami generated by the great Aceh/Sumatra earthquake. They knew the earthquake had occurred at a major collisional plate boundary forming a subduction zone, well documented by frequent and strong earthquakes. At least since the advent of the concept of plate tectonics the subduction zone at the Sumatra trench was known as a potential site for strong earthquakes with the possibility of accompanying tsunamis. Numerical models of earthquakes in the region (Soloviev and Ismail-Zadeh 2003) tell us about a possibility of great earthquakes in the region.

In this sense the tsunami of Aceh was predictable; failure to establish a warning system before the event is a blow to the professional self-esteem of Earth scientists (IUGG Commission on Geophysical Risk and Sustainability 2005). A tsunami warning system which became established in the Pacific was not installed in the Indian Ocean before the event. The experience of this disaster called many seismologists back to the roots of their discipline and profession: their responsibility towards society.

This is the history of the propagation of the tsunami after the Origin Time (OT) of the Aceh earthquake (Table 1) (6:59 a.m., local time on December 26, 2004):

This history provides ample evidence that there would have been enough time for an early warning, at least for Thailand, Sri Lanka, and the western part of the Indian Ocean, had an early warning system been installed with sufficient regional and local communication to the authorities and communities. Fast magnitude determination is an urgent requirement. The Sumatra/Aceh earthquake with a magnitude of 9.0 (USGS 2004) [2] to 9.3 (Stein 2005) was the third

Table 1 History of the propagation of the tsunami of the Aceh earthquake after the Origin Time (OT: 6:59 a.m., local time on December 26, 2004)

Time after OT	Tsunami propagation
8 min	the Pacific Tsunami Warning Centre (PWTC) issued a warning
15 min	the PWTC estimated the magnitude was to M = 8.0 andreversed its warning into “No Danger for the Pacific Ocean”
31 min	the coast of Aceh/Sumatra
1 hour	Thailand (same time the PWTC updated the magnitude to M = 8.5 and issued a tsunami warning to Thailand)
2 hours	Sri Lanka
2½ hours	India
3½ hours	Maledivia
4½ hours	Harvard updates magnitude to M = 8.9
7¼ hours	PWTC issues warning about tsunami danger for Africa and Madagascar via US State Department
7¼ hours	East Africa
14½ hours	PWTC warning: the tsunami has reached the Pacific

[2] USGS (2004): http://earthquake.usgs.gov/eqinthenews/2004/usslav/

largest during the last century. The fault had a length of 1200 km and the fault plane with 120,000 km^2 covered an area nearly as large as Greece. Modelling estimated the effective tsunami rupture with a length of 600 km and the average rupture velocity of 2.5 km/s, i.e. 9000 km/h (Ammon et al. 2005). The rupture started at a depth of 7 km with a slip of 20 m. The rupture surface became visible by the location of numerous aftershocks.

In comparison, the two earthquakes of Lisbon 1755 and Aceh 2004 were located at prominent plate boundaries, however, with different relative velocities of the colliding plates. The Australian plate is subducted with about 7–9 cm/year at the Sumatra arc, while the African Plate and Eurasian plate have a relative velocity of only 0.5 cm/year.[3]

The tsunamis following the earthquakes of Lisbon 1755 and of Aceh 2004 are comparable in strength, extent and impact. Both reached global dimensions crossing the Atlantic or the Indian Ocean, respectively, but they hit different societies in 1755 and in 2004. The news on the catastrophes spread within the societies at different speeds. In 1755 the news travelled at the velocity of the stage coach while in 2004 the catastrophe filled the TV-screens almost in real-time.

The shock wave which hit the society of the 18th century is now reaching the society of the 21st century. This society is not the same any more. Have we learnt our lesson as society? What have we learnt as seismologists?

4 Kant's Nutzen of Earthquakes in the 21st Century

Kant in his 1756-essay included a chapter *Von dem Nutzen der Erdbeben* (On the usefulness of earthquakes) (ibid. pp. 78–82). The title alone caused concern and disapproval. We can no longer follow his conclusions based on a concept of Earth's internal heat different from our knowledge. But like him we may also look for lessons (Nutzen) of present earthquake catastrophes. Let me propose two useful lessons somehow with direction and hope for the future.

4.1 "Nutzen": Enhancing Global Conscience

The first lesson from the great Aceh earthquake is the observation that a Shock-catastrophe following a large earthquake enhances – with the help of the media and the involvement of victims from "western" countries – a global conscience much more than the "creeping" catastrophes (aids, hunger, thirst etc.) which carry a much higher yearly death toll.

These "creeping" catastrophes of our century simply do not make it to the front-pages of the media. After the Aceh catastrophe there was a remarkable

[3] Nuvel calculator: www.ldeo.columbia.edu/~menke/plates2.html

global consciousness of solidarity. A "tsunami" of aid-measures, governmental and non-governmental, was flooding into the affected countries, sometimes more than they could take. Could Aceh lead to the birth of a sustainable strategy of global compensation? (Richard 2002) How does the global village react to the catastrophe of the hurricane *Katrina*? Or do we have to wait for the next catastrophe?

The Lisbon earthquake 1755 was the first modern disaster that compelled the state to oppose the notion of supernatural causation and accept responsibility for the reconstruction of the city. The late political theorist Judith Shklar (1990) noticed this and wrote:

> The modern age has many birthdays. One of them, my favorite, is the Lisbon earthquake of 1755. What makes it such a memorable disaster is not the destruction of a wealthy and splendid city, nor the death of some 10 to 15000 people who perished in its ruins, but the intellectual response it evoked throughout Europe. It was the last time that the ways of God to man were the subject of general public debate and discussed by the finest minds of the day.

She uses the public response to the Lisbon earthquake to illustrate:

> how people who once regarded certain kinds of suffering as misfortunes, "acts of God", came to view them as injustices caused by the action or inaction of the powerful.

The Aceh tsunami of the 2nd Christmas Day 2004 overwhelmed the people at the Indian Ocean without any alarm. The warning system in operation in the Pacific Ocean had sent late messages which did not reach the people living near the coasts nor the tourists at the beaches. No similar, even only rudimentary network existed for the countries at the Indian Ocean in time. The difference between the Pacific and the Indian Ocean is a clear paradigm for societal injustice inflicted by *action or inaction of the powerful* – sometimes driven by ignorance – within our global village.

After the Aceh event there were some suggestions that the extent of the catastrophe could have been caused by the absence of humility or by disrespect for nature. It is interesting to note that within 250 years the struggle between the notion of earthquakes as "*God's hand*" and its explanation as natural phenomenon was substituted by blaming mankind for disrespecting the laws of nature and announcing nature's punishment.

We must be careful not to be subjected to a new kind of inquisition with a new call for humility, this time not towards God but towards nature. Such a call for humility implies certain arrogance that we could have escaped the disaster if society had behaved according to the "*laws of nature*". Some calls are even heard asking "not doing everything which is technically feasible". This cannot be the lesson in the 21st century. Such an attitude does not help to prepare for the next LPEE. On the contrary: the parole of the day should have been "We cannot prevent large earthquakes and tsunamis; but we should be doing everything which is feasible to protect against their consequences".

4.2 *"Nutzen": Rationality and Morality – Aceh as "Early Warning" to Prepare for Future Events*

The second lesson from the Aceh disaster is the question whether the liberation from providence or from God during enlightenment by rationality did let us forget to carry morality with us. It is also the recognition that we should take it as "early warning" for similar extreme events which will hit our civilization in the future.

We realize that natural catastrophes uncover hidden societal instabilities. Rational prevention meets regionally – e.g. New Orleans – as well as globally – e.g. Indian Ocean – moral problems of societal injustice, i.e. how the rich are dealing with the poor.

We have to realize that we got off cheaply with the Aceh earthquake and tsunami. Other places will be affected much more disastrously: Tokyo, San Francisco, Los Angeles and Lisbon. Numbers of dead will move into the 100,000s and the property damage up to $\$10^{12}$. A global depression in the economy cannot be excluded (Keilis-Borok 2003). How will our society deal with such low probability extreme events (LPEE's)? What is the contribution of seismologists?

If we continue with the classical methods of hazard assessment and risk we will always be late with our advice mostly after the disaster (when the child has fallen into the well we advise to cover the fountain). We will end up taking the LPEE's as misfortunes and thus fall back into the pre-enlightenment period were natural catastrophes had to be taken as misfortunes. But our vulnerable society cannot afford this attitude, much less than previously in 1755, the risk connected with future LPEE's to our civilization becomes increasingly high. Prevention in the management of natural catastrophes requires urgently a sustainable strategy for the stability of buildings and infrastructure, as well as for the stabilization of society by justice, both globally and regionally.

5 Ranking Urgency of LPEE's for Decision-Makers

It is not only a question of global societal injustice. Even the well-to-do countries of Europe are without tsunami-early warning system in the Atlantic Ocean or in the Mediterranean! Not that they could not afford it, but these extreme events are just so rare that their recurrence time is not easily to be predicted for all practical purposes. This is not only a dilemma of Europe but of a civilization relying so strongly on a technical foundation.

Therefore the problem is much more fundamental. The decision-makers have not only a right to know from the experts how to protect beaches, harbours and towns against the maximum possible hazards but also to obtain professional advice on the *urgency* of the protection measures. Hazards and risks must be ranked according to their urgencies and that involves in one way or the other an *estimate of time* of occurrence.

This is the experts' dilemma for the LPEE's. Measures for disaster reduction require preparedness. Since we cannot be prepared for an arbitrary long time and for all possible places on Earth, forecasting of LPEE's has for practical reasons to be restricted to time and space windows. It requires also the definition of a lower magnitude of societal relevance. The LPEE's are so rare that classical statistical estimates of their recurrence periods fail, the error bars become as large as the estimated period itself.

Statistical methods are based on counts of numbers and not on physics. The only way out of the dilemma is to introduce physics (Knopoff 1999) into our efforts to forecast earthquakes. This means the introduction of a better knowledge of the intrinsic physical rock properties and state of the Earth's crust under deformation and stress.

How could Earth scientists in interdisciplinary alliance provide useful advice to decision-makers for the case of these LPEE's? Leon Knopoff (2005; personal communication) points to the really crucial question:

> Society asks why scientists cannot predict very large earthquakes such as the December 26, 2004 Sumatran earthquake. Scientists and engineers have been able to bring back samples from the moon, to eradicate smallpox, to put a television set in every home in the developed and in many homes of much of the developing world, to create computer games, to endow millions of users with cell phones, and to open doors to lobbies of great corporations automatically. . . . We should ask, "How successful has science been in solving really difficult, societally practical problems in recent years, given the above successes?" For example in medicine, why haven't the problems of finding the cures of malaria, Alzheimer's disease, HIV/AIDS, etc. been solved by now, since we already understand the underlying principles, since for example, we have known about DNA for 50 years?

Knopoff argues that the underlying basic physical principles for large earthquakes are also known and that the absence of success is mainly due to political rather than to scientific reasons.

New observations of material properties and behaviour have to supplement classical observations such as earthquake activities. New dense continuously recording GPS arrays with high sampling rate allow determining deformation rates in seismic active regions. Intrinsic rock properties at depth have to be obtained in situ by deep drilling into active faults (e.g. drilling into the San Andreas Fault with observatory at depth, SAFOD[4]). Hidden faults must be detected by Deep Seismic Sounding techniques. Powerful high-speed computers are needed for high resolution 3D models of stress and deformation patterns in complex communicating fault systems for improved hazard and risk simulation, including fast estimates of earthquake magnitudes. (Table 2).

[4] SAFOD within ICDP: http://www.icdp-online.de/sites/sanandreas/index/

Table 2 Improvement of Hazard & Ranked Risk Maps and Early Warning supported by fast high-resolution modelling tested by new observations (SAFOD = San Andreas Fault Observatory at Depth; ICDP = International Continental Drilling Project)

Observables	Tools and Data
Strain & Stress	GPS-networks, Satellites
	World Stress Map
Rock Properties, Temperature, Fluids	Deep Drilling (SAFOD, ICDP)
	Laboratory Experiments
Active Fault Systems	Geological & DDS Refelction & Palaeoseismic Surveys
Geometry & Seismicity	

The goal is to forecast the future behaviour of the system to increase the predictability of large earthquakes. A unified approach will have to be developed which combines the most advanced observations with sophisticated modelling and simulation algorithms on powerful computers. The computer analysis will have to take into account most realistic models of the properties of fault systems in their geological settings. The forecast must include a definition of a window in space, time and magnitude, i.e. an element of probability for the occurrence of the event. This unified approach will lead to a dynamical estimate of hazard changing with time. Ultimately, this dynamic hazard estimate must then be combined with the monitoring of the Damage Inventory and its Vulnerability changing in time to arrive at a basis for a *ranked* risk estimate.

The recent hurricane *Katrina* drowned New Orleans on 28/29 August 2005. This and the subsequent disaster demonstrate that failure in prevention and disaster management occur in spite of early warning and requests for governmental action. Mark Fischetti (2005) wrote on 2 September:

> Watching the TV images of the storm approaching the Mississippi Delta on Sunday, I was sick to my stomach. Not only because I knew the hell it could unleash (I wrote an article for Scientific American (Fischetti 2001) that described the very situation that was unfolding) but because I knew that a large-scale engineering plan called Coast 2050 – developed in 1998 by scientists, Army engineers, metropolitan planners and Louisiana officials – might have helped save the city, but had gone unrealized.

This catastrophe was forecasted (see also: Rademacher 2005a), and even a prevention plan "Coast 2050" brought to the attention of the Congress in Washington, D.C., as a joint effort of engineers, scientists, disaster managers and politicians. The plan got delayed and reduced by other pressing needs. Therefore, the catastrophe became a vivid paradigm for the inaction of the powerful in modern time in one of the most developed country. Scientists, engineers and disaster managers will have to answer the question whether and how to exert their responsibility beyond their own realm of competence. Realm of competence and realm of responsibility are not the same. It is not enough to produce a plan and to submit it to parliament or government. Responsibility does not end until the goal is reached. You have to follow it through even in complex situations against political and other resistance.

> Of course, things are complicated –. But in the end every situation can be reduced to a simple question: Do we act or not? If yes, in what way (Burdick 1964).

6 Summary

The Great Lisbon earthquake 1755 shook the society of the 18th century and led to the birth of modern seismology. Responsibility towards society in the management of disaster and prevention of future catastrophes was at the roots of this new science.

250 years later earthquakes of equivalent magnitude shake the much more vulnerable society of the 21st century with its denser population and highly complex technical civilization. Seismologists in alliance with civil engineers and disaster managers are challenged to provide societally practical advice for decision-makers to deal with LPEE's.

Two main lessons have to be learnt from the comparison of the two disasters at Lisbon 1755 and at Aceh/Sumatra 2004, both with accompanying tsunami.

(1) The Great earthquake catastrophes remind our society that it must continue in the effort started at Lisbon 1755 to take responsibility for the management of disasters inflicted by natural catastrophes with the most modern tools available today, from improvements of buildings, provisions, early warning to rescue and reconstruction. Since the extreme events affect our society worldwide the responsibility has to be exercised globally. It is a continued battle to reduce global injustice. It appears that the great earthquakes can generate such a global conscience much more effectively than the "creeping" catastrophes which hound the society of the 21st century.

(2) Scientists must recognize that LPEE's are a special challenge today. Because of their rareness it becomes exceedingly difficult to estimate their reoccurrence time from statistical analysis. Probabilistic prediction has to be fed by physics. New observations of intrinsic physical rock properties and behaviour have to supplement classical observations such as earthquake activities. New dense continuously recording GPS arrays with sampling rate allow determining deformation rates in seismic active regions. Intrinsic properties of rock at depth have to be obtained in situ by deep drilling into active faults. Computational high-resolution models for stress and deformation in communicating fault systems should be developed. The goal is a more reliable and even a ranked estimate of hazard and risk for large earthquakes. In a unified approach most advanced observations

together with sophisticated simulations will allow to forecast the future behaviour of the system, possibly even to improve the identification of the probability of their occurrence in a realistic window of time, space and magnitude.

Both goals – worldwide responsibility and the reliable forecast of the hazard and ranked risk for large earthquakes – are not to be achieved in a simple way. It requires the ingenuity of the best of scientists equipped with the best tools society can provide.

The hurricane Katrina taught the community of scientists, engineers, disaster managers that it is high time to find a new way of communication among themselves, with politicians and society to transform plans of high priority into reality for the sake of humankind.

Realm of competence and realm of responsibility are not the same. We must decide whether and how to extend and put into effect our responsibility beyond our own realm of competence.

In summary, our society has still to learn after the Copernican revolution that we are riding a restless, dynamic and risky planet. It is an illusion that our planet offers to us a static "natural" home with a solid platform to which we can return to develop our society (Rademacher 2005b). As seismologists, individually and in our associations, let us work hard, convincingly and with persistence at the forefront of science and engineering ingenuity to achieve the realization of the dream of the late Bruce Bolt (1996):

> The goal is to have a situation where we can say
> "Let the Earthquake come!"

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Seismic Engineering Contributions and Trends to Face Future 1755-Events

T.P. Tassios

1 Introduction

(a) The day of 1st November 1755, Voltaire was in Geneva. The news of the tragic event of Lisbon shocks that much the European intellectuals, that a lot of philosophical questions became fashionable again. Voltaire, however, nourished no hope; in his poem on the destruction of Lisbon he writes *Here is our hope/everything will now go well/This is the illusion*! I will maintain that Voltaire's views were not confirmed by Seismic Engineering: very many things went well indeed; despite daresome modern constructions, and despite a remarkable increase of density of population, safety against earthquakes was greatly improved.

(b) It should however be made clear that Seismic Engineering is only one component of the broader Seismic Risk Mitigation organisation. Its great value can not be appreciated without a **simultaneous** (i) further rationalisation in Seismology, (ii) appropriate levels of seismic preparedness, and (iii) adequate political awareness and availability of funds. More specifically, no advancement in seismic engineering can be of any help, without a post-quake fire-protection or without a broad public educational plan against panic.

(c) This being said, the main developments in the field of seismic engineering will be enumerated in what follows. All branches of seismic engineering will be briefly reviewed, including city planning, materials' technology, structural engineering (mainly in buildings), geotechnical engineering and mechanical modification of damping/period/and mass-motions. In doing so, only developments of the last six decades will mainly be considered – although earlier improvements in aseismic conceptual design are recognized to be of major importance.

I will also maintain that all these spectacular developments in Earthquake Engineering are completely deprived of any **side-effects**, as opposed

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to several other branches of Science and Technology, which may be used for unethical purposes too. From this point of view, Seismic Engineering seems to be the most clean and sustainable Technology...

(d) Nevertheless, a lot remains to be done; a lot of new knowledge has to be acquired, and considerable uncertainties should be rationified in the field of Earthquake Engineering.

That is why, occasionally, during the enumeration of positive developments, some problems will also be brought up, for which we should be worried about.

Unfortunately, much to my disappointment, I will not elaborate (as I should) on the ethical/political issue how this new knowledge will be brought to the benefit of hundreds of millions of people living in undeveloped seismic areas around the world. The very frequent hecatombs of the poor under seismic debris, is but another form of the Global Social Injustice.

2 City Planning

Within a broader sense of Engineering, we will first consider the progress made in Urban design, as another means of seismic risk mitigation.

2.1 New Buildings

Since our main scope is the final protection of human lives, the following urbanistic parameters have been broadly recognised as very important.

a) Architectural concept: Better internal escape roots (from inside to outside space) are favoured.
b) Broader and more frequent streets, both as a refuge and as a traffic facilitation, may reduce losses during evacuation.
c) Land use: Restrictions regarding the location of hazardous installations (such as fuel dealers, storage of inflammable items, etc), may drastically reduce post-quake collateral hazards.
d) Availability of open urban spaces (public squares, groves etc.) are recognised as basic components for seismic risk mitigation, both as refuge areas and as temporary settlement sites.
e) The same scope is served by provisions for collective uncovered spaces within each urban block (lawns, grass plots etc).
f) Streets should be large enough to facilitate escape of inhabitants and to accommodate debris of the houses of both sides. It is remarkable that the Italian Aseismic Code of 1909 required 10.0 m wide streets in seismically vulnerable cities. A Spanish Committee after the Granada earthquake (1886) was as demanding as to require that *the width of each street be the double of the maximum permitted height of the houses* (in Barucci 1990, p. 171).

g) Confined and disciplined public utility networks may drastically reduce collateral damages after an earthquake.
h) Minimal width of interspace between adjacent buildings (aseismic joints) annihilates the unforeseeable consequences of impact.
i) Last but not least, city planning regulations tending to reduce the density of population directly reduce seismic risk, for a given average structural vulnerability of buildings.

2.2 Existing Buildings

City planning regulations are also an important tool for pre-seismic interventions on existing buildings:

a) In case an existing building is highly vulnerable and disproportionately costly to rehabilitate, appropriate incentives may be given for its demolition and expropriation paid by means of plot-ratio transfers to other developing urban areas (the right for such transfers being commercially negotiable as well).
b) Similar provisions may encourage the possible demolition of one or two upper storeys of middle height buildings, in order to reduce seismic loads.

Fanatic structural engineers may underestimate the importance of such measures, but several studies and post-quake reports have confirmed their importance. Moreover, all these city planning measures offer additional societal profits in environmental and quality-of-life terms.

3 Materials

For the sake of completeness, some developments in the field of building materials will also be mentioned here; after all, a structure is a system of purposely organised material-elements.

a) **Concrete** in buildings, just after the 2nd World, was not that much resistant; compressive strengths around 20 *MPa* were prevailing. Besides, inadequate mixing and hand-placing resulted in rather frequent construction gross-errors, selectively brought to light by earthquakes.

   Against such a situation, the following developments have contributed to a considerable increase of seismic safety:

   – Average compressive strengths of concretes used in buildings were **doubled**, within half a century; for small house-buildings, where minimal code-requirements are governing the design, higher concrete strengths result in higher ductilities (for given transverse reinforcement and axial load values).

- Chemical additives have made possible good concreting, to face the considerable increase of steel ratios.
- The use of pumped concrete was generalised: Uniform placing and more efficient construction were enhanced; besides, the detrimental role of frequent cold joints was minimised.

Here again, some fanatic Analysts of finite elements may not be convinced about the paramount importance of these decisive developments. However, thousands of post-seismic inspections reveal that earthquakes are maniacs in . . . identifying local gross-errors!

b) **Steel reinforcements** have undergone several developments:

- Smooth bars are not anymore used. Thus, serviceability requirements were better observed. On the other hand, enhanced bond, tends to reduce the post-yield rotational capacity of critical cross sections; however, this is not the case for large plastic deformation under cyclic conditions (since under such conditions, bond is drastically reduced anyway).
- Steel strengths were increased; consequently, higher steel ratios can be better accommodated in slender building elements.
- The real problem, however, was the considerable decrease of the available uniform plastic elongation (ε_{su}) of the modern categories of steel, as compared to those available twenty years ago; similarly, the strength-to-yield limit ratio was also drastically reduced. Both these developments were not at all favourable to the ductility of structural elements. During the drafting of the CEB-FIP Model Code 90, we had the opportunity to vividly discuss the issue with the European manufactures of steel reinforcement. The outcome was positive: The actual production of the steel category S seems to optimise the benefits of high strength, considerable hardening, sufficient ductility and, above all, weldability.
- Industrialisation of sets of reinforcements is another positive development. Not only in economical terms but, mainly, as a way to avoid gross errors in detailing, and to guarantee fixed positions of stirrups against gradual sliding – when longitudinal bars tend to buckle. And this is another (numerically unpredictable!) good result.

c) **Fiber reinforced polymers** have also considerably contributed to better mitigation of seismic risk, especially in the field of structural interventions (repair or strengthening) of existing RC structures. It is worth noting here the apparent paradox of producing a ductile system, combining two brittle components (concrete and FRP, see Section 5b). We should however be worried about some hasty use of some of these materials, by inappropriate personnel and in applications hardly covered by regulatory documents, scrutinizing available scientific evidence rather than mercantilistic pressures.

d) **Masonry blocks**, specifically manufactured for reinforced masonry structures, is another positive development (comp, Section 4c, iii) provided that the requirements of EC 8 are observed.
e) **Modern dampers and isolators** were developed (see Section 7) making use of modern materials employed in military and aerospace engineering.

4 Design of New Structures

4.1 Analysis

4.1.1 The Times of Innocence

a) To go from seismic actions to seismic action-effects acting on building elements, a logical jump was needed: how an Act of God could be understood as a tangible load? Fortunately enough, D'Alembert[1] forces were conceived about the same time; thus, to translate an earthquake into inertia forces acting on the mass of a structure, a nominal maximum seismic acceleration was needed. Such accelerations were first estimated by means of some simple calculations based on fragments of large amplitude recordings of seismographs[2] (assumption of sinusoidal curves, double differentiation). It seems that as early as in the end of 19th century, seismic acceleration close to 0.30 g (Mino-Awari 1891) were estimated this way in Japan, and they were confirmed during the 1923 Tokyo earthquake. Thus when F. Omori (1900, according to Polyakov 1974) established the basic equation

$$F = m\,.\,\ddot{x}_{max}(1)$$

it was very clear that such a maximum ground acceleration $\ddot{x}_{max}$ should be imposed to the structures.

b) However, the Structural Engineering community was not very keen to endorse such large values;[3] some kind of structural feeling regarding acceptable inelasticities might have inspired such a daresome attitude of Engineers. Besides, static interpretation of stability of small height rigid-bodies during strong earthquakes, corroborated such an underestimation of seismic accelerations – although we know very well today that under cyclic dynamic displacements, such bodies may resist considerably larger accelerations (Fig. 1).

[1] Another coincidence: D'Alembert (1717–1783) was also a contemporary of the great quake of Lisbon.

[2] Luigi Palmieri (apparently based on a R. Mallet proposal 1898) built in Naples, Italy (1899), an electro-magnetic seismograph, which recorded several seismic events for the first time.

[3] It is worth to note that even among seismologists (Sielberg 1941 and Gutenberg 1942, see Roussopoulos 1949) the translation of the Mercalli-Sieberg intensity scale into ground accelerations varied about 300%.

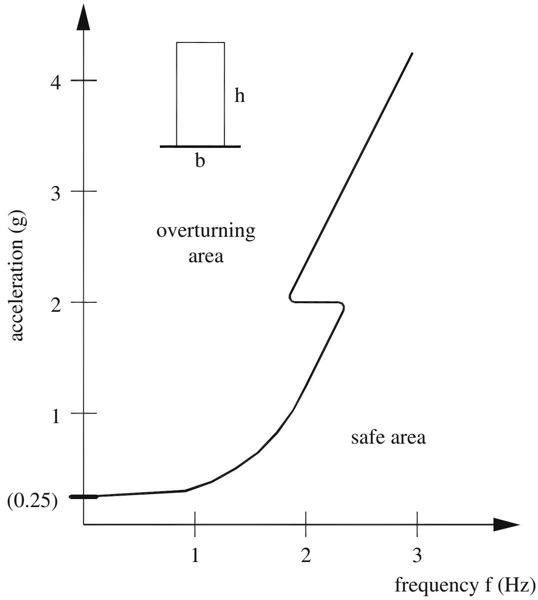


Fig. 1 A free rigid body $b = 0.64$ m, $h = 2.58$ m submitted to one cycle sinus acceleration pulse ($\alpha c = 0.25$ g, $p = 2.14$ rad/s) with frequency $f = 2$ Hz is overturned only if PGA = 1.1 g, instead of the static critical acceleration 0.25 g (Gerolymos et al. 2005)

Thus, the seismic design Recommendation suggested by some of the best world specialists of that time (1908), the Italian Professors in Torino, Italy, Modesto Panetti and Arturo Danusso, specified horizontal accelerations[4] of the order of 8% to 12%.

This was also the case (10%) with the first Japanese Regulation, following the 1923 Tokyo earthquake or with the Uniform Building Code, USA, 1927 (7.5 to 10%) or the first Greek Regulation (10%), after the 1928 Corinth earthquake. Or even with the Los Angeles (mandatory) Code (8%), after the 1933 Long Beach earthquake, and the Portuguese Code (1958).

c) In the meantime, Engineering Seismology was progressing: In 1933, for the first time, a strong motion was recorded in USA (Long Beach earthquake), and a bedrock acceleration value a bit lower than ¼ g was measured (Housner 1984). And this was four times larger than values specified in existing Codes (in response terms and accounting for the permissible stresses used at that time).
d) Notwithstanding this numerical discrepancy, once this logical jump made, Engineers were feeling more familiar with earthquake loading; they now had to deal with a load – a horizontal load. Its triangular distribution was initially suggested by Panetti; but even a uniform distribution was all too natural (after all, near the ultimate condition of a damaged structure, this is closer to the real distribution).

[4] It is remarkable that a few years later (1914), Sato, a Japanese engineer, proposed this seismic coefficient method for seismic design (Usami 1988).

The next problem now was the analysis of a multi-storey multi-bay frame: The same problem under vertical loads was not solved either; is was bypassed by the fragmented model of the famous continuous beam, neglecting the frame action, and introducing some nominal bending moment values to the external columns only.

A similar solution was not possible in the seismic case: there were no stable supports. Under horizontal loading, the simplest solution was only to consider each storey separately, under a shear force acting on each floor. Columns were initially supposed to be doubly fixed (Fig. 2a) in the case of the first Italian Regulation, 1909: however, some empirical corrections were introduced later on, regarding zero-moment points different than the mid-height of columns (e.g. see approximate greek method, Roussopoulos 1929). We may estimate today that shear forces and flexural moments derived by means of such a quasi-quantitative method, were only nominal values far from reality. Nevertheless, up to the middle of the 20th century, this was already a big progress, despite the anticipation of the American Prof. Derleth (after the great San-Francisco earthquake, 1906) that *An attempt to calculate earthquake stresses is futile; such calculations could lead to no practical conclusions of value* (Housner 1984)!

e) Yet, since vertical elements had different stiffnesses, the problem of two dimensional distribution of seismic forces along the diaphragm remained unsolved. A first correct step was made by the Japanese (Prof. Jacku-Naito 1920) considering that, along a plane frame, shear forces are undertaken by vertical elements in proportion to their stiffness. The most elegant two-dimensional mathematical solution given to this problem is due to Ath. Roussopoulos: Translational and rotational movements of the diaphragm were calculated, for all seismic directions. The locus of displacement of the head of each vertical element, was found to be an ellipse; its larger axis corresponded to the most adverse loading of this particular element (Fig. 3).

Obviously, these solutions were covering only **one-storey** elastic systems. Consideration of real multi-storey buildings was made only by means of rough correction factors; the initial assumption of doubly fixed slender columns

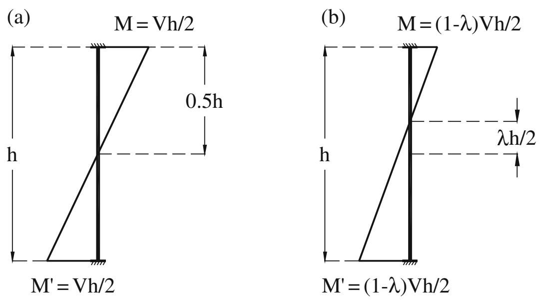


Fig. 2 (**a**) Doubly fixed columns at each floor; (**b**) The location of the zero-point depends on several conditions described in the approximate greek method, 1929

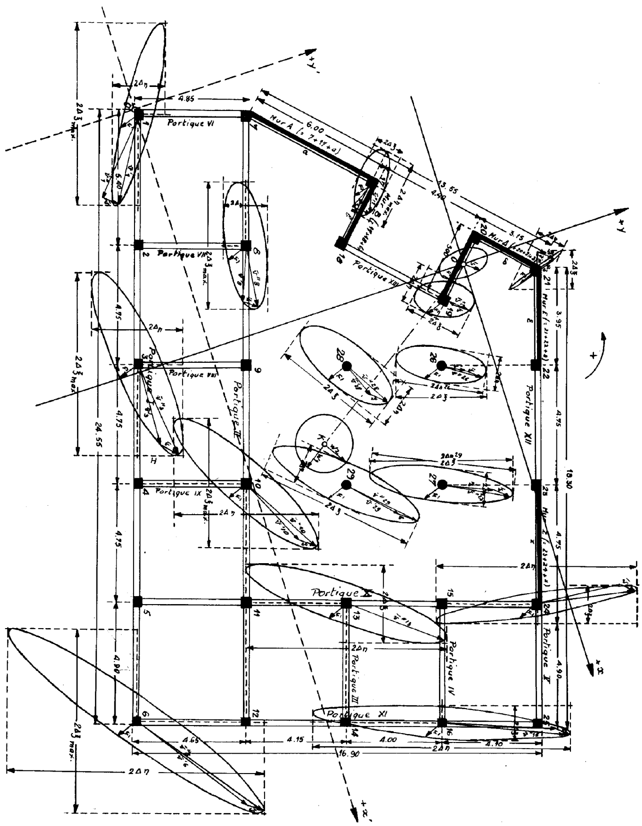


Fig. 3 The locus of displacement of the head of each vertical element is an ellipse (based on Roussopoulos 1932. Application: Roussopoulos 1958)

had occasionally led to dangerous gross errors, especially in the case of stiff columns or structural walls....

The road to the truth was still paved with several mistakes (as it is the case with other scientific sectors too).

4.1.2 The Progress

a) Two basic events acted as catalysts for a rapid development of Seismic Engineering. The first was the proliferation of **strong motion** recordings, initially[5] at Long Beach (1933), but practically after El Centro (1940) up to, say, 1970: Now is was possible, throughout the globe, to measure

[5] However, Prof. K. Suyehiro, of the Japanese Imperial College of Tokyo, had (1920) "clearly outlined the type of accelerographs that would be needed" (Hudson 1963, in Bozorgnia, Bertero 2004).

vibration periods and accelerations induced to the foundation of the building. This gave the possibility to connect Seismic Engineering with the rich knowledge of Dynamics; and, whenever circumstances allow for two scientific branches to get in contact, a rapid fertilization is taking place.

At this stage, a better understanding of the role of the **natural period** of vibration of the building was acquired, as compared to the governing period of the quake. And made the city of Los Angeles Building Code (1943) to relate seismic coefficient to the total height of the building (i.e. to its natural period).[6] Similar views were however forwarded by A. Danusso 1909 (b), who had demonstrated the theoretical relationship between seismic coefficient and flexibility of the structure. These views were already reflected in the 1.5-multiplyer of ground floor acceleration to be imposed to higher storeys (Italian Regulation 1909).

b) Now, the second basic event was ready to appear: Engineers were not interested in every detail of the vibration of the building; their concern was only about maximum values of the characteristics of this vibration: relative displacements, relative velocity and absolute acceleration of a single degree of freedom (which was supposed to describe approximately a regular building).

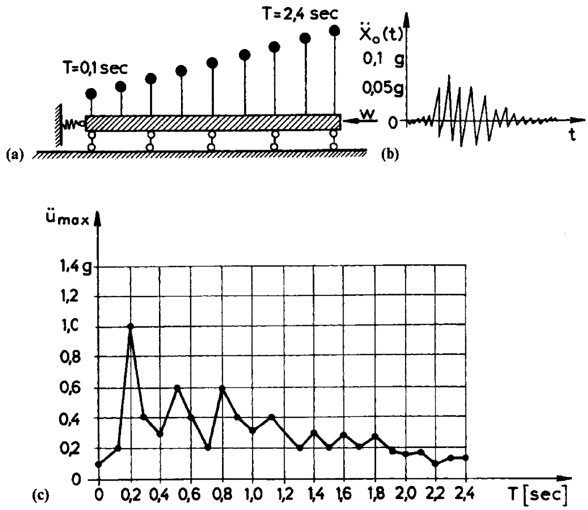
Thus, the concept of a **response spectrum** entered the game. Biot was the first (in 1935) to produce such spectra (Fig. 4) experimentally on a shaking table (Penelis and Kappos 1997), but it was only after the possibilities offered by the developments of computers (Housner et al. 1953) that response spectra were analytically produced. This was the second big event of progress during the second half of the 20th century, introduced for the first time in the Code ASCE-SEAONC 1952.

c) A third decisive step in this new period of Seismic Engineering was also due to the progress of computerized Numerical Analysis: **Three-dimensional** (static or dynamic) analysis of structural was now feasible, using as input appropriate response spectra (recorded, synthetical or code-specified). Thus, rough approximations and the respective mistakes (Section 4.1.1 (d)) could now be avoided.

d) Up to this stage, developments in Structural Analysis under seismic loading were almost independent of the final behaviour of the structure: everything was supposed to be and remain elastic during the earthquake. Thus, real inelastic behaviour of normal buildings under strong motion was not considered; despite their sophistication, such analytical tools did not always serve reality.

[6] Natural periods of buildings were already measured by E. Hall (Berkeley, 1912) and P. Byerly (San Francisco, 1931), according to Bozorgnia, Bertero 2004; but as well as by G. Alfani 1909, who measured several oscillatory characteristics of the municipal tower of Florence.

Fig. 4 Biot produced (1935) the first spectrum experimentally on a shaking table



The only simple way[7] to account for spread inelasticities, and continue taking profit of advanced linear Analysis, was to conceive an elasto-plastic response spectrum, derived by a simple reduction of the ordinates of the elastic spectrum (Fig. 5) by a quasi-constant reduction factor (**behaviour factor**) q, essentially an estimator of the available global **ductility** of the system. This was the outcome of a long observation of the survival of weak but ductile structures. Now, for a multi-degree-of-freedom system

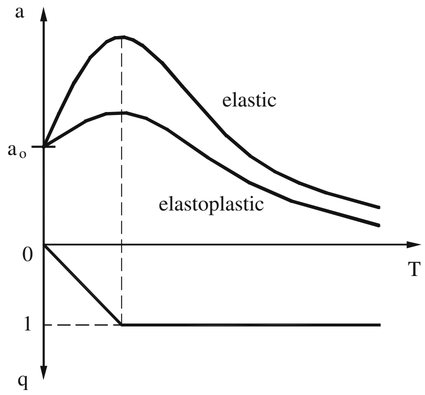


Fig. 5 A simply derived elastoplastic spectrum

[7] I am not referring here to explicit nonlinear analysis; its impact on practical design was not significant.

under elastoplastic conditions, to be still treated linearly as a single degree of freedom mechanism, an extensive series of requirements were needed (see Section 4.3.1) to allow designers to use the global reduction factor q-approach. This necessitated a strong interaction between Analysts and Behaviourists (see Section 4.2) in the sixties and seventies;[8] and the result was extremely fruitful in designing new structures reasonably conceived and precisely analysed. Once again, a big step of progress was made possible only thanks to a counter-fertilisation of ideas between Macro-analysis (action-effects' determination) and Micro-analysis (behaviour of critical regions).

e) The most recent development, the **displacement-controlled** nonlinear Analysis, was almost a timely consequence of the understanding of the fundamental role of local ductilities: In fact, despite the introduction of the q-approach, design up to the nineties continued to be based on force response – hence on resistances of critical regions. Now, it was better understood that, after all, earthquake is by nature an action imposing displacements (i.e. deformations) – not forces! Consequently, safety-verifications should be rather made in terms of post-yield deformational capacity of critical regions, under a given resistance. Seismic actions impose relative displacements (drifts) in the system, these displacements require rotation-angles at the critical regions, and these rotational demands should not be larger than local rotational capacities. As simple as that – but it took one century to come to it and to be able to check it numerically....

This being said, we must however admit that the state-of-the art of this approach of Analysis is not yet fully mature. A critical presentation of the numerous methods proposed is presented in the 25th Bulletin of fib, May 2003 (Ed. M. Calvi).

4.2 Conceptual Design

As it is well known, quantification should only be based on a sound qualitative understanding of natural phenomena. The same holds true in studying technological systems: first, making use of our knowledge of physical laws, and previous experience,[9] we conceive an appropriate system to serve our goals; subsequently, an experimental and/or analytical control will be carried out. The opposite sequence is extremely rare, if not dangerous.

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[8] Nevertheless, the first recognition of the importance of ductility (energy dissipation capacity) was given by the SEAOC Code, 1959, via a "K" factor modifying the base shear force for four categories of building structures.

[9] "Experience" in our field may be gained by observation of (i) the consequences of previous earthquakes or (ii) the results of experiments. Thus, empirical Technology precedes scientific Technology.

I will now maintain that in the history of seismic engineering, several basic concepts proved to be more important than any mathematical insight. In Technology, what really counts is the physical soundness of the ideas – not necessarily the glamour of their analytical handling. After all, this is the basic difference between a good Engineer and a genius student in Engineering.

Along the same lines, the best modern formulation of the importance of a correct Conceptual Design is found in the preface of the well known book Paulay, Pristley, 1992 Emphasis is on Design rather than Analysis, since conceivable uncertainty associated with describing expected ground notion characteristics, make details and sophisticated analysis of doubtful value, and indicate the scope and promise in **telling** the structure how it must respond under potentially wide range of earthquake characteristics, by application of **judicious design principles**. This should possibly be combined with the other view (a bit exaggerated as it may be) of the same authors, that *the extent and detail in Seismic zoning maps is more a reflection of the . . . density of geologists in the area, than of actual fault locations*.

I submit that these views summarise a good part of our progress in the field. Such **basic concepts** is seismic engineering are enumerated hereafter.

4.2.1 Aseismic Morphology

a) The concept of resistance against **horizontal seismic forces** supposingly acting on the structure, seems to be much older than D'Alembert's inertia forces. As an example, I will mention the hybrid aseismic structural system in **prehistoric Santorini** (Greece, 1600 BCE), consisting of timber moment-resisting frames, imbedded in masonry (Fig. 6).

   The most explicit example however was given by the **Authorities of Lisbon** just after the 1755 event (Azevedo 2005):

   – Provisions were first formulated for good construction materials and procedures.
   – The recommended Gaiola Pombalina with its triangular timber elements, insinuates a feeling of horizontal seismic forces. That is why the same system was adopted by the Italian Seismic Regulation after the earthquake of *Calabria 1783* (Fig. 7) – (see Vivenzio 1783) – although it seems that in Italy a similar structural system was already in use after the previous Calabrian quake of 1638 (A. Bumaldi 1638, in C. Barucci 1990).
   – Regularity was also governing the limited number of architectural solutions proposed to the inhabitants.
   – The limitation of number of storeys was perhaps the result of the knowledge that for that rigid category of structures, energy dissipation was only possible in the basement.
   – Moreover, experimental work of aseismic configurations was demonstrated in Lisbon main square, consisting of gaiola-systems put on platforms and subjected to various types of vibrations, in public!

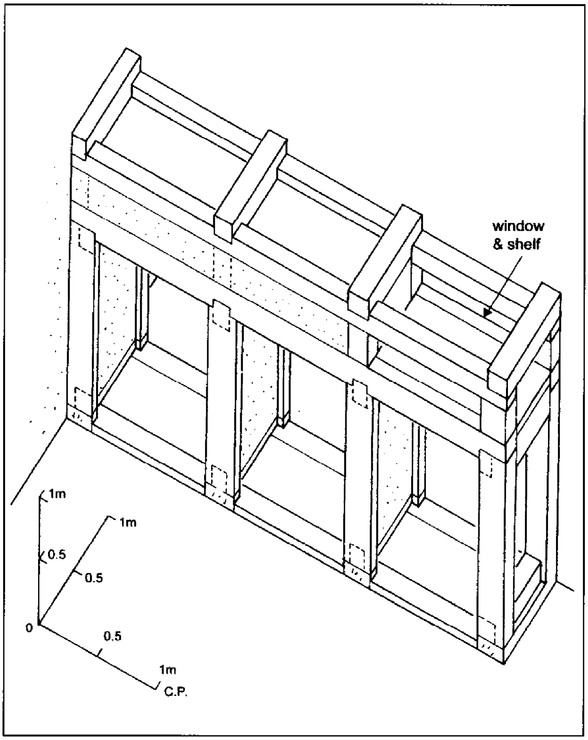


Fig. 6 Hybrid aseismic frame in prehistoric Santorini, (Greece, 1600 BCE)

These very sound and still valid provisions of the Portuguese State, came to a contrast to some other scientific attitudes reflected in Fig. 8. But we should show our understanding in these two directions of scientific endeavours; Isaac Newton himself had written 2000 pages in Alchemy.

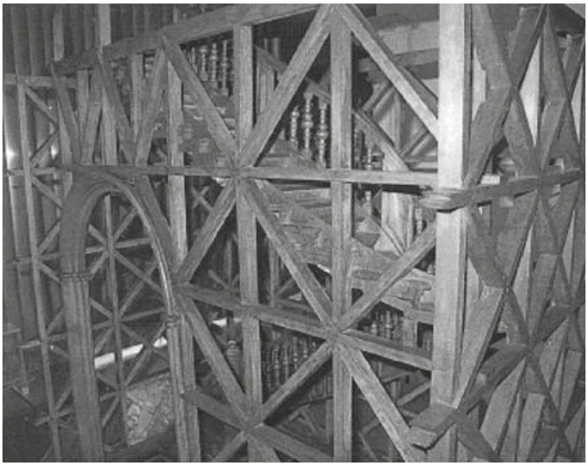
b) In more modern language, I consider as a basic progress the scientific understanding and rationalization of the old simple **regularity principles**.

- **In-plan regularity** is nowadays well understood,

    i) as in-plan stiffness and resistance of the floor-diaphragm. Relevant calculations are now feasible in case of irregularities or anomalies: The **diaphragm** as a plane building element, is submitted to its own inertia-forces and to the actions and reactions of the vertical elements connected to it. Simple elastic finite elements suffice for rational checking, additional reinforcements etc. Thus, a diaphragm as a plane quasi-rigid body, is able to reliably redistribute inertia forces to all vertical elements.

    It seems that this was also the purpose of some old rules (e.g. Italian Instructions of the reconstruction of Reggio, 20/03/1784) requiring the *iron confinements to tighten the building in all its parts*, Giangreco, 1983).

(a)



(b)

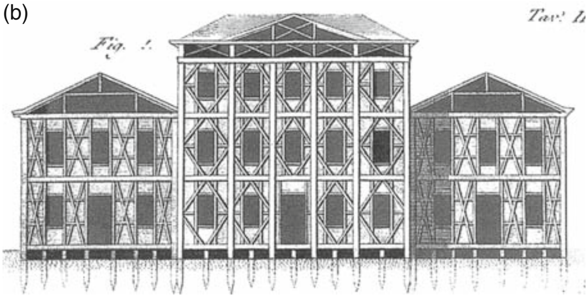


Fig. 7 (**a**) The Gaiola Pombalina (Portuguese recommendations after 1755) was inspired by the concept of horizontal seismic forces acting on the structure. (**b**) Casa Baraccata (Vivencio 1783) after the calabrian earthquake of 1783

Fig. 8 Other scientific preoccupations regarding the 1st Nov. 1755 earthquake…

– as an avoidance of **torsional** effects. Appropriate reduction of the distance between the center of mass and the center of rigidities[10] is the way to quantify and check those undesirable effects.

- **In-height regularity** nowadays, means the following:

  ii) A uniform vertical distribution of **safety margins** (V_R-V_S and M_R-M_S) is desirable, so that any unpredictable increase of seismic actions should not produce local failures (i.e. only local and inadequate dissipation of energy). I maintain that this principle is as fundamental as the requirement for local ductilities, and it can encompass several more specific rules, such as no mass concentration, no setbacks, no base or intermediate soft storeys, and the like.

  iii) A **natural flow of forces** should be ensured, avoiding staggered beams or (worse) staggered columns. Otherwise, local (horizontal or vertical) hammerings will produce completely unpredictable brittle conditions.

  iv) **Structural bridging** between adjacent buildings is just impossible: Even identical buildings may be found out of phase!

  v) **Short (low shear ratio) columns** (by their own dimensions or because of restrains imposed by partial infills) must be avoided. Not only because they attract higher action-effects but mainly because their low shear ratio radically modifies their failure modes, drastically reducing ductility (see i.a. Moretti and Tassios 2006).

Let me simply repeat how much the rationalisation (and subsequent quantification) of old principles, is an achievement as important as any sophisticated determination of action-effects (i.e. Analysis).

4.2.2 Ductility of Critical Regions

We have seen (Section 4.1.1 (b)) the tendency of the engineering community to rather underestimate the value of maximum acceleration produced by strong earthquakes. A first explanation was that, with the available analytical tools of those times, consideration of high seismic accelerations would have led to very heavy and costly structures. Nonetheless, Engineers might also have observed that some buildings had escaped really strong quakes with acceptable damages, but without collapse; the idea of **energy dissipation** has then emerged as a basic concept. After all, earthquakes do not apply external forces to be equilibrated: They merely impart energy to a structure; and the structure reacts in terms of forces, depending on the way the seismic energy is fed into it. The **nature** of the energy taken by the structure is not restricted; it may be elastic or elastoplastic. Thus (Fig. 9) the maximum reacting force is NOT a physical reality: it is a

[10] A further progress is this respect is due to T. Paulay (1997) who re-considered the issue of torsion under inelastic ductile conditions (see also Tassios 1998).

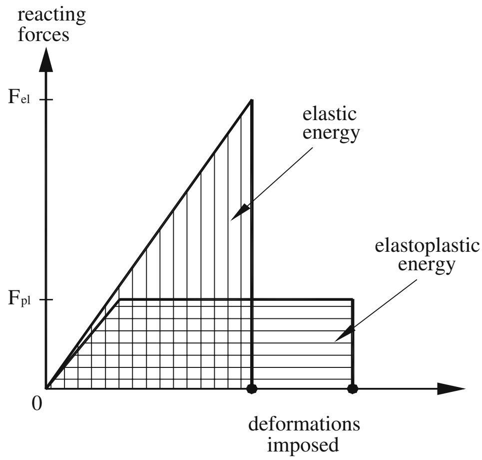


Fig. 9 Earthquakes impart energy to the structure; the structure translates energy into reacting forces, depending on its elastic or elastoplastic behaviour

technical byproduct; it may be large (F_{el}) or small (F_{pl}), at the expenses of small or large safely exhibited deformations of the system. Safely exhibited, in terms of available **ductility**. For a given earthquake, brittle systems are condemned to react with high resistance F_{el} – whereas **ductile** systems may be required to withstand forces as low as F_{pl} (in Fig. 9). It is a matter of available materials and know-how to build the seismic resistant system. Once again, the Engineer was reinstated in his role of a Magister: Instead of designing for resistance, we have now to design for Ductility too. This concept opened a completely new avenue in seismic engineering. In traditional terminology, the acting seismic force (F in Fig. 9) could now be a function of the energy dissipation capacity of the system, at it was recognised in the Code of the Structural Engineers Association of California, SEAOC 1959. This code-provision however was apparently anticipated as early as in 1920 (by Mononobe)[11] or in 1960 (by Blume).

More specific progress about how to achieve global and local ductility, will be reconsidered in the Section 4.3.1.

4.2.3 Dictated Sequence and Mode Failure

The so desirable global ductility of the structure (Section 4.2.2) cannot be achieved only by means of appropriate detailing of potential critical regions. Additional measures should be taken for the purpose:

i) Brittle failure modes (such as shear or anchorage) at critical regions should be avoided. Their respective resistances should be reliably higher than flexural resistances (e.g. $V_R > V_{MR}$).
ii) For obvious overall stability reasons, vertical building elements should not yield before yielding of horizontal elements. To this purpose, resistances of

[11] As maintained by Polyak of 1974.

vertical elements should reliably exceed the action effects induced to them at the moment of beam failure, taking into account hardening effects.

iii) Similarly, beam-column joints (potentially brittle plane elements) should reliably resists the maximum action-effects on their borders.

iv) In designing the detailing of columns' critical regions for ductility, the adverse effects of axial loads should be considered, taking into account a reasonable simultaneity of over-strengths of the end-sections of all beams above.

The generic term for the ensemble of these additional measures is **capacity design**, as it was included in SEAOC 1973, but mainly developed by Park and Paulay 1975 and further.

To use the words of Pauley himself, capacity design is not an Analysis technique – it is a powerful design tool, to overcome the rather crude estimates of earthquake structural actions, (irrespective of the degree of sophistication on which Analyses may be based), (Pauley and Pristley 1992).

That is why we considered this concept as another milestone in history of seismic engineering.

4.2.4 Performance Levels

The persisting uncertainties in almost all inputs in seismic engineering, resulted in a similar ambiguity of the targeted consequences of the design earthquake on structures:

- Were they expected to be intact and completely functional after the design-earthquake?
- Or, perhaps, we would be satisfied if at least human lives will be saved, whereas damage and property losses will be limited?
- Or, even, may we accept heavy damages and losses, provided that collapse of the structure is finally avoided?

Roughly speaking, these three performance levels were considered early enough as design limit states. They were made more explicit by the NEHRP 1997, and this is a considerable progress in conceptual design.

Nevertheless, such a rather refined definition of performance levels may not be completely justified in practice: a seismic action cannot be easily described by the actual code-provisions; a lot of very influential characteristics of an earthquake remain beyond description (duration or number of large amplitude cycles, directivity of the quake, local ground morphology etc.). Their consequences can hardly be predicted and codified.

4.3 Structural Behaviour

Nowadays, the old fashion term detailing, means several very basic design aspects, such as:

- Assurance of durability (e.g. calculation of the adequate concrete cover on steel bars, as a function of its permeability).
- Verification of the limit-state of anchorage against brittle pullout (i.e. calculation of the anchorage length of steel bars, as a function of concrete quality, concreting method and available transverse reinforcements).
- Available plastic strain of concrete under compression (as a function of the density of confining stirrups).
- Avoidance of stress concentration in welded areas of steel structures, etc.

These examples show that detailing was a **disparaging** term, reflecting the empiricism of the past; happily enough, in actual structural engineering we directly address the refined issues hidden under that term.

And it is not a coincidence that almost every progress made in structural behaviour under seismic conditions, is somehow related to such issues of paramount importance.

4.3.1 Reinforced Concrete

Because of the alleged brittleness of concrete, experimental and theoretical research on seismic behaviour of R.C. elements and structures was very intensive, mainly after the sixties. Some of the well known progress made in this field is very briefly reminded hereafter without details; but it is important to recall that, in all cases, rational **quantitative models** are available:

a) Local ductility of critical regions is enhanced

- for low normalised axial load values
- for high compressive-to-tensile reinforcement ratio
- for high available strain of concrete at failure (see CEB-FIP Model Code 90).

b) Available ultimate strain of concrete under triaxial compression due to confinement, is a direct function of the effective volumetric mechanical ratio of confining reinforcements

$$\alpha.\mathrm{w_w} = \frac{(\text{volume of closed stirrups})}{(\text{volume of confined concretes})} \cdot \frac{\mathrm{f}sy}{\mathrm{f}c} \tag{2}$$

where α denotes the efficiency of confining reinforcement, depending on its patterns, and f_{sy}, f_c are the strengths of materials (see CEB-FIP Model Code 90).

c) Available ultimate angle of rotation of building elements under cyclic conditions, is now quantified as a sum of three components:

- elongation of tensile steel
- pullout of anchorage
- shear distortion of the element (see i.a. Tassios and Stathatos 2005)

d) Shear resistance degradation under cyclic conditions being more rigid than flexural resistance degradation, shear failure may intervene during the earthquake, even in cases where initially $V_R > V_{MR}$ (see i.a. EC8, Part 1.4)

e) A rational design and check of column-beam joints was developed (see i.a. EC8, Part 1.3)
f) Structural R.C. walls became ductile, thanks to important conceptual and analytical developments; thus the invaluable aseismic importance of walls was reinstated (contributing to reduce psychological effects because of frequent earthquakes, enhancing further local dissipation of energy at beam-ends, mobilising stabilising membrane reactions of floors, etc.), (see i.a. Pauley and Pristley 1992).

Here again, the most characteristic achievements in R.C. seismic design, refer to developments in conceptual design, i.e. in Micro-analysis rather than in sophistication of Macro-analysis.

On the other hand, it is important to recall that long before the rational understanding and the successful modelling of the aforementioned developments, intuitive engineering had preceded: As early as in 1910, the ideas of Armand Considère (see Iori 2001) on confined concrete (béton fretté) were patented by the German firm Wayss und Freytag and used (Fig. 10) in the construction of the orphanage Regina Elena of Messina (Italy, after the seismic

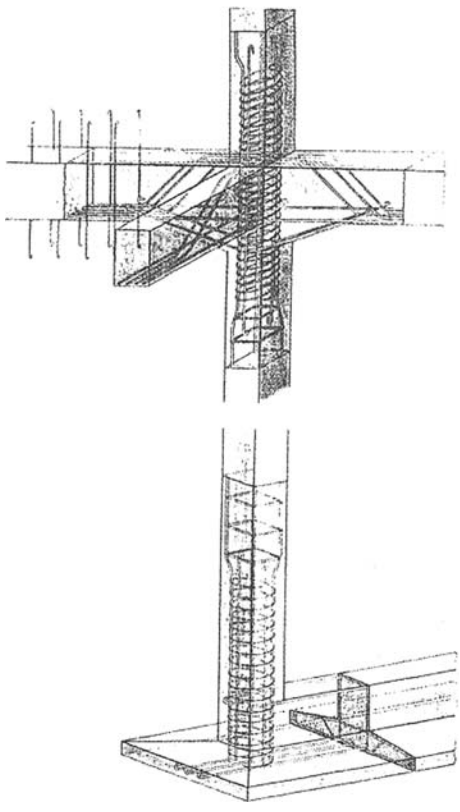


Fig. 10 Critical regions of columns (and column-beam joints) were heavily confined in aseismic buildings constructed as early as in 1920: Orphanage of Messina, Italy, system Wayss u. Freytag

disaster of 1908). Apparently, this prophetic system was subsequently forgotten for at least half a century. . . .

4.3.2 Steel Structures

This is another very interesting story of pathos kai mathos[12] – process. Steel, as opposed to concrete, is by nature a ductile material; consequently, the use of steel shapes as building material was considered as obviously aseismic.

a) And, in fact, riveted moment –resisting frames with normal masonry infills, around the 1900, has proved to be earthquake resistant (e.g. in the great San Francisco, 1906, earthquake). After all, this was a repetition of the old and successful mixed masonry – and – timber frames system (Section 4.2.1 (a)).

   High strength bolted frames (in the fifties) and jobsite-welded frames (in the sixties) were the subsequent developments, with (i) corner R.C. walls for further lateral stability (proved to perform well in Alaska quake 1964) or (ii) without walls, masonry being replaced by rather deformable infills or claddings). This latter system proved also to perform well (San Fernando 1971) because of the full welding of **all** seismic-resisting connections in **all** frames.

b) Economy, however, and speed being also basic engineering requirements, a double expediency was observed after 1970: The number of full moment resisting frames was gradually reduced; they appeared only in the perimeter of the building and, subsequently, only in one or two bays of the perimeter the connections were made seismic-resisting.

   By way of consequence, it was not a surprise to observe that the alleged ductility of the potential critical regions of beams and columns was **not** mobilized during the strong earthquakes of Loma Prieta, 1989, of Northridge, 1994, and of Kobe, 1995; instead, **brittle failures** were observed, and 10% of these structures in Kobe collapsed (J. Malley et al., in Bozorgnia and Bertero 2004). The most vulnerable area proved to be the joints between the bottom flange of the beam, welded to the supporting column flange: cracks were observed through

   – the weld or
   – the beam or
   – the column web or
   – through a combination of them.

c) Nowadays, however, thanks to FEMA 350 (2000) and to EC 8 (Part 1, Section 6), a considerable improvement of knowledge and technology was gained, after a successful consideration of the following aspects of design.

[12] Greek expression for "through suffering learning".

(i) Materials:

- ductility of steel shapes
- toughness of the weld metal
- geometry of the weld

(iii) Design:

- Capacity design principles in the beam-column joint area
- Due consideration of shear failure of panel zone

Research is continued on several other minor issues.

All in all, moment-resisting steel frames now may be safely designed for a targeted performance level.

d) On the other hand, **concentrically** braced frames (V, Λ, K or X type) may suffer of unbalanced vertical load at the brace-to-beam (or column) connection, when tension braces may safely yield but compression braces are buckled. Besides, buckling of braces is not contributing to the dissipative capacities of the system.

A considerable progress was therefore achieved when, actually:

- eccentric configurations of braces are used, allowing for a small beam-length (the link) to do the dissipation, while all other elements remain elastic.
- or, alternatively, the peaks of V or Λ-bracings are not connected directly to the middle of the beam, but they are fixed on a vertical small element (seismic link) designed to yield first, according to the capacity-principles or, finally, X bracings in the crossing point of which a damper is inserted, yielding before any failure of the bracing itself (Fig. 11).

A more recent and decisive development is the use of buckling-restrained bracings (BRB): A ductile steel core, slides freely inside a steel casing filled with concrete; a very small air-gap is left between the steel core and the concrete in order to eliminate the transfer of axial forces between the two elements; appropriate unbonding material is inserted in the gap. Thus, only very small-amplitude buckling modes are allowed, without appreciable

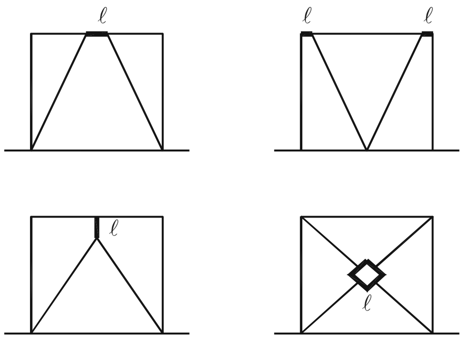


Fig. 11 Link lengths l are meant to dissipate energy, while brace elements remain elastic

modification of stable yielding of the core, under extensive compressive deformation of the steel core. An excellent post-yield hysteretic behaviour of the system is secured, under large cyclic imposed displacements (see i.a. Chia-Ming Uang et al., in Bozorgnia and Bertero 2004).

It is worth to observe, once again, that all these fundamental developments refer either to concepts or to Micro-analysis. Sophisticated macro-analytical tools for action-effects determination are applicable (and very useful, indeed) only on **already conceived** aseismic systems.

4.3.3 Masonry Structures

Masonry buildings constitute the 50% of existing buildings in Greece; this percentage goes up to 70% in USA. In other parts of the world, this figure may be as high as 90%. Although these very high figures do not reflect equal population percentages, they show however the extreme social and technical importance of masonry behaviour against seismic events. We are therefore wondering how much we have progressed, since 1755, in masonry seismic engineering.

I shall recall that a l l first lessons of aseismic behaviour were referring to masonry buildings, as discussed in Sections 4.1.1 and 4.2.1. Other than the limitation of the permissible number of storeys to one (or two), the main lesson was the importance of the hybrid systems of timber frames and masonry (Fig. 12) or simply the masonry reinforcement by means of timber elements.

In modern times, the **evolution** of these two old systems has generated the two actual masonry building systems.

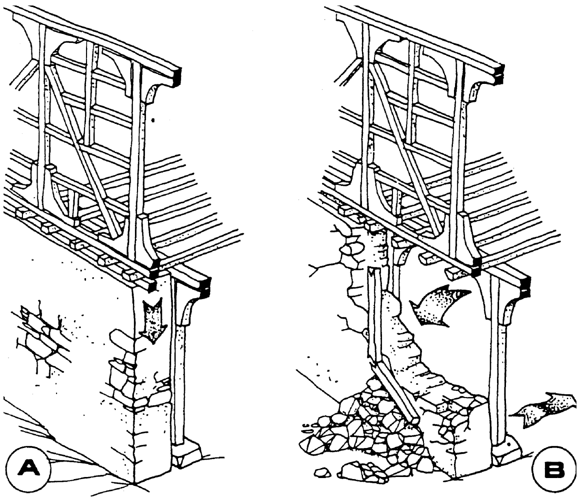


Fig. 12 Hybrid resistant system (timber frames and masonry) used in greek islands Lefkas and Lesbos (Touliatos 1995)

a) Completely Confined Masonry

A system of continuous and interconnected discrete R.C. or steel-shape linear elements (ties) located at the perimeter of masonry walls and around openings, offers the following structural functions (Fig. 13):

- Resistance against concentrated tension or compression (in-plane) forces, acting at the extremities.
- Connection of transversal walls against out-of-plane inertia forces
- In-plane ductility enhancement (resistance to web tensions, increase of the critical shear distortion angle of masonry)

Moreover, individual masonry blocks are prevented form falling out, thanks to numerous local friction and cohesion restraints offered by the ties.

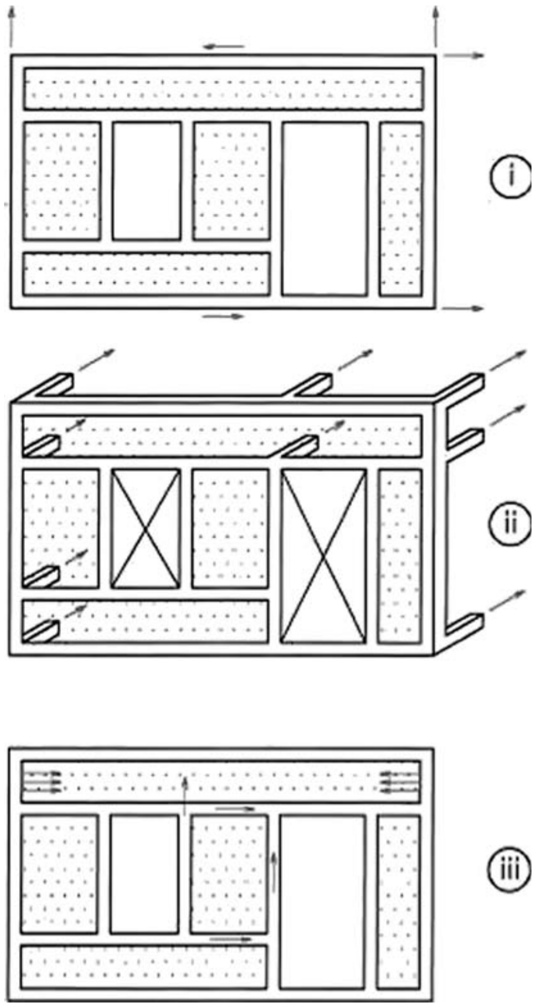


Fig. 13 The three structural functions of fully confined masonry

Consequently, the European Aseismic Design Code (EC8, Section 9.3) grants to completely confined masonries global behaviour factors equal to $q \cong 2.5$, provided that their diaphragmatic action is ensured.

b) Reinforced Masonry

Long iron-sheets or bars as a reinforcement of masonry are as old as the ancient greek temples (Thebe's treasure in Delphi, Propylea of Acropolis in Athens). Later on, copper sheets will be used as reinforcements, riveted along the meridians on the extrados of some vaults of the Saint-Sophia, Constantinopolis, (Tanyeli 1993).

In normal masonry however, spread metallic bars were proposed the first[13] time for seismic reasons by V. Gianfraceschi and G. Revere (2nd price of the Italian competition, after the Messina earthquake, 1906) Metal bars can be internally spread in masonries, horizontally and vertically, [...] through perforated concrete blocks, (in Monitore Tecnico, 20 Agosto, 1909); Fig. 14 shows the principle of the system proposed. The Italian Code of 1909 ordered the use of Reinforced Masonry *for 2-storey buildings, for churches, theaters and school buildings*, (T. Iori 2001).

Nowadays, R.M. systems are included in all Codes[14] across the globe, and they are granted with q-values almost as high as in the case of R.C. systems ($q_{max} = 3.0$). As an indication of the potentialities of Reinforced Masonry, Fig. 15 shows the hysteretic behaviour of a full scale R.M. wall, made of

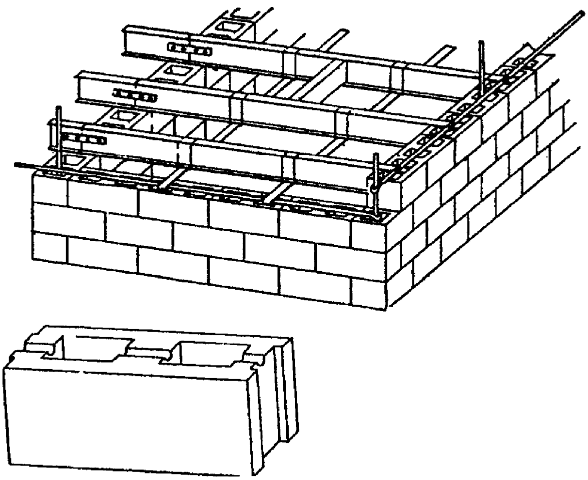


Fig. 14 Aseismic reinforced masonry system, proposed by Italian Engineers (1909), after Messina Earthquake

[13] Otherwise, two multilevel R.M. buildings were constructed in Paris, France, around the year 1900, but they did not resisted well durability problems.

[14] In the U.S.A., R.M. was introduced in the UBC, 1943. R.M. buildings behaved well during several Californian earthquakes. But this was not the case in Popayan, Columbia, where such multistory buildings collapsed: This system necessitates qualified workmanship.

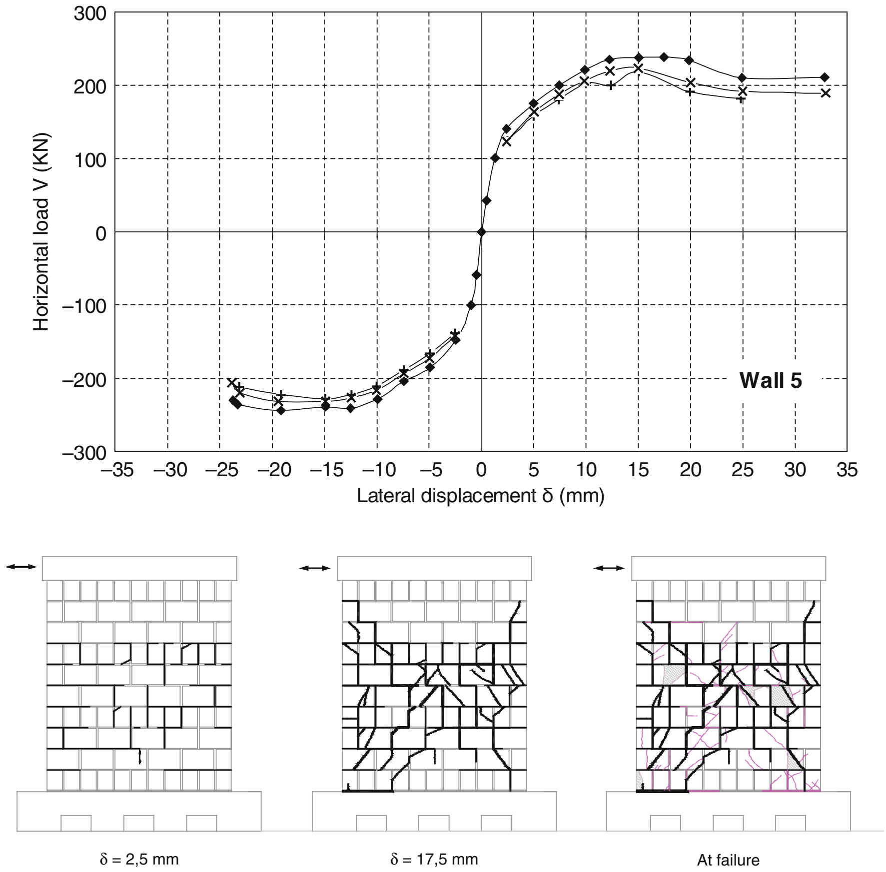


Fig. 15 Modern R.M. walls under large amplitude cyclic lateral loading showed their pronounced ductile behaviour (Psylla 2001)

self-insulating perforated bricks and minimal horizontal and vertical spread reinforcement of 0.5% (Psylla 2001).

5 Earthquake Geotechnical Engineering

Structural Engineers may occasionally tend to underestimate the importance of the properties of the ground: we simply bring down our loads to find support on it. With seismic actions, however, the soil was not anymore a passive receiver of loads, but it imposed dynamic loads to the structure, whereas at the same time the soil itself was modified by the same excitation. This completely new situation, produced quasi a revolution in geotechnical engineering, felt as early as after the Messina earthquake, 1906: The Italian seismic Code of April 1909 *prohibited construction in grounds marshy or subject to landslides or on the borders between soils of different nature and behaviour*.

Recently, a Renaissance of Soil Science has taken place (during the decade 1965–1975):

a) Newmark (Effects of Earthquakes on Dams & Embankments, Rankine Lecture, 1965) opened the road for understanding the fundamental difference between static and dynamic factor of safety of embankments. He showed that an instantaneous factor of safety well below unity, does not lead to failure but only to limited sliding deformation. Retaining walls, embankments and dams are presently been designed by computing their permanent sliding deformation as a function of the (smaller than 1) dynamic factor of safety, i.e., the ratio of the maximum sliding resistance divided by the peak ground acceleration. Thus, it remains each time to judge the acceptance of a given level of damage rather than a maximalistic safety factor in terms of forces.

   The concept is similar to the ductility concepts applied to seismic design of structures.
b) The development of the *Resonant Column under Cyclic Shear Tests* for measuring low-strain and large-strain dynamic soil properties (Richart, Hall, Woods, Stokoe, Seed, Whitman ≈ 1965–70), is another milestone. Followed the publication of experimental relationships between effective (secant) shear modulus G and of damping constant ξ, as functions of the shear strain amplitude (Seed & Idriss 1970). The consequences of these developments were very positive for the scientific progress which followed.
c) The following important development was the *one-dimensional wave propagation* (Soil Amplification) *theory*, for assessing the effect of soil on the intensity and frequency content of ground shaking (Seed and co-workers, Roesset and co-workers ≈ 1970), as well as the introduction of computer code SHAKE (Schnabel, Lysmer, & Seed 1972), still in use today.

   These Engineers opened the road for rational evaluation of soil effects on ground motions – which until that time was only done empirically, with very limited success.
d) Next came the development of *(Simplified) Theory for Liquefaction*, using field measurements of soil resistance (SPT N values) and empirical estimation of induced seismic shear stress (Seed & Idriss, Whitman ≈ 1965–69). Thus, a further step of rationalisation was made, and a new tool in avoiding dangerous construction-areas was invented.
e) Followed the development of analytical and numerical (finite element) solutions for *dynamic stiffness and damping of shallow and embedded foundations* (Luco, Veletsos, Lysmer, Roesset, Kausel, Gazetas: 1970–1974), as well as the development of theories for *soil-structure interaction analysis* applied to 1-dof system and to massive structures, such as nuclear power plants, (Veletsos, Bielak, Lysmer, Seed, Roesset: 1972–76). They opened the road

for rationally analyzing the effects of dynamic soil-structure interaction on a variety of structures and foundations.

6 Assessment and Redesign of Existing Buildings

6.1 Introduction

a) We may assume that repair of seismic damage and/or strengthening of existing structures was always performed. However, as a special discipline, the technology of **structural interventions** took its scientific character only after 1975; several national Codes,[15] provided specific rules to this end, while the first edition of the Eurocode EC8 (Part 1.4, 1988) seems to be the first systematic consideration of the subject: Structural Engineering was at last expanding from Embryology and Pediatrics, to Pathology/Surgery/Gerontology. And this was a sign of maturity: almost 70% of the inventory of existing building in western countries were designed and built on the basis of old fashioned Codes. Inhabitants and users of these buildings seem to be constitutionally unequal in terms of expected lifetime, as compared to those staying in new buildings. Structural intervention is therefore a socially indispensable and scientifically challenging issue.

b) The necessary general knowledge to this end was already available: Sound design principles and analytical tools for verification regarding **new** structures, may very well be applied when we are planning our interventions to **existing** buildings.

   There are, however, four basic difficulties:

   – An existing building may not comply with the prerequisites for the application of modern design Codes: It is not regular, it does not respect capacity design rules and it does not exhibit local ductility in hundreds of potential critical regions; consequently, we cannot follow the global behaviour factor approach specified by actual Codes.
   – The action-effects determined with the oversimplified methods of the past, as well as the old dimensioning methods (especially regarding shear resistance), are not reliable.
   – The seismic input data employed in the past were (almost always) considerably lower that the actually imposed values.
   – Last and worst, our knowledge of structural behaviour at the interfaces existing-to-added material were not much developed. This kind of micro-modelling (resulting in rational redimensioning of repairs and

[15] The Romanian Code was one of the good examples in this respect. Besides, the intensive post-war repairs of damaged structures in Soviet Union, has resulted in an extensive series of relevant publications and recommendations.

strengthenings) was not a glamorous field of research; it cannot ensure patents and it was not financed by most public research supporting agencies....

Nevertheless, here again, through continuous efforts, experience and devotion, Structural Engineers throughout the globe (see also the Proceedings of a Symposium of the Portuguese Society for Seismic Engineering, 2001), succeeded to accumulate knowledge, sufficient to feed an appropriate State of the Art on the subject – an extremely brief presentation of which will be attempted here below.

6.2 Vulnerability Studies and Assessments

a) Accumulated experience, logical analysis and numerous calibrations versus seismic events, have resulted in several simple methodologies to assess a pseudo-quantitative level of seismic vulnerability of buildings by means of data rapidly collected, mostly by visual observation and file examination. The relevant vulnerability matrices are prepared in appropriate probabilistic terms, and offer predictions of damage level, as a function of some estimator of the expected seismic action.

   Despite numerous doubts and rather frequent failures of such techniques to predict future seismic behaviour of existing buildings, vulnerability studies nowadays are considered to be a powerful tool for decision making of possible pre-seismic interventions, regarding sets of similar buildings rather than individual structures. However, because of the inevitably empirical character of such methodologies, their internationalisation is not yet easy.

b) As opposed to the aforementioned approximate vulnerability methods (statistically applicable to sets of similar buildings), assessment of seismic capacity of **individual** buildings is quite a different rational and completely quantified process.

Such assessment is based on the same analytical methods nowadays available for new structures, supplemented with appropriate modern modelling of critical regions:

- A performance level is decided and corresponding seismic actions are selected.
- Under specified conditions (see i.a. FEMA 350), linear analysis may be carried out; an appropriate global behaviour factor may be selected to this end, taking into account regularity of resistance margins along the building, local ductilities and secondary resistance mechanisms (see e.g. Bardakisand Tassios 2006).
- Otherwise, simplified non-linear static or dynamic analysis should be used, and available plastic rotation capacity of building elements should be checked (comp. Section 4.3.1 (c)).

6.3 Categories of Interventions

Experience gained on the subject, allows to draw the following inventory of the available categories of structural interventions; actually this is very helpful for decision making and in searching the respective appropriate design and construction techniques.

- Repair or strengthening of critical regions of existing structural elements, versus inadequate:
  - Lap splices
  - Shear resistance
  - Flexural resistance
  - Local ductility
  - Column-to-beam resistance ratio and versus heavy cracking and/or sliding.

  Most of these inadequacies may be remedied by appropriate confinements.
- Addition of new separately founded structural elements:
  - R.C. frames
  - R.C. walls
  - External steel frames or bracings
- Infilling of panels:
  - Strengthening of existing non-engineered infills
  - Infilling of R.C. walls
  - Infilling of steel frames or bracings (preferably buckling-restrained)
- Addition of dampers, isolators or mass actuators

Happily enough, the respective technologies are nowadays well developed, although their costs remain rather high.

6.4 Modelling

For each of the above enumerated intervention techniques, several behaviour models for dimensioning are needed. To this end, new constitutive laws at the interfaces of existing-to-added materials were developed, as illustrated in Table 1. The respective design-models may therefore be as **rational** as possible, avoiding an initial stage of empiricism. A set of such design models is contained in the new version of EC8, Part 1.4. 2005; more detailed models are presented in the Greek Code for Structural Interventions, 2005.

It is worth noting that FEMA-ASCE 2000, a complete and innovative document for decision making and Analysis, **does not** cover the second part of design, i.e. the determination of available resistances and stiffnesses of

Table 1 Force transfer along interfaces

	Concrete	Steel	Fiber reinforced polymers
Concrete	Compression friction	bond dowel confinement	bond anchorage confinement
Steel		welding	restraining of buckling
Fiber reinforced polymers			(thick layer ineffective)

composite critical regions. This may be an indication that scrutinised knowledge may not yet be that easy in this field, or that the second term of the inequality of safety is not considered as important at its first term. . . .

Nevertheless, here too, progress is in the making.

6.5 Strengthening of the Ground

The importance of soil conditions regarding the seismic behaviour of existing building was long ago recognised (see Section 5). On the other hand, technologies and design of soil improvement (including landslides' stabilisation) are well developed, and they may be used for an *aposteriori* strengthening of the ground against seismic effects.

Recently, a european funded research program (New geotechnical methods for mitigation of seismic risk of existing foundations, coordinated by Soletanche-Bachy, France, 2005), offered several theoretically and experimentally supported recommendations for such ground strengthenings. Among others, the effectiveness of a surrounding stiff containment (not connected to the building and going deeper than the liquefiable layer) was repeatedly confirmed, in terms of reduced settlements, and reduced excess pore pressure.

7 Response Modification Systems

7.1 Seismic Isolation

(a) It has been rightly said that the most earthquake-safe object is a balloon, retained by a string: the flexibility of the string uncouples the object from any ground-induced motion. That is why when in 30 January 1909 the italian engineer M. Viscardini (Barucci 1999) submits his patent request for an Aseismic Foundation System (Fig. 16), authorities were fascinated; but finally promoted seismic traditional aseismic systems of superstructure. The seismic isolators proposed by Viscardini possessed both restoring capacity geometricly ensured (thanks to the curvature of the sliding support) and energy dissipation. Only a few days later (of February 1909) the **Portuguese** Engineers A. Coffino and A. Rodrigues da Silva, submit (to the same italian Patent Office of Rome) another seismic isolation system intitled Building supports able to reduce the effects of seismic motions.[16] Two other plane-sliding systems were also proposed in Italy, during that extremely productive year of 1909; the fruitfulness of that marvellous year will be further confirmed by the letter of Mr A. Calantarients (an English medical doctor) to the director of the Seismological Service, Santiago of Chile, (August 1909) *The building will be built on a free joint (a layer of fine sand, mica or talc) that would allow the building to slide in an earthquake, thereby reducing the force transmitted to the building itself*, (Naeim and Kelly 1999).[17]

And in fact, several years later, some small masonry buildings in India (Bihar) and in China (Tangshan) survived disastrous quakes after having slided several centimetres on their shallow foundations. Besides, the Imperial Hotel of Tokyo (1921) was built on an intermediate layer of soft mud, performed very well in the 1923 Tokyo earthquake, having slided on that soil layer (Kelly 1986): Seismic isolation was approved by Nature. . . .

When the first nuclear power plant in South Africa (1970) will be built on a hybrid system of sliding neoprene bearings, the concept of seismic isolation will enter its age of maturation.

(b) It remains now to see how the filter of practice has sorted out the innumerable inventions of base isolation systems publicised every year. Some of them never surpassed the stage of a good idea, eventually proven to be dangerous, in terms of

- in-time instability of structural characteristics (durability, maintainability),
- sensitivity to temperature and to improbable large amplitudes of motion,
- unforeseen behaviour under combined cyclic actions and rate effects.

For the time being however, the following great progress has been made.

- Systems based on **sliding**: After the initial ideas of 1909, sliding systems were developed, normally coupled with elastomeric bearings. The Friction Pendulum System (FPS) (an articulated slider, coated with low-friction material, moves on a lower stainless steel spherical surface and sits on an upper concave spherical cavity) is having several large-scale applications, including LNG tanks and Hospitals in Greece.
- Several others spring type and rocking systems were proposed.
- However, the most extensively used are the **elastomeric**-based systems:

[16] Since then, several types of roller bearing systems were proposed but they had very low damping.

[17] However, it seems that the very first scientific proposal for such isolation was that of Prof. J. Milne (a british, teaching mining engineering in Japan) who presented it in a report to the british association for the advancement of science, in 1885.

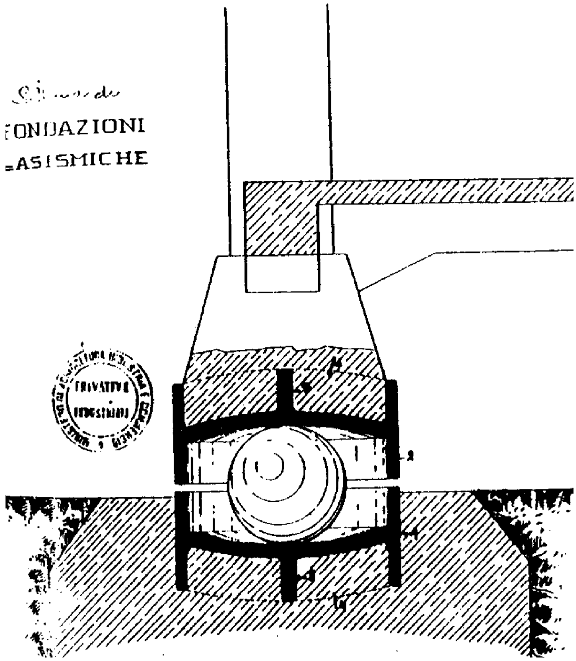


Fig. 16 The patented seismic isolation system of the Italian M. Viscardini (January 1909)

- Low damping, steel-reinforced rubber, coupled with a supplementary damping system or, better, with lead-plug inserted into a central hole of the elastomeric pad.
- High damping reinforced rubber (without additional damping system), exhibiting a mixed viscous and hysteretic behaviour.

The hope for longevity of such systems is supported by the 35 years of use of elastomeric bridge-bearings.

(c) Many seismic-isolated buildings were subjected to earthquakes and behaved as designed. But it is only in three base-isolated large buildings that seismic action was strong enough to give us the final certificate we were needing on the whole concept:

University Hospital (Northridge, 1994)
West Japan Postal Computer Center (Kobe, 1999)
Ojiya Hospital for the aged, (Mid-Niigata, 2004), see Fig. 17 (from Tazoh et al. 2004).

Their excellent performance, under earthquakes (equal to or exceeding design-values), has triggered a widespread application of well designed base-isolated buildings – mainly in Japan, where between 1995 and 2005, thousands of such buildings were constructed, as compared to hundreds between 1985 and 1995!

Last but not least, the significance of base-isolation for retrofitting of Monuments cannot be overemphasised.

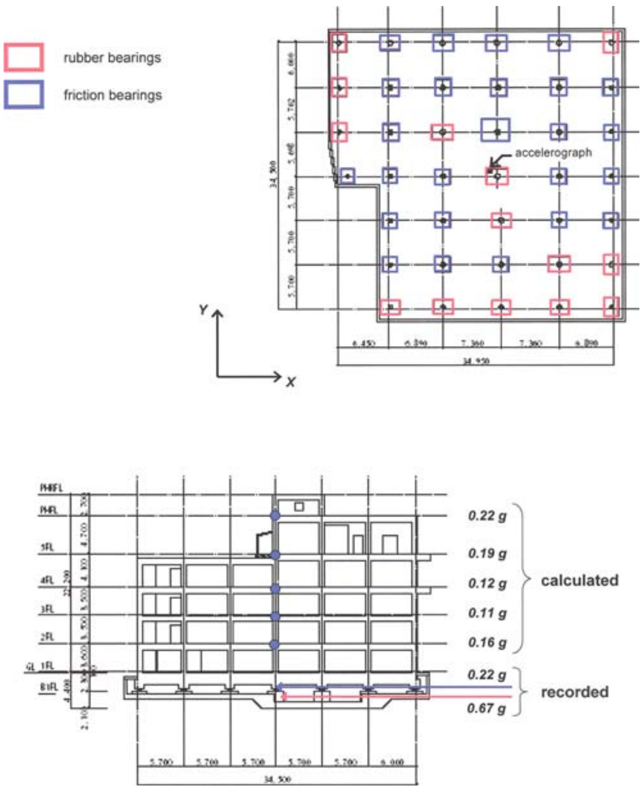


Fig. 17 The Ojiya Hospital (near Niigata, Japan) suffered no damage or disfunction during the mid-Niigata, 2004, quake, despite a recorded ground acceleration of 0.7 g. The Hospital was isolated with a combination of rubber and friction bearings (Tazoh et al. 2004)

7.2 Dampers

Passive energy-dissipators incorporated into the structural system of a building (or combined with base isolation), is the other very important category of seismic response-modifiers (Soong and Spencer 2002). Hundreds of buildings and bridges are actually equipped with such devices, worldwide:

- **Metallic** yield dampers (plates or bars, slit dampers or shear panels etc) made of mild steel, lead or shape-memory alloys. The unbonded brace system mentioned in Section 4.3.2 (d), may also be considered as a passive energy dissipator.
- **Friction** dampers (two solid bodies sliding to each other, provide energy dissipation). Such dampers may be inserted at the crossing point of X-braces incorporated in frames.
- **Viscoelastic** dampers consist of viscoelastic material (copolymers or glassy substances), bonded with steel plates sliding to each other.

– **Viscous** fluid dampers, widely used in aerospace industry, consist of a piston acting in a housing filled e.g. with silicone: The piston has some small orifices, through which the silicone may pass from one side of the piston to the other.

This is a fast expanding technology, applicable to new designs or to pre-seismic intervention of existing buildings.

7.3 Active Control Systems

Structural response control is a relatively new field of research in civil engineering (less-than two decades old). Various disciplines (such as data processing, control theory, sensing technology and structural dynamics), were integrated to the purpose:

a) **Hybrid** mass damper systems, the most common control device, consists of a tuned mass damper (tuned to one structural frequency) and an active control actuator, and it is able to reduce structural response close to the natural frequency of the mass damper. Small energy and forces are required to operate the system.
b) **Active mass** damper systems, need more space and much more energy to be operated. Restoring force may be trusted to mechanical springs or to the actuators themselves.
c) **Semi-active** damper system, combine the best characteristics of passive and active control systems, and may operate on battery power alone. Extensive research works have proved that such systems may effectively reduce structural response, under a large variety of dynamic excitations. In many cases, a semi-active device is using an electromechanical variable-orifice valve, in order to modify the flow-resistance of an hydraulic fluid damper.

An important variation of semi-active systems consist in using a controllable fluid damper, with no moving parts other that the piston.

Nowadays, approximately on hundred buildings are equipped with such response-modifiers. Research and experience is accumulated.

8 Are We Strong Enough Against the Giant Enceladus

> Et fessum quoties mutat latus, intremere, omnem
> murmure Trinacriam, et coelum subtexere fumo »
> Virgilius, Ain., III

As we know, earthquakes are produced whenever Enceladus moves, buried as he is under the mountain Etna (Sicily) thunderstruck by Zeus (Fig. 18). Can we, poor mortals, tame him?

Fig. 18 Enceladus buried under the island of Sicily, may occasionally move and produce earthquakes…

The answer is in the pediment of the Siphnos treasure in Delphi: Enceladus lying down under the lance of Athèna, insinuates that **wisdom** is his only vanquisher. A double wisdom:

a) **Knowledge** how to mitigate seismic risk:

 – Civil defense measures (including preparedness, rescue mechanisms, urgent settling plans, alternative employment plans, psychological resistance)
 – Rational city-planning and earth use.
 – Continuous re-drafting of our Regional Aseismic Design and Redesign Codes, resisting (professionally understandable, but socially unacceptable) inertia of our minds.
 – Revolutionary procedures securing total Quality Assurance in design construction, supervision and maintenance of our structures.

b) Moral-political wisdom.

- We should not forget! The psychological tendency to be relieved of painful memories, is only a small luxury for individuals – not for communities. The cost of unpleasant memories is largely counterbalanced by the profits generated thanks to our readiness to spend today,[18] in order to avoid future losses. Thus, preseismic strengthening of existing buildings must be a systematic long-range component of our national economical Plans.
- Ingratitude is a big sin! We are grateful to medical doctors (and occasionally we show them our extra appreciation). Society should think to take a similar attitude versus Engineers; not by means of extra individual payments, but via additional public research and development funds in the fast evolving field of seismic engineering. Short-sighted market policies cannot offer societal benefits in the field of Natural Disasters.
- Human transnational fraternity feelings were regenerated during disastrous quakes (e.g. Turkish/Greek or Pakistan/India collaborations). Let us create an International Mutual Seismic Relief Fund through U.N., in order to alleviate our guilty feelings due to the global inequality in front of Death.

And so be it!

Acknowledgements This short report (clearly inadequate as it may be) on the history of a very important branch of Technology, could not be written without the friendly assistance of Prof. J. Azevedo (Lisbon), Prof. M.A. Chiorino (Torino), Prof. G. Gazetas (Athens), Prof. P. Touliatos (Athens), Prof. G. Macchi (Pavia), Dr J. Logiades (Athens), Prof. A. Kappos (Thessalonikè) and Prof. J. Vayas (Athens). Their generous contribution is thankfully acknowledged.

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Bruce Alan Bolt 1930–2005 Professor of Seismology, Emeritus

David Brillinger, Joseph Penzien and Barbara Romanowicz

Fig. 1 Professor Bolt

Bruce Alan Bolt, Figs. 1 and 2, Professor Emeritus of Seismology at the University of California, Berkeley, died suddenly of pancreatic cancer at Kaiser Permanente Medical Center in Oakland, California on July 21, 2005.

Professor Bolt was born on February 15, 1930 in the small town of Largs, New South Wales, Australia. He attended East Maitland Public School, Maitland Boys' High School, Newcastle Technical College, New England University College of the University of Sydney, Sydney Teachers' College. Majoring in mathematics and physics, he received a B.Sc. degree with honors in mathematics in 1952 from the University of Sydney, and also M.Sc., Ph.D., and D.Sc. degrees in 1955, 1959, and 1972, respectively, from the University of Sydney.

After a year at Sydney Teacher's College (Diploma of Education, 1953) Professor Bolt taught at Sydney Boys' High School in 1953. He was then appointed to the faculty in the Department of Mathematics (Applied Mathematics) at the University of Sydney and progressed through the ranks as Lecturer, 1954–1959, and Senior Lecturer, 1959–1962.

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After completing his Ph.D. thesis in elastic wave theory, he won a Fulbright scholarship to Lamont Geological Observatory at Columbia University in 1960 and to Cambridge University (U.K.) in 1961. Meeting there with Perry Byerly, Professor of Seismology at U.C. Berkeley led to an invitation to a chair in seismology at U.C. Berkeley in 1963.

During the period 1963–1993, he served as Professor of Seismology in the Department of Geology and Geophysics at the University of California, Berkeley (UCB) and as Director of UCB's Seismographic Stations. In his early years at UCB, Professor Bolt developed strong research interests with faculty members in structural and geotechnical engineering, which resulted in his serving as Professor of Civil and Environmental Engineering during the period 1983–1993. Upon retiring from UCB in 1993, he received the campus' highest honor, the Berkeley Citation, then became Professor Emeritus of Seismology and Professor in the Graduate School, thus continuing his academic activities until his death.

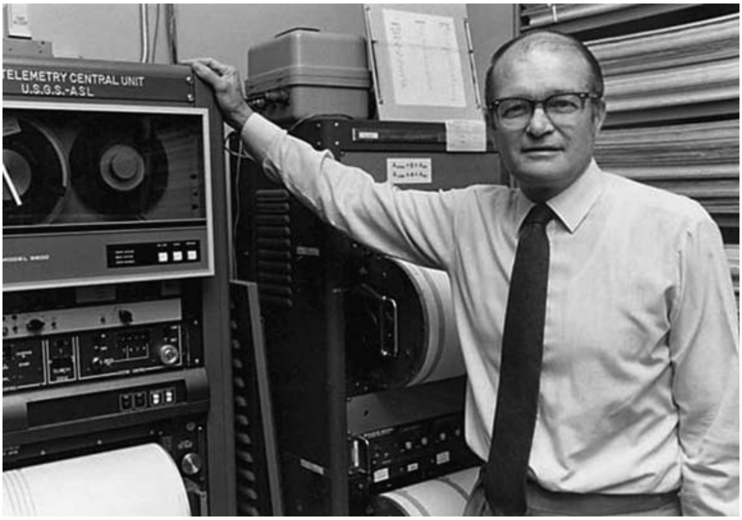

Fig. 2 Professor Bolt

While teaching at the University of Sydney, Professor Bolt developed outstanding expertise in the specialty areas of applied mathematics, statistics, and geophysics. As a result, he continued to make valuable contributions to advancing knowledge in these areas throughout his career. His strongest desire was to understand natural phenomena, particularly their mathematical and statistical descriptions. He wrote numerous novel papers pertaining to the deep earth, dispersion, earthquake engineering, free oscillations, seismology, and statistics. His first published paper was a 1957 note in *Nature*, followed by one in

Geophysical Journal of the Royal Astronomical Society, on seismic observations of the 1956 atomic explosions in Australia. In 1960, he published a paper with John Butcher on the dispersion of seismic waves, which began his deep involvement with large data sets and digital computing. His creativity in and knowledge of statistical methodologies, with influence from Harold Jeffreys, led to the estimation technique for the revision of earthquake epicenters still in use today (2006). As part of that work, he led statisticians by some ten years in developing the method of robust regression. His many contributions to seismology, including the development of earth models, have involved finite element methods, elastic wave-propagation theory, broadband and digital recording, strong-motion array development, data collection and interpretation, attenuation relations, and earthquake statistics.

Professor Bolt wrote six and edited eight textbooks on earthquakes, geology and computers among other topics, as well as almost two hundred research papers. His numerous publications on topics in seismology include *Nuclear Explosions and Earthquakes: The Parted Veil,* 1976; four very popular books: (1) *Earthquakes: A Primer*, 1978, (2) *Inside the Earth: Evidence from Earthquakes*, 1982, (3) *Earthquakes and Geological Discovery*, 1993, and (4) five editions of *Earthquakes*. An updated version of the 2003 5th Edition was published in 2006. In recognition of his many contributions to seismology, he was elected Fellow of the American Geophysical Union and the Geological Society of America, Associate of the Royal Astronomical Society, and Overseas Fellow of Churchill College, Cambridge. He served as President of the Seismological Society of America in 1974 and editor of its *Bulletin* from 1965 to 1972, President of the International Association of Seismology and Physics of the Earth's Interior from 1980 to 1983, President of the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS), and President of the California Academy of Sciences.

Notably, Professor Bolt was actively involved from the early 1960's through the mid-1980's in the development of reference earth models and towards this goal. contributed many measurements of body wave travel times as well as free oscillation eigenfrequencies and attenuation. He was particularly interested in constraining the average earth structure near two major interfaces: the solid/liquid core mantle boundary (CMB) and the liquid/solid inner core boundary (ICB) and focused many of his studies on the challenging topic of density. He provided some of the first robust measurements of the density jump at the ICB (1970) and also confirmed the density jump at the CMB (1985) and provided insights on the resolution of the density profile throughout the earth's interior (1975). He studied the shear velocity structure near the CMB, and the compressional velocity profile in the outer core, with a continued interest in characterizing resolution and uncertainty, which developed through a fruitful collaboration with Professor David Brillinger in Statistics. He was one of the proponents of the existence of an anomalous layer at the bottom of the outer core and made prominent observations of compressional waves (PKnKP) bouncing multiple times inside the outer core, which demonstrated that the CMB must be very spherical, and was illustrated in his textbook "Inside the Earth".

Many of Dr Bolt's observations were made on records from the Berkeley Seismographic Stations (BSS), which, in the good tradition of previous Directors, he modernized over the years and kept apace of current technology. He introduced broadband recording at Berkeley in 1963 and started replacing paper recording by magnetic tape recording in 1964. Continuing in the innovative vein, under his directorship, the BSS developed a regional broadband digital network based on inexpensive PC microcomputers, with telemetry to Berkeley over ordinary phone lines, and including three stations equipped with state-of-the-art broadband seismometers at Berkeley, Mt Hamilton and the San Andreas Geophysical Observatory near Hollister. Because of his involvement in the BSS and the related information service on northern California earthquakes, Dr Bolt's research interests gradually shifted towards characterizing ground motions from regional earthquakes in relation to earthquake hazards in the built environment.

In addition to many contributions to seismology, Professor Bolt made invaluable contributions to the field of earthquake engineering through teaching of basic seismology to graduate students in structural and geotechnical engineering, conducting research characterizing strong ground motions for engineering design purposes, serving as consultant on important engineering projects, and participating as a member of numerous panels, boards, and commissions. He was an active participant in the UBC Earthquake Engineering Research Center.

Professor Bolt's consulting work focused primarily on setting seismic criteria for new and retrofit designs of important critical structures, such as dams, nuclear power plants, large bridges, underground structures, and pipelines. These structures included the Aswan Dam, Diablo Canyon Nuclear Power Plant, Golden Gate Bridge, Bay Area Rapid Transit (BART) underground stations, BART transbay tube, and the Alaska Pipeline. His consulting work in 2005 included characterizing the controlling seismic sources and assessing tsunami risk for use in designing the now-planned (2006) suspension bridge crossing the Messina Strait between Italy and Sicily.

His setting of seismic design criteria for critical structures involved identifying seismic-source zones, guiding the conduct of seismic hazard analyses, generating site-specific response spectra and corresponding free-field ground motions, characterizing the spatial variations of ground motions, and predicting expected future fault offsets. Further, he participated in evaluating the seismic performance of such structures. His strong background in applied mathematics and mechanics made it possible for him to effectively communicate with structural and geotechnical engineers on seismic-design and damage-assessment related issues.

The numerous seismic-related panels, boards, and commissions on which Professor Bolt served include the California Department of Water Resources Consulting Board, California Department of Transportation Seismic Advisory

Board, San Francisco Bay Conservation and Development Commission (BCDC) Engineering Criteria Review Board, Metropolitan Transportation Commission (MTC) Engineering and Design Advisory Panel (EDAP), Golden Gate Bridge Seismic Instrumentation Panel, and the California Seismic Safety Commission (CSSC). As a member of CSSC, he actively participated in the Commission's sponsoring of numerous bills introduced into the legislature which became California law, thus greatly enhancing seismic hazard mitigation in the State.

In recognition of Professor Bolt's many valuable contributions to earthquake engineering, he received the Earthquake Engineering Research Institute's 1990 George W. Housner Medal, the California Earthquake Safety Foundation's 1995 Alfred E. Alquist Medal, and was elected to the National Academy of Engineering (NAE) in 1978. His NAE citation reads as follows: "For the application of the principles of seismology and applied mathematics to engineering decisions and public policy."

Having served many years as Chair of the UCB Academic Senate and as President of the Faculty Club, Professor Bolt seemed to know everyone on the Berkeley campus. His close relationship with individuals extended to a myriad of scientists and engineers worldwide. He was sought after as a speaker. Always in meeting one of his many close friends, he would extend a warm greeting with a big smile. His personal character has been admired by all who have had the pleasure of knowing him. He will be greatly missed by his friends, colleagues, students, and all who knew him.

Professor Bolt is survived by his wife Beverley (Bentley) of Berkeley, CA; three daughters, Gillian Bolt Kohli of Wellesley, MA, Helen Bolt Juarez of Fremont, CA, and Margaret L. Barber of Orinda, CA; a son, Robert C. Bolt of Hillsborough, CA; a sister, Fay Bolt of Sydney, Australia, and sixteen grandchildren.

Part III
Social-Economic Impact on Communities Exposed to Earthquakes and Tsunamis

Catastrophe Risk Management in Developing Countries and the Last Mile

Haresh C. Shah

1 Introduction

The last three decades have seen a revolutionary improvement of our understanding of catastrophic events such as earthquakes, hurricanes, floods, tsunamis, etc. These improved understanding of the phenomenological aspects have not made any visible dent in the consequences of such events, especially in developing countries. There are three main reasons.

1. The first one is that we have not substantially improved our ability to assess social, economic and political consequences of such events, when they occur. There are still major uncertainties in physical and social vulnerabilities of exposed communities. Also the quality of information about the exposure – physical infrastructure as well as demography of population at risk – is quite limited.
2. The second reason is that even if we can estimate social, economic and political consequences, our ability to translate that information into public policy options is very amateurish. Most of the work on phenomenological aspect of catastrophes, exposure and vulnerability analysis and risk assessment is done by science and technology based professionals. They are not qualified to take the information on risk and convert it into public policy because that part of the problem requires a robust understanding of the economics, social and political realities and policy formulations.
3. The final stumbling block in improving catastrophe risk profile of a developing country is the lack of connection between experts and the society at risk. This problem of disconnection is what is referred to in this paper as the problem of 'The Last Mile'.

One would assume that with all the technological progress in countries such as USA, Japan, Western Europe and other developed countries in the field of

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risk management, their ability to transfer knowledge would play an important role in mitigating catastrophe risk in developing countries. Unfortunately, that is not happening. Over the last three or four decades, great strides have been made in understanding the science, engineering and socio-economics of earthquakes and strategies to mitigate the effects of such catastrophes. On the one hand, due to easy availability of the internet, the scientific, engineering and economics knowledge bases are readily accessible to all experts around the world, yet the implementation of the known strategies and the benefits from those strategies have been confined to only few developed countries. To a large extent there has been increasing death and economic disruptions on a global scale rather than reduction in risk. Figure 1, (Munich Re Group; Knowledge Series, Natural Catastrophes 2004) shows increasing economic losses over the last five decades in constant dollars. Table 1 shows similar trend in life loss since 1999. One of the reasons for this discontinuity between available knowledge and 'on the ground' performance is the lack of communication between the knowledge generators and knowledge users with experts talking only to experts. There is also lack of understanding on the part of knowledge generators as to who are the sectors that need to know and who can make a difference. Creating the right partnership for implementation in a country is as important as creating the knowledge base for possible risk reduction. In addition, ownership of the problem and its solution is not shared by all the stakeholders. As a consequence, reaching out and connecting the last mile is not generally done. So what is needed to create true international partnership

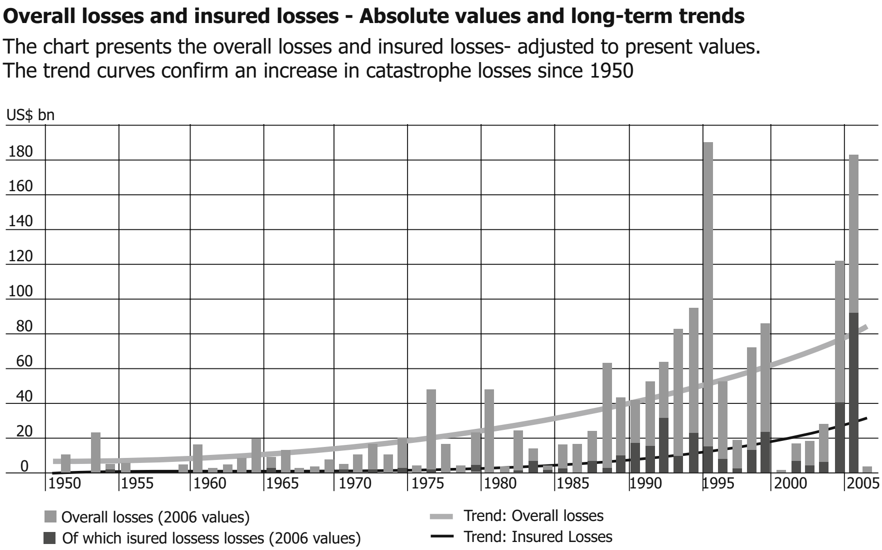

Fig. 1 Overall losses and insurance losses – Absolute values and long-term trends (Munich Re Group)

Table 1 How well are we doing? Recent EQ losses

Date	Event	Lives Lost	$Loss in Billions	%GDP
8/17/1999	Kocaeli – Turkey	17,000	$15–20	7–10%
9/21/1999	Chi Chi – Taiwan	2,400	$10–12	3–4%
1/26/2001	Bhuj – India	14,000	~$10	1%
5/21/2003	Boumerdes – Algeria	2,300	$5	10%
12/26/2003	Bam – Iran	26,000 +	$1	1%
12/26/2004	Asian Tsunami & Sumatra EQ	>170,000	>$10	–

to mitigate catastrophe risk? True impact will only be achieved when the issues described above are fully recognized and actions are taken to improve the situation. International partnership means understanding the socio-economic constraints of the two partners. It means understanding the incentive structure and working with that structure. Understanding individual and societal incentive structure could facilitate in developing strategies that would work for a nation in mitigating catastrophe risk. True partnership means working with the strengths of the two partners rather than one partner acting as a giver and one partner acting as a receiver. Working with partners may be somewhat unconventional from the perspective of the usual engineering and science-related disciplines. However, this approach is very much needed. Insurance and reinsurance companies, banks, media, grass roots organizations, political and commercial leadership and similar other partners can really make a difference to the economic consequences of natural disasters. They should be part of the partnership for disaster mitigation. The old ways of creating and working with international partnerships have not clearly worked. It is time to be bold and different and to try new ways of collaborative efforts. We need to take a careful and critical look at our old ways and determine what needs to be changed. Through such introspection and through innovative means we can greatly improve global natural disaster risk mitigation.

It is interesting to note that the last mile is not connected even in highly developed countries. The Kobe earthquake in Japan clearly pointed out the gaps between knowledge and practice of earthquake risk management. Hurricane Katrina and its impact on Gulf Coast states of USA showed a total disconnection between intellectual and material resources available and their use in helping the impacted citizens. Not only the pre-event preparation was less than desirable, but the post event response and reconstruction were so poorly managed that the impact will linger on for many years to come. Again, the last mile was not connected.

2 The Problem at Hand

With all the knowledge at our disposal, our score card for catastrophe risk reduction is poor at best. As mentioned earlier, Fig. 1 shows economic losses and insured losses world wide over the past five decades. The losses are

Table 2 Economic & insured losses comparison of decades 1950–1999

Decade	1950–59	1960–69	1970–79	1980–89	1990–99
Number of Events	20	27	47	63	91
Economic Losses	42.1	75.5	138.4	213.9	659.9
Insured Losses	–	6.1	12.9	27	124
Ratio of Insured to Economic Losses %	–	8.1%	9.3%	12.6%	18.8%

Table 3 Economic & insured losses comparison of decades 1950–1999

Factor	70's – 60's	80's	90's – 60's
Number	1.74	2.3	3.4
Economic Losses	1.83	2.8	8.7
Insured	2.11	4.4	20.4

increasing exponentially. There is hardly any dent in life loss figures. In other words, it is clear that societies impacted by these catastrophes have not reaped the fruits of technological and scientific advances. In fact, things have gotten worst. This can be attributed to the three causes of disconnection between knowledge and ground realities. Table 2 shows the progress (or lack of it) we have made between 1950 and the year 1999 on economic and insured losses. Table 3 shows that the economic losses have increased almost 9 fold and the insured losses have increased twenty fold. This is hardly a score card worth showing to any society seeking assistance to mitigate their catastrophe risk.

Looking ahead, the picture seems even more pessimistic. Figure 2 shows the fifteen most populous cities in the world in 2015 (United Nations; State of the World's Cities, 2004/2005, 19/8/2004). 12 of the 15 mega cities are in developing countries. These are also the cities with moderate to high catastrophe risk potential. A mega event near any of these cities could be disastrous.

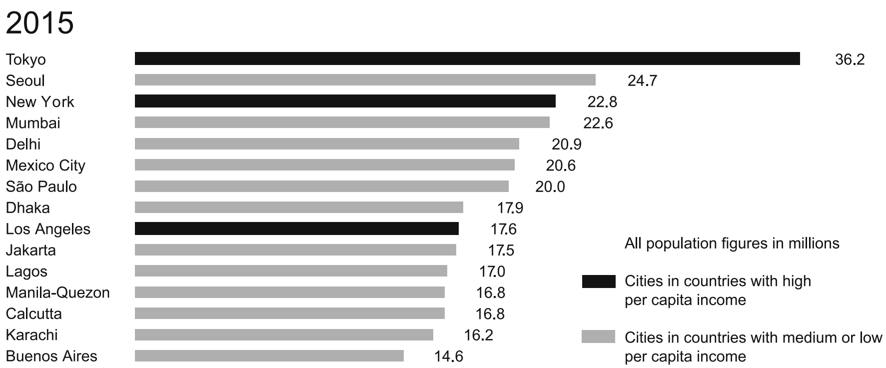

Fig. 2: 15 Most populous cities in the world in 2015 (United Nations)

From above discussions, it is obvious that the increasing knowledge in risk management has not reached the implementation stage where it can make a difference. The discontinuity is more pronounced due to second and third causes articulated in the previous section. It is particularly severe in our inability to connect the last mile.

In the telecommunications industry, they define the most crucial link between available technology of narrow and broadband communication and the use of that technology by a typical homeowner as the problem of the 'Last Mile'. The concept is that, unless the last connection between the homeowner and the most sophisticated available technology is not there, all the available technology cannot be effective for the vast market of consumers. The problem of the last mile continues to be a challenge in that industry and giants of the telecommunications industry are still struggling for the control of the last mile.

It seems from the evidence that we have similar problem in mitigating catastrophe risk globally in general and developing countries in particular. It is not clear whether the work many organizations have done over the past few decades in mitigating (say) earthquake risk in developing countries is not appropriate. However, it is clear that the strategies that have been used to improve risk profile have not been as effective as one would expect. Also, it is quite possible that the past efforts are not yielding desired results because we have not understood what motivates or does not motivate individuals and societies to plan mitigation efforts even before the catastrophe takes place. It is often said that unless one understands the incentives that drive individuals and societies to take action, it would be hard for any strategy for risk mitigation to be accepted and implemented. So, may be enough is not done to educate society on this concept.

As mentioned earlier in this section, with the massive global urbanization, the problem of catastrophe risk is likely in increase rather than decrease, unless innovative approaches to risk mitigation are initiated and the last mile is connected. Figure 3 (United Nations; State of the World's Cities, 2004/2005, 19/8/2004) provides an interesting trend. For the first time in human history, there will be more people living in urban areas than in rural areas. This concentration of population has direct impact on increasing natural disaster

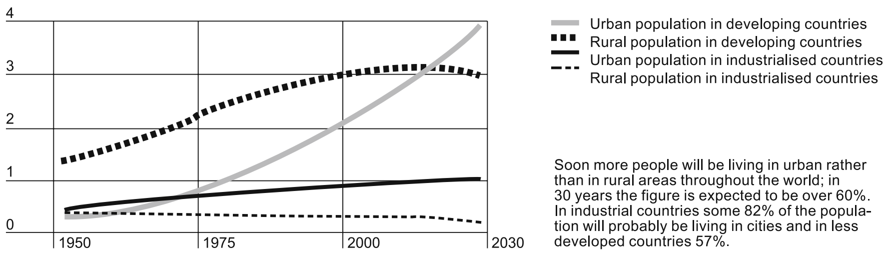

Fig. 3: The gap is widening (United Nations)

risk. It bears repeating here that the leadership in helping societies manage their catastrophe risks, whether in developed countries or in developing countries, has been taken by engineers and scientists. Very few social scientists, economists, finance professionals or public policy experts have been involved. This has been a major stumbling block in coming up with workable, effective and acceptable strategies to manage catastrophe risk due to natural events. This has to change.

3 Some Observations

Based on past experience, it seems like the following actions do improve an ability of a country or a region to make measurable improvement in their catastrophe risk. As an example, for earthquake related disasters, the following actions have positive impact.

1. Awareness at the individual, family and community level of earthquakes and what to do before, during and immediately after an earthquake.
2. Social and political preparedness for the next catastrophic event at all levels of public agencies and private enterprises.
3. A positive role and attitude of government in 'educating' people about the cost and benefit of mitigation measures. Policies that develop proper incentives for people at risk to take mitigation measures.
4. Earthquake codes and their strict implementation in urban and rural communities for engineered structures.
5. 'How to' type of instructions for building non-engineered and rural structures.
6. Help at the village or community level to build safer homes, safer schools, safer hospitals and safer community infrastructures.

Actions such as the ones suggested above, and if properly implemented, do help communities to reduce their earthquake disaster risk. An equally important aspect to help mitigate catastrophe risk is to make sure that team working towards connecting the last mile is made up of multidisciplinary professionals. Once the risk is quantified and identified, it is important that public policy experts and risk management experts work as a team. In the past, too often, politically and financially unrealistic strategies are suggested by technical people. Such a one dimensional view (technical solutions) of complex social, political and economic problem has very little chance of being applicable or implementable. This type of assistance or strategy development has what created a problem of disconnection at the last mile.

Personally, author of this paper has been involved in many professional, non-governmental, philanthropic, academic and similar organizations. Many of these organizations have worked internationally with great passion and dedication to help reduce the problem of earthquake risk in many developing countries. Many of these efforts have resulted in visible improvements in the earthquake risk profiles of communities in developing nations. These efforts have involved meetings, conferences, workshops, seminars, courses, public lectures or just simple networking. All these efforts and means for a noble cause of risk reduction are fine and laudable. However, how many of these efforts pass the test of linking with the last mile? Many of these efforts are between people who are like minded. These meetings are attended by academics and senior governmental officials, who are usually the most educated and 'aware' people. How many times have we seen that the connection between such groups and the people in the last mile are not complete? The connection is often broken after a meeting or a workshop or a conference ends. As an example, how often does a $100 000 grant to help reduce earthquake risk in a specific developing region of the world ends up with 90% of that amount being spent in conducting a workshop or a conference with hardly anything remaining for the last mile? In many developed countries, quite often, hundreds of thousands of dollars are spent in developing a program, in travel, in discussion meetings, in workshops, etc. with very little left for the ultimate beneficiary who is supposed to be helped by the experts. How will the risk profile and the risk culture ever change in developing countries unless we are connected all the way through to and including the last mile? It often seems like many projects and meetings are held as an end in itself. Many of these projects or meeting proceedings are under the illusion that the responsibility of experts ends with the publication of the paper or conclusion of a symposium. In catastrophe risk management field, one must travel not only the first 99 miles but must travel the last mile to make any difference.

4 Major Issues Related to the Last Mile

It is generally accepted by many who have worked towards earthquake risk reduction in developing countries that non-scientific and non-technical issues play a major role in implementing known risk reduction strategies. Let us look at some of these issues.

1. Perception of risk is an important issue. Without the society's understanding of the type and level of risk, it is very difficult if not impossible to develop and implement strategies for earthquake risk reduction. Many developing societies live their daily lives with many different risks. Unless it is clear to them how catastrophe risk fits into their hierarchy of risks, it is very hard for them to either 'get excited' or do something about that risk. So the first and foremost requirement for a developing society to implement needed risk reduction strategies is to understand the catastrophe risk and how it relates to other human-made or natural risks. In the experience of the author of this paper, many developing societies have not properly understood catastrophe risk. The experts in these countries have done a relatively poor job of raising

the awareness of the citizenry about the problem and possible solutions. Most experts in those countries have not taken special effort to travel the last mile.

2. In a society with many competing demands on available resources, it is not clear to many as to how one can balance the risk/reward equation. What level of resources needs to be committed to achieve an acceptable level of safety is a complex problem. Even in industrialized countries the answers to such questions are not obvious. So, in an economically developing country, it is even more difficult to justify the time and resources needed for catastrophe risk reduction.
3. There is a widespread perception that to do anything about mitigating catastrophe risk, the immediate or short-term cost is enormous. The technical community has mainly propagated this perception. The message has been that earthquake-resistant structures require specialized knowledge and that the cost is not trivial to build earthquake-resistant structures or to upgrade existing structures to some acceptable level of performance. This may be true, but there are also many non-capital intensive risk mitigation options. They include raising awareness of the citizens and self-help solutions. Community based retro-fitting of schools and other important structures can also be achieved without great costs. Development of disaster management plans and implementing those plans can help post-disaster recovery. Risk transfer options such as insurance pools can be developed. Implementing some of these options can make a great impact on the risk profile of that community.
4. There is relatively little communication between researchers, academics and a few well-known professionals on the one hand, and the rest of the country, which is at risk, on the other. The few 'world class' individuals in the country have not been able to make their citizens, their engineering community, their governmental organizations and their regulators aware about the type and level of risk and what measures would buy maximum benefit at minimum cost. This has created an awareness vacuum. Without a 'bottom up' interest in implementing risk management strategies, it is very difficult to make any headway towards catastrophe risk reduction.
5. Professionals, such as architects, structural engineers, contractors, government inspectors, etc. have very little professional accountability for poor performance of structures. Even in countries where good building codes exist, there is very little effort to implement and enforce those codes. As a result, we have seen great death and destruction in many recent earthquakes. Ability to practice these professions is not based on licensing or accountability checks.
6. In developing countries, usually an organizational infrastructure that allows a good working partnership between academics, engineering practitioners, government regulators, financial institutions and social activists does not exist. Thus, the time between the generation of knowledge and its implementation on the ground is excruciatingly long.

These and many such reasons can be cited for a lack of progress in many countries. One of the most frustrating observations is that there are groups of countries where there is knowledge, there are resources and there is awareness. China, India, Turkey and Iran are some of these counties where modern understanding of earthquake resistant design (as an example) and mitigation strategies exist. However, for the reasons cited above, the knowledge is not converted into practice. Therefore, there is very little hope that a major urban centre in India or China or Turkey will perform well in a future earthquake. What can we do in those countries to make a difference? Is the problem of connecting the last mile making it difficult to achieve desired outcomes? For too long, individuals in some of those countries have been 'preaching' to their own kind. Moreover, it could be said that foreign 'experts' have been preaching to local experts and the resulting information exchange terminates at that level. It is time to change the way risk mitigation and risk management has been approached in these countries. It is important to understand the last mile that will make all the worthwhile efforts connect fully, all the way from articulation of problems to possible solutions and actions.

5 Concluding Remarks

There are many collaborative projects between the developing countries and developed countries in the field of catastrophe risk mitigation and management. Many of these projects are funded either by the governments or philanthropic foundations. Considerable funds are invested in such projects. Unfortunately, it is very difficult to demonstrate the efficacy of such efforts when one looks at life and economic losses. The reasons for this lack of progress are complex. However, we must investigate the root causes of why our efforts are not bearing fruits. It seems that vast sums of money are spent by experts talking to other experts. Conferences and workshops are held in which like minded people with similar academic and professional backgrounds talk to each other. Also, there is a miss-match between expertise and how and where that expertise is applied. Science and technology based experts try to take leadership in social and public policy issues. Social scientists and public policy experts try to dabble in developing scientific and technology strategies for risk mitigation. Instead of working in synergy, there is often crossing of disciplinary turfs, resulting in ineffective implementation strategies and cost over runs. The original vision and mission of assistance programs are forgotten and running a conference or a workshop becomes an end in itself. All these well intentioned but ineffective strategies must change. There needs to be a paradigm shift in how these programs are conceived, executed, and finally implemented. There needs to be a connection with the Last Mile. Unless we change our strategies for risk mitigation in developing countries, there is very little hope of making a difference.

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A Phenomenological Reconstruction of the Mw9 November 1st 1755 Earthquake Source

Robert Muir-Wood and Arnaud Mignan

1 Introduction

The 1755 'Great Lisbon' earthquake is one of the two or three most studied earthquakes in history. However, unlike other iconic earthquakes, such as Tokyo in 1923 or San Francisco in 1906, there is still no consensus as to the location and extent of the originating fault rupture.

This review sets out to synthesize all the different contemporary information sources on the event itself and then to build from these the outline of what can be understood about the structure of the source. This synthesis of information is complicated because the reports on the 1755 earthquake are heavily biased towards the north of the affected region even though the areas of significant (MSK VIII and higher) damage are comparable north and south of the Gulf of Cadiz.

Large earthquakes defy simple scaling relations for determining their magnitude (Frankel 1994). Based on felt area radii, Johnston (1996) extrapolated a moment magnitude Mw of 8.7 +/– 0.39, while Abe (1979) estimated a magnitude based on the logarithm of farfield tsunami heights to arrive at a Tsunami Magnitude Mt = Mw of 8.75 or greater. In order to match the ratio between the heights of the 1755 and 1969 tsunamis at local ports Baptista et al. (1998a) required an energy release about 40 times greater than that of 1969 (Mw7.9), implying a 1755 magnitude between 8.9 and 9.4. As discussed in this chapter, a magnitude of at least 9.0 is indicated from the energy radiated into the farfield at long periods, which was stronger than in other recent magnitude 9+ earthquakes.

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2 Summary of Proposed Sources

A number of tectonic structures have been proposed as the source (or a component of the source) of the Nov 1st 1755 earthquake (see Fig. 1).

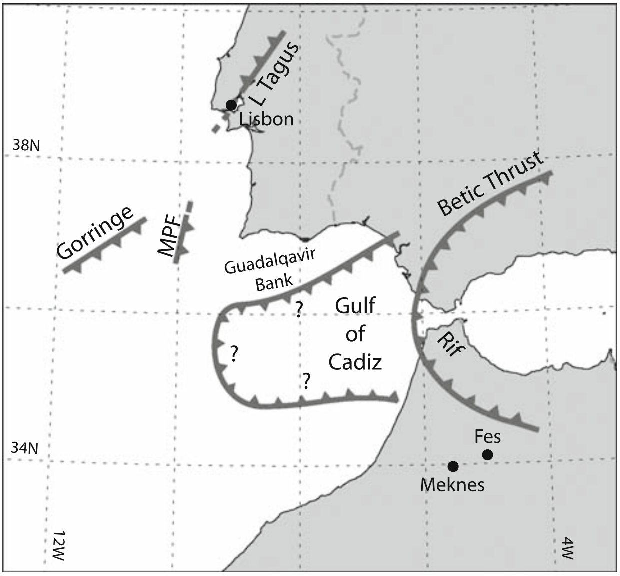

Fig. 1 Principal tectonic structures identified around SW Iberian continental margin. Note that the Gulf of Cadiz overthrust structure proposed by Gutscher et al. (2002) still remains controversial

2.1 Gorringe Bank

The Gorringe Bank is a northeastern trending asymmetric ridge of oceanic lithosphere approximately 180 km long and 60–70 km wide, that rises to within 25 m of sea level. The reasons and timing for the uplift are not resolved, but most authors conclude that the ridge has been uplifted by two bounding reverse faults implying very high levels of horizontal compression (Sartori et al. 1994), with an estimated 50 km of crustal shortening since the mid-Miocene (Hayward et al. 1999).

Through the 1990s, the source of the 1755 earthquake was widely considered to be a bounding fault to the Gorringe Bank; as for example by Johnston (1996) who proposed 12 m of displacement on a 200 km long reverse fault extending down to a depth of 50 km. There were two principal arguments in support of this source: (a) that the ridge was by far the most prominent feature with a presumed tectonic origin in the area to the southwest of Portugal; and (b) the occurrence of the Feb 28th 1969 Ms7.9 earthquake was taken to indicate activity at the ridge. However the 1969 earthquake was located to the southwest of Gorringe Bank beneath the Horseshoe Abyssal Plain, on a fault without pronounced pre-existing topography, although the focal mechanism showed reverse displacement with a minor strike-slip component on a N35W striking fault plane with dip angle of 52 degrees (Fukao 1973) similar to the orientation of the Gorringe Bank. The well studied macroseismic field from the 1969 earthquake was also considered to have some similarities with the intensity field from the 1755 earthquake although typically two intensity grades lower However Baptista et al. (1998b) explored a potential 120 km long reverse fault source for the 1755 earthquake on the Gorringe Bank and found from tsunami arrival times that this location was too far to the west.

2.2 *Marques de Pombal Fault (MPF)*

The acquisition of multichannel seismic reflection profiles from the offshore continental margin of SW Iberia has made it possible to identify and map active tectonic structures cutting through the Quaternary sedimentary section and intersecting the sea floor. Inevitably only significant vertical displacement is easy to see in these sections. The 100 km long N-S trending Marques de Pombal thrust, located 100 km offshore SW of Cape St Vincent, midway between the Gorringe Bank and the coast was first identified as an important neotectonic structure by Zitellini et al. (1999 and 2001) in a multi-channel seismic reflection survey. The structure is located along the 1755 tsunami source zone, proposed by Baptista et al. (1998b) from tsunami travel times.

2.3 *Guadalqavir Bank – Northern Gulf of Cadiz Reverse Fault*

Offshore to the south of the Algarve coast in the northern Gulf of Cadiz another active compressional N75E reverse fault structure was identified in seismic reflection profiles at the northern edge of the accretionary wedge (Jimenez-Munt et al. 2001; Negredo et al. 2002). This fault was considered a potential source of the M6.5 1964 earthquake (Udias and Arroyo 1970) and has an orientation similar to that of the N55E fault identified to be the source of the 1969 earthquake. Baptista et al. (2003) proposed that the 1755 earthquake source was a compound of the two separate fault sources of the Marques de Pombal thrust and the Guadalqavir Bank fault. However what is observed on seismic reflection profiles intersecting the sea floor is likely to be a short section of high angle reverse stepover structure of fault systems that also have a less easily detectable low angle or strike-slip configuration.

2.4 *Gulf of Cadiz Subduction Zone Overthrust*

The existence of a 180 km N-S shallow easterly dipping overthrust fault system below the Gulf of Cadiz was first proposed by Gutscher et al. (2002), forming the top of a proposed subduction zone overthrust system passing to the east under the Straits of Gibraltar. Thiebot and Gutscher (2006) considered evidence from seismic reflection profiles in the western Gulf of Cadiz basin, suggested ramp faults cutting through to the sea floor. The existence of active compressional tectonics in the region, characteristic of a subduction zone forearc setting is revealed by the population of mud volcanoes in the eastern Gulf of Cadiz sea floor, indicative of the active dewatering (Gardner 2001; Pinheiro et al. 2003; Somoza et al. 2003; Rooij et al. 2005). The majority of the 30 volcanoes so far identified are located along a front extending to the SSE from close to Faro on the Algarve Coast of Portugal as far south as the continental margin of Morocco. While the proposal that there is a deep subduction zone beneath the Gibraltar arc remains controversial, (see Platt and Houseman 2006), the existence of a shallow easterly dipping overthrust fault system beneath the Gulf of Cadiz, fits a number of features of the 1755 earthquake source.

2.5 *Lower Tagus Valley Fault*

A Lower Tagus Valley LTV fault zone was trenched and found to present geological evidence of recent displacement by Fonseca et al. (2000), who estimated 0.5–0.7 mm/yr displacement over the past 1500 years (Vilanova et al. 2003; Vilanova and Fonseca 2004). Based on the degree to which high intensity ground shaking in 1755 occurred in the vicinity of the fault, as well as contemporary reports suggesting localized deformation and tsunami generation, Vilanova and Fonseca (2004) propose that the LTV fault was involved as an element of the 1755 earthquake source.

3 Phenomenological Evidence on the 1755 Earthquake Source

While all the potential fault sources that have been proposed can explain some of the data on the 1755 earthquake, no proposal has been capable of explaining the totality of observations. This paper explores nine separate lines of phenomenological evidence that can be retrieved from contemporary accounts concerning:

1. Event duration and source complexity
2. Levels of ground shaking as reflecting distance to the rupture
3. Farfield long period effects reflecting the generation and transmission of long period ground shaking
4. Evidence for coseismic deformation
5. Nearfield tsunami amplitudes and travel times
6. Farfield tsunami polarization and amplitudes
7. Triggered seismicity and the implications on coseismic far-field stress changes

All the different classes of phenomenological evidence have then been synthesized to constrain what can be projected as to the size and configuration of the fault rupture source.

3.1 Duration and Complexity – Ground Motion

At any location the maximum duration of strong ground motion will be sensitive to the disposition of the fault in relation to the observer as well as how the rupture propagates. Consider an idealised 300 km 'line' fault with realistic terrestrial physics – a rupture velocity of 3 km/sec, radiating the strongest modes of surface wave vibration at a group velocity of 4 km/sec for the lower frequencies, reducing to 3 km/sec for the higher frequencies. Meanwhile the fastest P waves travel at 6 km/sec. What is the maximum duration of the earthquake at different observation points – assuming the observer detects the full sequence of vibrations?

For an observer at the epicentre immediately above the start of line rupture, the final vibrations will be radiated from the other end of the fault 100 seconds after the start of rupture (and after vibrations have first been felt) and the slowest surface waves will then take 100 seconds to arrive, implying a duration of 200 seconds. For an observer at the opposite end of the fault – the first P wave vibrations will be felt after 50 seconds, and the rupture will itself arrive at 100 seconds implying a duration of 50 seconds of strong shaking. At the midpoint of the fault the duration will be 125 seconds. In either direction beyond the line of the fault, the duration of the full suite of vibrations will extend by the difference in velocity between the fastest P waves and the slowest surface wave group velocity, of one minute per 360 km, and slightly less than this for directions orthogonal to the fault.

Real faults have a 2D surface, can undergo bilateral as well as unilateral rupture, the initial P waves may not be felt at significant distances while vibrations may not be perceived radiated from far sections of the fault. For example while the Mw 9.2 1964 Alaska earthquake, involved 800 km of fault rupture starting at Valdez and ending beneath Kodiak Island 300 seconds later, an observant geologist at Valdez only felt vibrations for 210 seconds implying that vibrations radiated from further than about 300 km down the fault rupture were too weak to be observed. In the Indian Ocean earthquake of Dec 26th 2004 the fault rupture extended for 1200 km and had a duration of 8 minutes, but the fault continued too far from any observer for the whole wavetrain of vibrations to be felt, at a single location.

In terms of the reported durations from the Nov 1st 1755 earthquake (see Fig. 2). At Cadiz and Lisbon the duration of ground shaking was reported as 6 minutes (Gentlemen's Magazine, Feb 1756). At Tangier and Tetuan in Morocco the duration was 7-8 minutes, and involved three violent shocks. At Oporto about 300 km to the north of Lisbon it lasted 7 minutes and 600 km to the northeast at Madrid it was reported as 8 minutes, suggesting that the wave

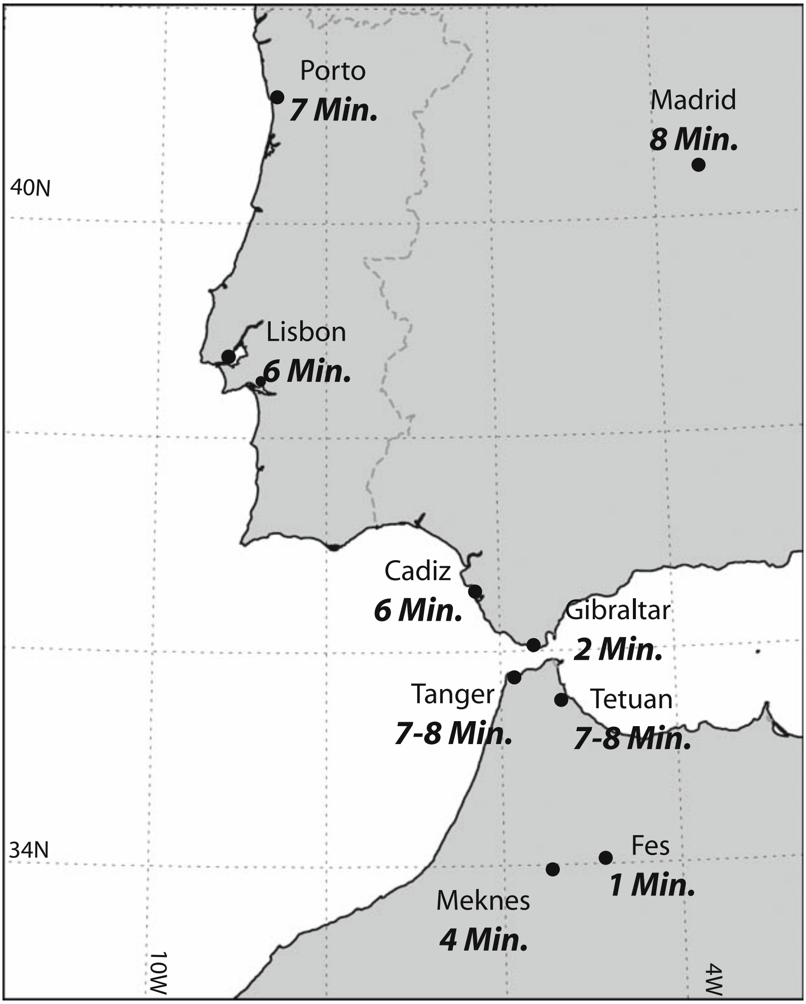

Fig. 2 Reported durations of the Nov 1st 1755 (09.30) mainshock

train was extending much as would be predicted and also that the full sequence of ground motions continued to be strong enough to be observed. However further to the southeast the amplitude of ground motion passed below detectability for some part of the wave train.

Not knowing the point of the beginning or end of the fault rupture, or how far the fault was located from each of these locations, if the 6 minute duration at Lisbon and Cadiz was within 300 km of the actual fault rupture then if there was a single episode of fault rupture without interruption this must have had a minimum duration of 150 seconds (i.e. half of 5 minutes). Given that the observations at two distinct azimuths from the fault in Cadiz and Lisbon were the same, and that the rupture was almost certainly less than 300 km from Lisbon implies a longer rupture duration, of 200 seconds or longer. At standard rupture velocities of 3 km/sec (the 2004 Indian Ocean earthquake had an average rupture velocity of 2.8 km/s while the 1964 Alaska ruptured at an average speed of 3.5 km/sec: Christensen and Beck 1994). 150 seconds therefore gives a minimum 450 km long fault, while 200 seconds implies at least 600 km.

These lengths could be overestimated if rupture was interrupted between fault segments. Reports from Lisbon often stress that there were three distinct phases of shaking, with a weaker phase of long period motion for the first minute, followed by stronger vibrations that led to significant building damage. However this is similar to the reports from Valdez in 1964 in which a phase of weaker motions gave way to much stronger ground shaking, simply as a function of the amplitude of the different wave trains and the location of specific areas of strong energy radiation along the fault. As discussed below, the nature of the long period seiching in the 1755 earthquake demonstrates that the wave train had become coherent and continuous in the far field, implying that there was in fact a single episode of fault rupture.

3.2 The Macroseismic Field – and What it Reveals About Proximity to the Fault Rupture

There is general agreement that the fault rupture associated with the 1755 earthquake was predominantly located offshore. Observations from ships indicated the strongest impulse (from water-transmitted T waves) in a region extending from latitude 38.30 N down to 36.24 N and between 7 and 12 W (Rudolph 1887).

Detailed reports of the onland affects of the 1755 earthquake were collected across Spain and Portugal soon after the earthquake, from which macroseismic intensities have been interpreted and mapped (Martinez Solares et al. 1979; Moreira 1983). However no comparable survey was performed in Morocco and the observations in that country have never received the same scrutiny. (Intensity maps often misleadingly incorporate the high intensities of another damaging earthquake that occurred in northern Morocco later in November 1755.)

At the time of the earthquake Morocco was split into two caliphates: 'Fez' to the north and 'Morocco' to the south (with its capital at Marrakech), separated by the Oun Er-Rbia River. A small number of detailed accounts of the earthquake have survived (see Aboulqasem ben Ahmed Ezziani (1886), Cigar N. (1981), Gazette de Cologne, Jan 11th 1756. Gentlemens Magazine (Jan 1756), Gentil L. and Pereira de Sousa F.-L, (1913), Rolland F.A. (1923), Taher 1979 as well as Manuscript letters from Franciscan Missionaries in Morocco back to Madrid). From these accounts it is notable that there is an increase in the severity of the earthquake in passing to the southwest. In the north where a strong E-W shaking was reported accompanied with 'a noise like millstones', descriptions are consistent with a general MSK intensity of VII: at Tangiers 'a great pile of ancient buildings near the gate of the town tumbled down, but damage throughout the town was otherwise fairly limited', while at Fez – 'Buildings were injured but only 2 or 3 people were killed. Bricks fell out of walls.' However along the coast to the southeast, at Sale – 'it did vast damages – numbers of house having tumbled down' while inland at Morocco (Marrakech): 'The majority of the houses and public buildings of the town were totally flattened, and a great multitude of the inhabitants were buried in the rubble'. At Safi and St Croix (Agadir) on the southwest coast 'many houses and other buildings were destroyed, which buried a large number of people'. All of these accounts are consistent with intensities VIII-IX – see compiled map for the whole macroseismic field Fig. 3. There were major landslides in the mountains of

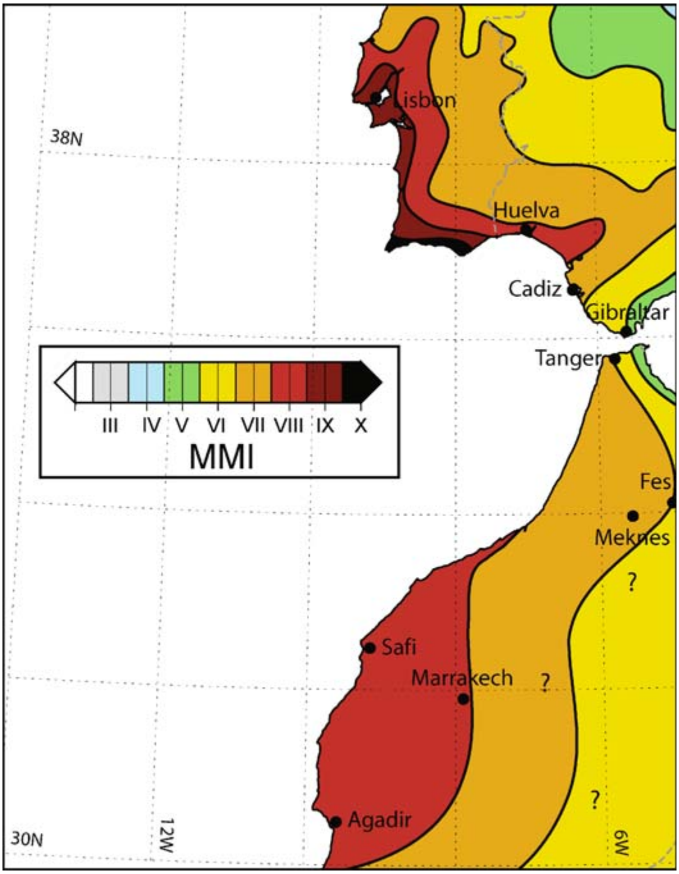

Fig. 3 Macroseismic MSK intensities of the Nov 1st 1755 mainshock. Compiled isoseismic map from Grandin et al. (submitted 2007), modified for Morocco. Data provided by Pereira de Sousa (1919) and Moreira (1984) for Portugal and Martinez-Solares (1979) for Spain. The MSK intensities for Morocco are determined from several accounts and differ from Levret (1991) (see text)

southern Morocco: at 8 leagues from Marrakech one report mentions up to 10,000 soldiers killed by a combination of building collapse and rock falls.

Strong ground motion is a good indicator of proximity to the fault rupture. In recent major 'comparable' (M8 onland shallow thrust fault earthquakes), such as the 'Great Kwanto' Tokyo M8.2 earthquake of Sept 1st 1923 and the M7.9 Gujarat earthquake of Jan 26th 2001, the highest earthquake intensities (in particular MSK IX and X, reflecting general destruction) were only found overlying the fault rupture. The region with the highest levels of damage in the 1755 earthquake was the western Algarve coast as far to the east as Tavira, where MSK intensity levels are consistently assessed as X, reflecting the fact that almost all buildings were leveled by the earthquake. By comparison with Gujarat in 2001 or southern Tokyo Bay in 1923 the western end of the Algarve must have overlain (or been located in close proximity to) a section of the 1755 fault rupture. Southern Morocco is also clearly closer to the fault rupture than northern and northeastern Morocco, implying that the fault rupture extended far to the south, even as far as the coastline of Morocco at 32–33 N? The levels of damage and the reports of intense high frequency vibration in the City of Lisbon also imply that there was nearby fault rupture (as proposed by Vilanova and Fonseca 2004).

3.3 Farfield Long Period Affects

The most extraordinary feature of the 1755 earthquake remains the range and intensity of farfield long period effects (see spatial extent on Fig. 7). Such effects were completely new to the experience of Europeans and to scientific observation, and 250 years later still remain the most widespread and varied examples of earthquake seiching known from any earthquake. At distances of a few hundred kilometers from the earthquake source, long period strong motion could be damaging: as at Malaga where the tops of some high buildings fell. At distances greater than 1000 km there were many observations of chandeliers hung from cathedral roofs oscillating, as in Milan and Amsterdam. Across a broad area of the coastal plain of northern Germany, on this Sunday morning, 'branches' hanging from the roofs of churches were seen to vibrate: as at Emshorn, Bramstadt, Willster, Kellinghusen and Melidorf. At Glucksdorf, three large branches, each weighing a ton, were set into slow oscallation from East to West for the Space of an Hour.

Across Holland and northern Germany, many rivers and canals were sent into pronounced oscillation (see accounts in the London Evening Post, Dec 6–9th 1755). The River Eidar which separates the old Town of Rendsburgh from the new – rose to a great height. The water of the Staehr was very much agitated at Itzehoe, as was the water that surrounds the Garrison at Fort Steinbourg, while the Schwinge and the Ost and were greatly agitated at Cuxhaven. In southern Britain observations came from ponds and lakes: as at Pibley Pond in the county of Derby, English Midlands: where in a 30 acre body of water: the water rose two feet (0.6 m) and continued flowing backwards and forwards for 2 hours. Smaller ponds in the neighbourhood of Bury St Edmunds, in Suffolk continued oscillating for 8 or 10 minutes, while in a pond at Dunstal in a different mode of resonance 'the water rose successively for several Minutes in the form of a Pyramid, and fell down like a Water spout' (London Evening News Dec 6th–9th 1755). Further to the north many lakes and fjords were sent into motion in northern Britain and Scandinavia (Kvale 1955). At Loch Lomond in west Scotland water levels rose and fell 0.8 m with a period of 10 minutes, with the principal phase lasting for 45 minutes. Similar behaviour was seen at Loch Long and Loch Katrine, and the rise in the waters at Loch Ness was 'so violent as to threaten destruction to some houses built on the sides of it' (Scots Magazine 1755). The area of seiching extended south through Switzerland and canals were sent into prominent oscillation around Milan.

The best modern scientific studies of far-field long period affects were made following the 1964 Alaska earthquake (McGarr and Vorhis 1968) where seiching was measured widely from water gauges but observed less often – as along the Gulf coast of Texas and Lousiana where it affected small enclosed lakes, bayous and coastal navigation canals, with a predominant resonant period of 10–15 second. Widespread seiching was also observed from the 2004 Indian Ocean earthquake (Amateur Seismic Centre 2005) in ponds and tanks in Assam, Jharkand, Maharastra, Manipur, Orissa and West Bengal as well as in northern Thailand and eastern Nepal. However in comparison with the observations made in 1964 or 2004, the 1755 earthquake showed a stronger signature in terms of the ubiquity of observations across a very wide range of resonant periods, implying a greater amplitude and spectrum of long period energy, within a coherent low frequency wave train. This implies a single phase of rupture on a very large seismic source.

3.4 Coseismic Deformation

With a source predominantly located offshore it is not surprising to find few accounts suggesting coseismic deformation (Fig. 4). However Mr Stoqueler the Hamburg Consul at Lisbon was walking outside at Colares at the westernmost point of land near the Rock of Lisbon (Cabo da Roca) when the earthquake hit and recounted that: 'it is there apparent that (the sea) does not reach its usual bounds for you walk almost dry to places where before you could not wade'. (Gentlemens Magazine, March 1756), reflecting an estiumated 0.3–0.4 m of uplift. That there was preseismic and coseismic strain close to this location is suggested by the observation that a fountain at nearby Sintra that was greatly decreased in the afternoon of the 31st, in the morning of the 1st it ran very muddy, and after the earthquake it returned to its usual state, both in quantity

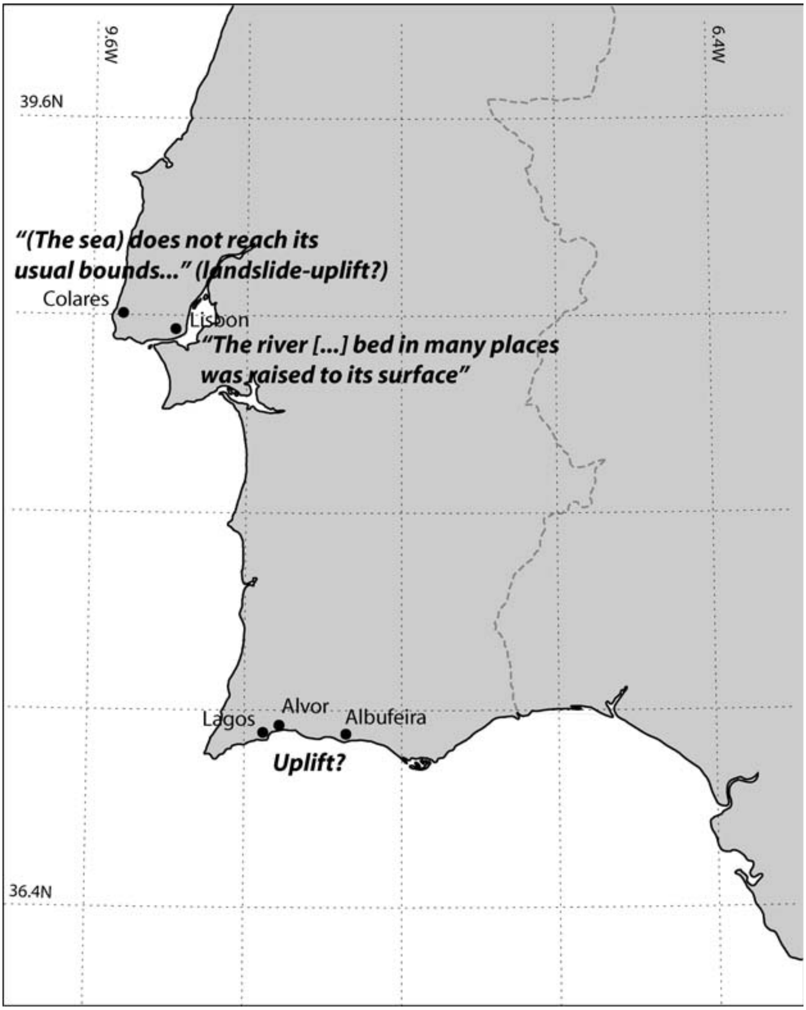

Fig. 4 Reports indicating potential coseismic deformation from the Nov 1st 1755 earthquake

and clearness. Another report suggestive of coseismic deformation came from Lisbon itself where it was said that 'the river which forms a great Bay opposite the town, was equally disturbed: its bed in many places was raised to its surface'.

Along the coast of the western Algarve there were pronounced geomorphological changes, which could imply some coseismic uplift, although it has not been possible to separate out the profound affects of sediment movements associated with the tsunami. After the earthquake and tsunami the harbour at Faro was so choked that the seat of administration was moved to Tavira, while at the harbour of Alvor only small craft could be handled where formerly boats of 45 tons had docked (Chester 2001).

3.5 The Local Tsunami

The 1755 tsunami was particularly destructive along the western Algarve coast where accounts are comparable to the experience of Banda Aceh on Dec 26th 2004. Water levels reached 15 m and more above sea level, as at Alvor where the water flowed 500 m inland and rose to the level of the village. In Nova de Portimao water reached 2.6 m high inside the church, while at Lagos the sea rose 10 m high and invaded the land more than 800 m (Sousa and Pereira). In Albufeira 16% of the population of the village was killed by the tsunami while in Armacao de Pera only one building remained standing. At Cadiz the tsunami reached an estimated 15 m above sea level overwhelming the causeway connecting the town with the shore.

The tsunami was also very destructive along the Coast of Morocco, in Tangier it flowed into the heart of the city, rising 50 feet (15 m) perpendicular, 'leaving behind it a vast quantity of fish and sand'. At Sale the sea flowed into the heart of the city and drowned several inhabitants, overwhelming all those who went outside the walls of the town and causing many deaths. In Algazait – several walls fell down and a great Part of the Town was overflowed. A caravan traveling towards Marrakech along the beach was overwhelmed, killing the animals and large numbers of people.

Baptista et al. (1998b) employed all the available accounts of elapsed time of the nearfield tsunami (relative to the earthquake shaking) along with reported tsunami heights (see Fig. 5), to determine what these revealed about the configuration of the tsunami source. This study provides some important constraints on the fault source. In order to explain the 45 minute arrival time of the tsunami to Figuera (40.14 N) to the north of Lisbon, the seismic source had to extend as far north as the latitude of Lisbon. The best fit in terms of tsunami arrival times was found to be a 300 km long NNW-SSE source located midway between the coast of southwest Portugal and the Gorringe Bank, and running as far south as the Guadalquivir Fault.

However even this source does not extend far enough to the south to explain the observed 30 minute travel time of the tsunami at Safi on the southwest coast of Morocco, (the modeled source predicted arrival times of 70 and 85 minutes – a greater mismatch than for any other observation point). This implies that the deformation must have extended significantly further to the south than proposed by Baptista et al. (1998b).

Also all contemporary accounts highlight the fact that tsunami heights were much greater on the Algarve coast than on the westerly facing coast to the north of Cape St Vincent (Pereira de Sousa 1911), suggesting that there was greater sea floor displacement to the south. In terms of tsunami heights, the modeling of Baptista et al. (1998b) produced reasonable fits with the height data for the northern coastal locations but has problems in generating suitable heights for Cadiz (modeled 5–7 m, observed 15 m) and Madeira (modeled 1.5–2.4 m v

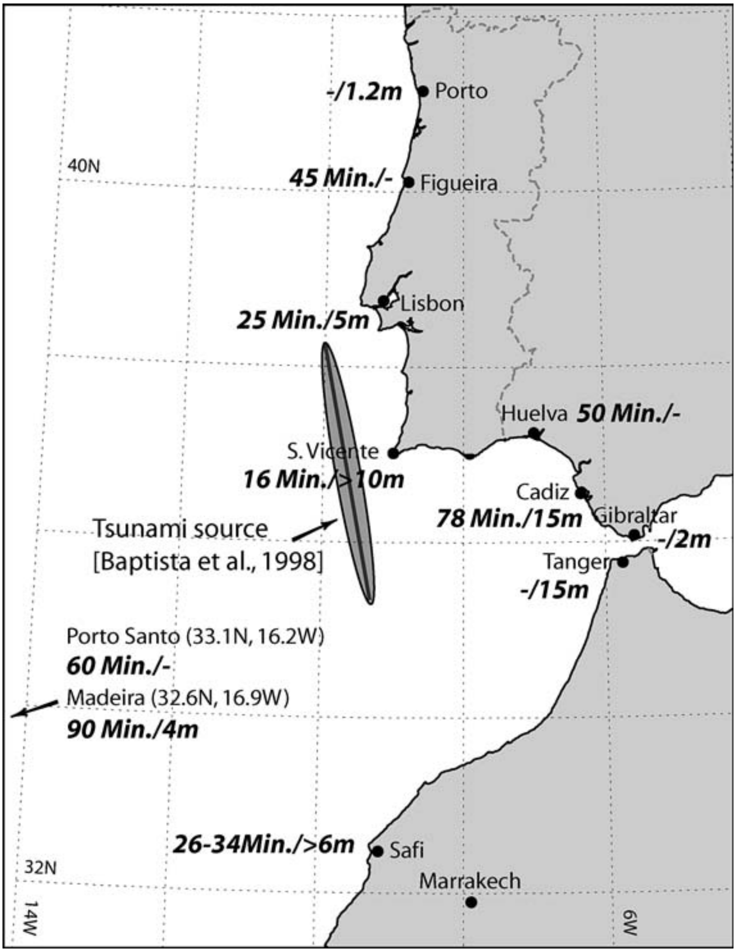

Fig. 5 Tsunami travel times, heights and proposed tsunami source, from Baptista et al. (1998b)

observed 4 m). This suggests again that the actual source must have exhibited higher amounts of seafloor deformation to the south.

There is also a question as to whether there were in fact two separate tsunamis in Lisbon – one generated by local deformation and the second reflecting the arrival of the tsunami from the main earthquake source. According to several eyewitnesses the sea in Lisbon 'rose up first within 10 minutes of the earthquake' (Gentlemens' Magazine, March 1756). At the time of a second great shock, within minutes of the first, a boat captain reported that the river rose at once near twenty feet and in a moment subsided immediately upon this extraordinary concussion.

3.6 The Farfield Tsunami

The occurrence of the 2004 Indian Ocean tsunami has significantly expanded the understanding of farfield tsunami amplitudes and what they reveal about the pattern of coseismic sea floor deformation. A fault extending for hundreds of kilometers creates 'lensing' – in which the long wavelength tsunami propagates coherently in directions orthogonal to the fault, while for directions parallel to the fault there is destructive interference of tsunami waves generated within 2–3 minutes all along the linear zone of seafloor deformation (as the rupture moves at speeds an order of magnitude faster than a deep water tsunami). Therefore the azimuthal variation in farfield tsunami amplitudes of the largest earthquakes should reveal the orientation of the causative fault. For example, the strong tsunami amplitudes observed along an E-W trending band across the Indian Ocean (including Thailand, Sri Lanka and Somalia) on Dec 26th 2004 demonstrate that the causative fault was oriented approximately N-S.

The tsunami from the November 1st earthquake was observed at a number of locations in southern Cornwall, England, occurring at low tide reaching maximum amplitudes of 1–2 m. Given the distance (of around 1500 km), these amplitudes are not very significant and suggest that the tsunami propagation was relatively incoherent to the north.

The timing of the 1755 earthquake and the propagation speeds across the North Atlantic meant that the event had the potential to arrive in daylight at all the colonized ports from Brazil, through the Caribbean and along the east coast of North America. However along all the ports of the East coast of North America such as at New York and Boston, the tsunami went unobserved and therefore presumably had an amplitude less than 0.5 m. In Charleston, South Carolina the rice merchant Henry Laurens, wrote on Jan 12th 1756 in response to a request for information from Gidney Clarke in Barbados, that it was not seen 'here'), The absence of observations along the North American coastline contrasts with what was observed in the islands of the northeast Caribbean (see Gray 1756). On Martinique the tsunami overflowed the low land entering the upper rooms of houses and also retreated a mile. On the island of Saba it flowed twenty one feet (6.5 m). At St Martin's 'a sloop that rode at anchor in fifteen feet of water was laid dry on her broadside' (>5 m). On Antigua the water rose twelve feet perpendicular (3.5 m). The tsunami heights were lower further to the south at Barbados where the amplitude was measured as five feet (1.5 m) – and where the 'water ran over the wharfs into the houses'.

The explanation as to why a tsunami arrived at heights of up to 6 m in the Lesser Antilles while remaining undetected for similar distance ranges along the US East Coast is most simply explained by the polarization of the tsunami amplitudes as a result of the shape of the originating sea floor deformation (see Fig. 6). To maximize the WSW radiation of tsunami wave energy towards the Caribbean the primary orientation of the sea floor deformation off the SW coast of Portugal must have been NNW-SSE.

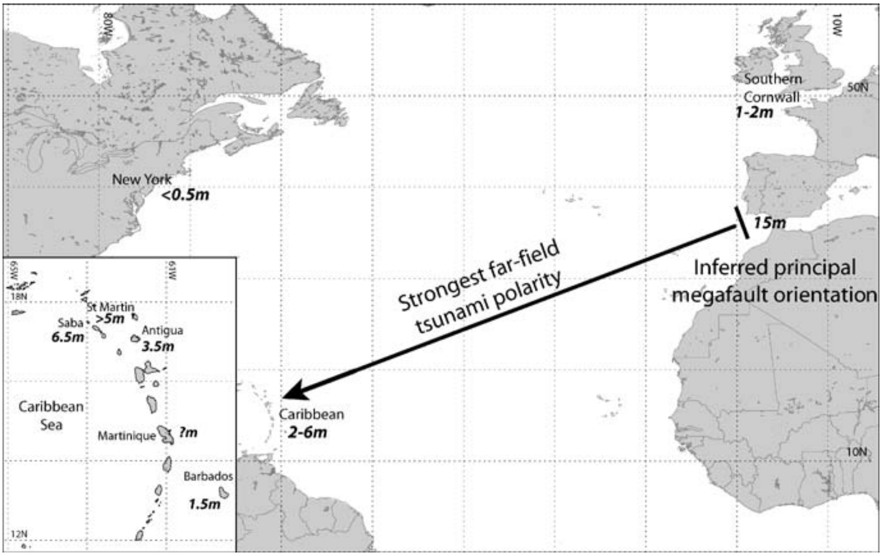

Fig. 6 Farfield tsunami heights reported in the Eastern Caribbean, and implications for tsunami directivity

4 Related Earthquakes

4.1 Preceding Earthquakes

Thirty three years before 1755 on Dec 27th 1722 a major earthquake (assessed as M7+), and accompanied by a local tsunami, caused very high levels of damage along the eastern end of the Algarve coastline – in particular affecting the port of Tavira as well as Loule and Faro, where intensities were mapped as IX (Moreira et al. 1993) . Further to the west, intensities were mapped as VIII. The zone of intense (MMI >VIII) destruction in 1722 lies adjacent to the zone of intense destruction in 1755, suggesting that these two earthquake ruptures might have been contiguous to one another and hence that the 1722 earthquake was preparatory to 1755.

4.2 Aftershocks

In the hours and days following the Nov 1st 1755 earthquake there were many aftershocks, all around the rupture area, so that the list of notable aftershocks at Gibraltar for example, does not overlap with those at Lisbon. There is some suggestion that aftershocks were migrating the rupture north – a major earthquake noted at Lisbon at midday on Nov 1st, was more intense at Oporto 300 km north of Lisbon, where it 'occasioned a good deal of damage, rent several churches from top to bottom and tumbled down one of the turrets of the church of the Congregadoes' (Gentlemens Mag p 562).

One report (London Evening News, Dec 11–13th 1755) suggests (if reliable?) that there may have been a triggered earthquake on Nov 1st close to the northern coast of Algeria, as would explain the account that 'at Algiers, Part of the City is destroyed, and considerable damage done to the Harbour'. Such an earthquake could also explain the observations that the Sea was violently agitated all round the island of Sardinia; that all the Rivers in that Kingdom overtopped their Banks, and drowned great tracts of the Ground, something like an Earthquake; that the damage done by this inundations is very considerable and that 'We have advice that on the first instance the Abundance of Barques, employed in the coral fishery on those Coasts, have been lost.' This could reflect a tsunami from a triggered earthquake off the coast of north Africa (as was observed in the 2003 Zemmouri earthquake off the northern coast of Algeria that caused tsunami damage in the Balearic Islands: Alasset et al. 2006).

4.3 Triggered Mainshocks?

A series of major earthquakes occurred across western Europe, north Africa and Eastern North America in the months and years after 1755. The closer the location in space and time to the Nov 1st earthquake the more that a physical connection can be proposed.

4.3.1 November 18th 1755: Cape St Ann, Massachusetts

The Cape St Ann earthquake situated offshore to the northeast of Boston Massachusetts had a magnitude of M6.2 and is the largest earthquake to have occurred in the New England area since European settlement began in the early 1600́s. Occurring within 17 days of the largest earthquake ever known in the Atlantic Ocean it is tempting to suggest a link, athough the 4000 km spatial separation of these events is too great to be explained within the current generation of earthquake stress transfer models.

4.3.2 November 27th 1755 Meknes, Morocco

On the evening of Nov 27th there was a major earthquake in northern Morocco, reported as 'far stronger' although 'not as long' as the event of Nov 1st. (Some local contemporary accounts are confident that this earthquake was in fact on the night of the 18th/19th). Highest reported intensities were in the city of Meknes where the majority of houses were destroyed, including the palace, many mosques and the tower of the Grand Mosque which was 'demolished right down to its foundation along with the majority of the mosque itself' 10,000 inhabitants of Meknes were counted as dead. The zone of intense MSK IX to X damage extended to Zarhun located 15 km to the north of Meknes, where the Roman site of Volubilis was badly damaged, fissures were noted and a large landslide destroyed the town of Moulay Idriss a few km to the south. The level of destruction was lower at Fez around 50 km to the ENE although there was widespread damage (MSK VII-VIII) and a small number killed. There was however no damage along the coast to the west at either Rabat and Sale. While the full extent of the high intensities in this earthquake is not known, the magnitude is likely to have been in the range M6.5-7. Moratti et al. (2003) propose that this earthquake was located on an E-W, northerly dipping reverse fault outcropping about 15 km to the north of Meknes (i.e. away from the city) but within 5 km north of Fez but this does not reconcile with the relative levels of damage at the two cities, or with the level of destruction at Meknes (which suggests fault rupture in the vicinity of the city) and the question as to the causative fault of this earthquake should probably be left open.

4.3.3 December 9th 1755 Brig in Switzerland

Mw 6.1, one of the major events in history in the Valais region, of southern Switzerland bringing intensities of MMI VIII also caused some minor damage in Milan (Gisler et al. 2004).

4.3.4 February 18th 1756 Aix la Chapelle, Belgium/Germany Border

Magnitude 6.1, (Mw5.8) intensity VIII - the largest earthquake since 1600 in the Lower Rhine Graben region.

4.3.5 December 23rd 1759 Earthquake Kattegat

Largest historical earthquake in the vicinity of Denmark (Ms 5.1-5.6).

4.4 March 31st 1761 Earthquake

The largest of all the earthquakes likely to be linked to Nov 1st 1755 occurred on March 31st 1761 (Borlase 1761). Even though this is the largest earthquake known in Europe since 1755 it is relatively poorly known, in part because in Portugal the Government suppressed accounts fearing that it would lead to ‘consequences of terror and fancy’.

The 1761 earthquake was felt widely onland from southern Ireland in the north (at locations where the Nov 1st 1755 earthquake was not felt), Bordeaux and Barcelona in the east, Morocco in the south and Fayal in the Azores to the west. Onland in Iberia and Madeira intensities were generally V–VI, in Lisbon demolishing some of the remaining 1755 ruins as well as some new buildings ‘to the amount of 20,000 moidores’. There were isolated locations of damage at intensity VII at Evora and Beja inland Portugal, while at Corunna in the northwest corner of Spain the shaking was so strong as to cause landsliding with several properties slipping downhill a few metres. At Madeira the shaking lasted 3 minutes long, and as in 1755 involved E-W motion, leading to rockfalls in the eastern part of the island and damage to some buildings. The earthquake was felt most strongly by vessels between 43 N and 44 N and around 11–14 W also suggesting that the source was situated to the northwest of the Nov 1st 1755 earthquake rupture.

In terms of size measures: the duration of the earthquake was 2 minutes in Morocco, 3 minutes in Madeira, 2.5 minutes at Madrid, 3 minutes at Aranjuez, but 5 minutes in Lisbon suggesting the rupture may have travelled north, starting close to the northern end of the 1755 rupture. The earthquake caused a major tsunami that was detected 1.9 m high in Cornwall, England, flooded quays at Cork, Ireland and arrived 2.4 m high 75 minutes after the shaking at Lisbon. The tsunami was also strong in the Azores and reached 1.2 m in Barbados. At Amsterdam and Maesland Sluis chandeliers swayed and Loch Ness was observed to seiche, rising two feet (0.6 m), indicating once again very significant long period ground motions.

The source of the 1761 earthquake must lie to the northwest of the 1755 source (Baptista et al. 2006): probably about 300 km offshore (although the latitude remains less resolved). From the tsunami a magnitude of Mw8.5 has been inferred, (Baptista et al. 2006) and allied with the far-field seiching and event duration suggests a source 200–300 km long.

4.5 How do Triggered Earthquakes Constrain the 1755 Nov 1st Earthquake Source?

The probability must be considered that some of the major earthquakes, which occurred across western Europe, north Africa and eastern north America in the months and years after 1755, were not independent to the Nov 1st 1755 earthquake (see Fig. 7). The primary candidates to be linked are (a) the proposed displacement on the Lower Tagus Valley fault as an expansion of the Nov 1st mainshock and (b) the earthquake of Nov 27th in Morocco. It also seems possible that the Dec 9th 1755 and Feb 18th 1756 earthquakes in central Europe may have been advanced as a result of the Nov 1st earthquake. Lastly the M8.5 March 31st 1761 earthquake must be considered closely related to the Nov 1st 1755 fault rupture.

Simple stress transfer models have been created in order to explore alternative source geometries insofar as they would be likely to have triggered these subsequent earthquakes. The stress field due to co-seismic displacement associated with the 1755 Nov 1st earthquake is determined for 4 different source scenarios, the Gorringe Bank, the Marques de Pombal thrust, the Gulf of Cadiz subduction zone overthrust and the proposed tsunami source of Baptista et al.

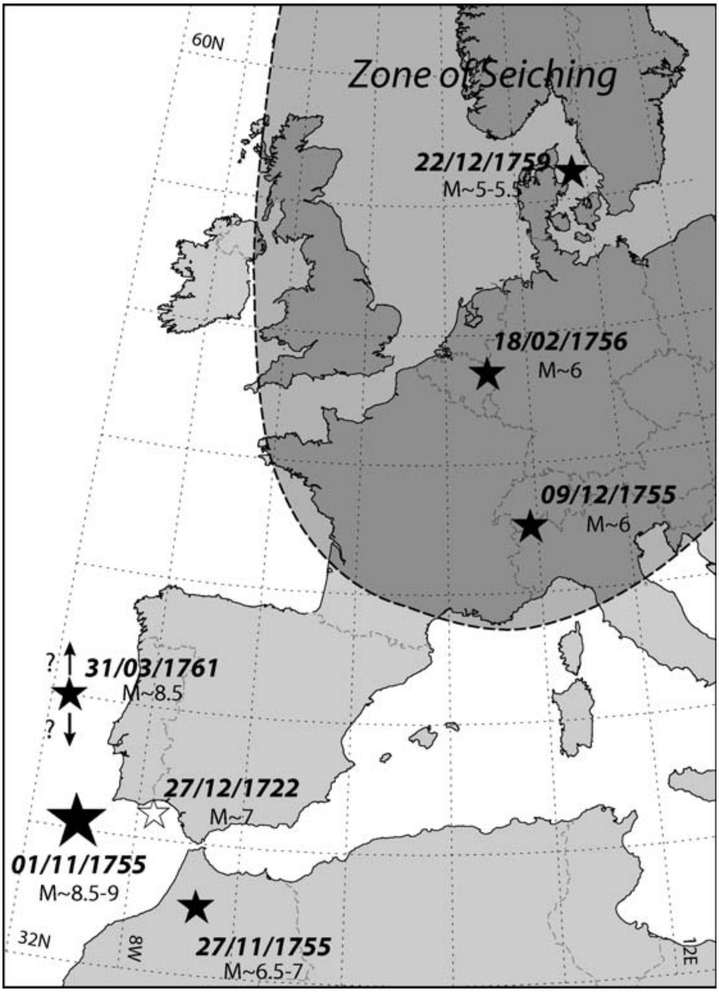

Fig. 7 Principal earthquakes observed before and after the Nov 1st 1755 earthquake, and the principal region affected by seiching of lakes, ponds and canals

(1998b). The source parameters can be found in Table 1 and correspond to a compilation of previous studies, from Johnston (1996) and Baptista et al. (1998a) for the Gorringe Bank, from Zitellini et al. (2001) and Terrinha et al. (2003) for the Marques de Pombal thrust, from Gutscher et al. (2002) for the Gulf of Cadiz overthrust (source simplified, and dip fixed to 25°) and from Baptista et al. (1998b) for the proposed tsunami source.

Figure 8 represents the positive stress changes associated with the 4 different source scenarios. Stress contours of +0.5 bar show the regions where triggering of earthquakes of similar mechanism is more likely to occur. In all cases, the

Table 1 Source parameters used in stress transfer models (apparent coefficient of friction $\mu = 0.4$)

Source	Length (km)	Width (km)	Strike	Dip	Displacement (m)
Gorringe Bank	175	50	55°N	45°	20
Marques de Pombal	150	50	10°N	25°	20
Gulf of Cadiz overthrust	200	200	–10°N	25°	20
Tsunami source	350	150	–10°N	25°	20

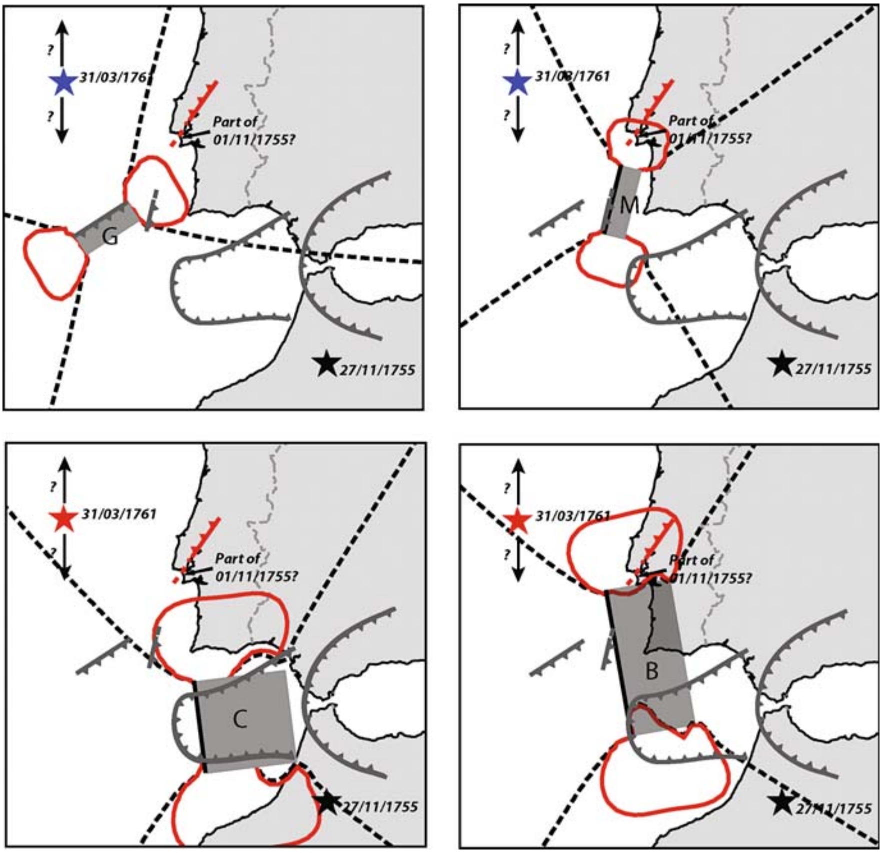

Fig. 8 Alternative fault models of the Nov 1st 1755 mainshock and associated stress changes. Sources G, M, C and B are respectively the Gorringe Bank, the Marques de Pombal fault (extended northward), the Gulf of Cadiz overthrust and Baptista et al. (1998) proposed tsunami source (see model parameters in Table 1). Stress contours are represented in black (+0.5 bar) and in dashed black (0.bar). The Lower Tagus fault as well as the 1761 offshore earthquake are represented in black when possibly triggered, whereas in grey if located in a stress shadow

Lower Tagus Valley fault is likely to have experienced a significant increase of stress. This is consistent with the proposal of Vilanova et al. (2003) that the LTV fault was triggered by the 1755 Nov 1st rupture, within the same episode. The location of the 1761 offshore earthquake is not fully resolved but if we consider the more plausible location represented on Fig. 8, this event could only be triggered by a source oriented NNW-SSE to N-S. A triggering from source C or B might be possible whereas it seems impossible from source G or M (clearly in the stress shadow). The other event that should have been triggered by the 1755 Nov 1st earthquake is the Morocco event of Nov 27th that occurred less than one month later. However, the mechanism of this event proposed by Moratti et al. (2003) is orthogonal to the displacement implied by the majority of the fault sources for the 1755 earthquake, and based on this mechanism it is not easy to see how stress transfer would have triggered the failure. However if the underlying mechanism involved a sinistral displacement along a NE-SW fault, as is typical of Morocco, this would have had the potential to be triggered by E-W stress reduction within the region to the east of the main 1755 fault rupture.

Concerning the Brig, Aix-la-Chapelle and Kattegat earthquakes, they seem too far away from the 1755 Nov 1st source to have experienced a significant stress increase (stress changes < 0.005 bar) and thus they cannot help in constraining the source geometry of the 1755 Nov 1st earthquake.

5 Summary – Constraints on the Nov 1st 1755 Earthquake Source

The purpose of this paper has been to review the full range of phenomenological observations from the 1755 Nov 1st earthquake in order to determine what they reveal about the fault source. Having summarized what can be inferred from each individual set of observations it is possible to explore how these conclusions can be combined into a coherent interpretation.

To summarize these findings.

- The extraordinary energy and spectrum of far-field long-period ground motion implies that there was a single principal episode of fault rupture with a moment magnitude of c. Mw9.
- The duration of the fault rupture implies a fault length of 450–600 km (consistent with the Moment Magnitude).
- The strong polarization in farfield tsunami heights implies that the fault that generated the seafloor deformation had a predominant NNW-SSE orientation.
- The nearfield tsunami travel times and amplitudes are consistent with the proposal that there was a N-S to NNW-SSE oriented zone of strong seafloor deformation located midway between the coast of SW Portugal and Gorringe Bank. This tsunami source corresponds with the location and trend of a prominent high angle reverse fault structure – the Marques de Pombal thrust

showing evidence of geologically recent displacement. However the tsunami source must have been significantly longer than this 100 km mapped fault. While the tsunami source proposed by Baptista et al. (1998b) appears to be consistent with observations of tsunami travel times and tsunami heights along the SW Portugal coast it appears to understate tsunami heights in SW Spain and Madeira and overestimate travel times to Morocco, implying that the tsunami source extended further south into the Gulf of Cadiz and towards the coast of Morocco.

- High levels of destruction (at MSK VIII) both inland and along the coast in southwestern Morocco suggest that this region was within 100–200 km of the causative fault.
- A 600 km fault would need to extend from 38 N (the latitude of Lisbon) down to 32.5 N (close to the coast of Morocco) – see Fig. 9.

As additional and corroborative evidence in support of this proposal:

- Many observers report strong E-W motion – consistent with reverse displacement on a N-S striking fault.

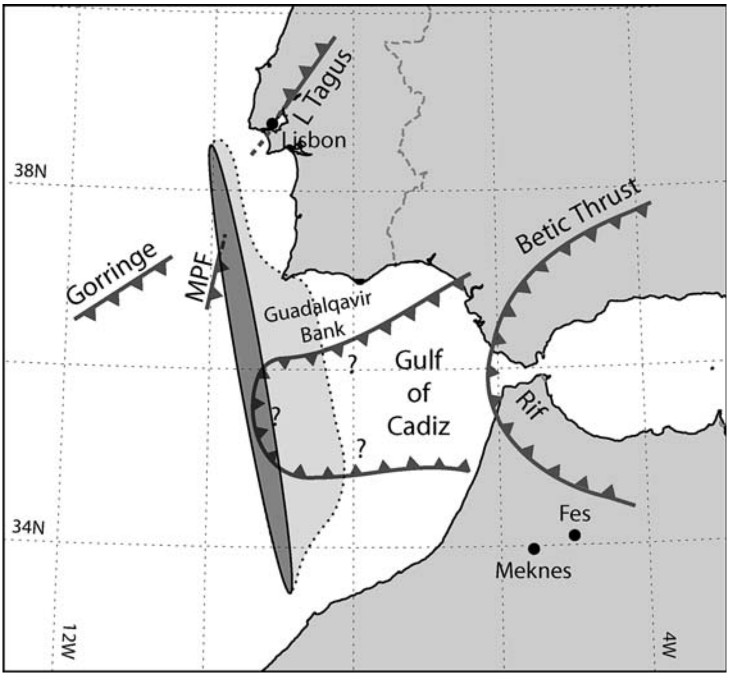

Fig. 9 Proposed zone of seafloor deformation and associated fault rupture in the Nov 1st 1755 mainshock. The dark grey ellipse represents the most likely orientation and extent of the source and the light grey zone represents the possible structure of the fault plane

- Observations from ships report the strongest impulse of water transmitted T waves in the area around the Marques de Pombal thrust. This may be because the fault emerged as a high angle structure on the sea floor in this area. Further south it is expected the fault may be a shallow dipping overthrust structure (dipping at 5–10 degrees towards the east?) in which seafloor deformation was more distributed.
- The pattern of very high intensities in the western Algarve suggests that an element of the fault rupture underlay this area. This could reflect the transition between a relatively steep reverse fault to the north and a shallow dipping overthrust structure to the south, with a much larger downdip extent. Alternatively this could be some kind orthogonal reverse fault structure absorbing the difference between the displacement on the main 1755 fault rupture and the overall NW-SE plate boundary displacement predicted in this region.
- It is likely that displacement on the Lower Tagus Valley fault system was triggered as part of the main rupture sequence.

In terms of the seismotectonic context of the region, the proposed NNW-SSE to N-S fault system appears to lie to the east of a zone of prominent recent seismic activity involving reverse displacement on faults trending NE-SW, including the Ms7.9 1969 earthquake and a more recent Mw6.1 earthquake on Feb 2nd 2007 (Borges et al. 2007). While these focal mechanisms are consistent with the expected plate boundary motions in this region, further to the east, beyond the proposed Nov 1st 1755 source structure, a different tectonic style exists, as around the Gulf of Cadiz and into northern Morocco apparently reflecting decoupling from the expected plate boundary motions. It is presumed that it is the faults along which the Nov 1st 1755 rupture occurred, that provide this decoupling.

Understanding the configuration of the seismotectonics is a pre-requisite for determining the current seismic hazard of this region, as well as helping identify other comparable situations worldwide capable of generating such regionally destructive earthquakes along with their accompanying megatsunamis. From the seismicity of the 20th Century and with the current generation of seismotectonic models for this plate boundary, there would be no suspicion that earthquakes such as the Mw9 Nov 1st 1755 and Mw8.5 March 31st 1761 could be generated in this region.

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The 1755 Lisbon Earthquake and the Genesis of the Risk Management Concept

A. Betâmio de Almeida

1 Introduction

Most of the historical writings about man response to catastrophes and to dangerous situations converge towards the identification of the 1755 Lisbon earthquake as the first modern disaster and as a landmark associated to a crucial change in the cultural and social perceptions of natural catastrophes. This fact can be explained by the magnitude of the human disaster and by the historical epoch. However, other factors, intrinsically associated to the effective response to the event, also justify the choice of the 1755 earthquake as the genesis of a modern crisis management and of what are now named as risk management.

Following G. Y. Kervern (Kervern, 1995, p. 8), this change can be characterized by the remark of J. J. Rousseau on the man responsibility and knowledge in what concerns the decision of the construction of cities in seismic zones. This position was published in 1756 as a contribution to the debate induced by the strong cultural 1755 aftershock that swept across the European continent and it was very different from the traditional contemporary positions concerning the strong religious belief or the philosophical skepticism against the Nature and the Providence (Voltaire Rousseau). This reaction of Rousseau can be considered as a symbolic beginning of the progressive development and strength of rational methodologies and tools for the control and management of natural and manufactured hazards. This human goal represents an epistemological rupture and challenge: the lonely man facing the uncertainty and the fate, trying to be an active actor and not just a potential victim.

Before 1755, other strong earthquakes were felt in Lisbon before the big one: eight (XIV c.), five (XVI c.), three (XVII c.) and in 1724 and 1750 (França, 1977, p. 59). However, the 1755 Lisbon earthquake introduce a dramatic change and remained in the people memory due to: 1) its own characteristics (violent earth

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shake, extensive and durable fire and deadly water wave or tsunami in several locations); 2) the European cultural environment and movement and 3) the emergency and recovery actions executed by the national government.

The cultural impact of the 1755 Lisbon Earthquake on the western society is well documented (e.g. Fonseca, 2004), in what concerns, among others, the moral, the philosophical, the scientific, the social and the artistic components.

Some previous authors made references to the relief action among other aspects of this event (e.g. Fonseca, 2004; Marques, 2003). In this text, the author briefly describes the context of the change and the evidence of a proto risk management structure associated to the 1755 Lisbon earthquake. The work is based on published bibliography and is framed by two key concepts – memory and knowledge – applied to the cultural history of the human response to catastrophes.

2 Risk Concept and its Meaning

Risk is nowadays a dominant concept in our societies and is associated to multiple conditions or factors: natural hazards; uncertainties involving science and technology and their effects on our health and quality of life; humankind vulnerability and lack of a consistent meta-discourse explaining life anguish and its meaning; and the appeal of a life game dealing with fears, chance and opportunities. The risk concept is now so important that our contemporary society is characterized by U. Beck as the "Risk Society" (Beck, 2003). The risk concept, as an evidence of fate, has its roots in the Antiquity but it acquires a growing importance and status in the Renaissance period as associated to the interference of natural hazards and fate uncertainties on the trade activity and on property[1]. Some scholars associated the consolidation of the concept and the origin of its name to the Portuguese or Spanish navigations (Giddens, 2002, p. 21).

The notion of risk is inseparable[2] from the ideas of probability and uncertainty (contingency). The rationalism and the Enlightenment change the previous human attitude when facing the uncertain fate: from observing God, the Creator and the perfect Observer, towards himself as an autonomous and finite observer.

[1] A trade risk ambience is presented by Shakespeare in his play "The Merchant of Venice" (1598?) which is an example of the travel contingency on the epochal European life.

[2] The so called "technical dimension" of the concept defines the conceptual entity risk R in the following way: R = probability of the hazard (causal chain of events associated to the hazard) • Consequences. Nowadays, risk is considered as a multi-dimensional concept beyond the technical one: it has a social behaviour dimension and a psychological dimension (risk perception), among others.

In order to control the future contingency, probability quantification based on the past experience and memory was an essential step that began to be possible in the domain of the games of chance (Bernstein, 1988, p. 49). However, games of chance were a pure human construction. Natural catastrophes were still considered in a different way.

The risk concept is strongly related to the process of decision: risk is a form for present descriptions of the future under the viewpoint that one can decide, with regard to risks, on the one or other alternative (Luhmann, 1992). A rational decision implies freedom and knowledge as well as confidence on world regularity and on the observer. Human risky decisions become dependent on probability and extent of loss or gain involved with external (from nature) or manufactured (from our action upon the world) risks. In contemporary societies, the causes of the events are veiled by the respective probability of occurrence that need to be trusted by the decision-makers. Rational options to make decisions can be strongly related to future consequences ("present futures") and to a new ethical responsibility: "there are no longer any dangers that are strictly externally attributable. People are affected by natural catastrophes, but they could have moved away from the endangered area or taken out insurance" (Luhmann, 1992, p. 71). Knowledge is useful when explaining the past and the present but is also vital for the prediction of future events because risky decisions are now those that can be regretted in the "future presents" should undesirable losses occur ((Luhmann, 1992).

To choose the best option in order to minimize losses is now a main goal of the risk decision-maker who becomes responsible for the future consequences due to "natural hazards". Memory and knowledge are supposed to furnish future scenarios that are felt as present virtual events to the decision-maker that will face the burden of the uncertainty management between the probable and the improbable.

3 Risk Management Structure

How to manage the future and the uncertainty become a growing demand of the social and economic activities associated to "progress" and "modernity". Meanwhile, the public safety standards and expectations forced the risk decisions and analysis to become a very complex and highly "risky" issue. The term "risk manager" was coined by the "Harvard Business Review" in 1956 and the risk management become well-known and accepted in the financial and the insurance activities. In the technological and engineering domains, the methods of reliability and risk analysis became scientific branches. The nuclear industry as well as the environment and health hazards strongly contributed to the present risk management acceptance.

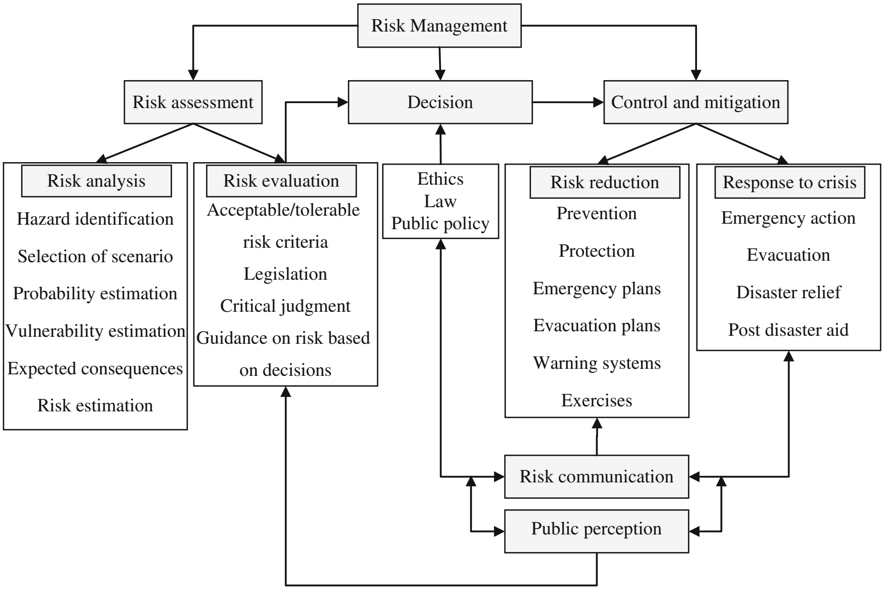


Fig. 1 A standard risk management structure

A general standard structure for risk management is presented in Fig. 1. This structure has the following main components:

- Risk assessment
- Risk analysis, the technical-scientific branch with the objective to make a detailed analysis of each identified scenario or danger in order to estimate the event probability and their consequences (quantitative risk estimation).
- Risk evaluation, for guidance on risk decisions, including acceptable or tolerable risk criteria and proposal of mitigation measures.
- Risk mitigation or control
- Risk reduction, to prepare and implement measures for risk abatement, planning of risk prevention or protection, including emergency and evacuation plans.
- Response to a crisis, to prepare emergency actions, including evacuation, disaster relief and post-disaster aid (civil protection actions).

The decision component is the key component of all this process. Among different disciplines, ethics and public perception are crucial for assistance to decisions about residual, acceptable, shared or imposed risks. In what concerns decisions associated to public risks, as those related to natural hazards (e.g. earthquake, floods, fires…) or technological hazards, the decision falls in the political domain. In fact, despite all the scientific and technical knowledge, risk concept is also a social construction in what concerns the future and the response to the epistemological and random uncertainties. In this context any decision about public risks must take into consideration the risk public (or social) perception based on people values, culture and fears.

From 1755 to nowadays, a deep change occurred in what concerns the organized response to natural hazards and disasters as well as to human manufactured dangers. Detailed analysis, based on scientific knowledge and rationalism, constitute a major part of the "corpus" of practice of the risk management but there is another part that depends on the public reaction to knowledge and to information. Risk communication becomes a powerful way to reflexivity and to the building up of an efficient public preparedness. In fact, the public risk aversion or the public response to a risk characterization, or a disaster scenario, strongly depend, among other factors, on the communication process and on the strong human utopian expectation and desire of an almost perfect ("no damage or no victims") risk management. However, it is now accepted that organization and preparedness can strongly reduce the losses and the damages. The criteria for risk acceptance by the public and for the selection of risk mitigation measures should not be considered as just a complex technical problem. They also involve people behavior and culture and the risk management need to be a hybrid process based on both natural and social sciences.

4 Memory and Knowledge Related to Earthquakes (Before 1755)

Any catastrophic event, involving heavy human and economic losses, in a dimension that surpasses the normal human scale, unchains a process of thought, reflection, interpretation and adjustment by the society. According to our contemporary reasoning, this process can be considered as a collective response to overcome the anguish created in those that survive to such event. It is a way to move aside the idea of living a future similar horror. It can be considered as a survival process of human societies and the response pattern to catastrophes induced by natural factors and as a paradigmatic cultural process.

In fact, a relevant component of the human culture can be identified by the way it finds an interpretation or a meaning for the violent disruptions of the life and of the orderly structure of the world including the response to a real catastrophe.

This topic is presented in a very clear and elegant way by Plato (428–348 BC) in the "Timaeus" where it is explained that several destructions of people occurred in the past and the same will happen in the future. The selected main causes for such events were the fire and the water: the earth is submerged by the gods in order to purify the inhabitants... and those who live in towns are pushed to the sea by the rivers.

"Timaeus" can be interpreted nowadays according to the contemporary risk management structure: hazard identification, selection of causes, characterization of effects and the contingent and probabilistic characteristic of the events:

"it happened sometimes and it can happen again, sometimes with large time intervals between" – perhaps anticipating the well known "period of return" concept.

Plato points to the fault of the Greeks in what concerns the prevention and protection because, "contrary to what the Egyptians do, other people do not preserve the knowledge through the writing".

The lack of memory concerning the causes and effects of events that "periodically" strike the humans would maintain the ignorance of the man, as "children who know nothing about what happened in the ancient times"... Memory and knowledge is the key for protection according to Plato.

In fact, among other catastrophes, reminiscences of strong and deadly earthquakes can be found in classic Antiquity culture. Biblical interpretations of earthquakes in Palestine are presented in the Old Testament (Boer and Sanders, 2005, p.22):

– 2 Samuel 22:8 – "Then the Earth shook and trembled; the foundations of heaven moved and shook, because he was wroth";
– Jeremiah 10:10 – "the Lord is the true God...at his wrath the earth shall tremble".

Earthquakes are also mentioned in the New Testament as catastrophic events involving powerful forces at work within and upon our planet. In Greece, the city of Sparta was devastated by a powerful earthquake that shook Sparta and Helike, a city on the Corinthian coast, was swallowed by an earthquake and a tsunami in 373 BC.

In what concerns the knowledge about the causes or the explanation of such kind of catastrophes, two types of theories can be found in the Classic Antiquity: those based on divine (Gods) action; and those based on physical grounds.

Several Greek philosophers presented different physical conjectures based on "active principles": the water (Thales of Miletus – 624–546 B.C.); the air ("blow theory") by Anaximenes, in the sixth century BC; the air and the fire (Anaxagoras, in the fifth century BC), among others. Aristotle (384–322 BC) discussed in his work "Metereologica" early theories and developed his own explanation through the "pneuma" concept associated to the wind generation and internal circulation inside the earth regulated by the external climatic conditions. Aristotle's ideas were still accepted in the early years of the XVIII century, more than two thousand years after his death.

Despite the epistemic uncertainty, the predominant representation of the seismic activity in the ancient Western World was that of a passive and cavernous earth (planet) traversed by active fluids that were the cause of earthquakes.

The development and consolidation of the Christianity reinforce the divine component associated to only one God. The theory of Aristotle was accepted in the Middle Age with one important safeguard added to the theory by St. Thomas Aquinas (1225–1274): "the main cause of earthquakes is God, and can only collaterally be attributed to subterranean winds". Before, Philastrios, abbot of Brescia (fall of the fourth century AC), wrote as the heresy number 102 ("Liber de Haeresibus") the belief in the natural causes of earthquakes. Natural phenomena including earthquakes could be interpreted as divine messages.

The Renaissance and the "Scientific Revolution" periods made a change and put again questions and reflections in what concerns the interpretation of the earthquakes. Do "the cause of the earthquakes will be within or over the earth?" smartly asks Galileo (1564–1642) possibly inspired by the Constantinople earthquake (1556) where a comet and an unsent set of stars had been observed in the sky. New physical explanations were presented since the XVII century, namely the "explosion theory", possible inspired by the developments in applied thermodynamics (Keller, 1998, p. 135). Around 1750, a new theory was proposed based on the electricity and inspired in the work of B. Franklin associated to atmospheric phenomena (Keller, 1998, p. 136).

The 1755 earthquake occurred in Lisbon in a key corner of the scientific knowledge path related to natural catastrophes and also of the history of risk management practices.

5 The 1755 Proto-Risk Management

5.1 Emergency Response to Crisis

In what concerns the risk management structure (Fig. 1), the first evidence of the 1755 change corresponds to a specific component of the risk management process: the event realization and post-disaster (emergency) response to crisis. After the 1755 earthquake this response had a strong and efficient political leadership under the power of the marquis of Pombal, the Portuguese head of the government.

This political action reflects one of the aspects of the structural changes which occurred in the XVIII century: the development of the modern state. Faced to a major disaster in its capital the modern state assumed the collective responsibility for their consequences and for the relief. The King of Portugal gave to Pombal full responsibility and power for dealing with the emergency response and for the Lisbon reconstruction. This leadership is symbolically remembered by the famous answer to the question "what should we do?": "bury the dead and feed the living" is supposed to have said Pombal.

The emergency response to the 1755 earthquake included different tasks, namely (Conceição, 1829):

– The disposal of bodies, in order to avoid the outbreak of plague - Pombal concluded that the best way to solve this problem was to collect them on barges and sunk them beyond the river Tagus; the Church agreed and the traditional religious rites were exceptionally disregarded.

- The treatment, the feeding and the housing of the survivors - emergency hospital were created and special measures were applied to provide the transport of food from different parts of the country and by ship in order to avoid the hunger in Lisbon, including the price control; to provide materials for the new houses and orders for the removal of debris; camps were set up to house the homeless (Fig. 2).
- The guarantee of public security by troops and courts in order to stop looting and to recover stolen wells - exemplar death punishments were applied and a pass system was set up to regulate entrance and exit from the city.
- The fight against superstition and prophecies that could induce a fatalist terror and also the fight against reactionary positions (e.g. the repressive actions against the rumor that a new earthquake would strike again on the anniversary of 1755 earthquake).
- Special financial resources were mobilized - legislation was prepared (1756) to regulate an extraordinary tax for imported merchandise (França, 1977, p. 69).
- International aid from different countries: food and supplies from England, timber from Hamburg, a generous gift in cash from the Queen of Spain (Francis, 1985, p. 124) and other came from Brazil, Italy, France, Sweden and Holland (França, 1977, p. 67).

Some of these types of actions are very similar to those considered in the present disaster and crisis management and civil protection planning.

Fig. 2 Lisbon after the earthquake. Camping outside the damaged town and executions of robbers and looters. (1755 Kozak Collection, KZ119)

5.2 Reconstruction and Risk Mitigation

The reconstruction of the damaged part of Lisbon began with demolitions and removal of debris and, one year after the earthquake, the area was disencumbered.

Very soon the works related to the rebuilding of Lisbon were in progress (Figs. 3 and 4) and a concerted set of measures and special legislation was approved along the following months:

– General Manuel da Maia, Chief Engineer of the Kingdom, was asked to submit a report ("dissertação") for the reconstruction of the city; this report comprises three documents dated 4 December of 1755, 16 February of 1756 and 31 March of 1756; in this report Maia presents different solutions.
– The survey and register of streets, squares and houses in the damaged part of the city in order to avoid future quarrels (Conceição, 1829, p. 73); and the prohibition to construct new houses and the order to demolish those that were out of the plan (Conceição, 1829, p. 75).
– A complex logistic planning involving the materials, the construction processes, human resources, financial aids and legal measures were prepared including the control of price speculation (França, 1977, p. 103).
– A team of architects and engineers developed plans for the reconstruction taking into consideration the structural safety, the urban aesthetics and public health.

As an example of what we can now call risk mitigation planning, based in the control of consequences (less vulnerability) for the inhabitants, Maia recommended in his report (December, 1755) that the new buildings should not be very

Fig. 3 Pombal implementing the reconstruction of Lisbon, M. S. Carmo Sendim (Municipal Museum of Lisbon)

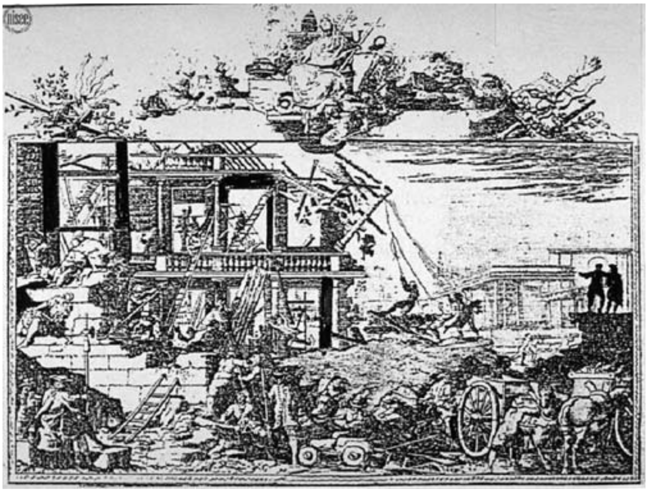


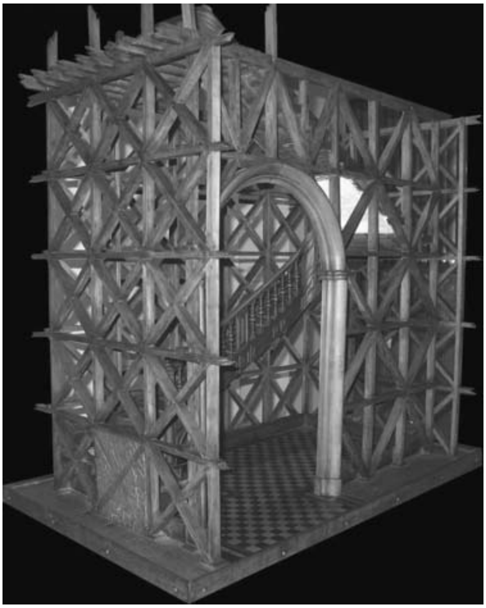
Fig. 4 Repairs and new construction after the Lisbon earthquake. (Kozak Collection, KZ78)

high (control of the number of floors), in order to diminish the consequences of their ruin should another earthquake occur and, also, that the new streets should be wider, in order to allow an easier way to flee from the debris.

Maia introduce a very interesting aspect related to public perception and memory (França, 1977, p. 80): it would be very dangerous to allow a free rebuilding of the city without constraints to the height of the buildings because the people would forget the horror of the 1755 earthquake and the prevention (technical) constraints. However, a strong economic pressure put away these strict criteria (França, 1978, p. 56). Maia also proposed the search of a less vulnerable area for the reconstruction of the damaged area of Lisbon and even referred the potential negative influence of water (e.g. sewage) in the dynamic behavior of the soil.

In what concerns the prevention against the collapse of the new buildings with three or four floors, under seismic forces, the most popular structural measure was the wooden structure or cage ("gaiola") as an embedded structure in the walls (Fig. 5). According to França (1977, p. 158) and Fonseca (2004, p. 93) experiments of this invention were made in a full-scale model, by using soldiers in March in order to test the efficiency of the structure under dynamic forces. These experiments can be considered as one of the first dynamic essays in the context of the earthquake engineering (Fonseca, 2004, p. 93). The reconstruction of Lisbon and the protection against seismic forces acting in the buildings left unforgettable memories and was a strong support to the development of new technical anti-seismic procedures in Portugal.

Fig. 5 A cage structure model. IST Museum (Lisbon, Portugal)



5.3 Risk Communication and Earthquake Descriptions

The description of what happened in Lisbon, the horror in the All Saints Day of 1755, was communicated to other countries and to a large number of people outside Portugal. As a consequence of the earthquake and of the emotional shock that was a felt, a popular literature was generated which described the event, the destruction and the death associated to it. This had a tremendous effect on people and provoked an intense critical analysis and discussion related to the meaning of such a catastrophe.

Foreign eyewitness (e.g. merchants and diplomatic) descriptions and books, essays, poems and theatre plays had a deep and durable influence in the European imaginary. "Candide", the book of Voltaire with references to the Lisbon earthquake, that was published in 1759, became an international best seller (30 000 copies in the first year), "which became astounding at the time for a work of fiction (Dynes, 2000)". Another extraordinary vector of communication and of spreading of dramatic emotions was the large number of historical depictions of the 1755 earthquake. Examples of these pictures can be found in the Museum of the City of Lisbon and in the Kozak Collection[3] (Kozak et al., 2005). Some of the pictures try to acutely depict the events and are based on eyewitness (e.g. Le Bas engravings based on Paris and Pedegache

[3] http://nisee.berkeley.edu/kozak/index.html.

drawings), others are fanciful depictions. However, they were very important in what concerns the spreading of the message and as documents showing aspects of the earthquake magnitude and power of destruction, the suffering of the inhabitants, the rescue actions and also the Lisbon reconstruction (Figs. 2 and 4).

The motivations of the written or pictorial messages are diverse: to inform, to record, to understand or just the perverse sensational desire to show human suffering. Although based on very different technological platforms the nowadays media and communication channels mobilize similar motivations are much more powerful. The 1755 earthquake was also a landmark in this component of risk communication as well as in the interaction between world and image associated to scientific works (Keller, 1998).

6 The Moral and Philosophical Debate

The 1755 earthquake originated a well-known moral and philosophical debate across Europe, involving some of the most prominent intellectuals of the Enlightenment movement: Voltaire (1694–1778), Rousseau (1712–1778) and Kant (1724–1804) among others.

The descriptions of the catastrophes shocked Voltaire and gave him the opportunity to manifest his deep moral perplexity in what concernes the Goodness and the Divine Providence when confronted with the destruction, the suffering and the death that happened in Lisbon.

The "best of the worlds" concept of Leibniz (1646–1716) and Pope (1688–1744) as well as the meaning of the "justice of God" (the theodicy problem) were the background motivation for violent and satiric writings. Voltaire did not put in doubt the God's existence but the coexistence of God with the evil on earth doing so horrendous works was something that could not make sense.

The "Poème sur le Désastre de Lisbonne" (Voltaire, 1755), written shortly after Voltaire knew what happened in Lisbon, was an opportunity to discuss the Providence and other justifications of the earthquake and its horror.

The epochal philosophical discussions about the Providence were very important because they touch the concepts of knowledge and forethought as well as the religious belief in the intervention and care of God on behalf of the man: the belief "that God (or the gods) not only created the world but also governs it and cares for its welfare, particularly for men's welfare" (Marques, 2003, p. 7).

In what concerns the development of risk analysis and management, a fracture of the Providence concept was necessary because:

– The disruption of nature's order as the result of the impenetrable divine wills and punishments would be incompatible with a rational inquiry in the "natural causes" (natural law, and in the search of the regularities of the "natural events" (frequencies).

The writings of Voltaire were a strong contribution for the change from the intellectual optimism and potential fatalism for a more **skeptical** position that is a necessary condition for the construction of future scenarios in a risk analysis context.

Rousseau's answer to opens a new social view of disaster when he writes:. . ."that nature did not gather twenty thousand houses of six or seven floors, and if the inhabitants of that great city had spread out more and built smaller houses, much less damage would have been done, may be none at all. How many power wretches died in this disaster because they wanted to rescue their clothes, their papers, their money?".

Rousseau shifts the responsibility towards the behavior or the actions of the man and identifies the concept of vulnerability that is related to the probability of damage or loss should a hazard occur. In the Rousseau's text it can also be identified the importance of a promptly evacuation of the population at the first tremors or alarm signals in order to avoid human losses. Similarly, I. Kant also makes a critical analysis of the old layout of Lisbon in what concerns the direction of the Tagus valley and the orientation of the potential damages provoked by an earthquake according to the physical theory, free of religious causes, presented by him (Kant, 1756).Rousseau also points to the idea that disaster is also a social construction according to existing cultural norms and whether an event is considered or not a disaster will depend on who is affected, where it occurs and the way it is known (disaster communication).

All these new ideas and concepts are important because they introduce a new social and ethical responsibility to the decision-maker as well as the idea that one of the meanings of disaster has to be found in the social and cultural context is now one of the dimensions of the risk management. This social dimension includes the public behavior under a risk environment and need to be considered in the mitigation and emergency (evacuation and rescue) actions. The problem that was unveiled by Rousseau is a very actual one, as can be concluded from the U. N. report on disasters published in 2004 where the problem of rapid urbanization and high human vulnerability to earthquakes is pointed out (UN, 2004, p. 36) as well as the fact that decisions related to very fast development areas do not incorporate the risk assessment and management in what concerns the planning and construction processes.

7 New Memories and Knowledge

7.1 Science and Public Perception

The 1755 earthquake was the origin of new memories associated to a specific catastrophe induced by a natural hazard as well as a catalyst for new attitudes and a new knowledge. The new memories were stimulated by the popular literature and depictions and by the study of the event in a restrict circle of people. Baptista et al. (1998) made a careful analysis of the several types of documents written between 1755 and 1759, including reports, chronics, letters and anonymous coeval sources. The total number of documents identified by the authors attained was 982.

An official inquiry that was sent to all parochial districts (February 1756) has a special importance (Portugal and Matos, 1974).

This national inquiry ("Pombal enquiry") was found in 1910 by Pereira de Sousa and it is an example, among others of a new way to think the physical events and the catastrophes[4]. From the scientific point of view, one of the most interesting questions in the first one: "At what time did the earthquake began and how long had lasted?" Another question asks about the direction of the shock. Both were strongly related to the problem of the seismic propagation and the associated velocity of propagation, a topic discussed by Michell and Kant (Coelho, 2007). Other questions are related to different physical effects (e.g. soil liquefaction, tsunami…), building damages and recovery actions.

According to some authors, this inquiry can be considered as a landmark in the history of the modern seismology (Fonseca, 2004, p. 122). In fact, this kind of inquiry is important because it is a set of encapsulated memories (database) related to physical evidences and to both quantitative and qualitative consequences of the event allowing a more scientifically based explanation for the processes involved in this type of catastrophe.

New efficient knowledge would only be possible if the human minds freely believe in physical conditions and processes that can happen, again and again, due to neutral laws, external to our "will and sins". This change was possible in the epoch of the 1755 event. In fact, the debate between those who believed in divine causes and those who defended the natural causes was a reality in the epoch. J. Wesley, in a pamphlet titled "Serious Thoughts occasioned by the Late Earthquake at Lisbon" (1772), wrote: "why should we not be convinced…that it is not chance which governs the world…; why we have a general answer always ready, to screen us from any such conviction; all these things are purely natural and accidental; the result of natural causes". "But there are two objections to this answer: first, it is untrue, secondly it is uncomfortable… what is nature itself but the art of God? Or God's method of acting in the material world?" (Boer and Sanders, 2005, p. 85–86).

Some were convinced as, for example, Cavaleiro de Oliveira, a Portuguese refugee in London, that the catastrophe was the result of a divine punishment applied to Portugal as the result of superstition, idolatries and persecutions and of the actions of the Inquisition (Oliveira, 1756). The Jesuit Gabriel Malagrida also writes a text (1756) about "the true causes" of the earthquake and threatened with a divine punishment those who have worked in the reconstruction of

[4] In 1756, a similar enquiry was made in Spain by Fernando VI, King of Spain (Coelho, 2007).

Lisbon. Meanwhile, Pombal would have secretly ordered a French religious to develop an explanation based on "natural causes" (França, 1977, p. 72).

The new way of thinking is exemplified by John Michell, a professor at Cambridge University and one of the "fathers" of the modern seismology. Michell analyzed reports of the 1755 earthquake and, in 1760, published a book titled "Conjectures concerning the Cause and Observations upon the Phenomena of Earthquakes". In this book, the author proposed "a method for determining where the earthquakes originated". He suggested that the directions of wave propagation at different locations in plotted as lines on a map and that the lines be extended until they intersected. "He thus confirmed that the Lisbon earthquake had originated in the Eastern Atlantic" (Boer and Sanders, 2005, p. 95). Until now, the 1755 earthquake has been a permanent challenge for the scientists in order to understand the physical process of the earthquake. New scientific theories were developed after the 1755 earthquake (e.g. Kant, 1756). Seismology began its development and a long way was made until the contemporary knowledge about the general sources and mechanisms of the seismic activity. The enquiry proposed by Michell has been pursued by others, now with powerful computer techniques (e.g. Baptista et al., 2003, 1998). However, it remains some important uncertainties and there are several proposals for the location of the origin of the earthquake. But it is

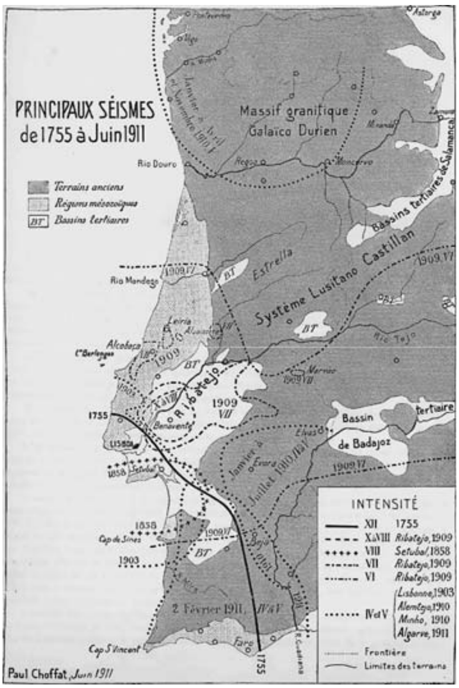


Fig. 6 Intensity distribution of relevant earthquakes that occurred in Portugal (1755–1911), presented by Paul Choffat (Choffat and Bensaude, 1912, p. 105)

sure that the 1755 earthquake remains a reference for the scientists in the seismological domain (Fig. 6).

The 1755 earthquake also remained in the memory of the Lisbon inhabitants for a long time. According to Choffat and Bensaúde (1912), “the 1755 earthquake left in the inhabitants of Portugal a terror associated to quakes, and the majority of the population expects the repetition of this catastrophe”. . . and the author refers the public the public reaction during the 1909 (23 April) earthquake (Benavente earthquake): “In Lisbon, many people remained outside during the night” (Choffat and Bensaúde, 1912, p. 19).

7.2 Risk Management Heritage

In Fig. 7 it is presented in a simplified way the association of historic facts related to the 1755 earthquake and some of the components that nowadays belong to a standard risk management structure. Memory (data) and (scientific) knowledge are very important elements for the risk management:

a. Risk analysis (hazard identification and scenario selection)
b. Physical process (seismology) → process understanding and probabilities
c. Expected consequences (earthquake engineering) → risk assessment
d. Risk mitigation and control measures (zoning)

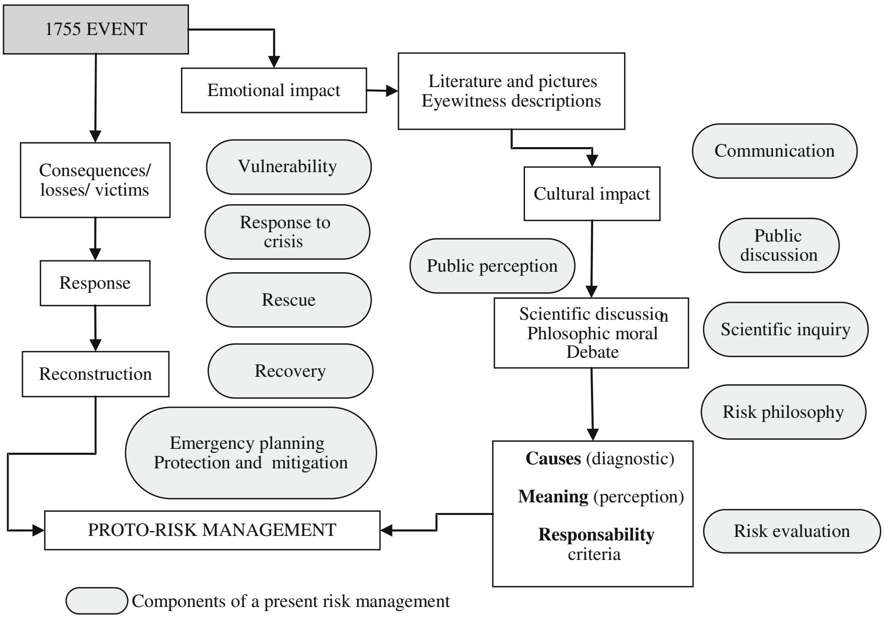


Fig. 7 1755 roots on nowadays risk management structure

e. Mitigation of consequences → vulnerability control → anti seismic buildings (earthquake engineering) and urban planning
f. Prediction and detection of the event → early warning
g. Evacuation → relief assistance (civil protection)

Due to the impact of 1755 earthquake and based on the seismic history of Lisbon, some Portuguese authors presented a few statistical predictions of the return period associated to great earthquakes in Portugal (Coelho, 2007): two centuries, according to Mendonça (1758) or between 1977 and 1985.

Unfortunately, a feasible earthquake prediction is still not fully guaranteed in all situations but the vulnerability control and mitigation based on dynamic response of the buildings and on the control of damages is now possible (earthquake engineering and building codes). A good urban planning based on risk mapping can also reduce the vulnerability as well as a good organization and preparation (civil protection).

A better protection can be obtained by a good memory and knowledge management (as Plato wrote in the "Timaeus"). However this memory management need to preserve a sufficient and healthy public oblivion; but some, among us, need to know, to remember and not to forget the memories, and to be responsible for the implementation of a good enough system of protection and for an efficient public information and education.

8 Concluding Remarks

The author is confident that the historical evidence is consistent enough (and not anachronistic) to sustain the conjecture that links the response to the 1755 earthquake to the genesis of what is now called the risk management. The risk management concept implies a strong rational position facing the past (memory and knowledge) and the future (planning, prevention and protection) catastrophic events. The Enlightenment movement and the philosophical discussions concerning the moral responsibility associated to the Lisbon earthquake opened the way to the acceptance of rational (scientific) enquiry about the causal conditions of this kind of natural catastrophes.

Among the several changes induced in the European culture by the 1755 Lisbon earthquake, the genesis of the risk management as an integrated methodology applied to severe accidents and catastrophes can be considered as one of them. This heritage results from both the cultural epoch and contemporary political and philosophical conditions and also from the specific characteristics of the earthquake. After two hundred and fifty years, the 1755 event can be considered as one of the foundational landmarks of risk management when a new phase began: the "age of reason" based on the political will and knowledge as well as on the human responsibility. From an ethic of fatality the world began to change towards an ethics of responsibility a main characteristic of the risk management is the acceptance of human responsibility in what concerns the magnitude of the consequences of an event (local vulnerability).

From 1755 to nowadays the way how to see and to react to public catastrophes changed very much: risk management is based on rational knowledge (logos), on the public response and perception (pathos) and is also an organized attitude (ethos) that the political power can not forget and need to be well prepared.

The historical memory of the 1755 landmark should constitute a strong motivation for a continuous effort concerning the public protection against catastrophes in Portugal and in the world.

Acknowledgements Figures 2 and 4 – "Courtesy National Information Service for Earthquake Engineering, University of California, Berkeley".

Figure 3 – "Courtesy Museu da Cidade – Câmara Municipal de Lisboa".

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Holistic Urban Seismic Risk Evaluation of Megacities: Application and Robustness

Martha Liliana Carreño, Omar D. Cardona, Mabel-Cristina Marulanda and Alex H. Barbat

1 Risk from a Holistic Perspective

Disaster risk has been defined, for management purposes, as the potential economic, social and environmental consequences of hazardous events that may occur in a specified period of time. However, in the past, the concept of risk has been defined in a fragmentary way in many cases, according to each scientific discipline involved in its appraisal (Cardona, 2004). Based on the formulation of disaster risk of UNDRO (1980) several methodologies for risk assessment have been developed from different perspectives in the last decades. From a holistic perspective, disaster risk requires a multidisciplinary evaluation that takes into account not only the expected physical damage, the number and type of casualties or economic losses (direct impact), but also the conditions related to social fragility and lack of resilience conditions, which favour the second order effects (indirect impact) when a hazard event strike an urban centre (Cardona and Hurtado, 2000; Masure, 2003; Carreño et al., 2007a).

Cardona (2001) developed a conceptual framework and a model for disaster risk analysis of a city from a holistic perspective. It considers both "hard" and "soft" risk variables of the urban centre, taking into account exposure, socio-economic characteristics of the different localities (units) of the city and their disaster coping capacity or degree of resilience. The model was made to guide the decision-making in risk management, helping to identify the critical zones of the city and their vulnerability from different professional disciplines. Carreño (2006) developed an alternative method for Urban Risk Evaluation, based on Cardona's model (Cardona, 2001; Barbat and Cardona, 2003). The urban risk is evaluated using composite indicators or indices. Expected building damage and losses in the infrastructure, obtained from future loss scenarios are basic information for the evaluation of a physical risk index in each unit of analysis.

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Often, when historical information is available, the principal hazard can be usually identified and thus the most potential critical situation for the city.

The holistic evaluation of disaster risk is achieved affecting the physical risk with an impact factor, obtained from contextual conditions, such as the socio-economic fragility and the lack of resilience, that aggravate initial physical loss scenario. Available data about these conditions at urban level are necessary to apply the method. A brief explanation of the model is made forward to illustrate the benefits of this approach that contributes to the effectiveness of disaster risk management, inviting to the action identifying the hard and soft weaknesses of the urban centre. Figure 1 shows the theoretical framework of the alternative model.

From a holistic perspective risk, R, is a function of the potential physical damage, D_j, and an aggravating coefficient, I_f. The former is obtained from the susceptibility of the exposed elements, γ_{Di}, to hazards, H_i, regarding their potential intensities, I, of events in a period of time t, and the latter depends on the social fragilities, γ_{Fi}, and the issues related to lack of resilience, γ_{Ri}, of the disaster prone socio-technical system or context. Using the meta-concepts of the theory of control and complex system dynamics to reduce risk, it is necessary to intervene in corrective and prospective way the vulnerability factors and, when it is possible, the hazards directly. Then risk management requires a

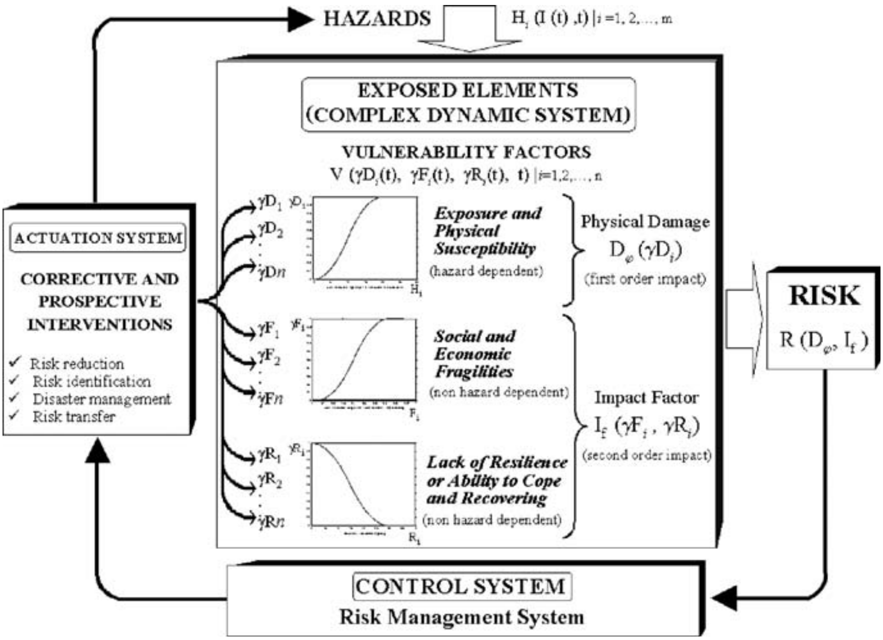

Fig. 1 Theoretical framework and model for holistic approach of disaster risk (adapted from Cardona and Barbat, 2000)

system of control (institutional structure) and an actuation system (public policies and actions) to implement the changes needed on the exposed elements or complex system where risk is a social process.

1.1 The Urban Seismic Risk Index – USRi

The USRi is a composite indicator –or index– that measures seismic risk from an integrated and comprehensive perspective and guides decision-making identifying the main multidisciplinary factors of vulnerability to be reduced or intervened. The first step of the method is the evaluation of the potential physical damage (hard approach) as the result of the convolution of seismic hazard and physical vulnerability of buildings and infrastructure. Subsequently, a set of social context conditions that aggravate the physical effects are also considered (soft approach). According to this procedure, a physical risk index is obtained, for each unit of analysis, from existing loss scenarios, whereas the total risk index is obtained by factoring the former index by an impact factor using an aggravating coefficient, based on variables associated with the socio-economic conditions of each unit of analysis.

The proposed holistic evaluation of risk is performed using a set of input variables, herein denominated descriptors. They reflect the physical risk and the aggravating conditions that contribute to the potential impact. Those descriptors are obtained from the loss scenarios and from socio-economic and coping capacity information of the exposed context (Carreño et al., 2005a).

The USRi depends on the direct effect, or physical risk, and the indirect effects expressed as a factor of the direct effects. Therefore, the USRi for each unit of analysis is the total risk that can be expressed as follows:

$$USRi = R_T = R_F(1 + F) \tag{1}$$

expression known as the Moncho's Equation in the field of disaster risk indicators, where R_T is the total risk index, R_F is the physical risk index and the indirect effects given by an impact factor $(1+F)$, based on an aggravating coefficient, F is the aggravating coefficient.

The physical risk index, R_F, depends on the weighted sum of a set of component factors

$$R_F = \sum_{i=1}^{p} F_{RFi}\, w_{RFi} \tag{2}$$

where p is the total number of descriptors of physical risk index, F_{RFi} are the component factors and w_{RFi} are their weights respectively. The factors of physical risk, F_{RFi}, are based on the gross values of physical risk descriptors such as the number of deaths, injured or the destroyed area, and so on. It has to be mentioned that the calculation of physical risk scenarios is not the objective of the methodology developed in this chapter, but the physical risk index is obtained starting from existing loss evaluations.

The coefficient, F, is another index evaluated in the same way. It depends on the weighted sum of a set of aggravating factors related to the socio-economic fragility, F_{FSi}, and the lack of resilience of the exposed context, F_{FRj}

$$F = \sum_{i=1}^{m} F_{FSi}\, w_{FSi} + \sum_{j=1}^{m} F_{FRj}\, w_{FRj} \tag{3}$$

where w_{FSi} and w_{FRj} are the weights or influences of each i and j factors and m and n are the total number of descriptors for social fragility and lack of resilience respectively. Figure 2 shows the process of calculation of the total risk index for the geographic units of analysis, which could be districts, municipalities, communes or localities. See Carreño et al. (2007a) for measurement units of each descriptor and a detailed explanation of the method.

It is estimated that the indirect effects of hazard events, sized by the coefficient F in Equation (1), can be of the same order than the direct effects. According to the Economic Commission for Latin America and the Caribbean

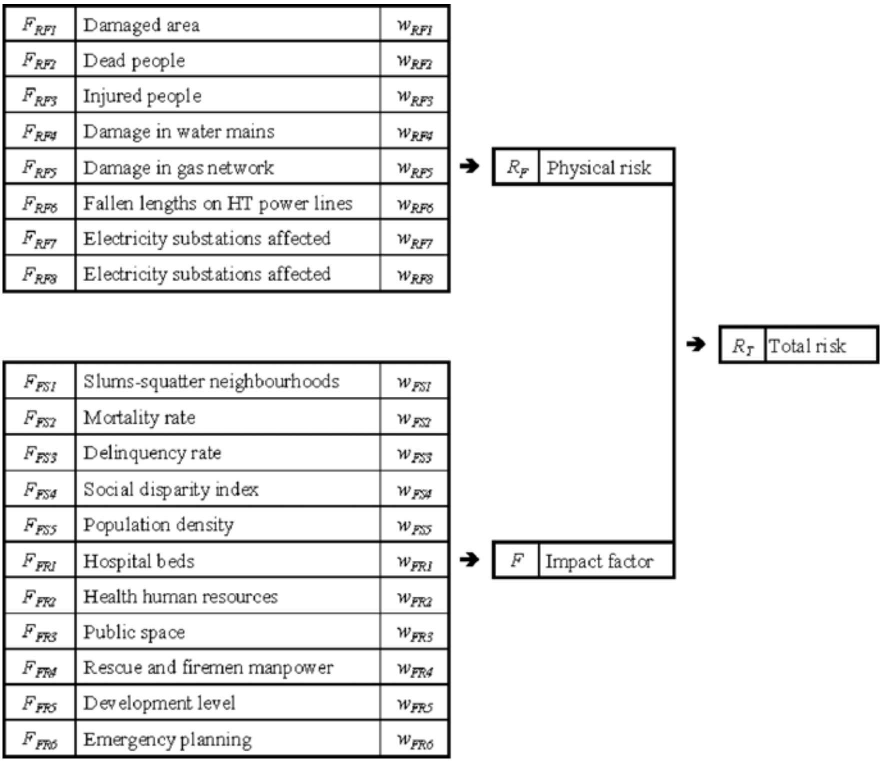

Fig. 2 Factors of physical risk, social fragility and lack of resilience and their weights

(Zapata, 2004), it is estimated that the indirect economic effects of a natural disaster depend on the type of phenomenon. The order of magnitude of the indirect economic effects for a "wet" disaster (as one caused by a flood) could be of 0.50–0.75 of the direct effects. In the case of a "dry" disaster (caused by an earthquake, for example), the indirect effects could be about the 0.75–1.00 of the direct effects, due to the kind of damage (destruction of livelihoods, infrastructure, housing, etc.). This means that the total risk, R_T, could be between 1.5 and 2 times R_F. In this method, the maximum value selected was the latter. For this reason, the aggravating coefficient, F, takes values between 0 and 1 in Equation (1).

The factors of physical risk, F_{RFi}, and the aggravating factors F_{FSi} and F_{FRj} are calculated using transformation functions as the shown in the Fig. 3. These examples of transformation functions for physical risk, social fragility and lack of resilience (or coping capacity) show the values of the descriptors in the *x-axis* and the corresponding factors, or scaled values, in the *y-axis*.

These functions describe the intensity of the risk for each descriptor and standardise the gross values of the descriptors transforming them in commensurable factors. Sigmoid functions were used in most of the cases to develop the transformation functions. All their maximum and minimum values (corresponding to the values 1.0 or 0.0 of each factor) were fixed using existing information about past disasters as well as the expert opinion. For example, the transformation function for damaged built area defines a minimum risk (0.0) when this descriptor is zero and, the maximum risk (1.0) was established for a potential damaged area of 20% of the overall constructed area.

The weights w_{RFi}, w_{FSi} and w_{FRj} represent the relative importance of each factor and are calculated by means of the Analytic Hierarchy Process (AHP),

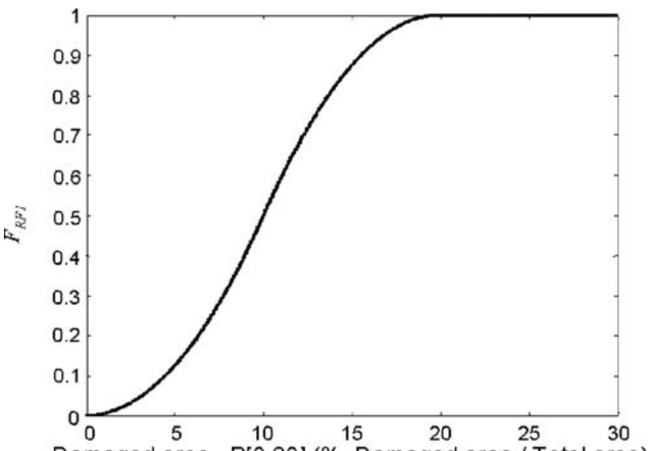

Fig. 3 Example of transformation functions used to standardise physical risk, social fragility and lack of resilience factors

which is used to derive ratio scales from both discrete and continuous paired comparisons (Saaty, 2001). This process has been performed starting from experts opinions collected by means of the Delphi method. This is the most adequate way of judging the relative importance of variables having different nature and calculating their relative weights.

1.2 Improvement of the Previous Approach

The model of holistic urban risk evaluation proposed Carreño et al. (2007a,b) improves conceptual and methodological aspects of the first proposal of Cardona (2001), refining the applied numerical techniques and turning it into a more versatile tool. The conceptual improvements provide a more solid theoretical and analytical support to the new model, eliminating unnecessary and dubious aspects of the previous method and giving more transparency and applicability in some cases. Cardona's model allows the evaluation of the seismic risk in an urban center taking into account the characteristics of the physical risk, seismic hazard, physical exposure, socio-economical fragility and lack of resilience, what permits to identify those characteristics of the city that increase the level of risk and also the critical areas. This model studies different types of information by means of indicators and uses a normalization process of the results based on the mean and on the standard deviation which is applied to each indicator. As a consequence, the results obtained with Cardona's method allow a comparison of the holistic seismic risk among the different areas of a city in a relative way, but not a comparison in absolute terms with other urban areas. Cardona's model uses of a neuro-fuzzy system, with fuzzy sets which identify the linguistic qualifications of the descriptors, but the necessary information for the calibration of this system do not exist.

The new method proposed in this chapter conserves the approach based on indicators, but it improves the procedure of normalization and calculates the final indices in an absolute (non relative) manner. This feature facilitates the comparison of risk among urban centers. The exposure and the seismic hazard have been eliminated in the method proposed in this chapter because they have been included into the physical risk variables calculation. The descriptor of population density, a component of the exposure in Cardona's model, is now included as a descriptor of social fragility. The new approach preserves the use of indicators and fuzzy sets or membership functions, proposed originally by Cardona, but in a different way. Other improvements of the proposed model refer to the units of some of the descriptors; in certain cases it is more important to normalize the input values respecting the population than with respect of the area of the studied zone. This is, for example, the case of the number of hospital beds existing in the studied urban area. The socio-economic fragility and the lack of resilience are a set of factors (related to indirect or intangible effects) that aggravate the physical risk (potential direct effects).

2 Application to Megacities

The Urban Seismic Risk Index was proposed and applied to Barcelona (Spain) and Bogota (Colombia) by Carreño (2006) and by Carreño et al. (2005b, 2007a) and by Suarez (2007) to Manizales (Colombia). Recently, the USRi has been applied to Metro Manila (The Philippines) by Earthquake Megacities Initiative (EMI, 2006). In the case of Bogota the scenarios of losses building by building were developed by Universidad de Los Andes (2005). For the city of Barcelona probabilistic and deterministic scenarios were developed by ICC/CIMNE (2004). In Manizales the damage scenarios were made by ERN (2005). This section presents the summary of the USRi results for the all cities of Metro Manila to illustrate its application to other metropolitan urban centre.

2.1 Results for Metro-Manila

Metropolitan Manila, the capital city the Philippines is officially called the National Capital Region (NCR). Although it is the smallest region, it is the most densely populated region of the country. Metro Manila is composed by 4 municipalities and 13 cities thereof into an integrated unit with the manager or commission form of government. They are the cities of: Quezon, Kaloocan, Valenzuela, Muntinlupa, Las Piñas, Marikina, Manila, Parañaque, Makati, Mandaluyong, Malabon, Pasay, Pasig. And the municipalities of: Taguig, Pateros, San Juan and Navotas (see Fig. 4).

In order to evaluate the USRi for each city, the physical risk index was calculated using physical risk descriptors based on the earthquake damage

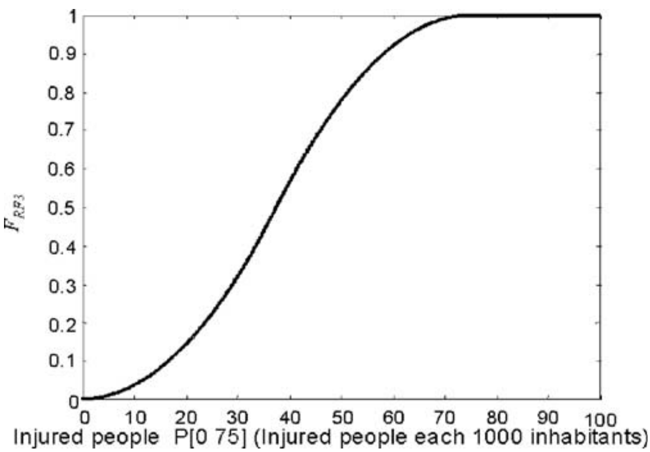

Fig. 4 Territorial division of Metro Manila, Philippines

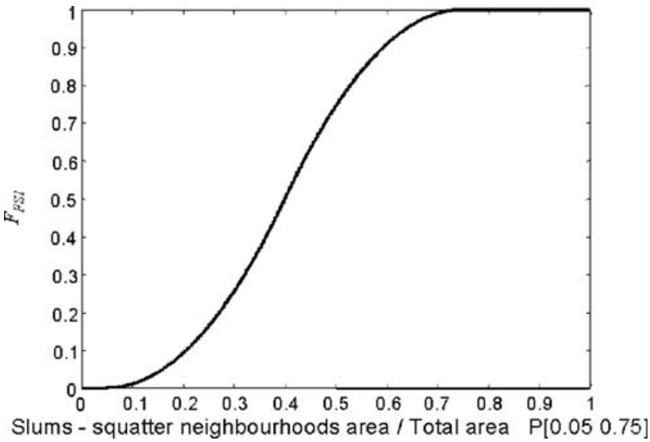

Fig. 5 Physical risk index for Metro Manila

MMEIRS-08, obtained from the Earthquake Impact Reduction Study of Metro Manila (MMEIRS). This scenario corresponds to an earthquake of Magnitude 7.2, in the West Valley Fault, with 2 km of depth. Figures 5, 6, 7 show the results for the physical risk index, the aggravating coefficient and the total risk index (USRi) for Metro Manila using the model above described.

Cities in Metro Manila were clustered according to their level of risk in four different arrays according to the total USRi and its components, physical risk and aggravating factor (social fragility + lack of resilience), this grouping is show, on Table 1.

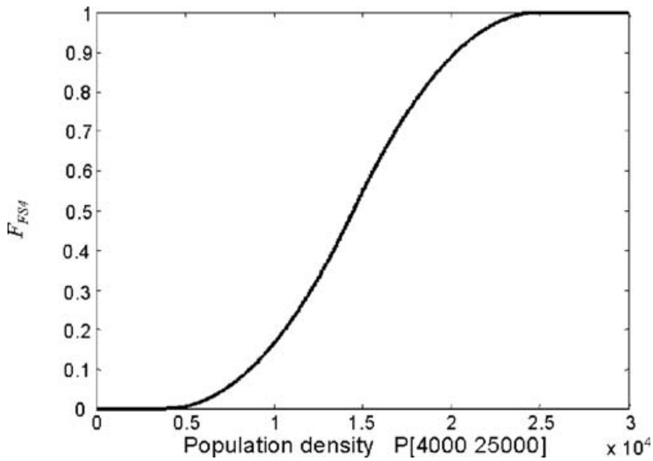

Fig. 6 Aggravating coefficient for Metro Manila

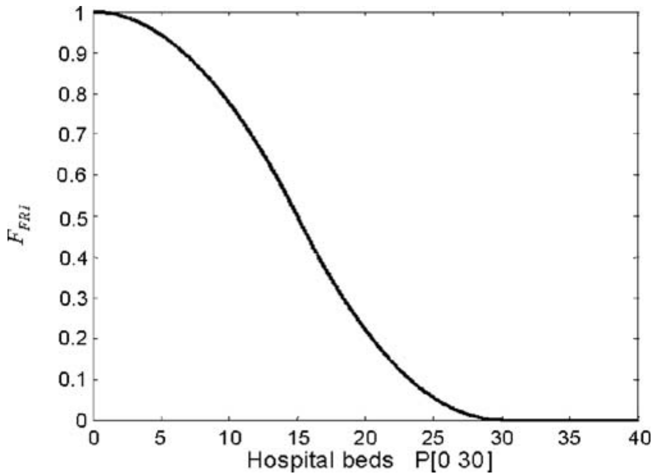

Fig. 7 Total risk index for Metro Manila

Table 1 Holistic urban seismic risk of Metro Manila

Feature	Ind.	Degree	Range	Cities of metro manila
Physical Risk	R_F	Very high	0.45–1.00	Pasig Pasay
		High	0.30–0.44	
		Medium-High	0.20–0.29	Pateros Muntinlupa Marikina Makati Manila Navotas Taguig Mandaluyong Paranaque
		Medium-Low	0.10–0.19	Las Piñas Quezon Malabon San Juan
		Low	0.00–0.09	Valenzuela Kalookan
Aggravating Coefficient	F	Very High	0.65–1.00	Navotas Malabon Taguig San Juan
		High	0.55–0.64	Kalookan Valenzuela Pasay Pateros Las Piñas Quezon Pasig
		Medium-High	0.40–0.54	Marikina Paranaque Mandaluyong Manila Makati Muntinlupa
		Medium-Low	0.20–0.39	
		Low	0.00–0.19	
Total Risk	USRi	Very High	0.70–1.00	Pasay Pasig
		High	0.45–0.69	Navotas Pateros Marikina Taguig
		Medium-High	0.30–0.44	Muntinlupa Manila Makati Mandaluyong Paranaque
		Medium-Low	0.15–0.29	Las Piñas Quezon Malabon San Juan
		Low	0.00–0.14	Kalookan Velenzuela

2.2 Sensitivity Analysis

Sensitivity analysis is also referred to as robustness analysis of the model. In this case, the sensitivity analysis studies how the variation in the values of the index of Total Risk, R_T, can be apportioned, qualitatively or quantitatively, to different sources of variation, and how this given composite indicator, depends upon the information fed into it. In other words, once the proposed model (composite indicator) is determined, it is important to analyse how much the results are influenced by uncertainty in the source data or uncertainty in the weights and transformation functions, due to the stakeholders' subjectivity or plurality of perspectives. For this analysis a Monte Carlo-based simulation was performed.

The Monte Carlo method is a technique that uses sets of random numbers as input parameters and probability distributions for iteratively evaluating a deterministic model. This method is often used when the model is complex, non-linear or involves more than just a couple uncertain parameters. The Monte Carlo simulation is one of many methods for analysing uncertainty propagation, where the goal is to determine how random variation affects the sensitivity, performance or reliability of the model. In this case, the values of the R_T for each territorial unit were obtained five thousand times, using random sets of input data, transformation functions and weights; each value sampled within an acceptable range of variability.

The random values, for each set previously mentioned, were obtained individually by using uniform distributions in each interval. In the same way, random values for simultaneous variation of the all sets were obtained. The minimum and maximum values were chosen according to expert opinions to define the interval of variation for each input parameters. By this way, thousand of stochastic results were created with random inputs of each parameter (input data, transformation functions, weights and all simultaneously) for each territorial unit of analysis. In order to provide a concise summary of the results, the mean, median, standard deviation, maximum and minimum values and a few other summary statistics to describe the resulting distribution were reported. Likewise, a histogram was created for visualizing the uncertainty of results, which illustrates the profile of results, the uncertainty degree and the existing distribution. In addition, the cumulative distribution function was included in each graph to illustrate the percentage of data points that are below a value or point of interest.

Figure 8 shows an example of a histogram and Table 2 illustrates an example of a statistic summary of the Monte Carlo simulation.

Once the results were calculated through Monte Carlo procedure, the variability or volatility graphics were built to compare the stochastic results of R_T to the fixed results obtained from the methodology. Figures 9 and 10 illustrate minimum ($\triangleright$), maximum ($\times$) and mean ($\blacksquare$) values and the bars represent the fixed values obtained for each territorial unit. It can be seen that for the

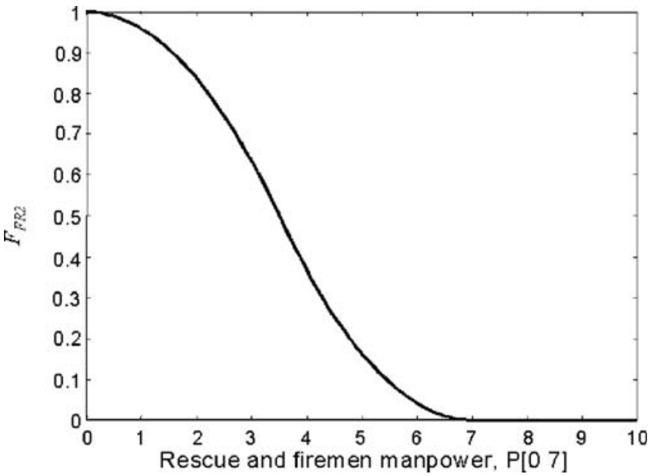

Fig. 8 Example of a histogram of the results of RT for Mandaluyong City with stochastic weights

variability of each parameter the volatility of values is not so big. The uncertainty is bigger in the case of the simultaneous variation of all parameters.

Table 3 shows that the results obtained through the simulation are very similar to the results obtained using fixed values of input data, weights and transformation functions of the method previously described and applied to Metro Manila. The overall results show that the cities of the metropolitan area vary slightly in their rankings. Some units fluctuate at the most by one position. In other urban centres, as Barcelona and Manizales, where the method has been applied and where a similar sensitivity analysis has been made, the results are similar than in Metro Manila. Only in the case of

Table 2 Example of statistic summary of the results for Mandaluyong City with stochastic weights

Central tendency (location)				Quantiles, percentiles, intervals			
Sample Size (N):	5000			90% Interval		95% Interval	
				Q(.05):	0.343	Q(.025):	0.34
Mean	0.40	**Median**	0.40	Q(.95):	0.459	Q(.975):	0.47
StErr:	0.00						
Skewness	0.0435			**Alpha** (a):	0.05	Q(a/2):	0.34
Kurtosis	–1.0225			% Interval:	95%	Q(1-a/2):	0.47
Spread				**Probabilities**			
StDev	0.04			**Pr** (**y** < 0.33) =			0.44%
Max:	0.49	Q(.75):	0.43	**Pr** (**y** > 0.40) =			49.93%
Min:	0.32	Q(.25):	0.37	Pr (0.33 < y < 0.4) =			49.63%
Range	0.17	**IQ Range:**	0.06	Alpha (a):			0.5037

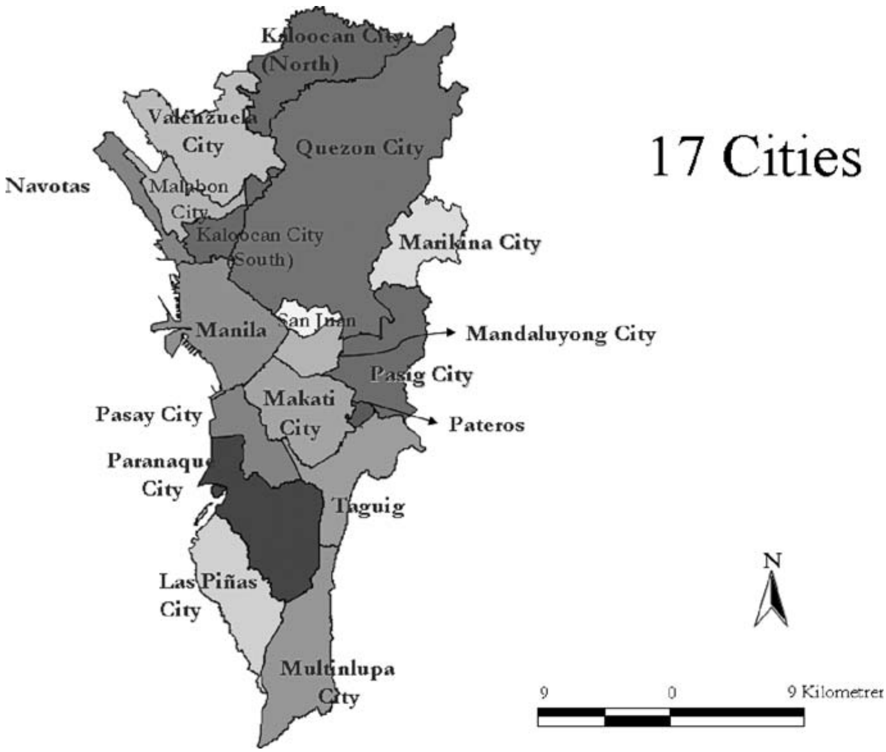

Fig. 9 Variation of the simulation results and fixed results for stochastic weights and stochastic data

Bogota some territorial units have been more volatile with position changes of two and three ranks; however it is not very relevant.

Therefore, according to the classification of the total risk by levels, all cities maintain the same level with exception of San Juan city which presents a change of range from medium-high to medium-low because the total risk result for fixed values is equal to 0.29 and for stochastic values the result was above 0.30. Classification by ranges of risk has special interest, because it is more relevant to have into account the level of risk where a territorial unit is located than its final numerical value for risk management implications.

According to the comparison of the results of sensitivity analysis, based on Monte Carlo simulations, and the results obtained by the holistic seismic risk evaluation here described, it is possible to conclude that the methodology is robust. It is not very sensible to slight variations in the input data and small changes in the modelling parameters, such as weights and transformation functions. The results do not present important or extreme changes. If the range of variation of data and parameters is reasonable, as it is in the case of seismic risk, in general the results of the model will be stable and reliable.

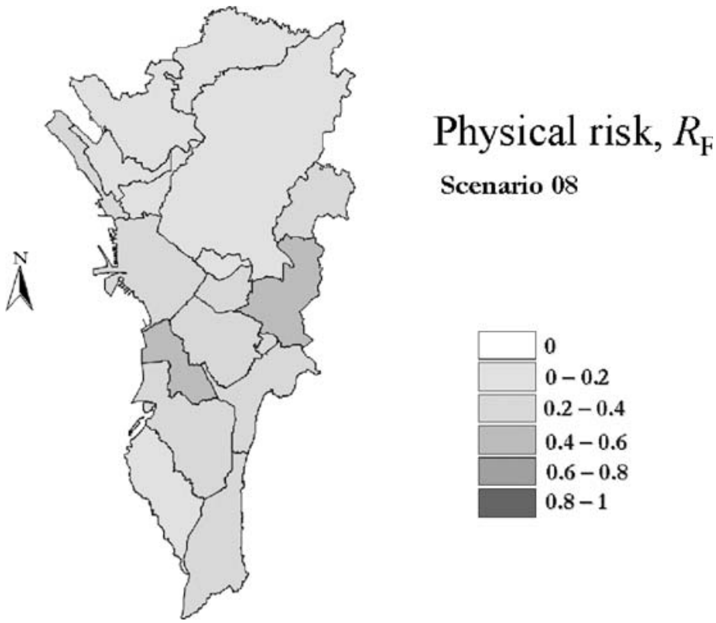

Fig. 10 Variation of the simulation results and fixed results for stochastic transformation functions and simultaneous stochastic data, weights and transformation functions

2.3 Comparison of Results

The results obtained for Metro Manila have been compared with those obtained for Barcelona, Bogota and Manizales. Table 4 shows the average risk values for the four cities, corresponding to the most significant scenarios in each case. Metro Manila and Bogota are located in zones with intermediate seismic hazard, whereas Barcelona is located in a zone with low to moderate seismic hazard and Manizales is placed in a zone with a high seismic hazard. The average values obtained for the physical risk index, R_F, reflect not only the seismic hazard but also the level of physical vulnerability in each city. It is interesting to remark that the results obtained for the aggravating coefficient, F, are not so different for the four cities. The highest value of physical risk is for Bogota, but the worst situation, taking into account the aggravating coefficient, is for Metro Manila.

3 Conclusions

Disaster risk estimation requires a multidisciplinary approach that takes into account not only the expected physical damage, the number and type of casualties or economic losses, but also other social, organizational and

Table 3 Comparison between fixed values and stochastic results of RT, positions and classification of Metro Manila cities by risk levels

Level of risk	Fixed values		Data		Weights		Fuctions	
	City	R_T	City	R_T	City	R_T	City	R_T
Very High	Pasay	0.72	Pasay	0.70	Pasay	0.71	Pasay	0.70
	Pasig	0.71	Pasig	0.70	Pasig	0.70	Pasig	0.70
High	Navotas	0.49	Manila	0.52	Manila	0.53	Manila	0.53
	Manila	0.48	Navotas	0.49	Navotas	0.49	Navotas	0.49
	Pateros	0.47	Pateros	0.49	Pateros	0.49	Pateros	0.49
	Marikina	0.45	Marikina	0.47	Marikina	0.46	Marikina	0.47
	Taguig	0.45	Taguig	0.46	Taguig	0.46	Taguig	0.46
Médium-High	Muntinlupa	0.43	Makati	0.43	Makati	0.43	Makati	0.43
	Makati	0.42	Muntinlupa	0.43	Muntinlupa	0.42	Muntinlupa	0.43
	Mandaluyong	0.39	Mandaluyong	0.40	Mandaluyong	0.40	Mandaluyong	0.40
	Paranaque	0.31	Paranaque	0.33	Paranaque	0.33	Paranaque	0.33
Médium-Low	San juan	0.29	San juan	0.31	San juan	0.31	San juan	0.32
	Malabon	0.26	Malabon	0.27	Malabon	0.27	Malabon	0.27
	Quezon	0.19	Quezon	0.21	Quezon	0.21	Quezon	0.22
	Las pinas	0.17	Las pinas	0.19	Las pinas	0.19	Las pinas	0.19
Low	Kalookan	0.05	Kalookan	0.09	Kalookan	0.09	Kalookan	0.09
	Valenzuela	0.03	Valenzuela	0.07	Valenzuela	0.06	Valenzuela	0.06

Table 4 Comparison between the worst scenarios for the studied cities

Index	Barcelona	Bogota	Manizales	Metro manila
Physical risk, R_F	0.08	0.32	0.27	0.24
Aggravating coeff. F	0.42	0.55	0.56	0.59
USRi = Total risk, R_T	0.11	0.50	0.44	0.38

institutional issues related to the development of communities that contribute to the creation of risk. At the urban level, for example, vulnerability seen as an internal risk factor should be related not only to the level of exposure or the physical susceptibility of the buildings and infrastructure material elements potentially affected, but also to the social fragility and the lack of resilience or capacity to cope of the exposed community. The absence of institutional and community organization, weak preparedness for emergency response, political instability and the lack of economic health in a geographical area contribute to risk increasing. Therefore, the potential negative consequences are not only related to the effects of the hazardous event as such, but also to the capacity to absorb the effects and the control of its implications in a given geographical area.

For the modelling, a simplified but multidisciplinary representation of urban seismic risk has been suggested, based on the parametric use of variables that reflect aspects of such risk. This parametric approach is not more than a model formulated in the most realistic possible manner, to which corrections or alternative figures may be continuously introduced. The consideration of physical aspects allowed the construction of a physical risk index. Also, the contextual variables (social, economic, etc.) allowed the construction of an aggravating coefficient. The former is built from the information about the seismic scenarios of physical damage (direct effects) and the latter is the result from the estimation of aggravating conditions (indirect effects) based on descriptors and factors related to the social fragility and the lack of resilience of the exposed elements.

This new model for holistic evaluation of risk facilitates the integrated risk management by the different stakeholders involved in risk reduction decision-making. It permits the follow-up of the risk situation and the effectiveness of the prevention and mitigation measures can be easily achieved. Results can be verified and the mitigation priorities can be established as regards the prevention and planning actions to modify those conditions having a greater influence on risk in the city. Once the results have been expressed in graphs for each locality or district, it is easy to identify the most relevant aspects of the total risk index, with no need for further analysis and interpretation of results. Finally, this method allows to compare risk among different cities around the world and to perform a multi-hazard risk analysis.

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Part IV
Urban Planning Facing Natural Hazards, Information and Warning

Risk Estimates for Germany

Friedemann Wenzel, Fouad Bendimerad and Bruno Merz

1 Definition of Risk

Three components need to be determined in order to quantify risk from a particular peril and in delimited region: Hazard, vulnerability and exposure. Hazard pertains to the natural or man-made sources that cause an event to happen. In the case of earthquakes, the hazard would be the physical conditions (i.e. the seismic sources, seismo-tectonic environment, etc) that trigger earthquakes of different sizes. A hazard thus has the potential to cause harm and loss to people, human activity, property and the environment. Vulnerability relates to the potential of the build environment to sustain loss due to the occurrence of a hazard event. Hence, vulnerability not only includes the damage potential to buildings and other man-made structures but also the impact on population, economy and environment. Exposure is the ensemble of the assets at risk including population, buildings, critical facilities, lifelines, transportation systems, and institutional and societal processes, or lack thereof that can be disturbed by an earthquake event. An example of the latter would be the inability of an institution to deliver services because of failure of emergency response procedures. Risk is the probability of loss (physical, monetary or non-monetary) due to a particular hazard for a given area and a reference time period.

The relationship between severity of loss and frequency of loss is essential in quantifying risk. For example, in financial application such as insurance the understanding of the frequency-severity relationship is essential in underwriting, transferring and pricing risk. The frequency component of the risk is also important when assessing risk from several hazards (e.g., earthquakes, floods, draught, forest fires, etc.), or when comparing risk from a single hazard over a large geographic area. The severity-frequency relationship allows for an understanding of the relative relationships between the risks caused by many hazards, thus helping in setting up priorities for disaster mitigation purposes. Typically,

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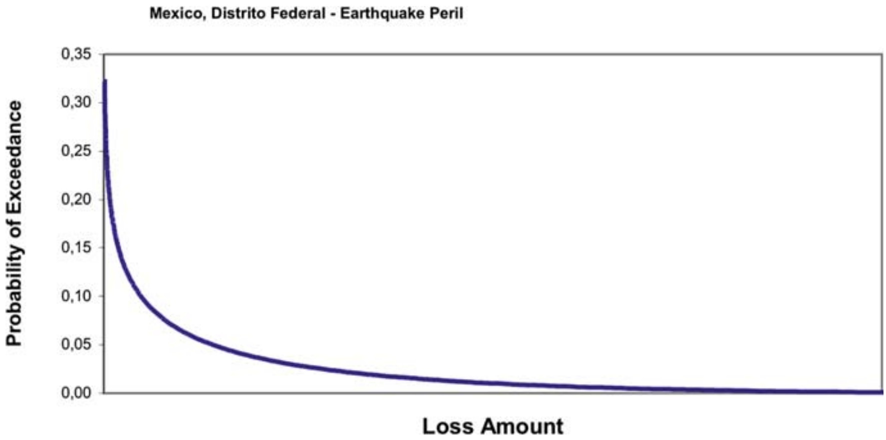


Fig. 1 Example of exceeding probability curve

this relationship is drawn as the probability of a exceeding a certain level of loss either from a single occurrence or multiple occurrences. The time period for calculating probabilities is generally one year. The occurrence exceeding probability (OEP) is a widely used concept in measuring risk. An example of OEP curve is shown in Fig. 1.

The integration of frequency and severity provides the average loss, which is the theoretical long-term measure of the value of the risk for a particular region and a particular peril. Average loss is an important parameter in measuring risk. It is the essential quantity for pricing risk (for example in insurance) and for performing relative comparisons of the various risks for purpose of mitigation. Generally average loss is measured on a one-year period and is referred to as Average Annual Loss or AAL.

$$\mathrm{AAL} = \mathrm{SumLoss(i)} \times \mathrm{prob(i)} \quad \text{For all events that cause loss}$$

The probability of occurrence can be estimated from the historical data. For rare events such as earthquakes, a probabilistic analysis needs to be undertaken in order to develop the severity-frequency relationship and estimate the average loss. The historical data is often too sparse and uncertain to produce a meaningful estimate of the frequency component. A complete treatment of the probabilistic modeling of risk has been published by the author elsewhere (Bendimerad et al., 2000; Bendimerad, 2001a).

2 Scenario Analysis

For disaster mitigation purposes, consideration of frequency may sometimes not be relevant. Typically, an event (e.g., earthquake) of a certain magnitude is assumed to happen and the disaster management planning is based on the consequences of such an event. This methodology is typically referred to as scenario analysis. Scenario analyses have been used for the last three decades to develop disaster management plans and to establish risk management policy. The event can be hypothetical or a repeat of an earthquake that happened in the past. Scenario analyses are a powerful tool for disaster assessment and management. They develop internal knowledge, provide the parameters for disaster management planning, and allow for an understanding of the vulnerabilities in infrastructure and population. In addition, scenario analyses are a tool for communication between the scientists (who develop the knowledge base for the scenario itself), the practitioners (who are in charge of the implementation of risk reduction measures), and the policy makers who decide on programs, budgets and priorities. Therefore, they are excellent policy-setting tools.

A complete treatment of scenario analysis is provided in Bendimerad (2001b), and Bendimerad (1997). The undertaking of a scenario analysis includes the following steps:

Step 1: The development and characterization of the inventory of built environment and the associated human exposure and economic value
Step 2: The analysis of the potential earth science hazards (PESH) to quantify the severity of ground shaking and ground failure for the postulated scenario earthquake
Step 3: The analysis of the potential damage (or vulnerability) to the inventory of structures under the projected ground shaking and ground
Step 4: The translation of damage potential into social and economic loss considering economic value and the inter-dependencies of the inventory

The general building blocks of a scenario analysis are shown in Fig. 2.

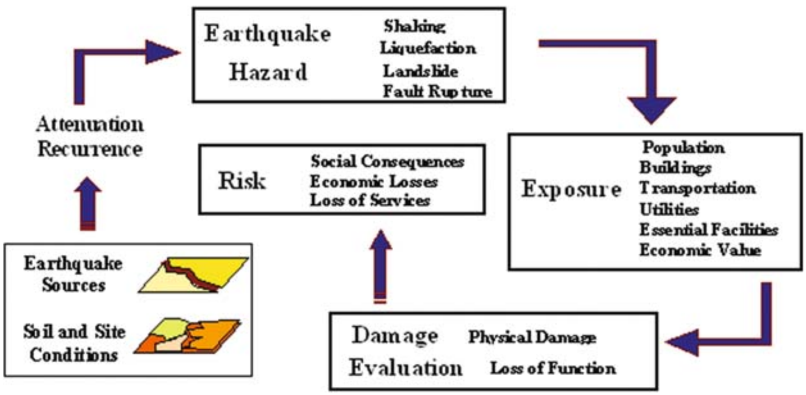


Fig. 2 Sketch of components and process for risk assessment

3 Seismic Risk of Germany

The Center for Disaster Management and Risk Reduction Technology (CEDIM, www.cedim.de), established as a joint initiative of the GeoForschungsZentrum Potsdam (GFZ) and the University of Karlsruhe, conducted an interdisciplinary study aimed at the assessment and mapping of different kinds of risks for die territory of Germany, including its seismic risk. Considerable parts of the country with its 357,000 square kilometres and 82 million populations are exposed to moderate seismicity, where earthquakes produce ground shaking intensity up to EMS intensity VIII. The seismic zones are densely populated, industrialized and display a high concentration of developed infrastructure, representing a typical example foe low probability high impact seismic risk.

The entire range of 14,000 communities in Germany was classified into five population classes P1 (less than 2,000 inhabitants), P2 (2,000–20,000), P3 (20,000–200,000), P4 (200,000–800,000) and P5 (more than 800,000) (Tyagunov et al., 2006). The seismic hazard input consist of the German part of the D-A-CH map (Grünthal et al., 1998), constructed for a non-exceedance probability of 90% in 50 years (average return period of 475 years) in terms of seismic EMS-98 intensity for Germany, Austria and Switzerland. For four vulnerability classes A, B, C and D the damage probability matrices were constructed following the ideas of die European Macroseismic Scale (EMS-98), where the description of damage distribution in terms of “few”, “many”, “most” is given in intensities.

Vulnerability functions were constructed for all vulnerability classes (A–D) in terms of the mean damage ratio (MDR) versus intensity of ground shaking. For computation of die MDR, which is considered as die cost of repair over die cost of replacement, the damage ratio range was assigned to the damage grades classified in the EMS-98. The combination of vulnerability models with the distribution of communities of different population classes and the seismic hazard input results in the specific damage distribution, e.g. the percentage of damage of the existing building stock. By assuming that the value in buildings in a community is proportional to the number of its inhabitants, we then estimate the risk as the product of the specific damage and the number of inhabitants in the communities. In order to obtain a rough estimate of the level of probable monetary losses we use an averaged value of construction cost per person of 50,000 €/person, which is approximately equal to the preliminary estimated average level of construction cost for residential buildings in the state of Baden-Württemberg (Kleist et al., 2006). The results of Fig. 3 can be interpreted as probable direct losses due to damage to residential buildings, corresponding to the considered level of seismic hazard for a non-exceedance probability of 90% in 50 years.

Scenarios for known historical events were designed: A repeat of the Albstadt event of September 3, 1978 with moment magnitude of 5.1 at a depth of 10 km

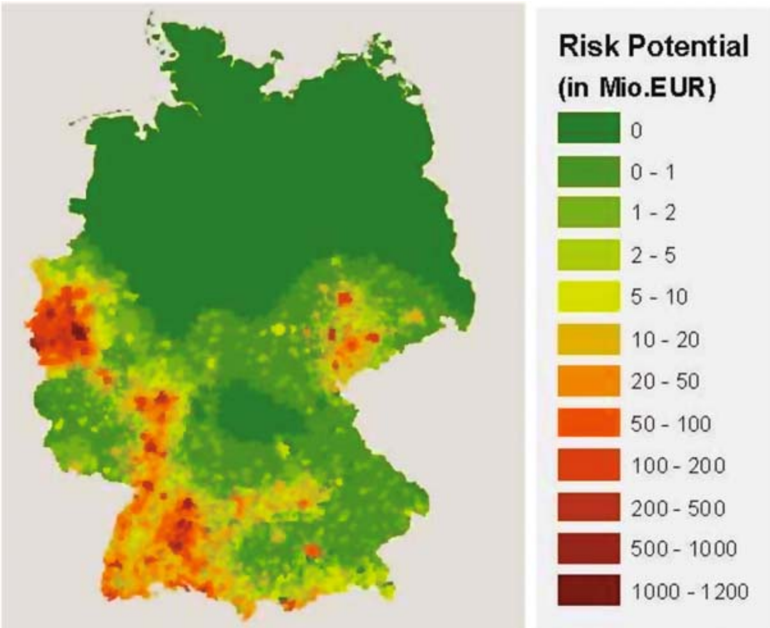


Fig. 3 Risk map for Germany representing a non-exceedance probability of 90% in 50 years

with maximum intensity of VII to VIII would generate damage to residential buildings in the range of 200 million Euros. A repeat of the Ebingen earthquake of November 16, 1911 with moment magnitude of 5.7 at a depth of 10 km with maximum intensity of VII would generate damage to residential buildings in the range of 1.0–1.5 billion Euros. If the epicentre of this earthquake moves closer to the city of Tübingen damage will be in the range of 8 billion rather.

4 Multi-Hazard Risk Assessment

Multi-hazard risk assessment is the process of estimating different impacts of hazards causing harm and loss to people, human activity, property and the environment. It includes the interdependency and interrelationships of natural hazards (landslide following ground shaking) and consequential risks (fire following earthquakes, toxic spills after flooding). Multi-hazard risk assessment requires a truly comprehensive concept and understanding of vulnerability. Risk assessment aims at identifying, measuring, quantifying and evaluating the worst effects of natural hazards in a comprehensible, comparable way. It develops answers on the following questions: What can go wrong? What is the likelihood that it could go wrong? What are the consequences? The latter questions addresses the specific issues of what values and quantities could be harmed, what is intensity duration and frequency of exposures and how the impact could be mitigated.

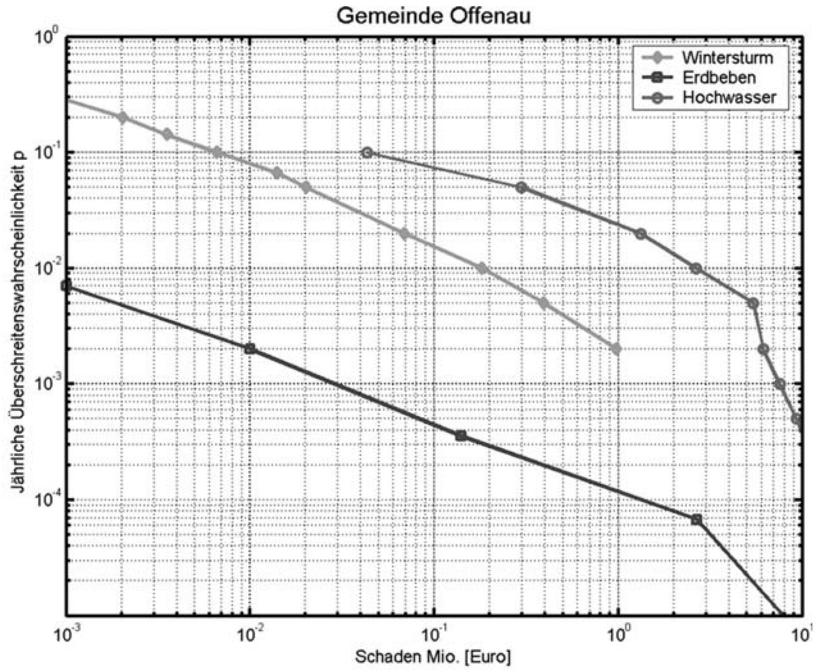
Multi-hazard risk analysis thus includes three issues:

1. Interaction of generically different types of hazards such as landslides or avalanches triggered by earthquakes or heavy precipitation, landslides that form dams and subsequent floods or that impact directly into lakes, again causing flooding.
2. Comparison of different types of risks (a) from a financial point of view (risk vs. probability of exceedance and annual average risk) and (b) from a disaster management view (synopsis).
3. Consequential risks for industrial production, infrastructure and economy at large, all types of indirect damage including complex vulnerability analysis considering different types of risks and including physical, economical, social, and environmental aspects.

Research on multihazard risk assessment has been done on national, European and global scale: National studies have been done e.g. in Germany (DFNK study for Cologne, Grünthal et al., 2006), Switzerland (KATARISK study), risk study in Australia performed by BMR, national risk assessment by HAZUS (FEMA, U.S.A.) and others. European projects were TIGRA, TEMRAP, and ESPON. On the global scale the World Bank sponsored GRID. An example for a regional study is the assessment of risks via index methodology for 12 Latin-American countries financed by the IADB. These projects can be classified according to different characteristics, such as

Methodology:	indicator-based, deterministic, probabilistic
Spatial Scale:	local, country-wide, regional, worldwide
Type of Hazards:	natural (floods, storms, etc), human induced (oil or toxic spill)
Exposures:	population, buildings, infrastructures, functions, others
Vulnerability:	GDP, population density, planning, warning systems
Risks:	loss of lives, economic losses, risk classes.

CEDIM developed a national risk map for earthquakes, winter storms and floods (Tyagunov et al., 2006; Heneka et al., 2006; Büchele et al., 2006). For several communities in Baden-Wuerttemberg risk curves and values for annual average risks for all three hazard types developed. Figure 4 shows this comparison for the Commuity of Offenau (2,700 inhabitants) located at river Neckar and the city of Tübingen (82,000 inhabitants) with no flood risk. The risk estimates for Offenau (Fig. 4a) indicate that flood risk is highest for all return periods and earthquake risk negligible. At a return period of 100 years flood loss is about 2.5 million Euros, whereas storm accounts for only 200,000 million Euro. For the city of Tübingen (Fig. 4b) we find: At a return period below 30 years storm damage exceeds earthquake damage. At 30 years they are equal (2 million Euro). At 50 years earthquake damage exceeds storm damage by a



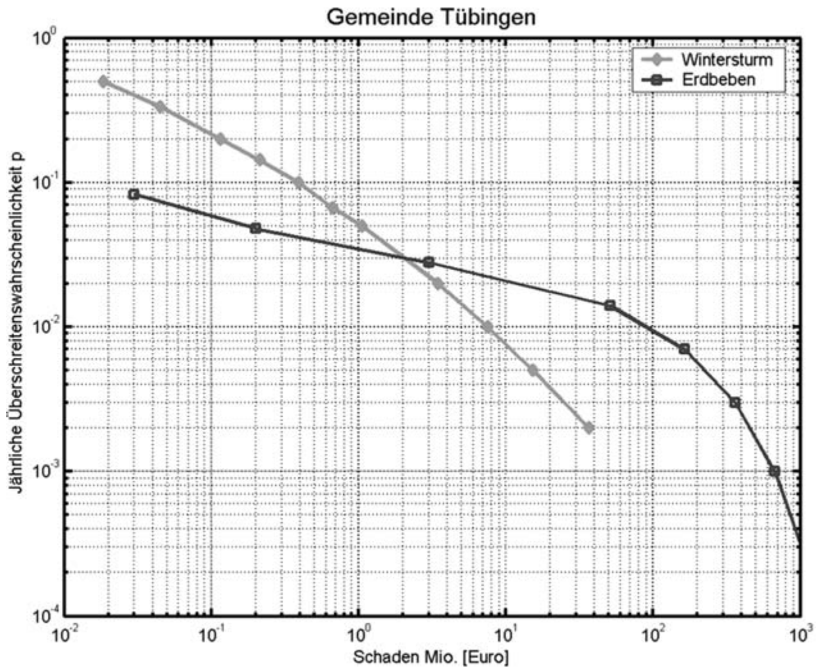
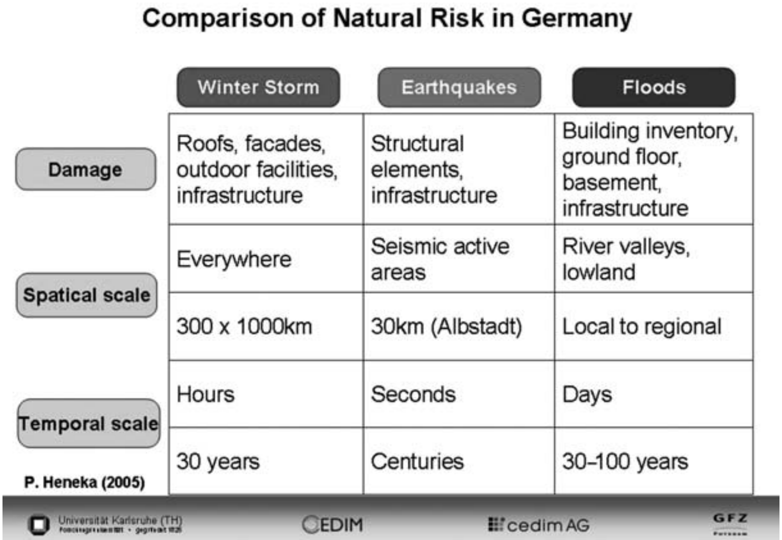


Fig. 4 Risk curves for earthquake, storm and flood risk for 2 communities in Germany (P. Heneka, 2005)


Comparison of Natural Risk in Germany

	Winter Storm	Earthquakes	Floods
Damage	Roofs, facades, outdoor facilities, infrastructure	Structural elements, infrastructure	Building inventory, ground floor, basement, infrastructure
Spatical scale	Everywhere	Seismic active areas	River valleys, lowland
	300 x 1000km	30km (Albstadt)	Local to regional
Temporal scale	Hours	Seconds	Days
	30 years	Centuries	30–100 years

Fig. 5 Comparison of the different damage types and pattern, and the different spatial and temporal scales involved in earthquake, storm and flood damage

factor of 3, whereas for larger return periods (475 years) storm accounts for 40 million and earthquakes for 500 million Euro.

Currently we develop a full-scale synopsis for the entire state of Saxony. This will be done at the community level with 539 communities and be restricted to damage to residential buildings. Understanding financial risks is only one components of risk management. A full understanding and modelling requires consideration of the different damage types and pattern, and the different spatial and temporal scales involved. Figure 5 provides a comparison of these aspects.

An issue of specific relevance is consequential risks: Modern life became highly dependent on networked infrastructure systems such as transportation, energy and water supply, and communication. Each of them is critical for societies to function, can cause significant, although mostly indirect losses, but is largely unexplored so far for multi-hazard impact. The table summarises the general and specific approaches to study the impacts of natural hazards on infrastructure networks, their spatial and time scales, and their mutual interdependencies. A significant challenge is to quantify the mostly indirect costs (social and economic impacts) of damage to infrastructure. Table 1 compares damage patterns and spatial and temporal scales of impacts on lifeline systems such as transportation, electric power supply, communication and water supply.

Systematic linkage between tools to model road traffic flow and the impact on infra-structure from earthquakes and flooding are under development for

Table 1 Comparison of damage patterns and spatial and temporal scales of impacts on lifeline systems such as transportation, electric power supply, communication and water supply

	Transportation	Electrical power	Communication	Water supply
Affected infrastructure	Streets, bridges, tunnels, railway tracks, hubs (airports, stations, etc.)	Transmission lines, substations and power stations	Fixed (fiber optics and cable) and mobile (radio components) infrastructure	Storage tanks, operation of pumps and control devices
Time scale	Real-time, days to years	Real-time, minutes to days	Real-time, minutes to days	Real-time, days, weeks
Spatial scale	Urban, regional, countrywide	Street, block, city, region	Street, block, city, region	Urban, regional
Input	Disrupted network infrastructure (degree and duration), traffic flows, activity patterns	Electrical power network structure before the hazard and destructed network structure for different disaster scenarios, actual and desired load flow	Throughput characteristic of the intact network, capacity necessary for different disaster scenarios	Water demand patterns, disrupted network, pump failure, contamination
Output	Identification of critical infrastructure and impact patterns, risk reduction and management strategies, emergency plans	Identification of critical nodes and components, rules for designing an optimum power grid, strategies and components for rapid installation of emergency power supply	Identification of critical nodes and components, survival strategies for a core network for rescue and security, reconstruction of communications infrastructure	Identification of critical infrastructure and impact patterns, risk reduction and management strategies, emergency plans
Interdependencies	Electrical power supply, communication	Transportation and communication infrastructure and to some extent water supply	Electrical power and transportation infrastructure	Electrical power supply, communication

the Rhinegraben Area. We apply a four-step approach that includes trip generation, trip distribution, modal choice, and assignment. The distortion of traffic (freight and passengers) can be modelled for scenarios and evaluated by a macro-economic tool (ASTRA developed by, Institute for Economic Policy Research, Karlsruhe University).

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Traditional and Innovative Methods for Seismic Vulnerability Assessment at Large Geographical Scales

G.M. Calvi, R. Pinho and G. Magenes

1 Introduction

Seismic risk can be defined as the possibility or probability of losses due to an earthquake, whether these losses are human, social, or economic; seismic risk can be quantified through the convolution of four individual factors:

$$\text{Siesmic Risk} = \text{Seismic Hazard} \times \text{Exposure} \times \text{Vulnerability} \times \text{Specific Cost}$$

The seismic hazard represents the effects that earthquakes can produce at the site of a structure or other engineering project (e.g. strong ground shaking) whilst exposure refers to the extent of human activity located in the zones of seismic hazard. The vulnerability represents the susceptibility of the exposed elements to earthquake effects and the specific cost represents the cost of the repair/restoration of a structure as a proportion of the cost of demolition and replacement of the structure.

Earthquake risk or loss modelling based on a single earthquake scenario can be very useful, particularly for communicating seismic risk to the public and to decision makers. Nevertheless, for many applications, including decision-making processes within the insurance and reinsurance industries, and in seismic code drafting committees, it is necessary to estimate the effects of many, or even all, possible future earthquake scenarios that could impact upon the urban areas under consideration. In either case, a significant component of a loss model is a methodology to assess the vulnerability of the built environment.

The seismic vulnerability of a structure can be described as its susceptibility to be damaged by ground shaking of a given intensity. The aim of a vulnerability assessment is to obtain the probability of a given level of damage to a given building type due to a scenario earthquake. The various methods for vulnerability assessment that have been proposed in the past for use in loss

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estimation can be divided into two main categories: empirical or analytical, both of which can then be combined in hybrid applications.

A vulnerability assessment needs to be made for a particular characterisation of the ground motion, which will represent the seismic demand of the earthquake on the building. The selected parameter should be able to correlate the ground motion with the damage to the buildings. Traditionally, macroseismic intensity and peak ground acceleration have been used, whilst more recent proposals have linked the seismic vulnerability of the buildings to response spectra obtained from the ground motion.

Each vulnerability assessment method models the damage on a discrete damage scale; frequently used examples include the MSK scale (Medvedev and Sponheuer, 1969), the Modified Mercalli scale (Wood and Neumann, 1931) and the EMS98 scale (Grünthal, 1998). In empirical vulnerability procedures, the scale is used in reconnaissance efforts to produce post-earthquake damage statistics, whilst in analytical procedures the scale is related to limit state mechanical properties of the buildings, such as inter-storey drift capacity.

A detailed account of the evolution of vulnerability assessment procedures for both individual buildings and building classes is given in the work by Calvi et al. (2006). Herein, the latter is somewhat summarized, with a view to provide readers with an overview of the most important applications and developments of the last 30 years, then followed by the introduction of an innovative and rational displacement-based approach to the same problem.

2 Empirical Vulnerability Assessment Approaches

The seismic vulnerability assessment of buildings at large geographical scales has been first carried out in the early 70's, through the employment of empirical methods initially developed and calibrated as a function of macroseismic intensities. This came as a result of the fact that, at the time, hazard maps were in their vast majority defined in terms of these discrete damage scales (earlier attempts to correlate intensity to physical quantities, such as PGA, lead to unacceptably large scatter). Therefore these empirical approaches constituted the only reasonable and possible approach that could be initially employed in seismic risk analyses at a large scale.

2.1 Damage Probability Matrices

Whitman et al. (1973) first proposed the use of damage probability matrices for the probabilistic prediction of damage to buildings from earthquakes; the concept of a DPM is that a given structural typology will have the same probability of being in a given damage state for a given earthquake intensity. Whitman et al. (1973) compiled DPMs for various structural typologies according to the damaged sustained in over 1600 buildings after the 1971 San Fernando earthquake, whilst Braga et al. (1982) based his DPMs on the damage data of Italian buildings after the 1980 Irpinia earthquake. This type of method has also been termed "direct" (Corsanego and Petrini, 1990) because there is a direct relationship between the building typology and observed damage. The use of DPMs is still popular in Italy, as the recent proposals of Dolce et al. (2003) and Di Pasquale et al. (2005) demonstrate.

Damage Probability Matrices based on expert judgement and opinion were first introduced in ATC-13 (ATC, 1985). More than 50 senior earthquake engineering experts were asked to provide low, best and high estimates of the damage factor (the ratio of loss to replacement cost, expressed as a percentage) for Modified Mercalli Intensities (MMI) from VI to XII for 36 different building classes. The low and high damage factor estimates provided by the experts were defined as the 90% probability bounds of a lognormal distribution, whilst the best estimate was taken as the median damage factor. Weighted means of the experts' estimates, based on the experience and confidence levels of the experts for each building class, were included in the averaging process, as described in Appendix G of ATC-13 (ATC, 1985). Examples of the use of DPMs based on the ATC-13 approach for the assessment of risk and loss include the city of Basel (Fah et al., 2001), Bogotá (Cardona and Yamin, 1997) and New Madrid (Veneziano et al., 2002).

A macroseismic method has recently been proposed (Giovinazzi and Lagomarsino, 2001, 2004) that leads to the definition of damage probability functions based on the EMS-98 macroseismic scale (Grünthal, 1998). The problems related to the *incompleteness* of the matrices (i.e. the lack of information for all damage grades for a given level of intensity) and the *vagueness* of the matrices (i.e. they are described qualitatively) have been tackled by Giovinazzi and Lagomarsino (2004) by assuming a beta damage distribution and by applying Fuzzy Set Theory, respectively. This macroseismic method has already been applied in the risk assessment of the City of Faro (Oliveira et al., 2004), Lisbon (Oliveira et al., 2005) and Barcelona (Lantada et al., 2004).

DPMs, based on intensity, have allowed the assessment of seismic risk on a large scale in both an efficient and cost-effective manner because in the past seismic hazard maps were also defined in terms of macroseismic intensity. Further, the use of observed damage data to predict the future effects of earthquakes also has the advantage that when the damage probability matrices are applied to regions with similar characteristics, a realistic indication of the expected damage should result and many uncertainties are inherently accounted for. On the contrary, however, the validity of their application to regions with markedly different building sock characteristics and/or hazard, is doubtful. Moreover, the use of macroseismic intensity scale implies that both the ground motion input and the vulnerability are based on the observed damage from earthquakes, which is far from ideal, and in fact recent developments have seen the derivation of DPMs using PGA maps as definition of seismic input.

2.2 Vulnerability Index Method

The vulnerability index method (Benedetti and Petrini, 1984; GNDT, 1993) has been used extensively in Italy in the past few decades (e.g. Faccioli et al., 1999; GNDT, 2000) and is based on a large amount of damage survey data; this method is "indirect" because a relationship between the seismic action and the response is established through a "vulnerability index". The method uses a field survey form to collect information on the important parameters of the building which could influence its vulnerability: for example, plan and elevation configuration, type of foundations, structural and non-structural elements, state of conservation and type and quality of materials. There are eleven parameters in total, which are each identified as having one of four qualification coefficients, K_i, in accordance with the quality conditions – from A (optimal) to D (unfavourable) – and are weighted to account for their relative importance. The vulnerability index ranges from 0 to 382.5, but is generally normalised from 0 to 100, where 0 represents the least vulnerable buildings and 100 the most vulnerable. The data from past earthquakes is used to calibrate vulnerability functions to relate the vulnerability index (I_v) to a global damage factor (d) of buildings with the same typology, for the same macroseismic intensity or peak ground acceleration (PGA). The damage factor ranges between 0 and 100% and defines the ratio of repair cost to replacement cost. The damage factor is assumed negligible for PGA values less than a given threshold and it increases linearly up until a collapse PGA, from where it takes a value of 100%.

The main advantage of "indirect" vulnerability index methods is that they allow the vulnerability characteristics of the building stock under consideration to be determined, rather than base the vulnerability definition on the typology alone. Nevertheless, the methodology still requires expert judgement to be applied in assessing the buildings, and the coefficients and weights applied in the calculation of the index have a degree of uncertainty that is not generally accounted for. Furthermore, in order for the vulnerability assessment of buildings on a large (e.g. national) scale to be carried out using vulnerability indices, a large number of buildings which are assumed to represent the national building stock need to be assessed and combined with census data (e.g. Bernardini, 2000); in a country where such data are not already available, the calculation of the vulnerability index for a large building stock would be very time consuming. However, of course, in any risk or loss assessment model a detailed collected of input data is required for application at a national scale.

2.3 Continuous Vulnerability Curves

Continuous vulnerability functions based directly on the damage of buildings from past earthquakes were introduced slightly later than DPMs; one obstacle to their derivation being the fact that macroseismic intensity is not a continuous variable. This problem was overcome by Spence et al. (1992) through the use of the Parameterless Scale of Intensity (PSI) to derive vulnerability functions based on the observed damage of buildings using the MSK damage scale. Orsini (1999) also used the PSI ground-motion parameter to derive vulnerability curves for apartment units in Italy. Both studies subsequently converted the PSI to peak ground acceleration (PGA) using empirical correlation functions, such that the input and the response were not defined using the same parameter. Other Italian applications of this methodology are, for instance, the work of Sabetta et al. (1998) and Rota et al. (2006), both of whom have used data obtained from post-earthquake damage surveys carried out in various municipalities over the past 30 years in Italy in order to derive typological fragility curves for typical building classes.

Alternative empirical vulnerability functions have also been proposed, generally with normal or lognormal distributions, which do not use macroseismic intensity or PGA to characterise the ground motion but are related to the spectral acceleration or spectral displacement at the fundamental elastic period of vibration; in general this has been found to produce vulnerability curves which show improved correlation between the ground motion input and damage (e.g. Rossetto and Elnashai, 2003; Scawthorn et al., 1981; Shinozuka et al., 1997). The latter has been an important development as it has meant that the relationship between the frequency content of the ground motion and the fundamental period of vibration of the building stock is taken into consideration; in general this has been found to produce vulnerability curves which show improved correlation between the ground motion input and damage. The introduction of vulnerability curves based on spectral ordinates, rather than PGA or macroseismic intensities, has also certainly been facilitated by the emergence of more and more attenuation equations in terms of spectral ordinates.

2.4 Screening Methods

In Japan, the evaluation of the seismic performance of existing reinforced concrete buildings with less than six storeys has been carried out since 1975 with the use of the Japanese Seismic Index Method (JBDPA, 1990). Three seismic screening procedures are available to estimate the seismic performance of a building with the reliability increasing with each screening level. The seismic performance of the building is represented by a seismic performance index, I_S, which should be calculated for each storey in every frame direction within the building using the following equation: $I_S = E_0 S_D T$, where E_0 is the basic structural performance, S_D is the sub-index concerning the structural design of the building and T is the sub-index of the time-dependent deterioration of the building.

The rapid assessment of the seismic performance of existing reinforced concrete buildings has received particular attention in Turkey in recent years. Several preliminary assessment methods have been proposed that require the dimensions of the lateral load resisting elements to be defined. Hassan and Sozen (1997) have proposed a procedure to define a Priority Index for each building which is a function of a wall index (area of walls and infill panels divided by total floor area) and a column index (area of columns divided by total floor area), damage data can be used to calibrate the Priority Index and define the level of vulnerability of the building. Yakut (2004) has recently proposed a Capacity Index which considers the orientation, size and material properties of the lateral load-resisting structural system as well as the quality of workmanship and materials and architectural features such as short columns and plan irregularities. The Seismic Safety Screening Method (SSSM) has recently been proposed by Ozdemir et al. (2005), which is an adaptation of the Japanese Seismic Index Method (JBDPA 1990). This rapid seismic safety evaluation method can be applied to structures with six storeys or less with either reinforced concrete frame, shear wall or dual frame-shear wall structural systems. The method has been calibrated to Turkish buildings using nonlinear static analyses of 12 buildings from Zeytinburnu, Istanbul (Ozdemir et al., 2005).

The use of rapid screening methods has an important role to play in the definition of prioritisation of buildings for seismic retrofit, but the use of such methods in large scale seismic risk models is limited due to the need to consider buildings individually in a deterministic fashion, and thus it would not be economically feasible.

3 Analytical Vulnerability Assessment Approaches

The emergence of more attenuation equations in terms of spectral ordinates and corresponding derivation of seismic hazard maps in terms of spectral ordinates, as opposed to macroseismic intensity or PGA, has not only catered for the aforementioned improvement of empirical methods, but also gave rise to the development of analytical/mechanical methods. These tend to feature slightly more detailed and transparent vulnerability assessment algorithms with direct physical meaning, that not only allow detailed sensitivity studies to be undertaken, but also cater for straightforward calibration to various characteristics of building stock and hazard (a definite disadvantage of empirical methods). Such characteristics place these type of loss assessment approaches, described below, in an ideal position for employment in parametric studies that aim at the definition/calibration of urban planning, retrofitting, insurance and other similar policies or initiatives (e.g. Bommer et al., 2005).

3.1 Analytically-Derived Vulnerability Curves and DPMs

Although vulnerability curves and damage probability matrices have traditionally been derived using observed damage data, recent proposals have made use of computational analyses to overcome some of the drawbacks of these methods that have been highlighted in the previous section. Singhal and Kiremidjian (1996) developed fragility (or vulnerability) curves and damage probability matrices for three categories of reinforced concrete frame structures using Monte Carlo simulation. The probabilities of structural damage were determined using nonlinear dynamic analysis with an ensemble of ground motions. For the DPMs, Modified Mercalli intensity was used for the ground-motion parameter, whilst spectral acceleration was used in the generation of fragility functions. Statistical analysis of the damage indices led to the evaluation of the probability of different damage states and thus fragility functions and DPMs were evaluated. The analytical vulnerability curves produced by Singhal and Kiremidjian (1996) for low-rise frames described were subsequently updated using observational data obtained from a tagging survey of 84 buildings damaged by the 1994 Northridge earthquake using a weighting system (Bayesian updating technique) to take into account the reliability of different data sources (Singhal and Kiremidjian, 1998).

Masi (2004) employed a similar procedure to characterise the seismic vulnerability of different types of reinforced concrete frames (bare, regularly infilled and pilotis) designed to vertical loads alone. The structural models employed in the study were representative of buildings designed and constructed in Italy over the past 30 years. A simulated design of the structures was carried out with reference to design codes, available handbooks and known practice at the time of construction. The seismic response of the designed prototype structures, subjected to various levels of ground-motion intensity, was estimated through nonlinear dynamic analyses with artificial and natural accelerograms, whilst the vulnerability was characterised through the use of the European Macroseismic Scale (EMS98), discussed above.

Rossetto and Elnashai (2005) constructed adaptive pushover curves of European buildings and applied the capacity spectrum methodology (see below) to obtain the performance point which was then correlated to a damage state through a damage scale calibrated to experimental data (Rossetto and Elnashai, 2003). This procedure was repeated using the acceleration-displacement spectra of many ground-motion records and the variability in the structural characteristics of the buildings was modelled using a response surface method leading to the derivation of analytical displacement-based vulnerability curves.

Dumova-Jovanoska (2004) produced vulnerability curves/damage probability matrices for reinforced concrete buildings built in the Skopje region. These earthquake damage-intensity relationships were derived by analytically modelling representative RC buildings and running dynamic nonlinear analysis with a set of 240 synthetic earthquake records. The damage to the structures was measured using the Park and Ang (1985) damage index and corresponding individual discrete damage states. A normal probabilistic distribution was assumed for the probability of occurrence of damage.

One of the principle disadvantages of the derivation of analytical vulnerability curves is that the procedure is extremely computationally intensive and time consuming and thus the curves cannot be easily developed for different areas or countries with diverse construction characteristics. However, analytical vulnerability curves have frequently been used to support rather than replace empirical DPMs and vulnerability curves based on observational damage data, leading to what are known as hybrid methods, as discussed in more detail in the next section.

3.2 Hybrid Methods

Hybrid damage probability matrices and vulnerability functions combine post-earthquake damage statistics with simulated, analytical damage statistics from a mathematical model of the building typology under consideration, as has been described in the previous section. Hybrid models can be particularly advantageous when there is a lack of damage data at certain intensity levels for the geographical area under consideration and they also allow calibration of the analytical model to be carried out. Furthermore, the use of observational data reduces the computational effort that would be required to produce a complete set of analytical vulnerability curves of DPMs.

Kappos et al. (1996, 1998) have derived damage probability matrices using a hybrid procedure in which parts of the DPMs for each intensity level were constructed using the available data from past earthquakes following the vulnerability index procedure discussed above. The remaining parts of the DPMs were constructed using the results of nonlinear dynamic analysis of models that simulated the behaviour of each building class. The time-history records were scaled to PGA values estimated by seismic hazard analysis; intensity and PGA were correlated using empirical relationships. A global damage index was derived to correlate the structural response from the dynamic analysis (ductility factors, displacements etc.) to loss, expressed in terms of cost of repair. A total of 120 analyses of typical Greek buildings designed to the 1959 code were run (6 structures, 10 ground motions and 2 intensities), and the statistical damage results were combined with the observed damage from the 1978 earthquake in Thessaloniki.

Barbat et al. (1996) used the Italian vulnerability index methodology for a hybrid vulnerability assessment of Spanish urban areas. A post-earthquake study was initially performed for two earthquakes with a maximum intensity of VII on the MSK scale. The structural and non-structural damage to masonry structures was analysed and correlated to the vulnerability and damage indices used in the Italian methodology. Statistical analyses were then performed to obtain the vulnerability function for intensity level MSK VII. A computer simulation process was subsequently used to obtain the vulnerability functions at other intensity levels.

The main difficulty in the use of hybrid methods is probably related to the calibration of the analytical results, considering that the two vulnerability curves include different sources of uncertainty and are thus not directly comparable. In the analytical curves the sources of uncertainty are clearly defined during the generation of the curves whilst the specific sources and levels of variability in the empirical data are not quantifiable. The method used to calibrate the analytical vulnerability curves using empirical data should depend on whether the aim is to include the additional uncertainty present in the empirical data which is not accounted for in the analytical data or whether the aim is in fact to improve the analytical model used to define the capacity of the building stock. If the principle goal is in fact the latter, then perhaps it would be preferable to calibrate the vulnerability curves considering the median (50 percentile) values and an adaptation of the analytical model could be made such that the two median values coincide. In this way, the observational data is used to calibrate the analytical model, but analytically-derived vulnerability curves with their known and specified sources of uncertainty are used in the loss model. This can be of use when only the uncertainty related to the capacity of the building stock is required and not that related to the ground motion; for example such curves are needed when a probabilistic seismic hazard assessment is used to define the ground motion as this will ensure that the latter variability is not double-counted (see for e.g. Bommer and Crowley, 2005).

3.3 Collapse Mechanism-Based Methods

Many recent proposals for analytical vulnerability assessment methods use collapse multipliers calculated from mechanical concepts to ascertain whether a mechanism will form and thus damage will occur; these procedures have been firstly applied by Bernardini et al. (1990), who proposed VULNUS, a method developed for the vulnerability assessment of unreinforced masonry buildings (URMB) using fuzzy-set theory and the definition of collapse multipliers.

FaMIVE, Failure Mechanism Identification and Vulnerability Evaluation method (D'Ayala and Speranza, 2002) is another procedure based on collapse multipliers, which is aimed towards the vulnerability assessment of historic buildings and town centres. The most probable collapse mechanism is found for both in-plane and out-of-plane failure by calculating the load factor or collapse multiplier through an equivalent static procedure; a number of possible out-of-plane collapse mechanisms are assumed, the equivalent shear capacity is calculated for each façade wall of the buildings under consideration for each collapse mechanism, and the most likely mechanism is identified as that with the lowest capacity.

Cosenza et al. (2005) present a mechanics-based approach for the assessment of reinforced concrete buildings that is also based on the formation of collapse mechanisms. First, the seismic capacity of a generic building model is defined, the assumed pre-defined mechanisms are established and the corresponding base shear is calculated assuming a linear distribution of horizontal seismic forces, and the ultimate roof displacement is determined as a function of the ultimate rotation of the structural elements. The seismic capacity of building stock will vary from the generic building due to different morphologic/geometric/structural configurations and material properties. Hence a number of building models are generated based on the probabilistic distribution of the structural parameters. A Monte Carlo simulation technique is applied to calculate "probabilistic capacity curves" which express the probability of having a capacity lower than the assigned value: this can be likened to the percentage of buildings with a value of capacity lower than a given threshold value.

A clear framework for the treatment of uncertainty in the geometric and material properties in an urban environment is accounted for in the method by Cosenza et al. (2005), whilst this is not the case for the VULNUS and FaMIVE procedures. However, the main disadvantage of the procedure by Cosenza et al. (2005), and also FaMIVE, is that no clear indication is given on how the capacity is to be convolved with the demand to calculate the probability of exceeding given limit states. The VULNUS procedure does allow the estimation of the probability of damage, but only for one limit state (the collapse limit state). Thus, the use of these collapse-mechanism methods within a loss assessment model appears to be somewhat limited at present.

3.4 Capacity Spectrum-Based

HAZUS (Hazard US) is the outcome of a project conducted for the National Institute of Building Science (NIBS), under a cooperative agreement with the Federal Emergency Management Agency (FEMA), to develop a nationally applicable methodology for estimating the potential losses from earthquakes on a regional basis (Whitman et al., 1997; FEMA, 1999, 2003). The vulnerability assessment component of the procedure is contained within the direct physical damage module and is based on the Capacity Spectrum Method of ATC-40 (ATC, 1996) wherein the performance point of a building type under a particular ground-shaking scenario (or PESH) is found from the intersection of an acceleration-displacement spectrum, representing the ground motion, and a capacity spectrum (pushover curve), representing the horizontal displacement of the structure under increasing lateral load (Kircher et al., 1997). A potential weakness of the method is that the capacity curves and vulnerability functions published in the HAZUS manual have been derived for buildings in the US having a limited range of storey heights, thus the application of this method to other parts of the world requires additional research to be carried out. Hence, capacity curves and vulnerability functions would need to be derived for the building stock under consideration; however, there is difficulty involved in obtaining a physically realistic representation of the inelastic response of the structure using pushover analysis. Although this aspect can be somewhat improved using displacement-based adaptive pushover techniques (Antoniou and Pinho, 2004), a faithful representation of the real structural behaviour requires a great deal of information about the structure, including reinforcement details, which are unlikely to be well known for a large building stock.

HAZUS has been adopted all over the world for the loss assessment of urban areas. The methodology in itself has not been adapted in any way, but the capacity curves and fragility functions have been calibrated to the building stock under consideration. Examples include the loss assessment of Turkey carried out by Bommer et al. (2002), the seismic risk assessment of Oslo documented by Molina and Lindholm (2005), the loss estimation of Taiwan (using a modified version of HAZUS called Haz-Taiwan) discussed in Yeh et al. (2000) and the RISK_UE project for the risk assessment of 7 European towns that has been discussed previously (see www.risk-ue.net).

The LNECloss tool is an automatic seismic scenario loss estimation methodology that makes use of the capacity spectrum method and has been implemented in a GIS environment (Sousa et al., 2004). This methodology is being developed and implemented within the LESSLOSS Integrated Project (Calvi and Pinho, 2004) under the earthquake disaster scenario predictions and loss modelling for urban areas sub-project, and is being applied to the city of Lisbon (see www.lessloss.org/main).

Giovinazzi (2005) presents a procedure that uses simplified bilinear capacity spectra (capacity or pushover curves converted to plots of spectral acceleration versus spectral displacement) which are derived using the equations and parameters available in seismic design codes. The base shear coefficient (which can be related to the yield spectral acceleration) is generally a function of seismic zone, soil conditions, building dynamic response, structural type and building importance, all of which can be obtained from the code. The yield spectral displacement is a function of the yield spectral acceleration and the yield period of vibration; the latter being calculated using the simple formulae available in seismic design codes which usually give the period as a function of the building height. The use of seismic design codes to calculate simplified pushover curves of the building stock is a cost-effective method for calibrating the vulnerability of buildings designed to different design codes. However, code-based definitions of the yield period of vibration and base shear capacity of RC structures could differ greatly from the actual properties of the building stock (see for e.g. Crowley, 2003) and in many countries the existing building stock has been constructed without even conforming to any design code. The modelling of the variability in the estimation of the damage suffered by the buildings (which is due to the uncertainty in the capacity curve, the damage state definition and the seismic demand spectra), needs also to be further verified and validated, given that the employment of a binomial distribution to fit damage data (e.g. Braga et al., 1982; Spence et al., 2003) may lead to unreliable results since the variation of the scatter around the mean damage band is not catered for.

4 Displacement-Based Vulnerability Assessment

The first steps towards the development of a conceptually sound, but still simple to apply and calibrate, seismic vulnerability assessment framework, can be found in the work by Calvi (1999). This paper proposed a holistic methodology that used displacements as the fundamental indicator of damage and a spectral representation of the earthquake demand. The procedure utilised the principles of the Direct Displacement-Based Design method (e.g. Priestley, 2003), wherein a multi-degree-of-freedom (MDOF) structure is modelled as a single DOF system (Fig. 1) and different displacement profiles are accounted for according to the failure mechanism or displacement profile at a given limit state, using the geometric and material properties of the structures within a building class. For reinforced concrete frames, the displacement capacity of column-sway (soft-storey) and beam-sway (distributed damage) failure mechanisms are considered (see Fig. 2) whilst for masonry structures different in-plane failure modes have been identified (see Fig. 3). The approach is particularly suitable for loss estimation studies, since, in addition to the direct customisation to any given building stock characteristics, it is very computational efficient thus allowing extensive and repetitive parametric studies to be carried out in a cost-effective manner.

Calvi (1999) considered the inherent variability in the structural properties within an urban environment by assigning maxima and minima to the variables and assuming a uniform probability distribution function. The period of vibration was calculated using the empirical formula in EC8 (CEN, 2003) which directly relates the height of a building to its period; again maxima and minima were applied to the parameters in this equation. The range of the limit state displacement capacity for the building class can be plotted against the range of the period of vibration, as presented in Fig. 4. This capacity area can then be directly compared with the displacement response spectrum; the area below the spectrum represents the proportion of buildings failing, or exceeding, the limit state. The displacement response spectra were adjusted to include nonlinear response, wherein a reduction of the spectral ordinates was applied to account

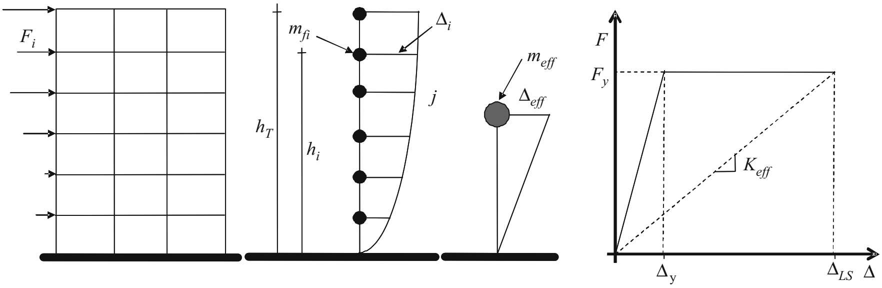

Fig. 1 Simplified model for an equivalent SDOF system

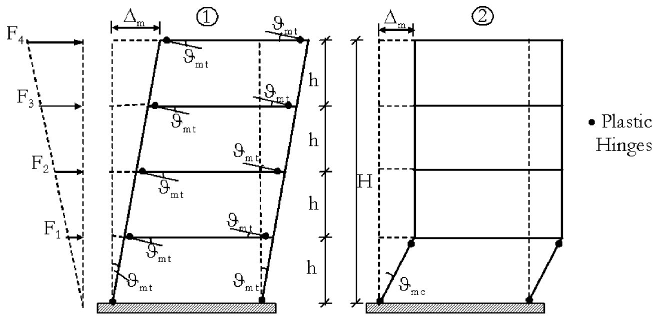

Fig. 2 Distributed damage/beam-sway (*left*) and soft-storey/column-sway (*right*) response mechanisms

for the energy dissipation capacity of the structure as a function of the target displacement and the structural response.

The methodology proposed by Calvi (1999) has subsequently been developed for reinforced concrete buildings by Pinho et al. (2002) and Crowley et al. (2004, 2006), leading to the Displacement-Based Earthquake Loss Assessment (DBELA) procedure. For the masonry component of the methodology, a parallel extensive development was carried out by Restrepo-Velez and Magenes (2004; 2005a,b), leading to the procedure MeBaSe. A brief description of the main components of both MeBaSe and DBELA is provided in what follows.

Three limit states are considered in MeBaSe for the case of in-plane failure mechanisms, namely *LS1*-*LS2* for which just slight structural and non-structural damages occur, *LS3* for which moderate structural damage and extensive non-structural damage occurs, and *LS4* for which the collapse of the building is considered (Fig. 3). Equations (1) and (2) are used to compute the limit state functions for the case of in-plane failure mechanisms, where the vector of random variables can be formed by the total height of the building class h_t, the height of the openings at the failing storey h_{sp}, the drift limit at yield δ_y, the drift limit at the specified limit state δ_{LS}, the resistant area of the walls in the direction of minimum strength A_m, the ratio of areas of walls in both directions

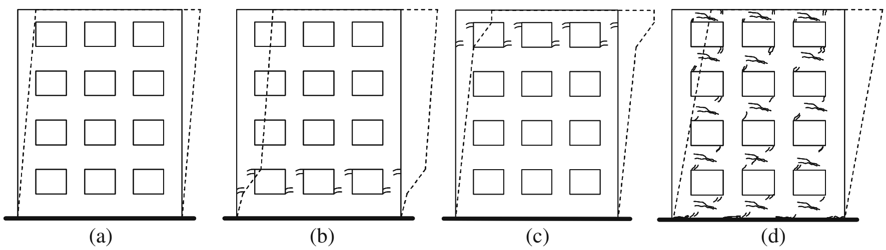

Fig. 3 Deformed shapes for different limit states and in-plane failure modes

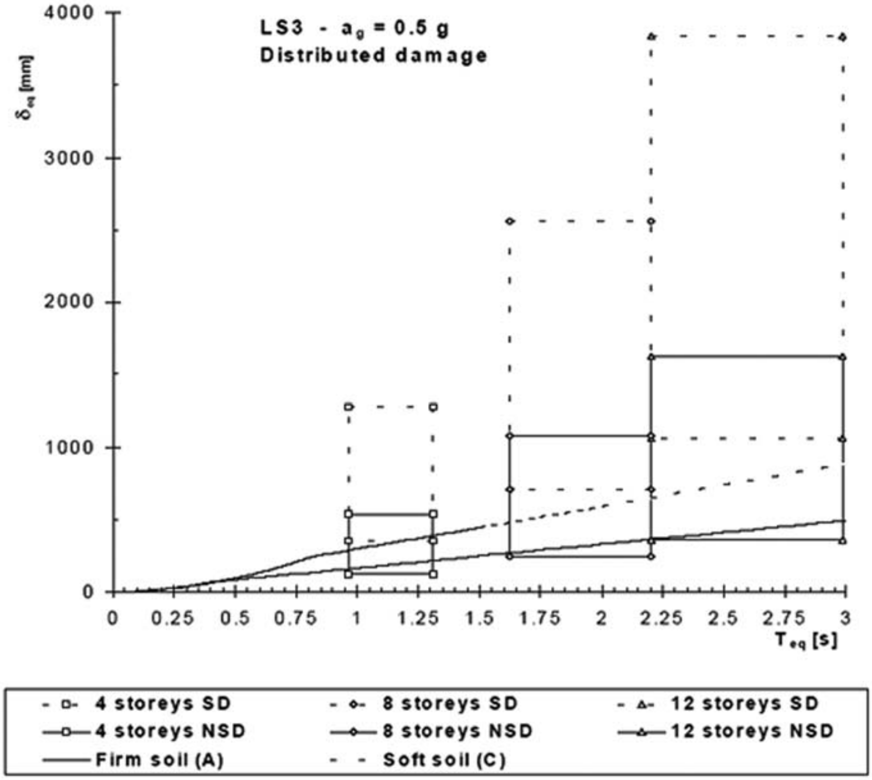

Fig. 4 Example of intersection of capacity areas and demand spectrum (Calvi, 1999)

γ_m, the referential shear strength of the masonry τ_{km}, and the correction factor ϕ_c. This correction factor allows to express the tri-dimensional response of a building by means of a simplified bi-dimensional model, based solely on the shear strength of the walls, and it is computed with Equation (3), where L_W is the length of the piers and L_T is the total length of the perimeter walls.

$$T_{LS} = 2\pi\sqrt{\frac{\kappa_1 h_T \delta_y + \kappa_2(\delta_{LS} - \delta_y)h_{sp}}{\frac{1}{\phi_c} A_m K_1 \tau_{km}\left(1 + \frac{K_2}{A_m(1+\gamma_m)\tau_{km}}\right)^{1/2} g}} \tag{1}$$

$$\Delta_{LS} = \frac{T_{LS}^2}{4\pi^2}\frac{1}{\phi_c} A_m K_1 \tau_{km}\left(1 + \frac{K_2}{A_m(1+\gamma_m)\tau_{km}}\right)^{1/2} g \tag{2}$$

$$\phi_c = 5.53\frac{\tau_{km}}{(L_W/L_T)} + 0.46 \tag{3}$$

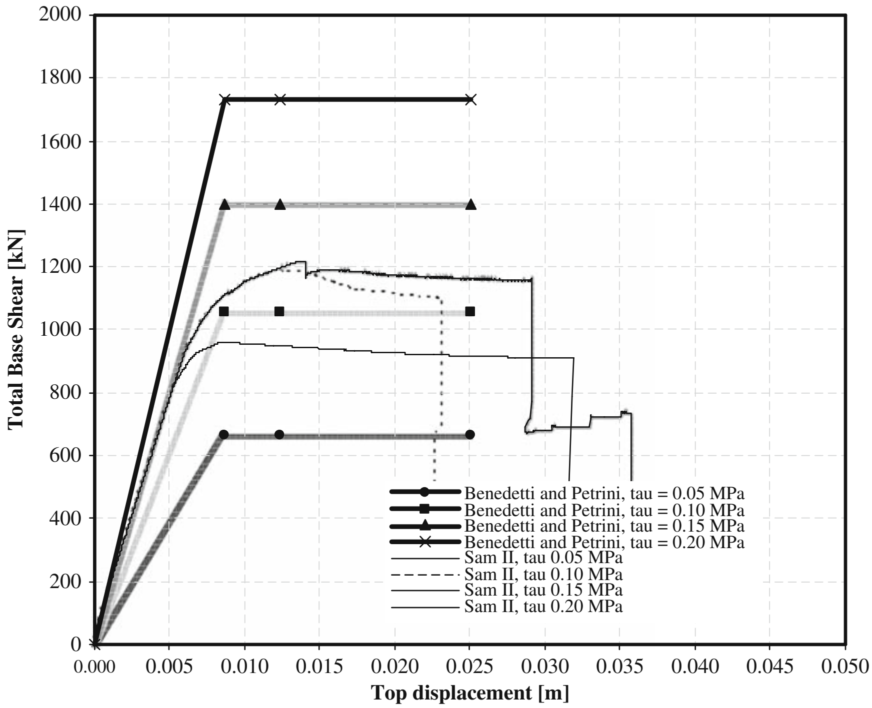

Fig. 5 Pushover results for a four-storey building with the simplified and three-dimensional methodologies, and for different referential shear strength values

To define ϕ_c (Equation 3), different building configurations, from two to five storeys, were analysed with this simplified approach and by using the three-dimensional nonlinear computer code SAM (Magenes, 2000). The buildings were selected trying to cover a realistic range of number of storeys, structural configurations and lateral strengths. As an example of the analyses, Fig. 5 shows the pushover results for a four-storey building obtained with the simplified and the three-dimensional methodologies, and for different values of τ_{km}. In Equations (1) and (2) κ_1 and κ_2 are used to compute the fraction of elastic and inelastic components of the displacements, according to the interstorey heights and the distribution of masses (see Restrepo-Vélez and Magenes, 2005b). Out-of-plane mechanisms, which were not considered by Calvi (1999), have been included in MeBaSe by considering one-way and two-way bending mechanisms, whose dynamic response has been described by Doherty et al. (2002) and Griffith et al. (2003) by means of a tri-linear model.

The main improvements that have been applied to the method by Calvi (1999) for reinforced concrete structures include theoretically improved structural and non-structural displacement capacity equations for ground shaking, the derivation of an equation between yield period and height for European buildings both with and without infill panels (Crowley and Pinho, 2004, 2006) and the consideration of the vulnerability to liquefaction-induced ground deformations (Bird et al., 2005, 2006). DBELA considers three limit states based on the sectional strains in the steel and concrete, for the case of structural limit states, and on inter-storey drifts, in the case of non-structural limit states (see Crowley et al., 2004). The structural displacement capacity of reinforced concrete members/structures is derived from structural mechanics principles; beginning with the yield strain of the reinforcing steel and the geometry of the beam and column sections in the building class, the yield curvature can be defined using the relationships suggested by Priestley (2003). These beam and column yield curvatures are then multiplied by empirical coefficients to account for shear and joint deformation to obtain a formula for chord rotation. This chord rotation is equated to base rotation and multiplied by the height of the equivalent SDOF system to produce the yield displacement capacity. Post-yield displacement capacity formulae are derived by adding a plastic displacement component to the yield displacement, calculated by multiplying together the limit state plastic section curvature, the plastic hinge length, and the height or length of the yielding member.

When the damage due to liquefaction-induced ground deformations is to be predicted, the displacement capacity is compared directly with the displacement demand imposed on members within the structure. If the damage due to ground shaking is to be considered in the loss model, the displacement capacity equations, which are functions of the geometrical and material properties and the height of the building class, are transformed into functions of period through the substitution of an equation relating the height of a structure to its limit state period. A direct comparison is thus possible at any period between the displacement capacity of a building class and the displacement demand predicted from a response spectrum (Fig. 6). For example, the first structural and non-structural

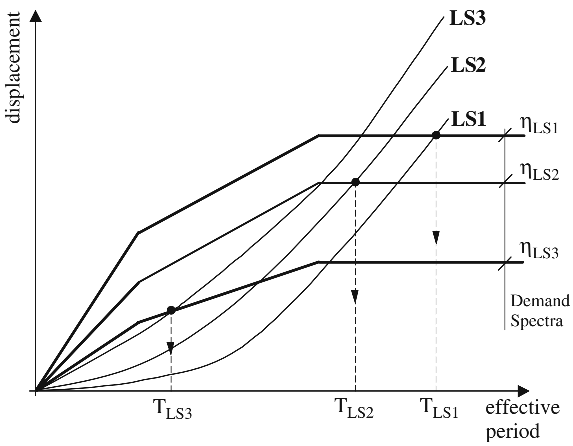

Fig. 6 An illustrative description of the deterministic comparison of limit state (LS) displacement capacity and displacement demand in DBELA

limit state capacities to ground shaking for bare beam-sway frames can be predicted using Equations (4) and (5), respectively:

$$\Delta_{Sy} = 5ef_h T_y \varepsilon_y \frac{l_b}{h_b} \tag{4}$$

$$\Delta_{NS1} = S\vartheta_1(10T_y) \tag{5}$$

where e_{fh} is the effective height coefficient, T_y is the yield period of vibration, ε_y is the yield strain of the reinforcing steel, l_b is the length of the beam, h_b is the depth of the beam, S is the shape coefficient relating the roof displacement to the displacement at the centre of seismic force, and ϑ is the inter-storey drift capacity of the non-structural elements.

One of the principle modifications to the methodology by Calvi (1999) that is common to both DBELA and MeBaSe is the consideration of a joint probability density function of displacement capacity and period (Fig. 7c) calculated from the variability of a vector of random variables present in the equations for displacement capacity, conditioned to period (Fig. 7a) and period of vibration (Fig. 7b); furthermore, the variability in the demand response spectrum is now also taken into consideration in the calculation of the probability of exceeding a given limit state.

As gathered already from the above, these displacement-based methodologies feature some characteristics that may be considered as ideal for seismic vulnerability assessment:

- use a continuous (e.g. PGD) rather than a discrete measure (e.g. intensity) of the earthquake demand, and use a more complete representation of the demand in the form of response spectra, rather than single parameters such as PGA

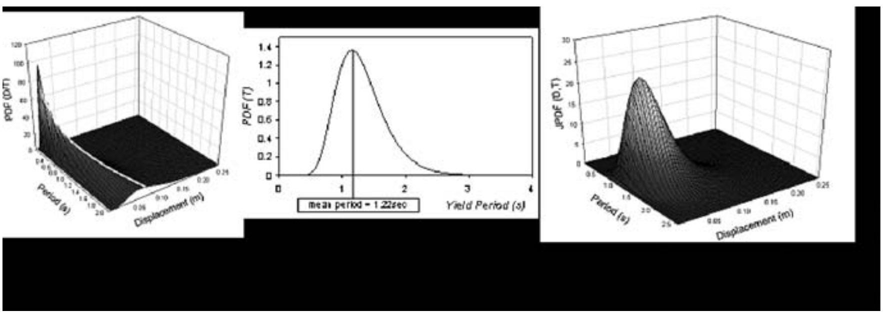

Fig. 7 (**a**) Illustrative probability density functions of yield displacement capacity, conditioned to period, (**b**) Illustrative probability density function of yield period for a building class due to variability in structural properties, (**c**) Example JPDF of capacity for a column-sway RC building class

- use displacement or deformation as an indicator of the demand level, which has a better correlation with the level of damage
- consider the uncertainty on the seismic demand, recognized to be one of the components of uncertainty that are required to be included in a complete seismic risk assessment
- consider the sources of uncertainty coming from capacity and response
- use mechanical criteria for the definition of the structural capacity
- include in the response the most common out-of-plane failure mechanisms (masonry structures only)
- include the foundation and non-structural components in the assessment
- may readily take into account the experience obtained from past earthquakes that have occurred in similar conditions, in order to improve the parameters and structural models used in the analysis
- have the possibility of evaluating the seismic vulnerability of a given class of buildings, considering different levels of quality and completeness in the data and in the refinement in modelling and analysis
- be of simple and fast application to real cases, avoiding to the maximum extent the use of expensive and time consuming computational tools
- avoid the employment of subjective definition of parameters, factors and relative weights
- require minimal adjustments for its applicability in different regions of the world.

5 Concluding Remarks

Traditional and innovative vulnerability assessment methods developed and used over the past 30 years have been presented herein. The review has not included all loss estimation procedures, since many are proprietary (see Bommer et al., 2006), and others are not easily classified into any of the categories considered herein (e.g. Ordaz et al., 2000), however it did provide an overview of the large majority of widely used methods for which clear insight into their inner workings exists.

Such review evidenced that an optimal or ideal vulnerability assessment methodology should feature the following main characteristics (i) the most recent developments in the field of seismic hazard assessment should be incorporated, (ii) all sources of uncertainty should be explicitly accounted for, (iii) the model should be easily adaptable to the different construction practices around the world, as well as allow for the inclusion of new construction types and the influence of retrofitting on the response of existing structures, (iv) a balance should be struck between the computational intensity and the amount of detailed data which is required and the consequent degree of confidence in the results.

It is unlikely that a single methodology can be produced which fulfils all of these requirements. For example, many analytical/mechanical models, though theoretically superior, require a large amount of detailed data, but the benefit of collecting such data is often not proven through validation of the methodology with empirical methods based on observed damage data. On the other hand, the derivation of vulnerability curves from observed data does not always consider the frequency characteristics of the building stock, and the influence of incorrectly modelling the seismic demand experienced by the buildings in the damaged region is normally unaccounted for. Furthermore, many methodologies do not explicitly model the various sources of uncertainty; this can be a problem when, for example, the uncertainty in the seismic demand needs to be removed from the vulnerability assessment calculations.

Nonetheless, it is believed that the recently proposed displacement-based methodologies for seismic vulnerability assessment do provides a significant improvement to traditional practice, as explicitly noted in the previous Section. Indeed, the equations that displacement capacity of members and structures to their geometrical and material properties, derived using the basic principles of structural mechanics, are found to be particularly suitable for loss assessment models due to their transparency, theoretical accuracy and computational efficiency. Whereas other loss methodologies require a great deal of effort to adapt their capacity and vulnerability functions to different building stock characteristics, with the proposed approach this can be easily and explicitly carried out through appropriate calibration of the mean, variance and probabilistic distributions of the geometrical and material properties in the capacity equations.

Notwithstanding its work-in-progress status, these displacement-based methods have already been applied in the assessment of the seismic vulnerability of entire regions or countries (e.g. Restrepo-Velez and Magenes, 2004; Iaccino, 2004; Faravelli, 2006) and in the carrying out of important sensitivity studies for definition of priorities in terms of data collection needs (e.g. Crowley et al., 2005) and also for evaluation of different approaches to the definition of the seismic hazard (e.g. Bommer and Crowley, 2005; Crowley and Bommer, 2005). Further, the method has also been proposed as a possible tool for calibration of future seismic design codes (e.g. Bommer et al., 2005). Currently on-going developments aim at including the effect of duration of the demand on the response of structures, the influence of infill panels on the displacement capacity of reinforced concrete frames, the inclusion of model uncertainty due to the simplifying assumptions that have been made in the modelling of RC buildings response, the prediction of acceleration-sensitive non-structural damage, and improvements to the displacement spectra predictions through the consideration of long-period ordinates, near-fault effects etc. Validation, and perhaps even calibration of these displacement-based methods using observed damage data from recent earthquakes, and considering also the outcomes of other analytical procedures (Borzi et al., 2007), is equally underway.

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Earthquake Early Warning: Real-time Prediction of Ground Motion from the First Seconds of Seismic Recordings

Maren Böse

1 Introduction to Earthquake Early Warning

The underlying idea behind earthquake early warning (EEW) is that seismic waves (stimulated by the earthquake) propagate at a lower speed than electromagnetic waves that are used to transmit possible warnings in case of strong events. This leading time can be used to trigger and execute automatisms to reduce likely damage that might be caused by the later arriving seismic waves (Fig. 1).

Substantial progress in seismic real-time acquisition and communication technologies aside from important developments and enhancements of seismic processing software pave the way for the design and implementation of EEW systems all over the world. Inhibited, however, by the high claim of reliability, there are only a few EEW systems in operation now, including systems in Japan (Nakamura, 1989), Taiwan (Wu and Teng, 2002; Wu and Kanamori, 2005a,b), and Mexico (Espinosa-Aranda et al., 1995). In other countries, such as California (Allen and Kanamori, 2003; Cua and Heaton, 2003; Kanamori, 2005), Romania (Wenzel et al., 1999; Böse et al., 2007), or Turkey (Erdik et al., 2003b; Böse, 2006) systems are under way.

Possible warning times are usually in the range of a few seconds to about one minute, depending on respective distances between seismic source, sensor and user sites. The maximum warning time that an EEW system can achieve is defined by the time difference between the detection of the faster P-wave of the seismic wavefield by appropriate ground motion sensors and the arrival of slower traveling high amplitude S- or surface waves at the user site (Fig. 2). In consideration of this extremely short time window EEW systems are most effective if used for triggering and execution of automatisms to prepare vulnerable systems and dangerous processes for the imminent danger. Seismic warnings can, e.g., be used to automatically slow down high-speed trains to avoid

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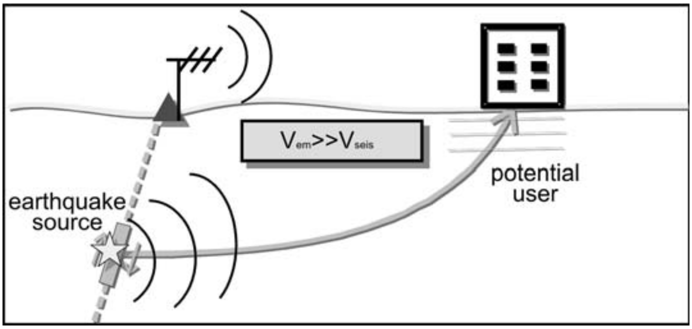

Fig. 1 Principle idea behind earthquake early warning: a seismic sensor (*triangle*) in the epicentral area detects seismic waves that have been stimulated by the earthquake source. Using electromagnetic waves that travel at a higher speed than seismic waves a warning is issued to a potential user before strong shaking at this site will occur

derailments, to shutdown pipelines and gas lines to minimize fire hazards, to shutdown manufacturing operations to decrease potential damage to equipment, or to save vital computer information to inhibit losses of data. A list of possible response strategies to warnings has been compiled by Goltz (2002).

EEW have to comply with primary two requirements: they have to be very fast and extremely reliable. The two main types of EEW systems, *regional* and *on-site* warning systems, meet these claims differently well. Regional warning systems use traditional sensor networks with real-time capability; seismic parameters are determined through application of common seismic methods. While regional warning systems mainly focus on potential users at epicentral distances of some hundreds of kilometers, on-site warning systems are supposed to serve a broader user community on local scale. On-site systems save warning times by decentralized processing and restriction on information derived from single stations instead of station networks; generally, only the initial parts of the seismic signals – the P-waves – are analyzed (Kanamori, 2005), – true to the motto: the P-wave carries information on the earthquake, the S-wave its energy (Fig. 2).

Olson and Allen (2005) and Allen and Kanamori (2003) suggest that the earthquake process may be deterministic and magnitudes therewith predictable

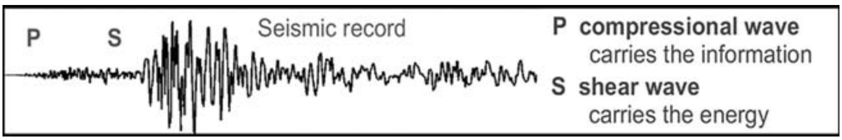

Fig. 2 Example of a seismic record illustrating the faster traveling P-wave (which carries information on the earthquake) and the slower high amplitude S-wave (which carries the energy of the earthquake)

from the faster P-wave. This assumption, however, has been heavily disputed in the seismological community. Rydelek and Horiuchi (2006), for instance, concluded from their study of 52 strong earthquakes ($6.0 \leq M \leq 8.0$) recorded by the Japanese Hi-net seismic network that magnitudes cannot be determined until ruptures are completed. Only for smaller events with $M_w \leq 6.0$, for which the first few seconds contain almost the entire rupture history, high correlations between parameters determined from the first few seconds of the P-wave and earthquake magnitudes can be expected and are actually observed (Rydelek and Horiuchi 2006).

In practice, on-site warning approaches based on single station observations, such as measurements of the predominant period as indicator for earthquake magnitudes (Nakamura, 1989; Allen and Kanamori, 2003; Olson and Allen, 2005), are quick, however, less robust than traditional regional warning methods that are based on station networks. High scatter in estimated magnitudes usually requires averaging over predictions at several on-site warning sensors (Wu and Kanamori, 2005b; Lockman and Allen, 2005), which is clearly in contradiction to the initial idea behind on-site warning.

Using the example of Istanbul, Böse (2006) developed a methodology for EEW – called *PreSEIS* (Pre-SEISmic shaking) – that takes advantage of both, regional and on-site warning paradigms. In contrast to the afore described on-site method, *PreSEIS* requires no assumptions on rupture determinism; it integrates the entire rupture history and determines the most likely source parameters at each time step after triggering of the first sensor using each the available information on ground shaking at the different sensors. Hence *PreSEIS* is independent from largest fault slips occurring at rupture initiation or at later time steps. The principle idea behind *PreSEIS* is illustrated in Fig. 3. As

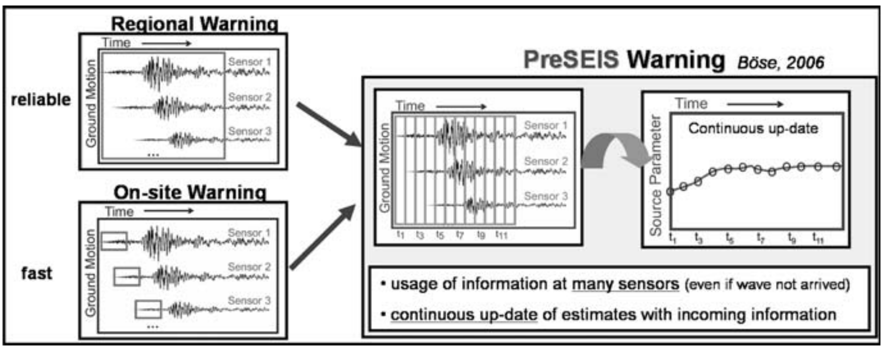

Fig. 3 PreSEIS (Böse, 2006) combines regional and on-site EEW with each other: it uses the available data at many sensors at each time step after triggering. From this information PreSEIS determines the most likely source parameters of the earthquake, including earthquake magnitudes, hypocenter locations, and rupture expansions. This approach leads to a fast and reliable algorithm for EEW

will be demonstrated in Section 2.1, this approach leads to a reliable and fast algorithm for EEW.

1.1 Seismic Hazard in the Marmara Region

Istanbul and the Marmara Region face a huge seismic risk: a historic earthquake catalogue for the Marmara Region over the last 2,000 years (Ambraseys, 2002) reveals that on average at least one medium intensity (VII–VIII) earthquake has hit Istanbul every 50 years; the average return period for high intensity (VIII–IX) events is 300 years. From previous earthquakes in the Sea of Marmara and the stress transfer of the 17 August 1999 Izmit earthquake ($M_w = 7.4$) Parsons (2004) determines a $53 \pm 18\%$ probability of strong shaking in Istanbul metropolitan area during the next 30 years. Erdik et al. (2003a) estimate that in case of a $M_w = 7.5$ earthquake in the Sea of Marmara destructions in Istanbul might amount to about USD 11 billion losses, with 40,000–50,000 deaths, and between 430,000 and 600,000 destroyed households.

The devastating earthquakes of Kocaeli and Düzce in 1999 have pushed the discussions about earthquake hazard assessment and possible strategies for seismic risk reduction in Istanbul metropolitan area and the Marmara Region in general. One outcome of these discussions was the common agreement on the development of key components of a real-time earthquake information system and implementation in the Istanbul Earthquake Rapid Response and Early Warning System (IERREWS). Details on IERREWS are given by Erdik et al. (2003b). The EEW system comprises ten tri-axial strong motion sensors that were deployed along the coast of the Sea of Marmara, a real-time communication radio- and satellite-link, and a central processing facility at the Kandilli-Observatory of the Bogazici University in Istanbul.

The tectonic background in the Marmara Region poses a big challenge for the realization of an EEW system: since the determination of earthquake hypocenters and magnitudes in the short time span between seismic detection at the EEW sensors and the arrival of high amplitude waves in Istanbul in a classical sense is impossible, impacts of earthquakes have to be estimated from the first seconds of seismic waves that have already arrived at the EEW stations. Against this background we will test the *PreSEIS* approach for EEW proposed by Böse (2006).

Aside from the tectonic situation in the Marmara Region we face another difficulty: even as the North Anatolian Fault is seismically rather active, the majority of recent earthquakes have occurred in the middle and eastern part of the 1,500 km long fault. Seismicity in the Marmara Region, on the other hand, is relatively low, – a fact that aggravates the required tuning of the EEW system. Simulations of ground shaking time series calibrated with observed earthquakes were used by Böse (2006) to overcome the lack of ground motion data in the Marmara Region.

2 Simulations of Seismic Ground Motion

To overcome the lack of moderate and strong motion data in the Marmara Region, Böse (2006) produced a database of 280 simulated time series of ground shaking by application of the Stochastic Simulation Method for Finite Faults (SSMFF; Beresnev and Atkinson, 1997, 1998). The SSMFF provides a simple and suitable technique for the simulation of seismic ground motions for moderate and strong earthquakes covering the full band of desired frequencies; its power is that detailed specifications of earthquake sources and propagation effects of seismic waves are not required. In contrast to the Stochastic Point Source Approach (Boore, 1983) the SSMFF allows for the inclusion of source dimensions. This extension is strongly required for simulations in the Marmara Region because as a consequence of short source-to-site distances source finiteness must be considered.

Seismic ground motion observed at a given site $\mathbf{x}$ is controlled by mainly three factors: the earthquake source S, effects of seismic wave propagation –such as geometrical spreading G and inelastic attenuation I–, and local site effects –such as high-frequency diminution H and site amplification F. The principle idea behind the stochastic simulation approach is the combination of the estimated Fourier amplitude spectrum of ground motion $|A(\mathbf{x},\omega)|$ at site $\mathbf{x}$ given by the multiplication

$$|A(\mathbf{x},\omega)| = \omega^2 C_S M_0 \left[1 + \left(\frac{\omega}{\omega_c}\right)^2\right]^{-1} \cdot I\,(\mathbf{x},\omega) \cdot G(\mathbf{x}) \cdot F(\mathbf{x},\omega) \cdot H(\mathbf{x},\omega) \quad (1)$$

with a random phase obtained from a time series of windowed Gaussian noise of finite duration (Boore, 1983).

The SSMFF considers the seismic rupture as a system composed of numerous point sources, each spectrally characterized by Equation (1). The simulated acceleration time series are obtained from summation of contributions of all subfault elements, each with a certain time delay due to rupture propagation (Beresnev and Atkinson, 1997). The used FINSIM code published by Beresnev and Atkinson (1998) is in the first place only valid for seismic S-waves. Böse (2006), however, developed a modified code that allows also for the simulation of P-waves.

Using a set of input parameters from the literature (e.g., Horasan et al., 1998; Pulido et al., 2004) Böse (2006) generated a database of synthetic ground motion records at the ten on-line stations of the Istanbul earthquake early warning system within IERREWS (Erdik et al., 2003b) for 280 earthquake scenarios in the Marmara Region with $4.5 \leq M_w \leq 7.5$. Source spectra for finite faults are highly sensitive to seismic P- and S-wave speed (α, β), as well as to maximum slip and rupture velocities (v_{max} and v_r). Böse (2006) used a simple homogeneous velocity model with $\alpha = 5.7$ km/s, $\beta = 3.3$ km/s, and $v_r = 0.8\,\beta$. Source parameters, such as v_{max}, were kept variable within physically

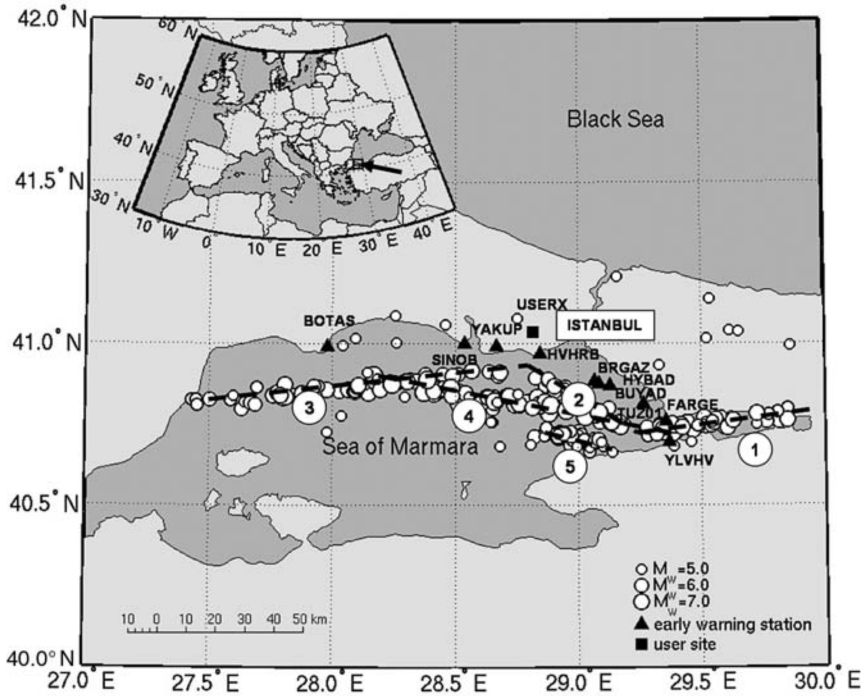

Fig. 4 Distribution of epicenters of stochastically simulated earthquakes for finite faults in the Marmara Region (Böse, 2006). Triangles mark the locations of sensors of the Istanbul earthquake early warning system within IERREWS (Erdik et al., 2003b)

reasonable limits. Bi- and unidirectional ruptures were modeled. For the integration of site effects Böse (2006) made use of mean amplification spectra for different soil types which were empirically determined by Boore and Joyner (1997). Due to varied source parameters for fixed magnitudes and distances, amplitudes of the simulated ground motions show variabilities in the order of a factor two. Figure 4 illustrates the distribution of epicenters of all simulated earthquakes.

2.1 PreSEIS – A New Approach to Earthquake Early Warning

PreSEIS (Pre-SEISmic shaking) developed by Böse (2006) is a method for the inversion of seismic source parameters from time-dependent ground motion data at sensors within a local seismic network (such as IERREWS). Only 0.5 s after P-wave detection at the first sensor, *PreSEIS* will issue first estimates of magnitudes and hypocenter locations. We can thus consider *PreSEIS* as a new approach to EEW that takes advantage of both, regional and on-site warning paradigms: *PreSEIS* shows a similar reliability like regional warning (as it uses station networks) and is at the same time as fast as on-site warning (as it requires the arrival of seismic waves at only one station). Within the primary seconds after rupture initiation the P-wave will have generally arrived at a minor subset of sensors, implying that the inversion problem will be ill posed. Significant improvement of estimates can be achieved through the integration of information on non-triggered sensors which will allow confining the space of possible solutions (Rydelek and Pujol, 2004; Horiuchi et al., 2005). With ongoing time longer time series of ground motion at more and more sensors will become available and will allow inferring further information on the earthquake. Based on the respectively available data, *PreSEIS* will up-date estimates of source parameters continuously every half second (Fig. 3).

For the inversion of seismic source parameters from seismic observations, *PreSEIS* makes use of Artificial Neural Networks (ANNs). Due to their massively parallel structure with high numbers of simple interconnected processing units – the so-called neurons – ANNs are suitable functions for complex nonlinear input-output mappings (Bishop, 1995). The importance of a link between one neuron to another is controlled by a weight parameter. The weight parameters of an ANN are iteratively adapted to the inversion problem by learning from a set of example patterns. Once learning is accomplished, ANNs are capable to process unknown data that follows the same statistical process like the training examples. ANNs are thus independent from formulations of explicit algorithms, and, in addition, show a high tolerance of noisy input data (Bishop, 1995).

The synthetic database described in Sect. 2 is repeatedly split at random into one training (70%), one validation (10%), and one test subset (20%). Validation and test subsets are used for the evaluation of network performances during and after training. The training itself is based on the training subset (Fig. 5, left).

For the *PreSEIS* approach Böse (2006) mimicked the seismic wave propagation for the synthetic data and trained the ANNs on the data that will be available at the ten early warning sensors of IERREWS at time steps between 0.5 s and 15.0 s after triggering of the first sensor in intervals of 0.5 s. To avoid over-fitting to the training data, learning is terminated once the errors (here the mean squared error MSE) for the independent validation subset increase (Fig. 5, right). This procedure is called "early stopping".

Figure 6 illustrates the performance of *PreSEIS* for training and test earthquakes in the database as a function of time after triggering of the first sensor. Reliabilities of predicted hypocenter locations and moment magnitudes show a clear increase with proceeding time, underlining the clear trade-off between accuracy of predicted parameters on the one hand and remaining warning time on the other hand. The median of location errors (the 50th percentile) goes down from 8.8 km at 0.5 s after triggering to 5.4 km during the subsequent 15 s interval (Fig. 6, top). This high accuracy results primarily from the fact, that the ANNs have learned the *a priori* information from the training subset that

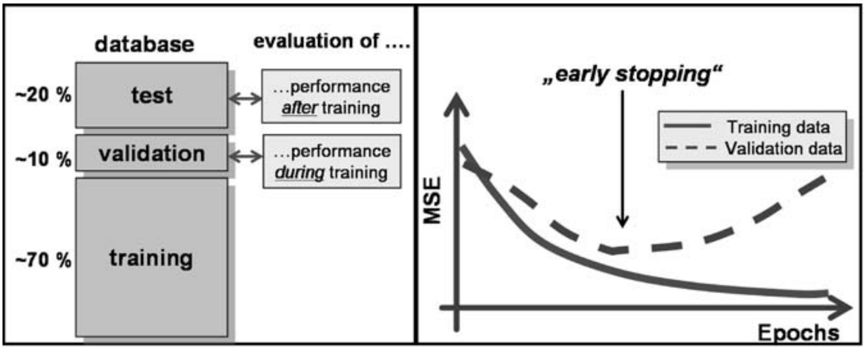

Fig. 5 The database of stochastically simulated ground motions (Section 2.1) is repeatedly split at random in training, test and validation subsets (*left*). Training of the ANNs is stopped once the mean squared error (MSE) for the independent validation subset increases (*right*). This "early stopping" procedure prevents that the statistical model provided through the ANNs will become over-fitted to the training data – at the expense of their generalization capability

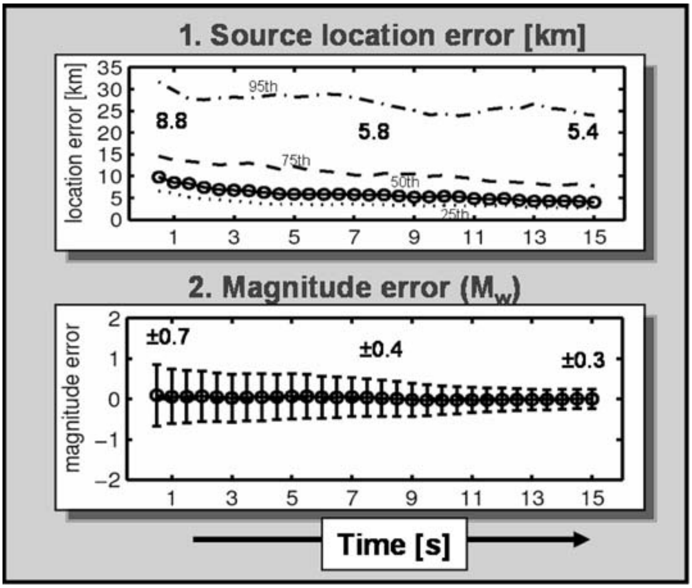

Fig. 6 Results of the PreSEIS approach. *Top*: 25th, 50th, 75th and 95th percentiles of error distributions for the determined hypocenter locations. *Bottom*: Mean and unit standard deviation of magnitude errors. There is a clear decrease of errors for predicted source parameters with proceeding time, underlining the trade-off between accuracy of predictions on the one hand, and remaining warning time on the other hand

earthquakes usually cluster along the major fault segments in the Sea of Marmara. Higher errors are mainly linked to earthquakes that occur beyond the fault segments or at the border of the sensor network. The mean magnitude error is at all time steps nearly zero, with the unit standard deviation of errors decreasing from 0.7 magnitude units at 0.5 s after triggering of the first sensor to 0.3 magnitude units after 15 s (Fig. 6, bottom).

3 Conclusions and Outlook

Böse (2006) developed a method for EEW that combines ground motion measurements at several seismic stations within a local sensor network for real-time estimates of earthquake source parameters. *PreSEIS* does not require that seismic waves have arrived at all sensors before estimates are issued. This makes the approach very fast and suitable for EEW. Unlike other methods for EEW – such as based on the predominant period determined of the initial part of seismic records as indicator for earthquake magnitude (Nakamura, 1989; Allen and Kanamori, 2003; Olson and Allen, 2005) – *PreSEIS* requires no assumptions on rupture determinism: due to its continuous up-date of estimated source parameters *PreSEIS* can also handle complex rupture histories (Böse, 2006). As illustrated in Fig. 7, *PreSEIS* can be considered as a new approach to EEW that bridges the gap between the existing regional and on-site warning systems.

Combing the information on ground shaking at different sensors in a station network, *PreSEIS* determines the most likely hypocenter locations, magnitudes and rupture expansions at each time step after triggering of the first station. The

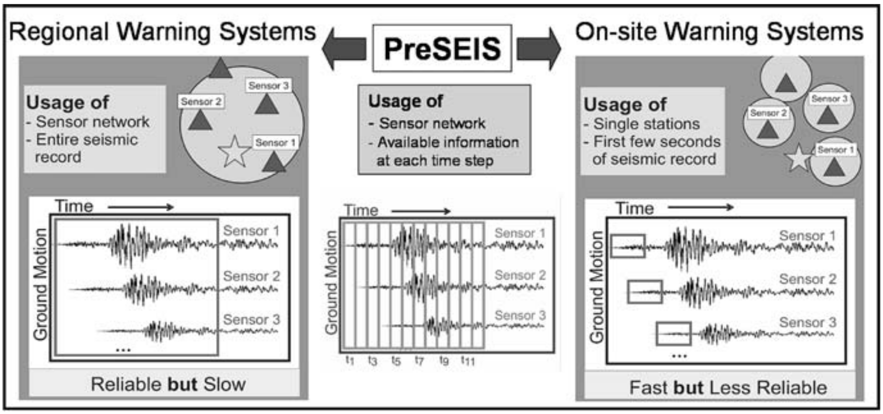

Fig. 7 PreSEIS bridges the gap between regional and on-site warning systems: it is based on sensor networks, though uses only the available information on ground shaking at each time step

estimated source parameters thereby show a clear and fast convergence towards correct solutions with proceeding time. For single events interim deteriorations might be observed (Böse, 2006).

PreSEIS (Böse, 2006) is currently tested and extended within two projects: “SAFER – Seismic eArly warning For EuRope” funded by the sixth framework program of the European Union, and “Earthquake Disaster Information system for the Marmara region, Turkey” (EDIM) funded by the German Federal Ministry of Education and Research (BMBF).

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Simulating Earthquake Scenarios in the European Project LESSLOSS: The Case of Lisbon

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1 Introduction

This paper is a completion of a previous paper published in the proceeding of the 1755 Lisbon Earthquake Conference and contain the results after the European Project LESSLOSS Risk Mitigation for Earthquakes and Landslides (Lessloss) regarding the evaluation of earthquake scenarios for the Metropolitan Area of Lisbon (MAL).

In the framework of the subproject 10, SP10 Earthquake disaster scenario prediction and loss modelling, finite-fault seismological models have been proposed to compute the earthquake scenarios for three urban areas: Istanbul (Turkey), Lisbon (Portugal) and Thessaloniki (Greece). The overall aim of SP10 is to create a tool, based on state-of-the art modelling software, to provide strong quantified statements about the benefits and costs of a range of possible mitigation actions, to support decision-making by city and regional authorities for seismic risk mitigation strategies. First element of this analysis process is the definition of the input as hazard parameter (Fig. 1).

The generation of earthquake ground motion scenarios involves both the particular choice of earthquake sources with associated fault rupture parameters, and the ensuing ground motion field calculated by an appropriate numerical tool, or empirically estimated, at a set of selected points within the urban area of interest.

For SP10 the aim was to define ground shaking hazard associated with particular scenario events, defined as earthquakes with a given magnitude and location, taken to be the worst event which would take place with a given return period, 50, 100 or 500 years. Defining that earthquake requires a close study of the capable faults, of the earthquake recurrence on those faults, and may also require a de-aggregation analysis of the local effects from all faults capable of generating damaging ground motions.

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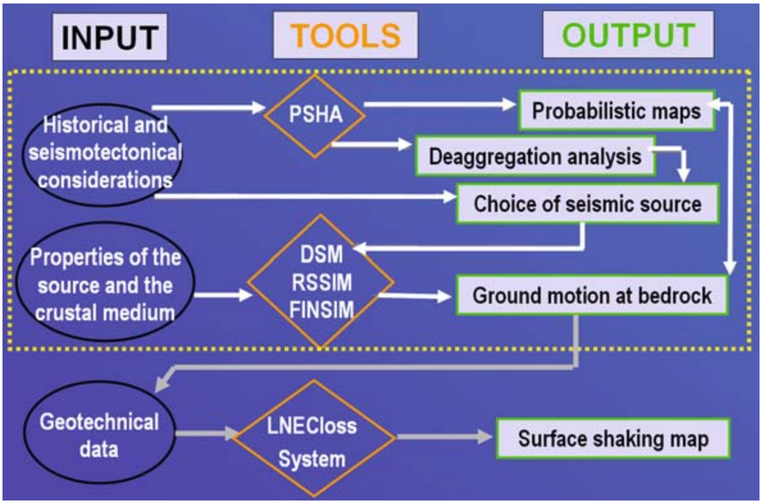

Fig. 1 Processes to achieve the definition of ground shaking hazard

Approaches to ground shaking hazard depend on the type and level of the analysis to be undertaken. For a Level 1 assessment, the site ground motion needs to be defined in terms of a macroseismic intensity, derived from the given magnitude and location of the scenario earthquake using intensity attenuation relationships. This approach was not used in SP10. For a Level 2 assessment specific parameters of surface ground motion are needed as inputs to the structural vulnerability, and these are derived from a two-step process. In the first step a set of expected bedrock ground motions is determined (with as fine a grid of points as needed for the structural analysis). This is obtained either by the use of empirical ground motion attenuation relationships, or with greater sophistication by the use of simulation techniques in which the mechanics of ground motion transmission from source to site is simulated. The bedrock ground motion is then used to determine the expected surface ground motion based on an understanding of the typical soil profile at each location. Again this can be done in different ways, either using either dynamic soil-column analysis, or using standard soil amplification coefficients applicable to the given soil type and depth.

In the case of Lisbon, the site effects are evaluated by means of an equivalent stochastic non-linear one-dimensional ground response analysis of stratified soil profile using the LNECloss system (Sousa et al., 2004). Given a seismic scenario (magnitude and location), the Bedrock Seismic Input and the Local Soil Effect software modules of the system, allow computing the Power Spectral Density Function (PSDF) of the strong ground motion at bedrock and at surface level of any site, respectively. A data base is available, containing information on stratified soil profile units for the region under analysis: in the framework of the project conducted by the portuguese civil protection authority (SNPC), it was carried out a geological – geotechnical survey that allowed the characterization of stratified soil profile units for MAL. The computer algorithms developed and implemented inside LNECloss system introduce some major improvements to take into account site effects due to soil dynamic amplification in rather efficient way (Serra and Caldeira, 1998).

2 Numerical Approaches

The first important distinction among recommended approaches for creating ground shaking scenarios is between simplified and advanced methods. Simplified methods make use of empirical attenuation relations of ground motion parameters and of local geological data. Within LESSLOSS project, advanced methods are extensively applied because of their capability of physically representing the ground motion. Indeed, finite-fault effects and directivity could assume a very important role. Moreover, the high resolution of ground motion scenario can match with the complexity of geotechnical characterization, vulnerability data and exposure factors involved in the urban level losses estimations.

Ground motion simulations were performed in the frequency band of engineering interest (0.5–20 Hz) by two numerical methods: a hybrid stochastic-deterministic approach (DSM-Deterministic-Stochastic Method; Pacor et al., 2005), used for all the investigated cases, and a non-stationary stochastic finite fault simulation method (RSSIM; Carvalho et al., 2004, 2007), applied in the case of Lisbon. Both methods allow computing synthetic time series for direct S-wave field at bedrock sites, and are suitable to generate shaking scenarios near an extended fault whereby the direct S wave-field is generally dominant in amplitude with respect to the reflected and superficial phases. Different rupture propagation models on the selected faults can be hypothesized and, even when input data regarding earthquake source, propagation medium, and site characteristics are of a very schematic nature, the complexity of near-source ground motion can be adequately reproduced. Extended fault simulations performed with different earthquake rupture models generally produce a high variability in the ground motion, mainly dependent on the assumed position of the hypocenter on the fault plane which controls the rupture directivity. During the project, a sensitive study was performed using different input parameters and different approaches in order to give the basic information to evaluate the range of uncertainty in seismic scenarios.

3 Definition of Scenario Earthquakes

Probabilistic seismic hazard analysis (PSHA) carries out integration over the contributions to the hazard from all the sources within a region, for certain ranges of magnitude M and distance R. It provides the estimate of a strong-motion parameter with a specified confidence level, during a given exposure period. Moreover it is able to account for the uncertainties associated with estimation of the seismicity and of the attenuation characteristics of the region. This latter property is taken into account by an additional integration over a given number of standard deviations, ε, of the adopted ground motion attenuation law (Bazzurro and Cornell, 1999).

Because of its integrative nature, PSHA does not provide a representative earthquake, in terms of magnitude and source-to-site distance, that can be used for engineering analyses and decision making. However this can be achieved through the de-aggregation of the PSHA results. The fundamental objective of this analysis is to compute the contribution to hazard, at a specific site, of every possible source S = (M,R, ε) considered in composite PSHA calculations. It allows to have a controlling earthquake, in terms of magnitude M and source-to site distance R, and to identify the location of the most probable source contributing to hazard at a well-defined site. Distance R can eventually be substituted by geographical coordinates and the measure of the deviation of the ground motion from the predicted value, ε, can also be considered in the de-aggregation process. Information about the controlling source can be used to generate a scenario earthquake, which is the basis of the deterministic hazard assessment (Harmesen and Frankel, 2001). Because deterministic scenarios are associated to representative earthquakes, they can be performed by advanced ground motion simulation methods allowing the reproduction of specific source effects like the extended fault properties, the earthquake rupture propagation, or the asperities distribution on the fault plane. It is noteworthy that if seismic source is accounted by only magnitude and distance parameters, none of these effects are considered.

4 The Case of Lisbon

The Metropolitan Area of Lisbon (MAL) has been historically struck by scarce, though intense, earthquakes, such as the 1755 Lisbon earthquake estimated as magnitude M = 8.7. The earthquake scenarios were defined based on de-aggregation of the probabilistic seismic hazard analysis (PSHA) for different return periods. For Lisbon area, LNEC (Campos Costa et al., 2002, 2006) performed the de-aggregation process in two stages. Both de-aggregation processes allowed independently to assess offshore and inland seismic sources (Fig. 2). Based on the revised deaggregation of PSHA it was possible to conclude that seismic hazard in the Metropolitan Area of Lisbon (MAL) is dominated by long distance scenarios (offshore sources), only for return periods greater than 50 years. The two previously defined scenarios with magnitude M 5.7 and M 7.6, associated with inland and offshore sources have been substituted by only one offshore scenario with magnitude M = 7.9, adopted for 475 yr RP.

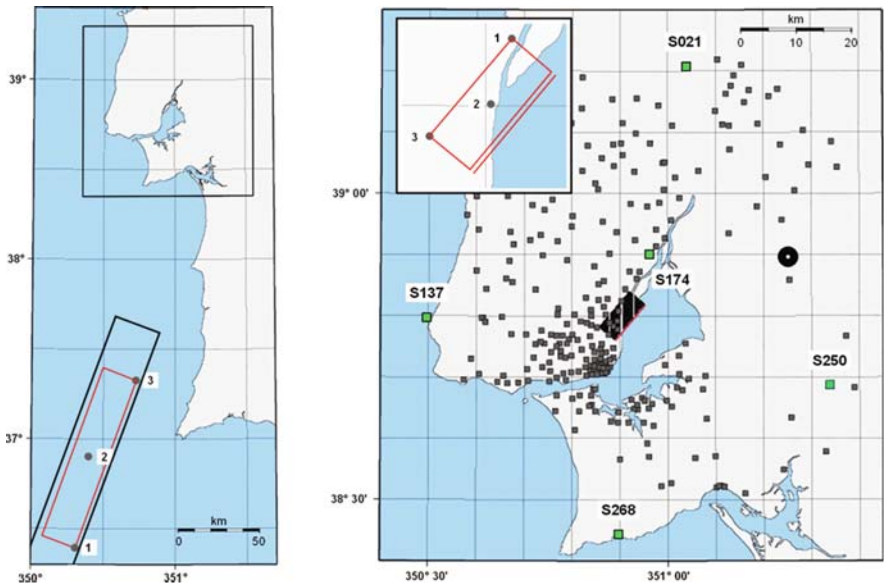

Fig. 2 Surface projections of the faults represented in Table 1. *Left*: position of the offshore sources MPTF M 7.6 and MPTF M 7.9 respect to the MAL (*black box*). *Right*: position of the inland sources LTVF M 5.7 (4.7) and LTVF M 4.4 (*black box* and *black circle*, respectively). A spot of the LTVF M 5.7 source with the assumed nucleation points is shown on the *top left*. The 277 parishes where time synthetic time series were computed are also shown (*black little squares*), together with the 5 test parishes selected for the sensitivity analysis (sites S021, S268, S250, S137 and S174)

Seismological studies dealing with the possible seismogenic structures associated with the 1755 Lisbon earthquake aimed to define plausible fault parameters for this source. Indeed, a direct display on a map of locations dominating the hazard as far as the knowledge of the most likely magnitude, allowed defining a specific set of earthquakes that present the greatest hazard to the site. The most recent studies on tectonic structures that could be associated with the 1755 Lisbon earthquake were employed to define plausible parameters for the simulation of offshore sources, while seismotectonic studies on the Lower Tagus Valley were used to define the simulation parameters of inland sources (Baptista et al., 2003; Cabral et al., 2004).

Since August 2005, when the selection of 50 year and 500 year scenarios for Lisbon Metropolitan Area (MAL) was accomplished, the work on seismic action scenarios selection and on the assessment of seismic motion achieved several progresses. These progresses resulted in adjustments in the magnitude and location of the scenarios initially studied by INGV. More specifically, seismic action scenarios were revised based on modal values derived from 3D deaggregation analyses in M and (X, Y) (magnitude and coordinates of bin source) (Sousa, 2006). However, for the 500 year return period, the revised scenario is still located offshore, validating the INGV preliminarily conclusions about the spatial variability of the ground shaking in MAL. For the 50 yr RP, the deaggregation analysis indicates a short distance scenario corresponding to inland sources of magnitude M = 4.4. The corresponding source was located beneath the Lower Tagus Valley (LTV), on the east side of MAL, and from previous study allows to infer both geometrical parameters and focal mechanism for a plausible source (Cabral et al., 2004).

4.1 Finite-Fault Model Parameters

Following this revised methodology the dominant scenarios found for Lisbon city are presented in Fig. 3. For the earthquake scenarios, the earthquake source locations, its geometrical, kinematical structure and parameters and the starting point of fault rupture are very important issues. Therefore, the finite-fault simulations require specification of the fault-plane geometry (length and the width, etc.), source (stress drop) and the crustal properties of the region (geometrical spreading coefficient, quality factor, etc.) and the site-specific soil response information. The fault dimensions as a function of moment magnitude are calculated using the empirical relations of Wells and Coppersmith (1994). The fault geometry and the source mechanisms are given in Table 1.

The model parameters calibration has been done with a dataset that includes horizontal components of ground acceleration records (hard sites) obtain by the national digital accelerometer network of Lisbon. (Carvalho et al., 2004). The other parameters, including crustal properties and sources calibration parameters to complete the finite fault simulation have been defined in previous papers (Zonno et al., 2005; Carvalho et al., 2007).

In this paper, we will present a bedrock ground motion simulation of acceleration (DSM-Deterministic-Stochastic Method; Pacor et al., 2005) for the scenario II: the inland LTVF, with length of 8.4 km and width of 6.0 corresponding to **M** 5.7 (see Table 1). The DSM method is a modern and sophisticated tools are applied to predict ground motions capturing the complexity of near-source ground motion, even when input data regarding earthquake source, propagation medium, and site characteristics are of a very schematic nature.

We will present also a bedrock and surface ground motion simulation of acceleration using a non-stationary stochastic finite fault simulation method (RSSIM; Carvalho et al., 2004) for the scenario V: the offshore MPTF fault, with length (along the strike) of 166 km and width (down-dip) of 30 km corresponding to an event of **M** 7.9 (see Table 1). The RSSIM method is a finite-fault method capable of capturing the offshore attenuation law calibrating the data of from national digital accelerometer network of Lisbon.

During the European Project both methods, DSM and RSSIM, have been used allow computing synthetic ground motion for direct S-wave field at bedrock sites and are suitable to generate shaking scenarios near an extended fault.

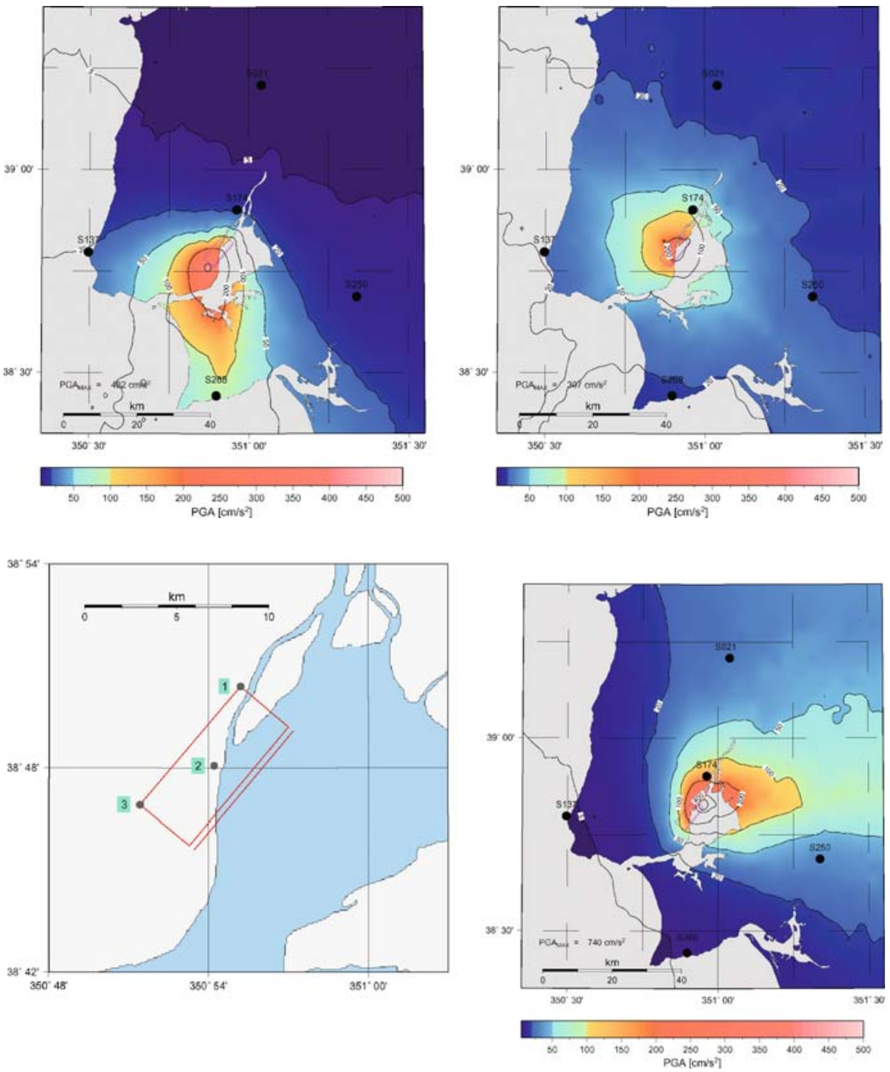

Fig. 3 PGA maps obtained in MAL through extended fault simulations using the M = 5.7 inland source (LTVF). Nucleation points of the NE/SW unilateral (1, *top left*), bilateral (2, *top right*) and SW/NE unilateral (3, *down right*) rupture propagations are indicated together with the surface projection of the fault (*down, left*). A rupture velocity VR = 2.7 km/s was adopted for all cases. Black dots indicate the position of the test sites used to analyse the variability of results

Nevertheless it is authors believe that FINSIM is worldwide known and a reference tool for a comparison of new approaches. A sensitivity analysis has been done performed using both adopted methodologies and FINSIM (Carvalho et al., 2007).

Table 1 Fault parameters of seismic sources used to perform the finite fault simulations in the case study of Lisbon

Name of seismic scenario	Fault Name	Mw	Return period (year)	Length (km) × Width (km)	Dip (degree)	Strike (km)	Ztop (km)
Scenario I	LTVF	4.4	50	1.4 × 2.3	55	220	0.5
Scenario II	LTVF	4.7	75	2.2 × 2.8	55	220	2.0
Scenario III	LTVF	5.7	200	8.4 × 6.0	55	220	0.5
Scenario IV	MPTF	7.6	200	110 × 24	24	20	4.5
Scenario V	MPTF	7.9	500	166 × 30	24	20	4.5

5 Evaluation of Seismic Scenarios

5.1 Onshore Analysis

The ground motion was computed at bedrock level through extended fault simulations, and then amplified by considering the site effects. The characterization of local soil effects was taken into account by computing the Power Spectra Density Function at the surface level and considering, for each parish, the non linear behaviour of the available stratified geotechnical soil profile (Zonno et al., 2005; Carvalho et al., 2007).

In Fig. 3 are shown the PGA maps obtained in MAL through DSM extended fault simulations using the M 5.7 inland source (LTVF). Nucleation points of the NE/SW unilateral (1), bilateral (2) and SW/NE unilateral (3) rupture. As evidenced, the unilateral rupture propagation (cases 1 and 3), produces the

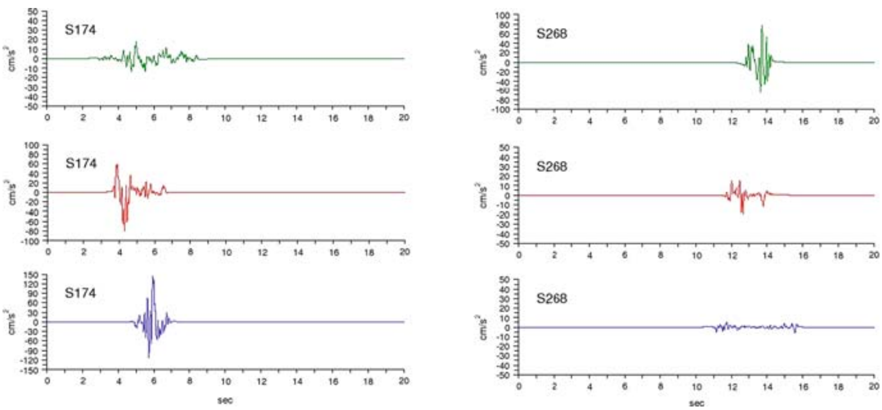

Fig. 4 DSM Time series Acceleration obtained at 2 test sites (S174 and S268) in the Metropolitan Area of Lisbon for the inland source of M = 5.7 with different rupture models. *Left panel*: unilateral nucleations (NE/SW, *green lines*; SW/NE, *blue lines*) and bilateral nucleation (*red lines*) with velocity rupture of 2.7 km/s

highest variability of the ground motion. Sites experiencing forward directivity (located South and East to the source in cases 1 and 3, respectively) are characterized by large amplitudes (and short durations).

In Fig. 4 are shown the DSM Time series Acceleration obtained at 2 test sites (S174 and S268) in the Metropolitan Area of Lisbon for the inland source of M = 5.7 with different rupture models. Left panel: unilateral nucleations (NE/SW, green lines; SW/NE, blue lines) and bilateral nucleation (red lines) with velocity rupture of 2.7 km/s.

The PGA variability due to uncertainties in both the nucleation point position and the rupture velocity is generally present over the entire simulated frequency band (f > 1 Hz) (Carvalho et al., 2007).

5.2 *Offshore Analysis*

Generally, ground shaking scenarios obtained through extended fault simulations present high variability mainly due to directivity effects. In fact, ground motion maps are very sensitive to the position of the nucleation point on the fault plane and unfortunately this is probably the most uncertain seismological information that we have to introduce in simulations. Otherwise, extended fault simulations are necessary in order to properly reproduce the earthquake ground motion from large and/or near sources. For the offshore analysis we have used the RSSIM method. In the Fig. 5 are shown the PGA maps obtained in the MAL for the 475 yr RP. Ground motion maps presented here show higher values of PGA in comparison with the acceleration maps obtained in the initial analysis (Zonno et al., 2005). Moreover, Fig. 5 evidences that the nonlinear soil amplification is quite pronounced,

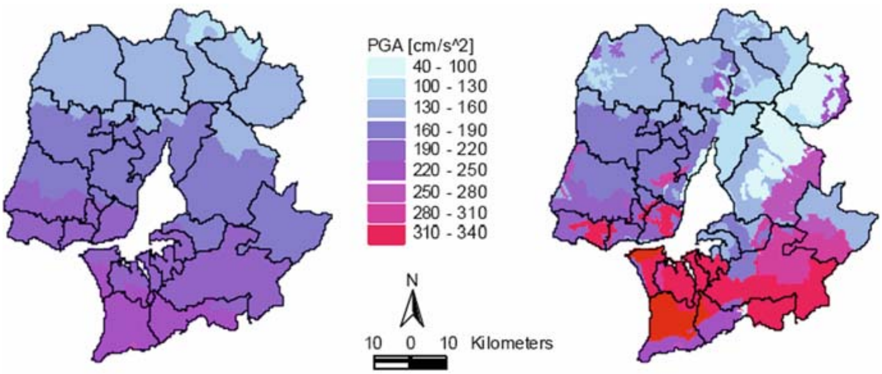

Fig. 5 PGA maps obtained at bedrock and surface level are compared. Ground motion amplification are evident especially in the South margin of Tagus River, where soft soils are

particularly in the south margin of Tagus River where soft soil sites are put in over bedrock (in this area peak ground accelerations range between 250 and 300 gal).

6 Conclusions

From the seismological point of view, many equally probable ground shaking scenarios were simulated by varying the nucleation point over the fault plane, the rupture propagation velocity and the slip distribution. The following criteria were used in performing the final choice:

- The proposed scenarios are those more compatible with the standard analysis of hazard in terms of peak values of strong ground motion parameters and spectral ordinates (empirical ground motion models and PSHA).
- In general, they should be suitable for application over the whole metropolitan area. In a second step, as in the case of large extension of the city (Istanbul), the scenarios have been finalized on a small zone.
- According to the final application, either the mean or the worst scenarios might be used. (for instance, in Lisbon area, the worst case was selected with purpose of generating precautionary scenarios only). The choice depends on the demand of the local administrators, the sensibility to the earthquake phenomena and, above all, the level of the seismicity of the area.

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How Distant Earthquakes Contribute to Seismic Hazard in Mainland Portugal

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1 Introduction. Seismic Hazard and Deaggregation Computation

The used method to compute the seismic deaggregation is that proposed by Bernreuter (1992) for assessing the control earthquake. This approach has been recommended by the SSHAC (1997) as a means of simplifying the understanding of the results obtained in a seismic hazard analysis.

We begin by calculating the seismic hazard, using spatially smoothed seismicity, for the Iberian Peninsula (Peláez and López Casado, 2002). This methodology was proposed by Frankel (1995), and was used here with certain modifications. The seismicity of the region was delimited in seismic sources, and the b and m_{max} parameters of the truncated Gutenberg-Richter relationship were smoothed.

The attenuation relationships in the Iberian Peninsula and its surrounding areas were also regionalised (López Casado et al., 2000a); this ensures a decrease in the uncertainties in seismic hazard evaluations. Five types of intensity attenuation relationships are used for eleven specific regions, with mean attenuation coefficient (absorption coefficient) values that range from 0.001 to 0.083 km^{-1}. The regionalization is clearly correlated with the seismotectonic characteristics of each region. When necessary, the relationship between macroseismic intensity and horizontal peak ground acceleration proposed by Murphy and O'Brien (1977) is adopted.

We were unable to include a characteristic earthquake model, since in the study region it was impossible to specifically associate the earthquakes with active faults at the moment. Instead, we incorporated a historical seismicity model, including the most destructive earthquakes in the area. We have not considered necessary to include a uniform background zone due to the historical extension and the spatial quality of the used catalog. Moreover, including this model would imply a hazard decrease in the most active zones. In short, four models were initially considered: those with a seismicity of M

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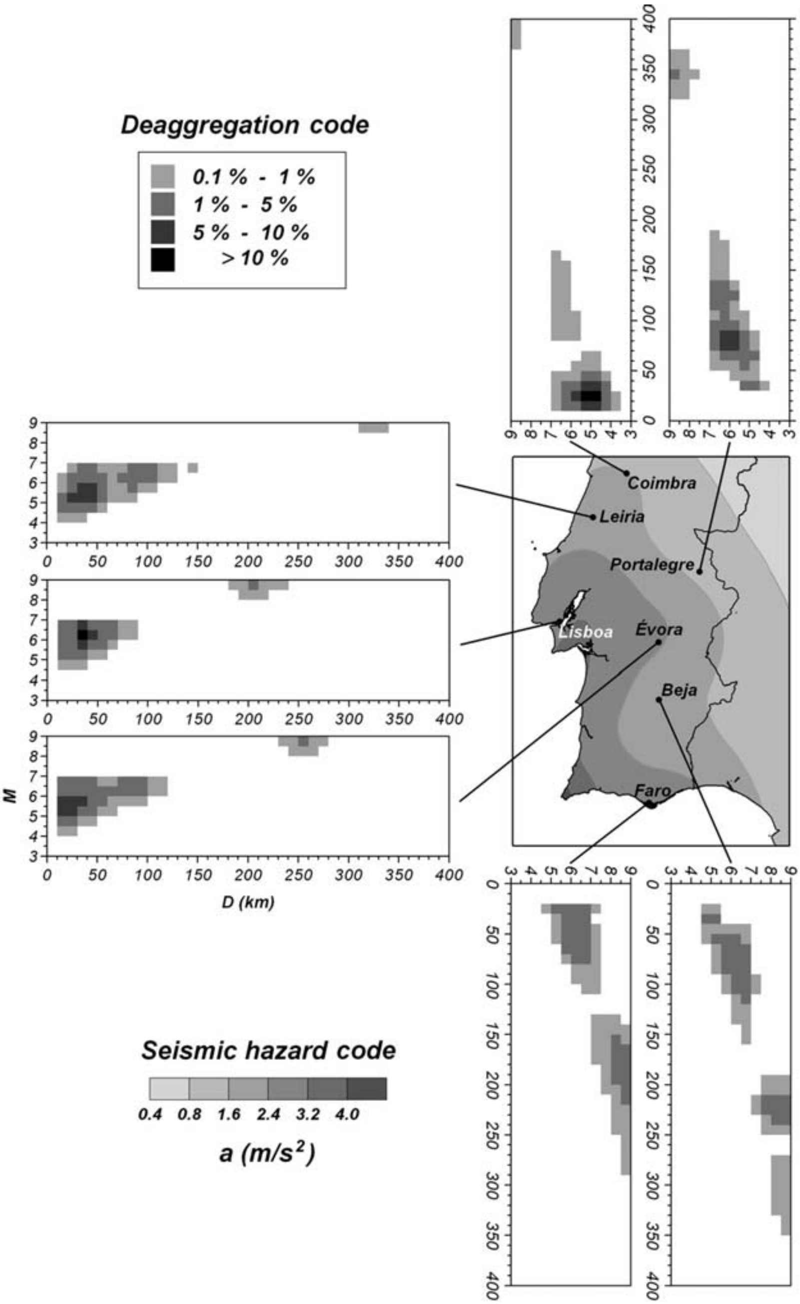

Fig. 1 Seismic hazard in terms of peak ground horizontal acceleration for the area, and deaggregation for the selected cities, with 10% probability of exceedance in 50 years. Modified from Peláez et al. (2002)

Table 1 Expected mean peak ground acceleration for the selected cities having a 10% probability of being exceeded in 50 years

City	PGA	
	cm/s^2	g
Coimbra	159	0.16
Leiria	199	0.20
Portalegre	140	0.14
Lisbon	308	0.31
Évora	244	0.25
Beja	199	0.20
Faro	274	0.28

$\geq M_S$ 5.5 after 1300, with $M \geq M_S$ 4.5 after 1700, with $M \geq M_S$ 3.5 after 1920 and with $M \geq M_S$ 2.5 after 1960. To the hazard generated by these four models, which included only shallow seismicity ($h \leq 30$ km), we added that generated by a fifth model that included the intermediate depth seismicity in the zone, that is, seismicity from 30 to 60 km depth with $M \geq M_S$ 2.5 after 1960.

Seismic hazard values (Peláez and López Casado, 2002) in the study area are shown in Fig. 1, where the mean peak ground horizontal acceleration is given with a 10% probability of being exceeded in 50 years, i.e., for a return period of 475 years. Table 1 presents the expected acceleration specifically for each of the seven Portuguese cities where the deaggregation calculation has been conducted.

The mean peak ground acceleration ranges from 0.14g for Portalegre to 0.31g for Lisbon. Table 2 shows detailed results for Lisboa and Faro for different levels of significance. Figure 2 gives the most important earthquakes felt that affect the hazard in the area. That is, those with a magnitude of over M_S 5.5 ($I_{MSK} \sim$ VIII–IX, in accordance with the relationships of López Casado et al., 2000b), that occurred after 1300, and those with a magnitude of over M_S 4.5 ($I_{MSK} \sim$ VII-VIII) that occurred after 1700. We have used the catalogues for the area of Mezcua and Martínez Solares (1983) and LNEC (1986), updated to 1999.

Table 2 Expected mean peak ground acceleration for Lisbon and Faro having a 10% probability of being exceeded in 50 years

α	PGA (g)	
	Lisbon	Faro
0.05	0.19	0.22
0.15	0.22	0.24
0.50	0.29	0.27
0.63	0.30	0.29
0.85	0.34	0.32
0.95	0.38	0.34

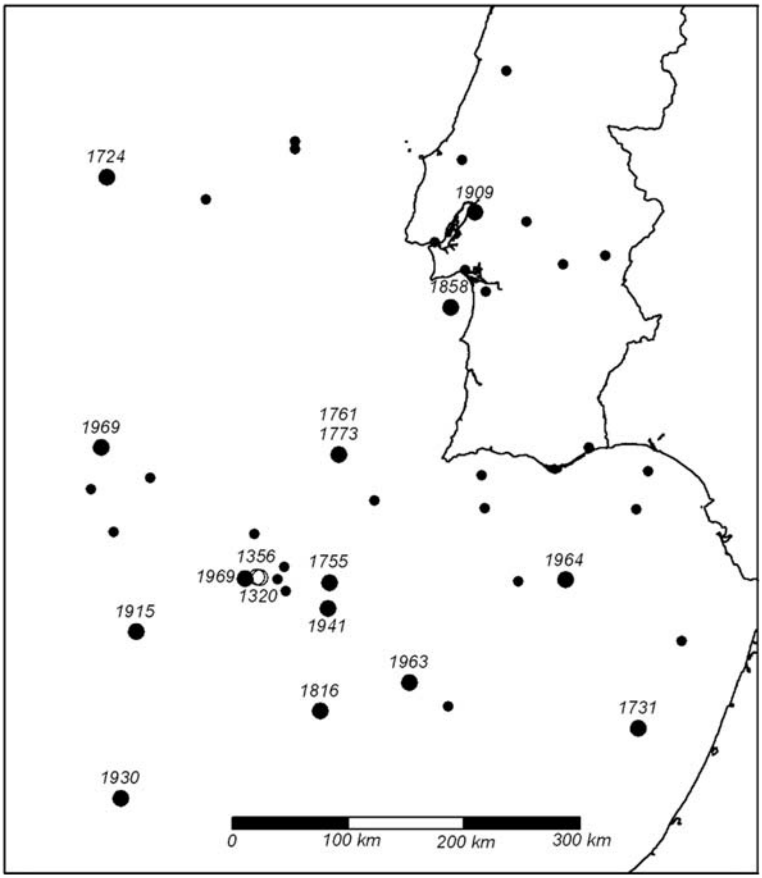

Fig. 2 Most important earthquakes that affect the hazard in the area. We can observe 4.5 $\leq$ MS $<$ 5.5 (*small filled circles*) and MS $\geq$ 5.5 (*large filled circles*) earthquakes since 1700, and MS $\geq$ 5.5 earthquakes since 1300 (*large open circles*). Modified from Peláez et al. (2002)

Since the used intensity attenuation model lacks spectral information, we have not performed the deaggregation in periods. Also, due to the regionalization of the attenuation relationships, we have not deaggregated in ε (ground-motion uncertainty). In this study we present the deaggregation of the mean peak horizontal acceleration only in terms of magnitude and distance, as well as in azimuth (in longitude and latitude). Based on these deaggregation results, we can determine, for different locations, the so-called control earthquake, design earthquake, modal event or dominant event. To do so, we use the average values of the magnitude and distance ($\bar{\mathrm{M}}, \bar{\mathrm{D}}$) and the modal values of these two variables ($\hat{\mathrm{M}}, \hat{\mathrm{D}}$). Bazurro and Cornell (1999) termed this the 2D hazard deaggregation technique. The expression proposed by Bernreuter (1992) to calculate $\bar{\mathrm{D}}$ is used in this work:

$$\log \bar{\mathrm{D}} = \frac{\sum\limits_{m}\sum\limits_{d} \mathrm{H}_{\mathrm{md}} \log \mathrm{d}}{\sum\limits_{m}\sum\limits_{d} \mathrm{H}_{\mathrm{md}}} \tag{1}$$

m is the magnitude, d the distance, and H_{md} is the contribution to the seismic hazard of the magnitude m ($m \pm \Delta m/2$) at a distance d ($d \pm \Delta d/2$) from the location.

The problem of using the average value of the magnitude and distance as a representative value of the control earthquake has been amply treated in Bazurro and Cornell (1999). We recommend the calculation of both the average and the modal values, taking for granted that the latter are more representative when applied to the seismo-resistant design, to the calculation of the safe shut-down earthquake, or to take into account in the design of response spectra. Nonetheless, a comparison between the pairs of values ($\bar{\mathrm{M}}, \bar{\mathrm{D}}$) and ($\hat{\mathrm{M}}, \hat{\mathrm{D}}$) is a simple and fast way of determining whether the sources generating the hazard in a particular area are many and heterogeneous, since different values will crop up depending on which criterion is used.

The method used to calculate seismic hazard (Frankel, 1995; Peláez and López Casado, 2002) is ideal for calculating deaggregation. The same cells and magnitude intervals used to calculate the aggregation of seismic hazard can also be used to calculate the deaggregation. The deaggregation is assessed by multiplying the relative contribution to the hazard of each cell by the weight assigned to each of the models for calculating seismic hazard. The results are presented, for computing the deaggregation in terms of magnitude and distance, using magnitude intervals of 0.5 units, and a linear distance increment of 10 km.

Although the minimum magnitude used to calculate and represent the deaggregation is the same as that chosen to calculate the hazard, the results are actually extremely interesting for magnitudes above M_S 4.5. However, the fact that we are using a macroseismic M_S magnitude, resulting from a relationship between macroseismic intensity and M_S magnitude (López Casado et al., 2000b), implies that we shall obtain magnitude values above M_S 9.0. For example, the 1755 Lisbon earthquake, reached a magnitude of M_S 9.4 on this macroseismic scale, considering *XII* (MSK scale) as their epicentral intensity for calculating the seismic hazard.

Finally, the calculation of the deaggregation in longitude and latitude is carried out using the same cells as in the hazard calculation.

2 Deaggregation Results

The plots of the results of the deaggregation in terms of magnitude and distance can be seen in Fig. 1. The results of the average and modal magnitude and distance values for the control earthquake are given explicitly in Table 3. Figure 1 shows the results obtained for the deaggregation in azimuth as plots, directly illustrating

Table 3 Mean and modal magnitudes (M_S) and distances (km)

City	$\bar{M}$	$\bar{D}$	$\hat{M}$	$\hat{D}$
Coimbra	5.4	47	4.5–5.0	20–30
Leiria	5.8	60	5.0–5.5	30–40
Portalegre	6.3	120	5.5–6.0	70–80
Lisbon	6.4	58	6.0–6.5	30-40
Évora	6.2	62	5.0–5.5	10–20
Beja	7.3	156	6.0–6.5	70–80
Faro	7.5	123	6.0–6.5	40–50

the contribution of each cell to the calculation of seismic hazard. Taking Figs. 1 and 3 together, it is easier to discuss the contribution of the different seismic zones to the appraised seismic hazard for each of the locations of interest. The relative contribution of the main and secondary seismic foci to the hazard at each location is detailed in Tables 4 and 5.

Two different deaggregation morphologies can be observed for the distinct cities (see Fig. 1). First, we can note deaggregations formed by a main lobe (single or double local seismic zone or focus, more or less extensive, surrounding the city) generating most of the hazard. However, one or two secondary lobes begin to appear that generate a small, though not inconsequential, amount of hazard. Such is the case of the cities of Coimbra, Leiria, Lisbon and Évora. These secondary lobes, of less importance with respect to their contribution to the hazard, can nonetheless mean a noticeable difference between the control earthquakes calculated using the average or modal values in some cases (Table 2).

The second group comprises cities that do not lie within the seismic zone or focus that generates the highest hazard in the location, lying instead at distances of 50–100 km away. In this category, secondary lobes that can have considerable importance also appear. The cities of Portalegre, Beja and Faro can be included in this group. In the case of Beja, the greatest contribution to the hazard is produced by a seismic focus more 200 km away from the city (Table 3). The fact that these cities are exposed to a wide range of potential damaging earthquake scenarios implies that the hazard and the deaggregation analysis remain uncompleted if only the peak ground acceleration is considered. Future efforts should be addressed to conduct these studies in terms of spectral acceleration values.

Referring to the control earthquake calculated for each of the locations (Table 3), and based on the modal values obtained, we have observed the following.

First, in 5 of the 7 studied cities (Coimbra, Leiria, Lisbon, Évora and Faro), the dominant event in these locations is produced less than 50 km away, with a magnitude ranging between M_S 4.5–6.5. In the case of Faro, using average values instead of modal ones would provide different result due to the appearance of somewhat important secondary lobes. In this case, the $\bar{D}$ value is on the

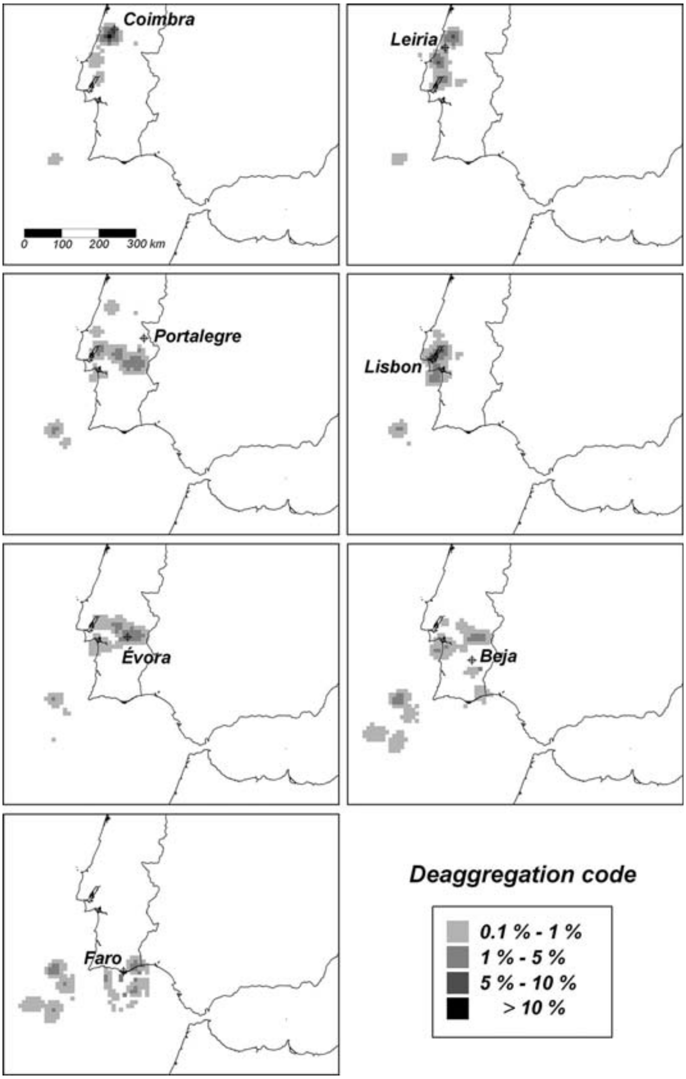

Fig. 3 Deaggregation in azimuth for the seven selected Portuguese cities. Modified from Peláez et al. (2002)

order of 125 km and the $\bar{M}$ value is around M_S 7.5. In contrast to the other cities mentioned, where nearby seismic foci dominate in the calculation of seismic hazard; in this case we cannot ignore this hazard generated at distances of about 150–250 km away from the location (see Fig. 1). The seismic foci responsible are those observed W and SW of Cape San Vicente.

In a second group of cities (Portalegre and Beja), there is a control earthquake generated 70–80 km from the location, with a magnitude M_S 5.5–6.5. The

Table 4 Main seismic focus, range of distance ΔR (km) to this source and contribution H (%) to the hazard for the selected cities

City	Main seismic focus	ΔR	H
Coimbra	Seismic focus southwest of the city: 1948 Condeixa (m_b 4.9) and 1969 Pombal (m_b 4.7) earthquakes	10–50	77
Leiria	Surroundings and seismic focus northeast of the city: 1948 Condeixa (m_b 4.9) and 1969 Pombal (m_b 4.7) earthquakes	<50	42
Portalegre	Seismic focus south-southwest of the city: 1757 Alentejo (*VIII*) and 1761 Évora (*VIII*) earthquakes	30–90	47
Lisbon	Surrounding and region north of the city: 1512 Lisbon (*VIII*) and 1531 Vila Franca de Xira (*VIII–IX*) earthquakes	<50	52
Évora	The city and its surroundings, and a region east of the city: 1757 Alentejo (*VIII*) and 1761 Évora (*VIII*) earthquakes	<40	57
Beja	Seismic focus at 100 km west of the Cape San Vicente: 1761 (*IX*) & 1773 West Cape San Vicente (*VIII*) earthquakes	210–240	29
Faro	Scattered seismicity in the Gulf of Cádiz: 1956 Southeast Cape San Vicente (m_b 5.0) and 1964 Gulf of Cádiz (m_b 6.2, *VII*) earthquakes	20–80	28

average values obtained for these cities give $\bar{\text{D}}$ values of 120–155 km and a $\bar{\text{M}}$ value ranging between M_S 6.3–7.3. In these locations the secondary lobes are more important than in the first group of cities. Once again, the seismic foci W and SW of Cape San Vicente are the source of these values for the control earthquake, when considering the average values of magnitude and distance.

Finally, we cannot disregard the hazard observed in some cities with magnitudes above M_S 8.0, which in some cases is truly significant.

Only four of the earthquakes included in the hazard and deaggregation analysis have a magnitude, instrumental or macroseismic, equal or exceeding

Table 5 Secondary seismic focus, range of distance ΔR (km) to this source and contribution H (%) to the hazard for the selected cities

City	Secondary seismic focus	*ΔR*	*H*
Coimbra	Seismic focus southwest of Leiria: 1940 Nazaré *(VII)* and 1989 Rio Maior (m_b 4.7) earthquakes	~ 90	5
Leiria	Surroundings and seismic focus southwest of the city: 1940 Nazaré *(VII)* and 1989 Rio Maior (m_b 4.7)	< 50	34
Portalegre	Seismic focus west-southwest of the city: 1909 Macas*(VIII)* and 1927 Vendas Novas (m_b 4.7) earthquakes	70–90	17
Lisbon	Surrounding and region south of the city: 1756 Setúbal *(VIII)* and 1858 Setúbal *(IX)* earthquakes	< 60	35
Évora	Seismic focus northwest of the city: 1909 Macas *(VIII)* and 1927 Vendas Novas (m_b 4.2) earthquakes	30–50	16
Beja	Seismic focus east of Évora: 1757 Alentejo *(VIII)* an 1761 Évora*(VIII)* earthquakes	60–80	22
Faro	Sesimic focus at 100 km west of Cape San Vicente: 1761 *(IX)* and 1773 West Cape San Vicente *(VIII)* earthquakes	170–200	20

M_S 8.0. The most recent was on 28/02/1969 (200 km SW of Cape San Vicente), with a recorded magnitude M_S 8.0 (U.S. Geological Survey / National Earthquake Information Center). The other three earthquakes have been assigned a macroseismic magnitude M_S, based on the epicentral intensity recalculated for the evaluated seismic hazard. These earthquakes occurred on 09/12/1320 (200 km SW of Cape San Vicente), with a M_S 9.4 macroseismic magnitude, on 01/11/1755 (150 km SW of Cape San Vicente), also with a M_S 9.4 macroseismic magnitude, and on 02/02/1816 (250 km SSW of Cape San Vicente), with a M_S 8.2 macroseismic magnitude. The locations of these last three historical earthquakes are given in Mezcua and Martínez Solares (1983) and LNEC (1986). They are therefore evidently affected by a degree of uncertainty that is not easily quantifiable, but that has been considered to a certain extent in the seismic hazard and deaggregation computation, since we have used an approach that spatially smoothes the seismicity. The four earthquakes are in the same area, a zone of some 20000 km^2 that lies 150–250 km SW-SSW of Cape San Vicente (Fig. 2).

There are some cities where this seismic focus, i.e., these four earthquakes, significantly contribute to the seismic hazard: Faro (this focus alone contributes 42% of the total seismic hazard in the city) and Beja (40%). Other cities are also affected, albeit to a less extent: Portalegre (12%), Lisbon (10%) and Évora (10%). The contribution is small for the cities of Leiria (4%) and Coimbra (3%).

3 Summary and Conclusions

We have showed the mean peak ground horizontal acceleration deaggregation results, for a return period of 475 years, in magnitude and distance and in longitude and latitude, at seven Portuguese cities, within the area of greatest seismic hazard in the Iberian Peninsula.

The results are presented in different ways: plots showing the deaggregation in terms of magnitude and distance, indicating the relative contribution of each cell (ΔM, Δd), plots showing the deaggregation in azimuth, giving the relative contribution of each cell ($\Delta\phi$, $\Delta\lambda$), and the control earthquake, calculated using both the average and modal values of the magnitude and distance variables.

The obtained results have allowed us to determine the distance and the azimuth at which the main seismic sources generating hazard are located from the different cities considered in this analysis, as well as quantifying the relative contribution to the total seismic hazard for each of them. The studied cities have revealed different morphologies in the results for deaggregation in magnitude and distance.

Finally, we should note the extreme importance of the seismic source SW of Cape San Vicente (the location of the 1755 Lisbon earthquake) for the hazard in the area. In particular for the cities closer to it, and in the range of magnitudes above M_S 7.5–8.0, the hazard generated by this focus is substantial.

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Part V
Propagation and Local Effects on the Seismic Destruction

Visualization of Seismic Wavefields and Strong Ground Motions Using Data from a Nationwide Strong-Motion Network and Large-Scale Computer Simulation

Takashi Furumura

1 Introduction

Following the destructive damage that resulted from the 1995 Hyogo-ken Nanbu (Kobe) earthquake (Mw 6.9), a nationwide network of 1,800 strong-motion instruments (K-NET and KiK-net networks) were deployed across Japan by the National Research Institute for Earth Science and Disaster Prevention (NIED). These networks were established to mitigate earthquake disasters by improving our understanding of regional seismic-wave propagation and site-specific amplification of ground motions in areas such as basins beneath highly populated areas.

Figure 1 shows the coverage of the K-NET and KiK-net strong-motion networks in Japan; the station interval is approximately 20–25 km. Each seismic station has a three-component accelerograph with a maximum scale of 2,000 cm/s^2. The ground acceleration is recorded at 100 or 200 samples/s with 24-bit resolution in A/D conversion (Kinoshita, 1998; Aoi et al., 2000). These dense seismic recordings enable the direct visualization of the wave field generated during large earthquakes. Snapshots of ground motion produced by interpolation of seismic record between stations illustrating clearly the seismic wavefield radiating from the earthquake source and amplifying in deep sedimentary basin, so that it is providing important insights into the complicated seismic wavefield and the development of strong ground motions during the past and future earthquakes.

Recent advances in high-performance computers, such as the Earth Simulator supercomputer constructed in 2002 at the Japan Marine Science & Technology Center (JAMSTEC), have made it possible to perform simulations of the regional-scale propagation of high-frequency seismic waves using a detailed subsurface structural model and a source-slip model for the earthquake itself. The results of such computer simulations, when coupled with seismic observations, provide an important means of understanding the detailed complexities

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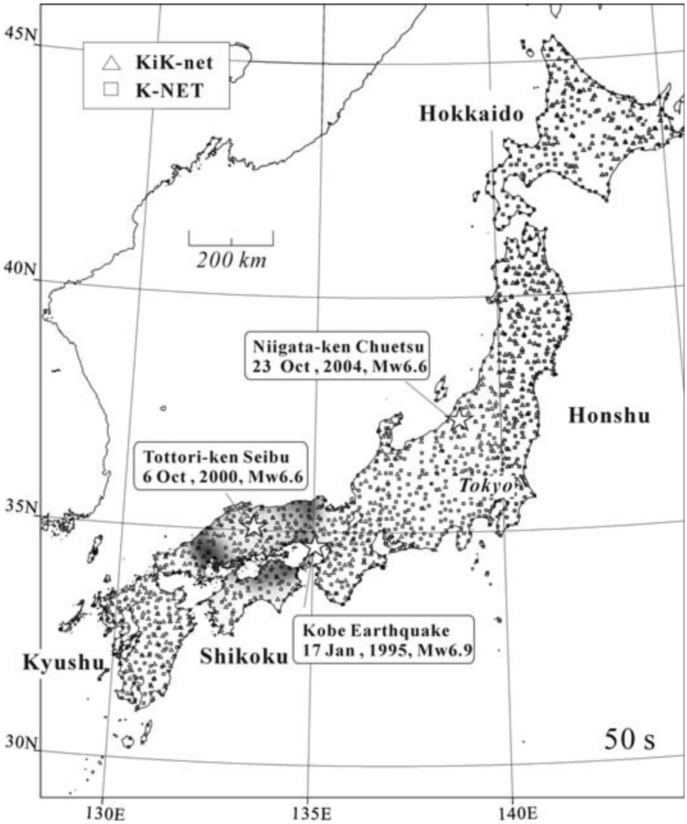

Fig. 1 Coverage of the strong-motion network (K-NET and KiK-net networks) across Japan and pattern of ground motion during the 2000 Tottori-ken Seibu earthquake (*gray area*) at 50 s after the source initiation. The hypocenters of the 1995 Kobe and 2004 Niitata-ken Chuetsu earthquakes are also shown

of the seismic wavefield, including amplification effects in sedimentary basins located beneath major population centers. Detailed comparisons of simulation results and observations from the dense network have led to steady improvements in the accuracy of the simulation model, enabling the model to be used to predict the strong ground motions expected for future earthquake scenarios.

The importance of obtaining detailed knowledge of subsurface structure in predicting strong ground motion disaster is illustrated by assessing the significant damage from the 1995 Kobe, Japan earthquake (Mw 6.9) and corresponding computer simulation using the Earth Simulator supercomputer. Modern cities are vulnerable to disasters resulting from long-period ground motions that develop within thick sedimentary layers beneath the cities following earthquakes in adjacent areas. Observations of the 2004 Niigata-ken Chuetsu, Japan (Mw6.6) earthquake, in combination with complementary computer simulations, demonstrate the significance of long-period surface waves that develop in central Tokyo due to multi-passing and the focusing of seismic energy toward the center of the sedimentary basin.

2 Computer Simulation of the 1995 Kobe Earthquake

The Kobe (Hyogo-ken Nanbu) earthquake of January 17, 1995 (Mw 6.9) was the most damaging in modern Japanese history, resulting in more than 6,700 deaths in Kobe and the wider Hanshin region. A notable feature of this earthquake was that most of the damage occurred within a narrow zone that came to be known as the "damage belt". The damage belt extends for more than 25 km across the city of Kobe, with a width of approximately 2 km defined by the extent of the area with a JMA intensity of 7. Fault rupture propagation generally produces strong ground shaking in the area directly above the fault, but in this case the damage belt is noticeably displaced from the fault traces (black lines in Fig. 2) toward the center of Kobe at an offset distance of approximately 1–2 km. This offset occurred because of bending of the ray paths and multi-passing effects of seismic waves interacting with the complex subsurface structure below Kobe.

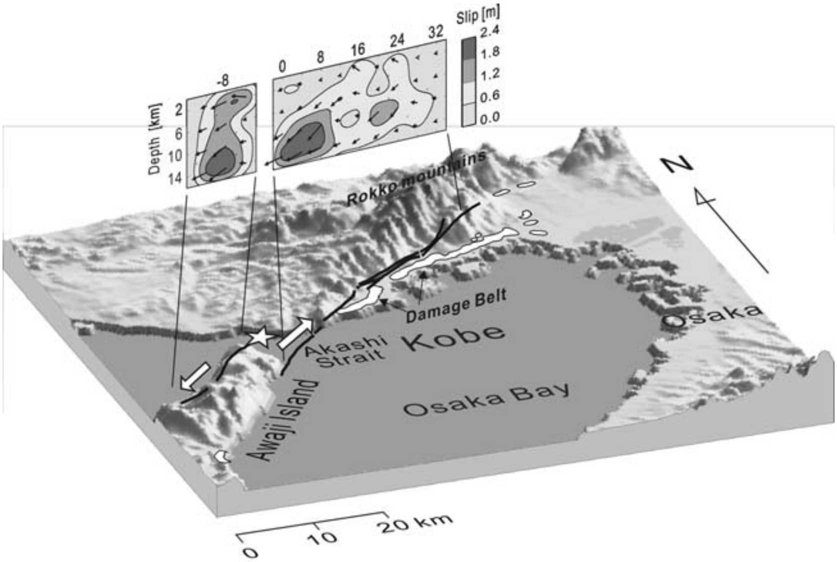

Fig. 2 Index map of the 1995 Kobe earthquake. The *star, black lines,* and *arrows* indicate the epicenter, source faults, and fault rupture directions for the earthquake, respectively. The *white* areas adjacent to the active faults represent areas of severe damage, with a JMA intensity of 7 (Damage Belt). The source slip distribution along the fault planes is projected above the relief map

Many studies have attempted to simulate the strong ground motions that occurred during the Kobe earthquake by using a subsurface structural model of the Kobe-Hanshin area and a source slip model for the earthquake itself; however, the patterns of the intensity distribution derived from computer simulations are not sufficiently clear to reproduce the sharply defined damage belt (e.g., Pitarka et al., 1998; Furumura and Koketsu, 1998, 2000). This problem probably reflects the insufficient resolution of the structural model at approximately 0.5–1 km depth and the high-frequency signals over 1 Hz employed in the previous simulations.

Ten years after the destructive Kobe earthquake, improvements in computing power have made it possible to perform large-scale simulations of strong ground motions with relatively high frequencies in excess of 1 Hz. In addition, the Geological Survey of Japan has compiled a high-resolution structural model of Osaka Basin with a horizontal resolution of 100 m and vertical resolution of 50 m. The model is based on a large number of reflection and refraction experiments, most of which where conducted across the zone of the damage belt immediately following the earthquake (Horikawa et al., 2003; Fig. 3a). The present high-resolution structural model and a source-slip model of the Kobe earthquake (Yoshida et al., 1996) were used in an attempt to reproduce the

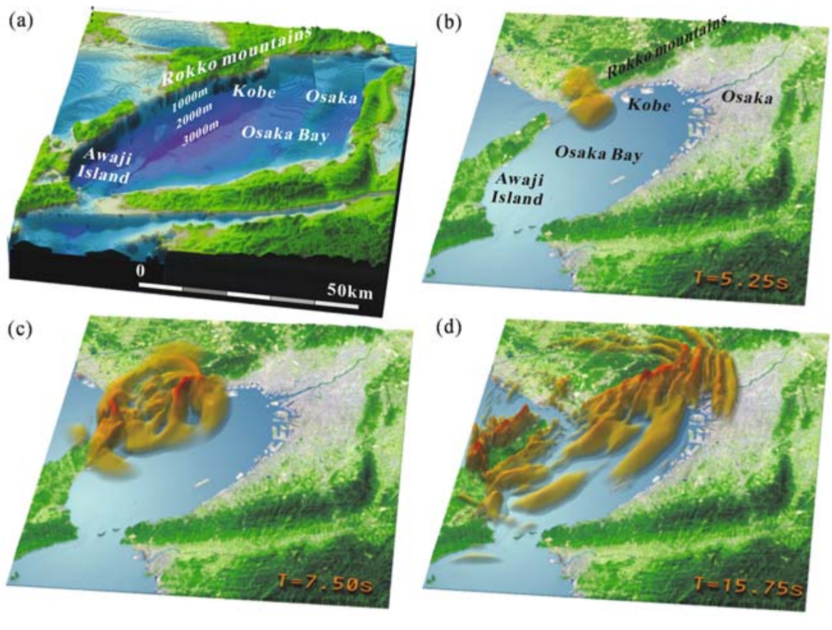

Fig. 3 (**a**) Subsurface structure of the Kobe-Hanshin area at the bedrock/sediment interface. (**b**–**d**) Temporal progression of the propagation of seismic waves of horizontal ground velocity motions derived from the simulation. The time from the start of the earthquake is shown at the *bottom-right* of each frame

measured pattern of seismic intensity and the narrow damage belt caused by the interaction of the seismic wavefield radiating from the source and significant amplification in the thick cover of sediments beneath Kobe.

2.1 Simulation Model

The present simulation model covers a volume of $90 \times 85 \times 40$ km discretized by a small grid spacing of $0.05 \times 0.05 \times 0.025$ km (Fig. 3). The grid size of the present simulation is four times smaller and the target area, and thus it is 20 times larger than those of the previous simulation (Furumura and Koketsu, 1998); the present simulation requires 62.6 billion grid points. Although the scale of the present simulation is approximately 1,500 times larger than that of the previous experiment that used a cluster of three workstations, it is easily handled by the Earth Simulator by parallel computing using a large number of processors.

The seismic wavefield that radiates from the seismic source and propagates through the heterogeneous crustal structure is calculated by solving the equation of motions and constitutive equations using a higher-order, staggered-grid parallel finite-difference method (FDM; Furumura and Chen, 2004). The minimum shear-wave velocity in the uppermost layer beneath Kobe is Vs = 0.25 km/s, and the small grid size (D = 0.05 km) used in the model enables the calculation of seismic waves of up to 2 Hz at a sampling of 2.5 grid points per shortest wavelength. The seismic source is introduced by assigning a number of double-couple point sources on the fault plane, each of which radiates seismic waves with a maximum frequency of 2 Hz. The fault rupture runs bi-laterally from the hypocenter to the northeast and southwest at an average rupture speed of Vr = 2.5 km/s. Since the source-slip models previously derived from the inversion of strong-motion waveforms and GPS data use the relatively lower frequency band below 0.5 Hz, the use of these models for high-frequency radiations in excess of 0.5 Hz is inadequate in the present simulation of high-frequency seismic wavefield.

Recent studies on source-rupture process claim that the radiation of high-frequency signals is largely controlled by irregularities in the source rupture speed. Consequently, in the present study the smooth slip model derived by the inversion described above is modified by introducing a random fluctuation in the fault-rupture speed. This fluctuation follows a Gaussian distribution function embedded at each sub-fault on the fault plane over an average speed (Vr). Such heterogeneities in the fault-rupture process efficiently produce high-frequency signals in the simulated wavefield. By comparing the simulated and observed waveforms, the parameter for the standard deviation of random fluctuation was finally selected as $\sigma = 2\%$.

The simulation model is surrounded by the perfect matching layer (PML; e.g. Marcinkovich and Olsen, 2003) of a 20-grid-point absorber in order to eliminate artificial reflections and wrap-around noises that occur at boundaries. The 3D simulation uses approximately 0.74 TB of computer memory and a CPU time of approximately 3 h, involving parallel computing using 240 nodes (1920 CPUs) of the Earth Simulator.

2.2 Simulation Results

The temporal progression of the simulated ground motions of horizontal velocity motion for the Kobe earthquake is shown in Fig. 3 using a visualization procedure of the seismic wavefield (Furumura et al., 2003). In the first frame (T = 5.25 s; Fig. 3b), the seismic wave begins to appear at the surface near Akashi Strait, and at T = 7.5 s (Fig. 3c) a directivity pulse generated from the rupturing fault is seen clearly approaching Kobe. The directivity pulses propagate northeastwards in the Rokko Mountain region at a relatively fast speed of approximately 3 km/s and eastwards on the Kobe side at a very low speed of less than 1 km/s (T = 15.7 s; Fig. 3d). In the middle frame (T = 7.5 s; Fig. 3b), another directivity pulse is generated from the second large slip (asperity) on the fault at 10 km below Kobe (see the upper panel in Fig. 3). These two pulses are amplified significantly in the thick, low-velocity sediments below Kobe and form longer tails in the basin (T = 15.75 s; Fig. 3d). A surface wave is produced at the edge of the mountain/basin interface by the conversion of an S-wave propagating upward from the source toward the Rokko Mountains. The surface wave then enters the sedimentary basin, resulting in stronger ground shaking due to constructive interference with the basin waves propagating upward from the source toward Kobe. These signals propagate through Kobe from west to east, resulting in intense ground shaking within densely populated areas.

The pattern of seismic-intensity distribution derived from the present simulation is shown in Fig. 4b, where a narrow and elongate zone of JMA intensity 7 is clearly seen to the southeast of the fault at Kobe and northwest of the fault at Awaji Island. The simulation result is similar to observations shown in Fig. 4a in terms of the distribution of areas with an intensity of 7: the width of the simulated belt is approximately 1–2 km, with pockets of intensity-7 areas east of the main damage belt, such as those observed at Takarazuka and Nishinomiya. In contrast to the present results (Furumura and Koketsu, 1998, 2000), previous simulations were unable to accurately reproduce these observations because of the relatively coarse mesh models and relatively low frequencies used in the earlier simulations.

The results of the present computer simulation, which employed a fine-mesh structural model and a relatively high frequency up to 2 Hz, clearly demonstrates the importance of the construction of a detail structural model for populated basins in order to mitigate strong-motion disasters expected for future earthquake scenarios.

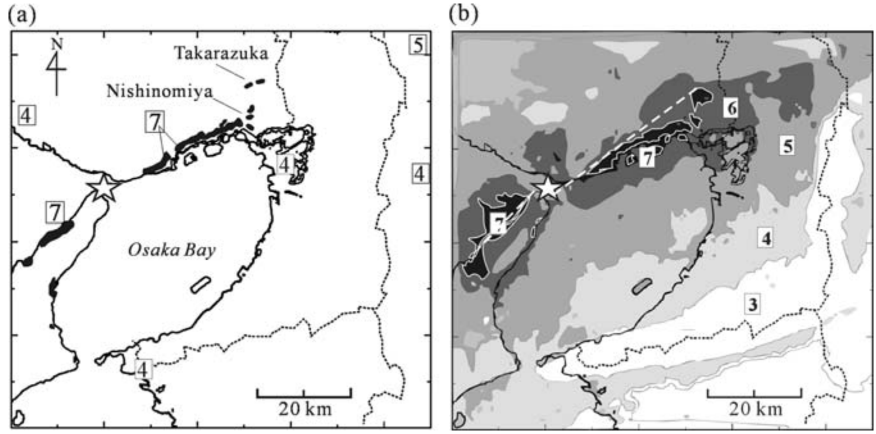

Fig. 4 (**a**) Distribution of JMA intensity for the 1995 Kobe earthquake and the area of the damage belt (*black areas*). (**b**) Simulated pattern of intensity. *Dashed lines* represent the surface projection of the fault traces. Numbers denote JMA intensity scale

3 Strong Ground Motions in Tokyo Associated with a Hypothetical Earthquake

The Tokyo Metropolitan Area has previously suffered strong-motion damage associated with large earthquakes. Earthquake-related disasters in modern history include the 1923 Kanto earthquake (M7.9), the 1703 Genroku earthquake (M8.4), and the 1855 Ansei Edo (M7) earthquake. The former two events occurred at the interface between the subducting Philippine Sea Plate and the North American Plate; the recurrence time of these events is known to be about 200–400 years. In contrast, the epicenter of the Ansei Edo earthquake is known to be located to the north of Tokyo Bay, but the source mechanism and depth are unknown.

Several attempts have been made to determine the source mechanism of the Ansei Edo earthquake by analyzing the pattern of seismic-intensity distribution in the area around Tokyo; however, the intensity pattern in Tokyo would have been strongly affected by the amplification effect of the thick sediments beneath the city. Thus, estimating the source depth on the basis of the local intensity pattern in Tokyo has proved to be very difficult.

Furumura and Takeuchi (2006) attempted to use the regional-scale intensity distribution to examine the possible source depth of the Ansei Edo earthquake. This approach was taken because large-scale anomalies in the distribution of intensity contours are strongly affected by structures in the deep crust and upper mantle rather than localized anomalous structures beneath Tokyo. The authors compared observed and simulated intensity patterns for the earthquake and demonstrated that the Ansei Edo earthquake would have been shallow event (h = 10 km) within the crust, as with the Kobe earthquake.

If such a shallow crustal earthquake occurs north of Tokyo Bay, the Tokyo Metropolitan Area will suffer a significant disaster due to the amplification of ground motions within the thick sequence of sediments beneath the Kanto (Tokyo) Basin. Accordingly, a simulation was performed of strong ground motions arising from a shallow crustal earthquake in the Tokyo area. The simulation employed a subsurface structural model of the Kanto Basin and the source slip model of the 1995 Kobe earthquake.

3.1 Simulation Model

The structure of the Kanto Basin has been extensively investigated since 1980 by employing a number of geophysical and geological experiments, including reflection and refraction surveys, array measurements of microtremors, analysis of gravity anomalies, and using data from deep drillholes (e.g., Yamanaka and Yamada, 2002; Tanaka et al., 2006). The basin model used in the present study consists of three sedimentary layers with shear-wave velocities of Vs = 0.5, 1.0, and 1.7 km/s, and underlying rigid bedrock (Vs = 3.0 km/s). The maximum thickness of the sedimentary sequence in central Tokyo is approximately 4 km at the center of Tokyo Bay.

The simulation model is 204 * 204 *128 km in size, and was discretized by a grid size of 0.1 *0.1 *0.05 km in a shallow layer to a depth of 8 km; the grid size is doubled (0.2 *0.2 *0.1 km) in deeper layers. This multi-grid approach to FDM simulations can reduce the required computer memory and processing time by a factor of 12 compared with conventional FDMs using a uniform mesh. The seismic source of the 1995 Kobe earthquake is placed north of Tokyo Bay at a depth of 14 km. The source rupture spreads bilaterally from the hypocenter to the northwest and southeast, parallel to the major trend of the active faults in the vicinity of Tokyo.

3.2 Simulation Results

The results of the simulation are shown in Fig. 5 for the propagation of seismic waves from T = 16.8 s to 43.2 s following the start of fault rupture at the hypocenter. The progression of the seismic wave demonstrates the process involved in the generation of strong motions at the surface and the significant amplification of ground motions within the thick sedimentary basin. The amplification and elongation of the ground motions is noticeable in central Tokyo, which lies above a thick cover of soft sediment upon rigid bedrock. Very large ground motions with a long duration are apparent at the northern and eastern edges of the basin where the ground motion encounters a sharp wall of basement rock when propagating out of the basin.

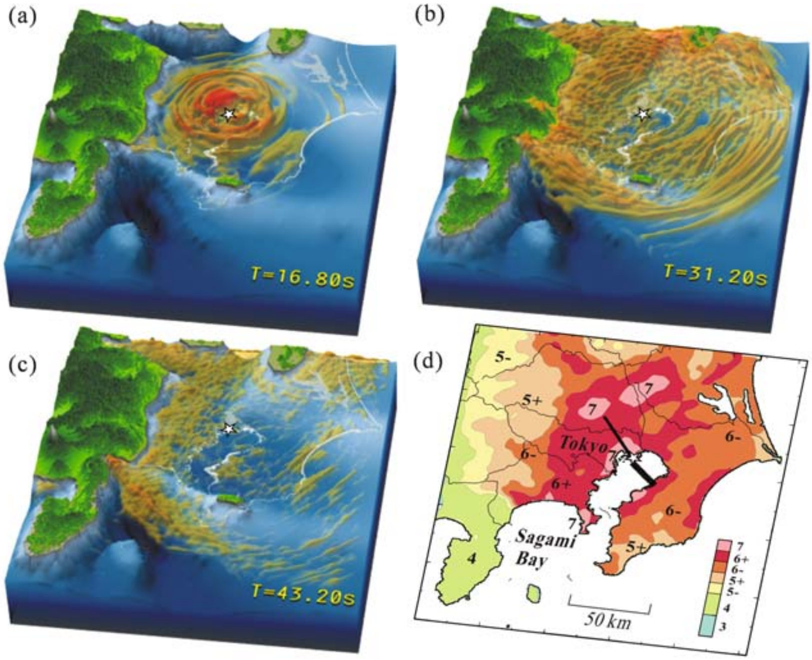

Fig. 5 Subsurface structural model of the Kanto area illustrating the topography of the sediment/basement interface. (**a**–**c**) Progression of the simulated surface ground motions of a hypothetical Tokyo earthquake. (**d**) Pattern of intensity distribution derived from the simulation. The *star* and *black rectangles* indicate the hypocenter and the surface projection of the source fault, respectively

The simulated pattern of the intensity distribution (JMA 7 point scale) for the hypothetical Tokyo earthquake is shown in Fig. 5d; intensities greater than 6 encompass almost the entire high-population area. The simulated acceleration seismograms are multiplied by site amplification coefficients that were derived empirically from the relationship between the surface geology and the site amplification effect (Kubo et al., 2003). This was done because amplification effects associated with shallow (< 30 m) superficial layers are unable to be accommodated in the present simulation model, which has a vertical resolution of 50 m.

The resulting intensity pattern reveals areas with largest intensity 7 around Tokyo Bay and in pockets at Saitama, Ibaraki, and Kanagawa. These areas of high intensity are caused by strong amplification of high-frequency signals in superficial layers, such as those extending along rivers and reclaimed land. The area with an intensity of >6 covers almost all of the population centers within the Tokyo Metropolitan Area; this area is several times larger than that observed during the Kobe earthquake in 1995. The large extent of this high-intensity area is largely due to the significant amplification of ground motions within the thick cover of sedimentary rocks, as these sediments trap the seismic energy within the thick basin structure beneath Tokyo.

4 Long-Period Ground Motions that Developed Within Kanto Basin Following the 2004 Niigata-Ken Chuetsu Mw6.6 Earthquake

The thick cover of sediments beneath the Kanto Basin causes another type of hazard related to the resonance of long-period ground motions, with a dominant period of about 7–12 s, in the sedimentary basin during large (M>7) earthquakes in nearby areas. An example of the development of long-period ground motions in central Tokyo was observed during the Chuetsu Mw6.6 earthquake at Niigata, Japan, on 23 October 2004.

The Chuetsu earthquake was the most damaging earthquake in Japan since the Kobe earthquake of 1955; it resulted in significant damage, 31 deaths, and more than 2,000 injured persons near the hypocentral region, with an extensive ground acceleration of over 1,700 cm/s^2 measured at Ojiya City.

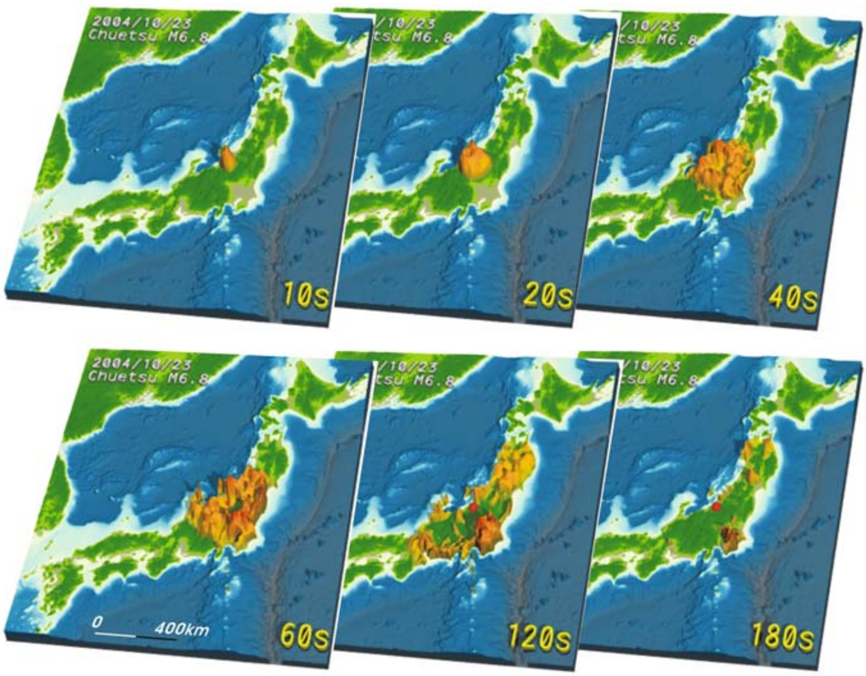

Fig. 6 Snapshots of observed ground motions associated with the 2004 Niigata-ken Chuetsu earthquake, Japan. The amplitude of the ground velocity motions are shown for times of T = 10, 20, 40, 60, 120, and 180 s after the earthquake rupture *Red circle* indicate the hypocenter of the Chuetsu earthquake

Tokyo is the largest population center within the Kanto Plain, and is located more than 150–200 km from the epicenter of the Chuetsu earthquake. The maximum intensity recorded in Tokyo was less than 3; however, the city was subjected to more than 5 minutes of intense (> 5 cm) ground shaking, with a relatively long dominant period of about 7 s. This large and lengthy shaking of long-period ground motions caused significant resonance within high-rise buildings of approximately 60–70 floors in height, thus warning of the high risk to modern constructions posed by large earthquakes. The prolonged duration of the long-period ground motions brings to mind the severe damage and accompanying fire in large oil-storage tanks that resulted from resonance associated with long-period ground motions at Tomakomai during the 2003 Tokachi-oki Mw 8.0 earthquake. These problems occurred even though the tanks were located more than 250 km from the epicenter of the quake (Koketsu et al., 2005). The 1985 Michoacan (Mw 8.1) earthquake in Mexico resulted in more than 20,000 fatalities in Mexico City, located 400 km from the epicenter, due to the attack of strong ground motions in relatively long-period signals of about 2–3 s and an extremely prolonged duration of more than 10 min (e.g., Anderson et al., 1986). These examples highlight the danger of long-period motions associated with large, distant earthquakes.

It is apparent from Figs. 6 and 7 that long-period surface waves develop at the northern margin of the Kanto Plain, induced by conversion from S-waves. The wave train of the surface wave is gradually elongated with propagation through

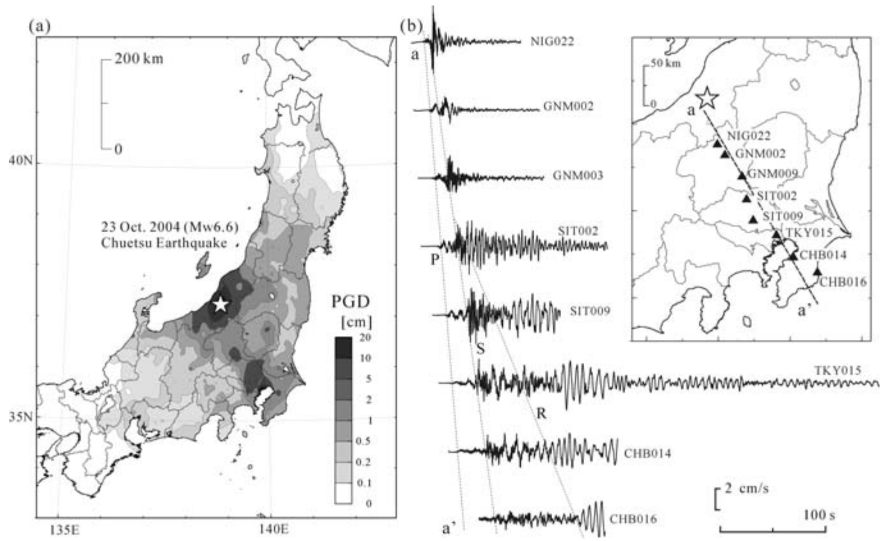

Fig. 7 (**a**) Distribution of peak ground displacement (PGD) during the 2004 Chuetsu earthquake. (**b**) Radial-component recorded section of ground velocity motions as recorded at seven stations from Chuetsu to Chiba (a-a′). Star indicates the hypocenter of the Chuetsu earthquake

the basin; this occurs due to the dispersion of surface waves in the low-wavespeed basin sediments (Vs < 0.5–1 km/s) that overlie high-wavespeed bedrock (Vs > 2–3 km/s). The dominant periods of the long-period surface waves in the center of the Kanto Basin are largely consistent in the period range of 6–7 s.

4.1 Development of Long-Period Ground Motion Within the Kanto Basin

The recordings of intensity meters located at city government offices and fire stations, among other sites, around Tokyo were examined to gain an understanding of the development of large and lengthy surface waves in central Tokyo from interaction to subsurface structure beneath the Kanto Basin. The integrated high-density seismic network (SK-net), which combines the K-NET, KiK-net, and a network of intensity meters, has a much denser station interval of about 8–10 km in central Tokyo.

The propagation of seismic waves recorded by the SK-net is illustrated in Fig. 8 as the particle motion of horizontal-velocity ground motions. A band-pass filter

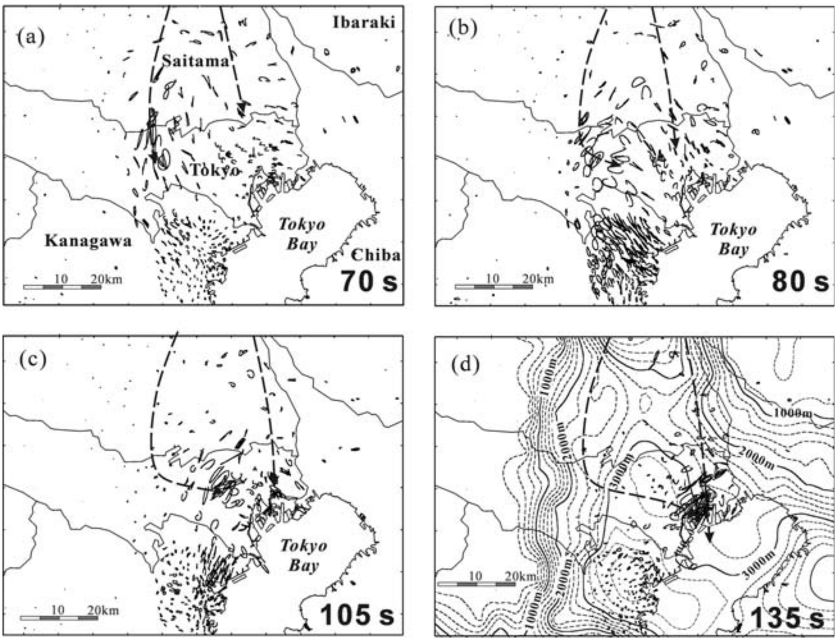

Fig. 8 (**a**–**c**): Particle motions of horizontal ground motions recorded by 495 intensity meters stationed throughout the Kanto Basin and 176 K-NET and KiK-net stations. The *large dashed lines* in each figure represent the estimated major wave-propagation paths of the surface waves that affected central Tokyo. The time from earthquake rupture is shown in the *bottom right-hand* corner of each figure. The configuration of the basement topography beneath Tokyo is shown by the contours in (**d**)

with a cut-off period between 3.5 and 14 s was applied to each seismogram trace to enhance the surface wave at 6–7 s.

The particle motions in the first frame (Fig. 8a; 70 s) show a family of large surface waves traveling into central Tokyo through Saitama Prefecture. The particle motions are largely polarized along the direction of wave propagation (i.e., radial), confirming that Rayleigh waves were the dominant component of the long-period ground motion recorded in central Tokyo during the Chuetsu earthquake.

In the middle frame (Figs. 8b,c; 80–105 s), large surface waves propagating along the western margin of the Kanto Basin in the direction of Kanagawa suddenly change direction to head toward central Tokyo as they drop from the steep slope of the bedrock interface to the deep basin bottom.

In the last frame (Fig. 8e; 135 s), the two sets of surface waves, one propagating directly from the north and the other rerouting from the area to the west of Tokyo, converge in central Tokyo. The merging of the two surface waves results in intense and prolonged ground shaking in central Tokyo, as observed in Figs. 6 and 7.

4.2 Computer Simulation of the Chuetsu Earthquake

To complement these observational data and gain insight into the complex seismic behavior within the heterogeneous structure of the Kanto Basin and the development of the large and prolonged long-period ground motions recorded in central Tokyo, the Earth Simulator supercomputer was used to run a simulation with a high-resolution subsurface structure model of central Japan and the Kanto Basin (Tanaka et al., 2006) and a source-slip model for the earthquake (Hikima and Koketsu, 2005). The simulation model covers a surface area of 440*250 km to a depth of 160 km, and is discretized by a small mesh of 0.2 *0.2*0.1 km for layers less than 10 km in depth and a double-sized grid for layers at depths of 10–160 km. The propagation of seismic waves at each grid point was calculated by solving the equation of motion using a parallel FDM (Furumura and Chen, 2004).

The results of the simulation are summarized in Fig. 9 as a series of three snapshots of particle motions at uniformly aligned locations that represent seismic stations located at 8 km intervals throughout the Kanto Basin; these simulation results can then be compared with observed ground motions. The simulated and observed waveforms of NS-component ground motions at four stations are also compared in the figure. The simulation accurately reproduces features of the observed ground motions, including the arrival of P- and S-waves and the elongation of long-period surface waves, although the fact that the simulation does not accommodate short-period signals of <1 s results in an underestimate of the amplitude of P- and S-waves.

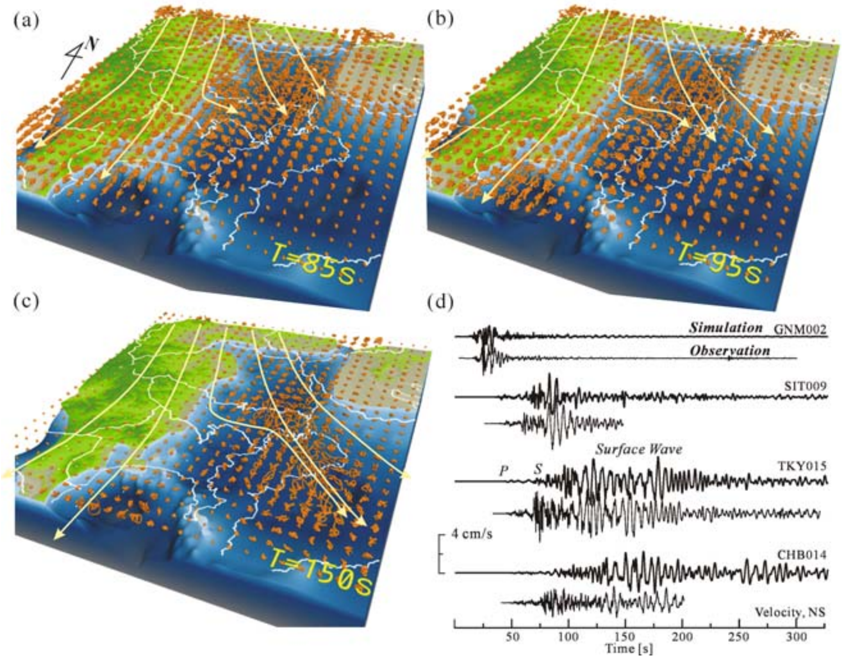

Fig. 9 (**a–c**): 3D view of simulated particle motions across the Kanto Basin and ray paths of the surface waves. (**d**) Comparison of simulated and observed waveforms of NS component ground motions at four K-NET stations

The simulation snapshot at 85 s (Fig. 9a) illustrates the generation of surface waves at the northwestern edge of the Kanto Basin and the significant amplification of ground motions in the soft sediments that overly the rigid bedrock. The simulation results at 95 s (Fig. 9b) show the rerouting of surface waves along the margin of the basin and subsequent focusing toward the center of Tokyo Bay via a saddle in the basement topography. The surface waves travel across Tokyo Bay to Chiba via the deepest part of the basement (Fig. 9c; 150 s) and then turning back toward Tokyo via the same route. The last snapshot (150 s) demonstrates the stagnation of surface waves in the deepest part of the basin.

The results of the computer simulation confirm that the mortar-like shape of the sedimentary basin and the thick sequence of sediments below Tokyo are the main causes of the large and prolonged ground shaking in Tokyo associated with long-period surface waves produced by the large earthquake. The anomalously large and prolonged shaking related to ground motions in Tokyo and Chiba result from multi-passing and the focusing of surface waves toward the bottom of the Kanto Basin with a cover of thick sediments and the subsequent stagnation of seismic energy in the basin.

5 Summary

Heterogeneities in the subsurface structure of populated basins, along with complex source rupture histories, result in highly complex seismic wavefields arising from large earthquakes. The seismic wavefields observed in Kobe during the 1955 Kobe earthquake and in central Tokyo during the 2004 Niigata-ken Chuetsu earthquake demonstrate the anomalous propagation of seismic waves when interacting with complex 3D subsurface structure beneath populated basins. The observations also reveal the significant amplification and elongation of seismic waves resulting from resonance within the thick sediments above the basin interface.

A computer simulation, undertaken using the Earth Simulator, of the propagation of seismic waves through the 3D structure of the Kanto Basin clearly demonstrates the risk to central Tokyo of the dramatic amplification of short-period seismic waves within the thick cover of soft sediments as well as the development of long-period surface waves resulting from the focusing and multi-passing of seismic waves toward central Tokyo.

The accurate prediction of strong motions during future seismic events therefore requires a good understanding of the seismic wavefield resulting from such heterogeneities in the subsurface structure and source rupture processes during large earthquakes. Visualizing the seismic wavefields of recent earthquakes using dense seismic-observation data and complementary computer simulations provides very important insights into the nature of the seismic wavefield. The improvements in the predictions of patterns of strong ground motions associated with future earthquake scenarios will depend on maintaining a close link between observation- and simulation-based studies in order to improve our knowledge of complex seismic behaviors in areas with heterogeneous structure.

Acknowledgments The author acknowledges the Earth Simulator Center for their support with computer resources. CPU time on the Earth Simulator was supported by the project Multi-Scale and Multi-Physics Integrated Simulation study (CREST) of the Japan Science and Technology Agency. K-NET and KiK-net data were provided by the National Institute for Earth Science and Disaster Research, Japan (http://www.bosai.go.jp). SK-net data were provided by the Earthquake Research Institute, University of Tokyo (http://www.sknet.eri.u-tokyo.ac.jp).

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Empirical and Theoretical Assessment of Upper Bounds on Earthquake Ground-Motions

Fabio Sabetta

1 Introduction

The issue of the assessment of the upper bounds on earthquake ground-motions has been recently identified as the "missing piece" from seismic hazard assessment, for both deterministic and probabilistic approaches (Bommer et al., 2004). If Deterministic Seismic hazard Assessment (DSHA) is to be used to define the maximum earthquake loading to which a structure may be subjected, then an estimate of the upper limit on the ground motion that a particular scenario could generate is needed. At the same time, the need for defining upper limits in Probabilistic Seismic hazard Assessment (PSHA) becomes evident when ground motions are calculated for very low annual frequencies of exceedance, using ground-motion prediction equations with untruncated lognormal scatter. For very long return periods, the hazard estimates are driven by the tails of the untruncated Gaussian distribution of the logarithmic residuals (Abrahamson, 2000) and may provide unrealistic high values of the ground motion. In fact both in DSHA and PSHA, as well as in the damage simulation scenarios for large earthquakes, the assessment of upper bounds in the ground motion is of fundamental importance to avoid the consideration of "physical impossible" ground motions. It is important to emphasize that the core of the problem is the need to identify ground motions that are not only unlikely (very small probability of realization) but actually impossible (zero probability).

This paper summarizes the results obtained by the author in the frame of the PEGASOS project (a comprehensive seismic hazard assessment for nuclear power plants in Switzerland; Abrahamson et al., 2002) where he participated as a member of the expert panel on ground-motion models (EG2). In particular an attempt to define the ground motion upper bounds on rock site conditions is proposed on the basis of recorded accelerograms, empirical prediction relationships, and numerical simulations. The maximum ground-motions that can be

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experienced at the ground surface are mainly controlled by three factors: (1) the energy released at the source of the earthquake; (2) the travel path of the seismic waves and the source-receiver geometry; (3) the soil non linear behaviour and the finite strength of the surface geological materials. In this paper the analysis is limited to the first two factors in order to predict the maximum ground motions in hard bedrock in terms of magnitude, distance, and spectral periods. The issue of estimating the level of this motion that can actually be transmitted through the soil deposits can be addressed subsequently. For example Pecker (2005) has developed a simple but robust method for estimating the limiting value on PGA by deriving a solution for a heterogeneous soil profile in which the shear modulus varies with depth.

2 Upper Bounds from Recorded Accelerograms

The first and most obvious tool for exploring upper bounds on ground motions is the ever increasing database of recorded accelerograms. In Fig. 1 the maximum recorded values of PGA as a function of time are reported. It is interesting to note the continuous increase with time of the recorded values of PGA. Until the 1960s the PGA had never exceeded about 0.35 *g*, the value obtained from the famous El Centro record of 1940. Since then, it has been proven that values in excess of 1.0 *g*, in both the vertical and horizontal directions, are common, particularly in proximity of the fault rupture.

In Table 1 are reported the 6 world highest Peak Horizontal Accelerations (PHA) recorded on rock or stiff soil sites. It is interesting to note that 3 of the 4 highest PHA values have been recorded on abutments of dams and are therefore probably affected by soil-structure interaction. Furthermore, many of these records are dominated by a single sharp acceleration spike: an example

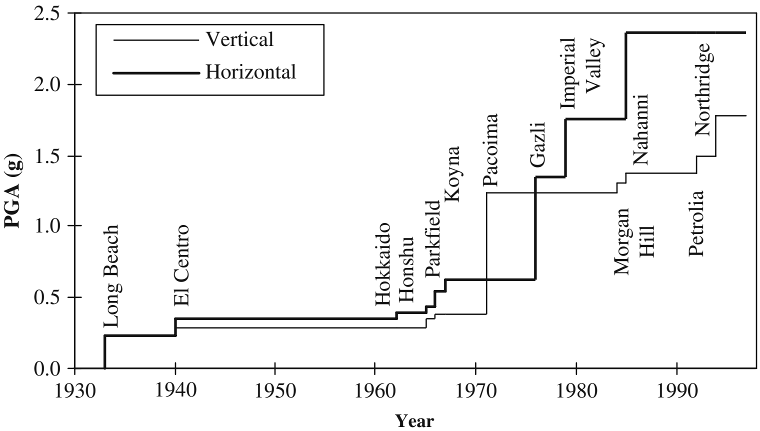

Fig. 1 Maximum recorded values of PGA as a function of time (from Bommer and Martínez-Pereira, 2000)

Table 1 List of the 6 world highest PHA recordings on rock or stiff soil sites

Earthquake Name	Date	Mw	Focal Depth	Fault type	Station Name	Local Geology	**PHA (g)** max hz.	PVA (g)	Epic. dist. (km)	Fault. dist. (km)
Northridge	17/01/1994	6.7	14 km	reverse	Pacoima Dam-Upper Left Abutment	rock	**1.58**	1.23	25.0	10.0
Petrolia	25/04/1992	7.2	15 km	reverse	Cape Mendocino	rock	**1.50**	0.75	4.0	0.0
Morgan Hill	24/04/1984	6.5	9 km	strike slip	Coyote Lake Dam	stiff soil	**1.27**	0.40	24.0	2.0
San Fernando	09/02/1971	6.6	8 km	reverse	Pacoima Dam-Upper Left Abutment	rock	**1.19**	0.71	9.0	0.0
Tabas	16/09/1978	7.4	5 km	reverse	Tabas	stiff soil	**1.10**	0.84	52.0	14.0
Nahanni	23/12/1985	6.7	6 km	reverse	Station 1 (Inverson)	rock	**1.02**	2.00	7.0	0.0

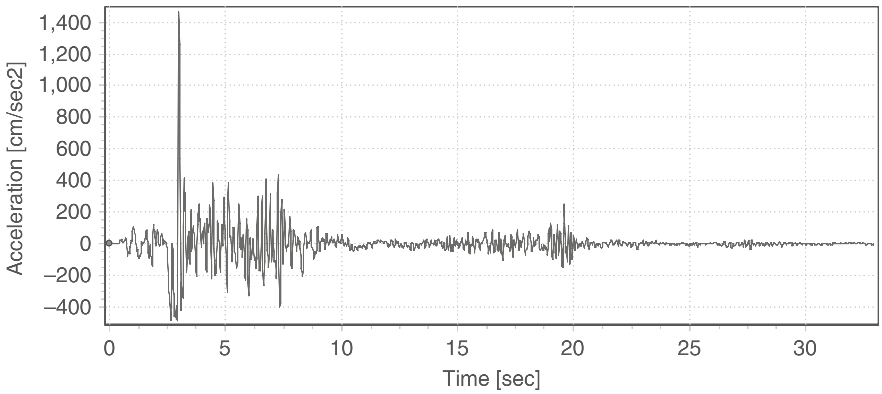

Fig. 2 Accelerogram recorded at Cape Mendocino (4 km epicentral distance) during the Petrolia earthquake of April 25 1992 (Mw = 7.2)

is given in Fig. 2 where the "Cape Mendocino" accelerogram, recorded during the 1992 Petrolia earthquake, shows a single strong spike of 1.5 *g* while the rest of the time-history is below 0.5 *g*. Maximum recorded values were also available, in the large ground motion database compiled in the PEGASOS project, for the acceleration response spectra (5% of critical damping) at several periods and different magnitude-distance bins. The maximum recorded motions are a good staring point because they provide lower bounds on the upper limits; in other words, the upper bounds must be at least as great as the recorded maxima for the same combination of magnitude, distance, and spectral period.

3 Upper Bounds from Numerical Simulations

In the absence of large numbers of recordings from very dense accelerograph networks triggered by many earthquakes, the best possibility of constraining the maximum levels that ground-motion amplitudes can reach is through the use of models to generate synthetic accelerograms. Whichever source model is chosen, the exploration of upper bounds requires, for each magnitude, mechanism, and source to site distance combination, a large number of simulations to be run, varying each of the variables such as the location, size and strength of asperities, rupture velocity, and nucleation point. This task is onerous in terms of computational effort and time; the real challenge is to ensure that all of the combinations of rupture parameters are actually physical.

Two different kinematic fault models have been provided in the PEGASOS project for numerical simulations aimed to evaluate the upper limit ground motion. The first, developed at URS-California by Arben Pitarka (Pitarka et al., 2000), is based on the hybrid Green's function method and uses a stochastic approach to generate the high frequency part of the ground motion.

The second, developed at OGS-Trieste by Enrico Priolo, is based on the Wavenumber Integration Method (Herrmann and Wang, 1985) with the source model proposed by Herrero and Bernard (1994) and uses a purely deterministic approach. Both models, in order to represent the worst-case, considered all the possible combinations of source geometry, fault mechanism, source-receiver geometry (constructive interference of waves), including the possibility of super-shear rupture velocity, a matter where there is still a lot of controversy in the scientific community (Bouchon and Vallée, 2003).

It has to be remembered that in the kinematic models the slip function is imposed, whereas in the dynamic modelling the limiting strength of the rock can be specified, thus allowing physical constraint on the radiated motions. In this respect, the upper bounds obtained from the kinematic source models are likely to be overestimated. Both models have therefore been subjected to a review by a "dynamic modeler" and, on the basis of this review, I only selected, among the solutions proposed by URS-Pitarka, those where the super-shear speed is restricted to the large asperities because the assumption of constant super-shear rupture along the entire fault, combined with a constant rise time, is considered unphysical by the author itself. Regarding the deterministic OGS-Priolo model, the reviewer's comments highlighted the overestimation of high frequencies at short distances due to the instantaneous slip front adopted in that model.

In Fig. 3 three simulated accelerograms from the two models are compared for $M_w = 7$ and different distances. The high frequency pulse with extremely large amplitude that dominates the acceleration at 5 km distance in the OGS-Priolo model, is caused by a coherent phase which is generated at the source and amplified during the rupture. The pulse is quickly attenuated at longer distances.

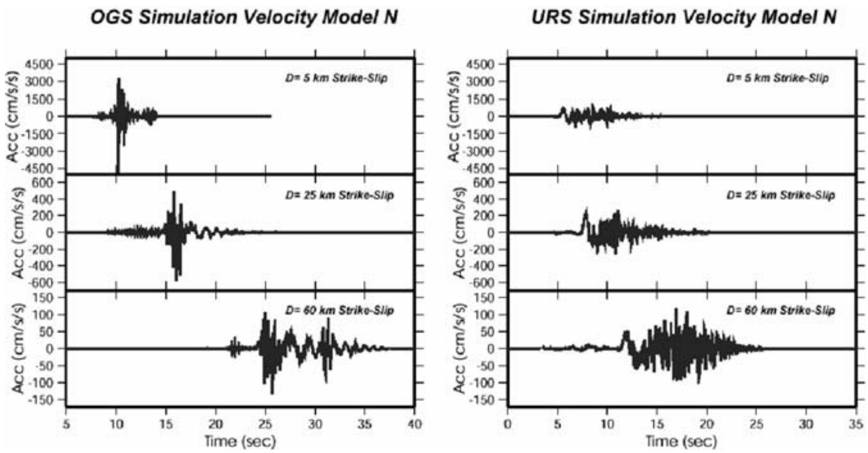

Fig. 3 Comparison of OGS and URS simulations for Mw = 7 and distance of 5 km (*top*), 25 km (*middle*) and 60 km (*bottom*)

Table 2 Largest PSA (horiz. cmp. geom. mean) from the numerical simulations

f(Hz)	M = 7 R = 5 km		M = 7 R = 25 km		M = 5.5 R = 5 km		M = 5.5 R = 25 km	
	Simul-1 Pitarka (g)	Simul-2 Priolo (g)	Simul-1 Pitarka (g)	Simul-2 Priolo (g)	Simul-1 Pitarka (g)	Simul-2 Priolo (g)	Simul-1 Pitarka (g)	Simul-2 Priolo (g)
0.5	1.50	0.29	0.35	0.23	0.11	0.02	0.03	0.01
1.0	3.00	1.20	0.60	0.92	0.55	0.07	0.06	0.03
2.5	6.00	1.91	1.80	1.51	0.55	0.50	0.11	0.69
5.0	8.00		2.50	2.15	0.75		0.19	0.53
10.0	5.50		1.30	0.78	1.10		0.28	0.11
20.0	4.50		1.00	0.78	0.80		0.22	0.39
PGA	3.50		0.60	0.78	0.50		0.13	0.39

At 25 and 60 km, the comparison between the two simulations is very favorable. For the above reasons, I did not consider the simulations provided by the OGS-Priolo model at distances less than 25 km and for frequencies higher than 2.5 Hz (blank cells in Table 2). The simulations were performed for four scenarios ($M_w = 7$ and 5.5; distance = 5 and 25 km) and from the simulated accelerograms the Pseudo-Acceleration (PSA) response spectra at different frequencies were also calculated as shown in Table 2.

4 Upper Bounds from Empirical Prediction Relationships

In principle upper bounds have to be assessed through the use of physical and not statistical considerations. However, in spite of the large increase achieved in the last years in the worldwide strong-motion database (Douglas, 2003), the recorded data are not enough to represent uniformly all the required frequencies, magnitudes, and distances. I chose therefore to make use also of some empirical prediction relationships (Ambraseys et al., 1996; Sabetta and Pugliese, 1996; Abrahamson and Silva, 1997; Spudich et al., 1999; Berge et al., 2003) covering continuously the M-R-f range. From the comparison with the values of the recorded data and numerical simulations discussed above and from the representation of the residuals of the prediction relationships in quantile-quantile plots, it comes out that the upper motion is of the order of the median plus 3 standard deviations. In Fig. 4 the response spectra of the above cited prediction relationships increased by three sigmas are shown. The relationships have undergone appropriate adjustments (Sabetta et al., 2005) in order to achieve a common definition of magnitude (M_w), style of faulting (strike slip), distance (closest point to the surface projection of the fault) and component of the ground motion (geometrical mean of the two horizontal). As the resulting values are sometime (large periods) higher than those of numerical simulations or recorded data, I also took into consideration the median values plus 2.5 σ.

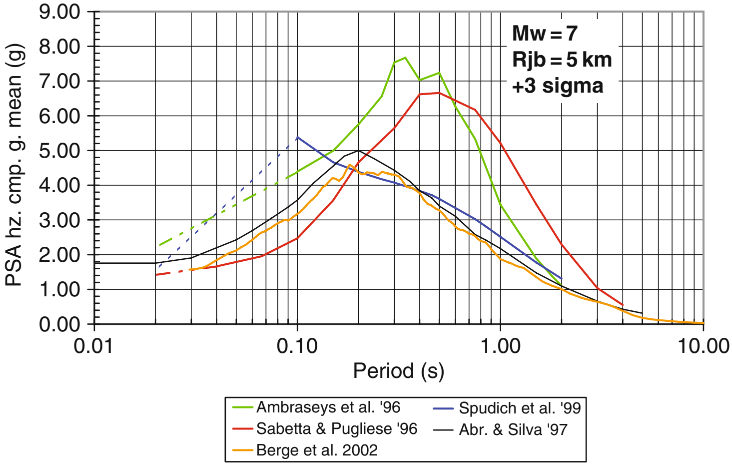

Fig. 4 Response spectra of some significant prediction relationships increased by three standard deviations for Mw = 7 and fault distance = 5 km

5 Results

In Table 3 a comparison among the results for the PSA upper bounds, obtained with the different approaches illustrated above, is presented for $M_w = 7$ and R = 5 km. The recorded data have been conservatively multiplied by a factor of 1.3 to take into account the continuous trend of increasing PGA with time (Fig. 2). The "weighted value" reported in the last column, comes out from a subjective weighted average of the previous values. The numerical simulations have a global weight of 0.5, with a net preference for the URS-Pitarka model and zero weight to the OGS-Priolo model at frequencies larger than 2.5 Hz and short distances, for the reasons discussed previously. The remaining weighting value is subdivided between recorded data and empirical relations with a preference for the former ones except when they are showing very low values due to

Table 3 Comparison of the PSA values (g) obtained from numerical simulations, recorded data and empirical relations for M = 7 and R = 5 km

f(Hz)	Simul. URS-Pitarka	Simul. OGS-Priolo	Recorded × 1.3	Emp. Relat. + 3 σ	Emp. Relat. + 2.5 σ	Max	Min	Weighted value
0.5	1.50	0.29	0.43	2.18	1.55	2.18	0.29	1.21
1.0	3.00	1.20	1.30	5.00	3.61	5.00	1.20	2.78
2.5	6.00	1.91	3.45	7.08	4.93	7.08	1.91	4.88
5.0	8.00		3.26	5.83	4.19	8.00	3.26	5.92
10.0	5.50		2.98	4.31	3.69	5.50	2.98	4.42
20.0	4.50		2.20	3.33	2.50	4.50	2.20	3.45
PGA	3.50		1.63	2.13	1.67	3.50	1.63	2.55

a lack in the corresponding M-R bin. It is worth to note that the numerical simulations provide always the highest values for frequencies larger than 2.5 Hz.

For application in seismic hazard assessment as well as in the simulation scenarios, upper bounds need to be expressed not as single extreme values but rather as continuous functions of the explanatory variables used in ground-motion prediction equations, e.g. magnitude and distance. As discussed previously the results of numerical simulations, and consequently estimates like those shown in Table 3, were only available for two magnitudes and distances (M = 7 and 5.5; R = 5 and 25 km). The recorded data too, do not cover continuously the M-R space. To extrapolate the estimates at different M-R values, the chosen approach is to use the empirical attenuation model of Ambraseys et al. (1996), corrected for Mw-Ms and for larger to geometrical mean horizontal component (Sabetta et al., 2005), and incremented by different fractions of standard deviation. As shown in Fig. 5 for Mw = 7 and f = 1 Hz, a good match of minimum, average weighted value, and maximum upper bound values (Table 3) is obtained using respectively 1.7, 2.8, and 3.6 sigma of the Ambraseys model.

Figure 6 is analogous to Fig. 5 but it refers to a lower magnitude and higher spectral frequency ($M_w = 5.5$ f = 10 Hz). In this case a higher fraction (4.2) of standard deviation is required to match the maximum values (red squares) at 25 km distance. It is interesting to note that, even for such a small magnitude, at zero distance an for a spectral period of 0.1 sec, the estimate of the maximum physically possible ground motion is of the order of 1–5 g.

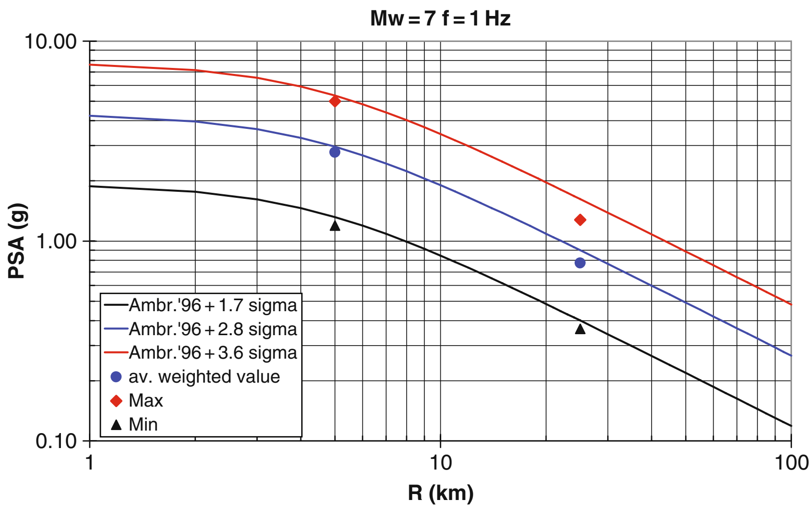

Fig. 5 Upper bound values (*solid symbols*) taken from Table 3 and analogous table for R = 25 km, compared with Ambraseys et al. 1996 relation incremented by different fractions of standard deviation

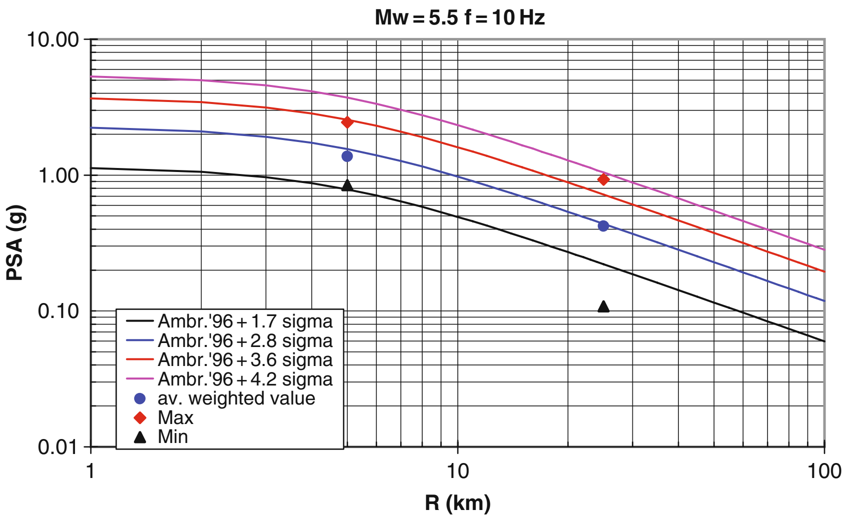

Fig. 6 Upper bound values (*solid symbols*) for Mw = 5.5 and f = 10 Hz, compared with Ambraseys et al. 1996 relation incremented by different fractions of standard deviation

6 Conclusions

In this paper different approaches have been explored in order to give an order of magnitude to the value of the maximum physically possible earthquake ground-motions on rock site conditions. In particular the results provided by recorded accelerograms, empirical prediction relationships, and numerical simulations, explicitly performed to evaluate the worst case scenario, were examined. The analysis took into account different magnitudes and distances and both PGA and response spectral values at several frequencies. The results show that, in case of high magnitudes and short distances, the numerical simulations provide always the highest values for frequencies larger than 2.5 Hz. In particular the max PGA for $M_w = 7$ and R = 5 km corresponds to 3.5 g. The empirical prediction relationships, incremented by 3 standard deviations, provide the highest values at low spectral frequencies, whereas the recorded accelerograms, even if conservatively multiplied by a factor of 1.3, never reach the maximum values. Finally, the values provided by the different approaches for specific magnitude-distance scenarios, can be extrapolated to any M-R value adopting the empirical prediction relationship of Ambraseys et al. (1996) incremented by about 3 standard deviations. This article is off course not intended to give definitive answers but just to stimulate the discussion, as suggested by the paper of Bommer et al. (2004) on the challenge of defining upper bounds, and to provide first numerical results.

Acknowledgments I thank all the management of the PEGASOS project and in particular Philip Birkhauser, Jim Farrington, René Graf, Andreas Hölker, Philippe Roth, Patrick Smit and Christian Sprecher for providing a stimulating environment and continuous support for the development of ideas presented herein. Special thanks are also due to Julian Bommer, Hilmar Bungum, Fabrice Cotton and Frank Scherbaum for the numerous and stimulating discussions.

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Suboceanic Rayleigh Waves in the 1755 Lisbon Earthquake

A. Vuan, A. Rovelli, G. Mele and E. Priolo

1 Introduction

SRW are large-amplitude, long-duration surface waves generated by shallow seismic sources in the ocean. These waves result from the coupling of crustal Rayleigh waves with acoustic water waves across the seafloor (Ewing et al., 1957). They propagate efficiently along oceanic paths and are well recorded at stations inland. SRW have been commonly observed for long distance paths (> 2000 km) and their effect has never been considered in earthquake engineering so far. However, recent observations in peninsular Italy during moderate earthquakes in the southern Tyrrhenian Sea (Rovelli et al., 2004) demonstrate that SRW attain large amplitude and long duration also at regional distances (300–2000 km).

In this study, we investigate moderate-size oceanic earthquakes recorded by the broadband station PMST in Lisbon to find the imprint of SRW. To confirm our interpretation we compare the data recorded from Tyrrhenian Sea earthquakes with PMST data by applying a frequency-time representation of the signals. Finally, we perform some simple numerical simulations to evaluate (i) if wave propagation in 1-D crustal and upper mantle models, including a water layer, can returns the amplitude and long coda of observed SRW, and (ii) if a possible occurrence of SRW during the Great 1755 Lisbon earthquake can explain the high level of damage and duration of shaking felt in southern Portugal.

2 Analysis of Recent Earthquakes in the Atlantic Ocean

We collected digital waveforms from moderate-size earthquakes that recently occurred in the Atlantic ocean along the Azores-Gibraltar Fracture Zone (AGFZ), recorded at station PMST in Lisbon (Fig. 1). Earthquake parameters are listed in Table 1. Figure 2 shows examples of vertical component

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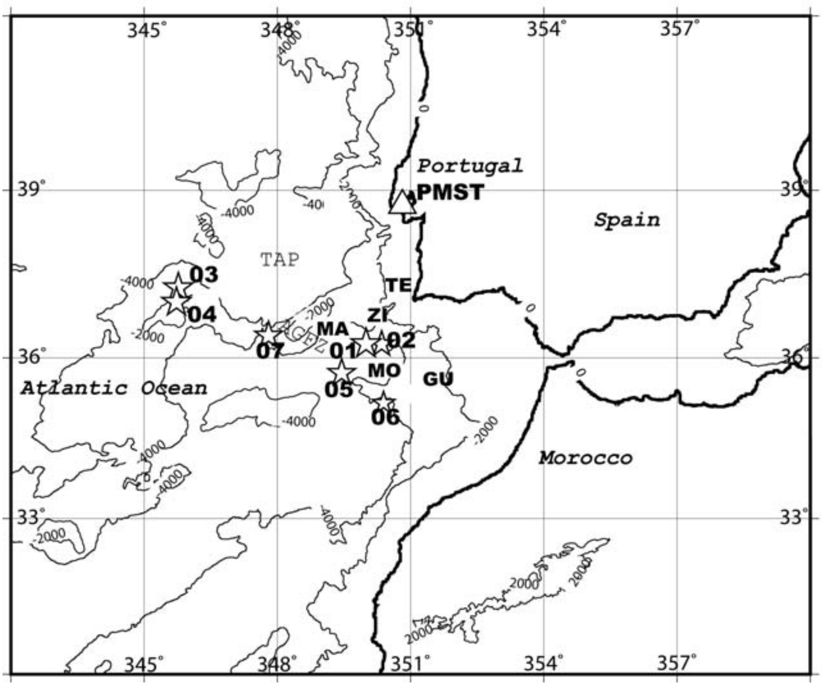

Fig. 1 Map of the Atlantic Ocean, the eastern Iberia and southwestern Africa showing the epicenters (*stars*) of seven earthquakes (see details on Table 1) and the PMST seismic station that recorded the events (*triangle*). Events 03 and 04, both occurred in the southwestern part of the Tagus Abyssal plain (TAP), generated suboceanic Rayleigh waves, whereas the other events did not. The 2000-m and 4000-m isobath is contoured in the Atlantic Ocean (TAP = Tagus Abyssal Plain, AGFZ = Azores Gibraltar Fracture Zone). Indicative epicentral locations of the 1755 earthquake proposed by different authors (GU = Gutscher et al., 2006, MA = Machado, 1966, MO = Moreira, 1989, TE = Terrinha et al., 2002, ZI = Zitellini et al., 1999)

Table 1 Events located off-shore in the Atlantic ocean and recorded by the PMST broad-band seismic station. Earthquake parameters are from ISC and EMSC (D = Depth(km), M = Magnitude, DIS = Distance(km))

#	DATE	HOUR	LAT	LON	D	M	DIS
01	2004/06/20	03:01:16.10	36.2475	−9.6545	10	4.6Ml	279
02	2004/12/13	14:16:08.59	36.2550	−10.001	10	5.0Mw	285
03	2006/01/10	01:09:34.19	37.0570	−14.133	10	5.5Mw	473
04	2006/01/09	16:40:44.26	37.0500	−14.121	10	5.3Mw	474
05	2006/06/21	00:51:15.60	35.7230	−10.545	33	4.8mb	356
06	2006/07/07	13:12:57.90	35.2000	−9.6075	40	5.1mb	394
07	2006/09/19	23:28:16.10	36.4000	−12.200	104	5.0mb	372

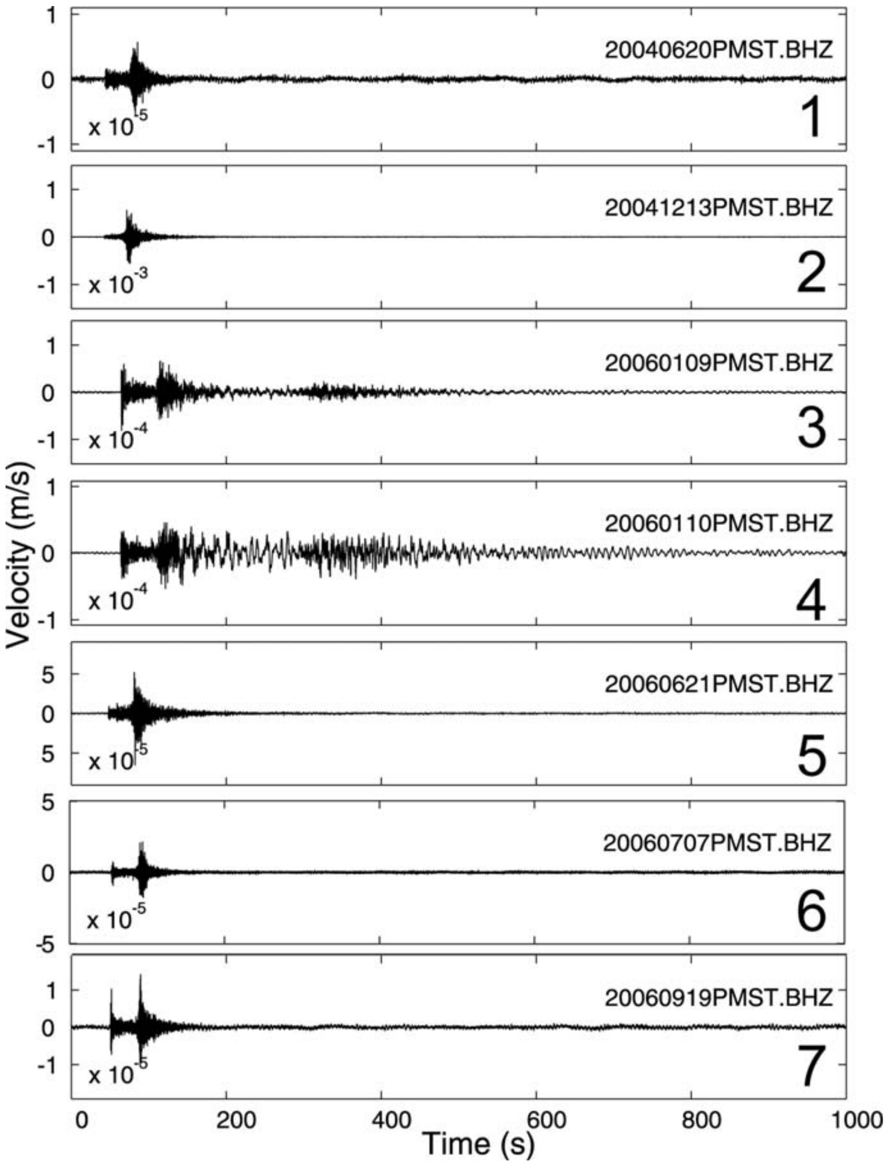

Fig. 2 Vertical components of PMST station recorded during the events listed in Table 1 and mapped in Fig. 1. Numbers on the *right* of each panel correspond to the event number of Table 1. Events number # 3 and 4 show a long duration compared to the other events

broad-band seismograms recorded by PMST. The station is managed by the Instituto Superior Tecnico of Lisbon and data are freely available through ORFEUS facilities. Seismograms of events #3 and 4 of Table 1 (see Fig. 2) are characterized by a long coda composed of dispersed wave trains with a predominant long-period component. They look as double events because of the arrival of short-period (0.5–2 Hz) T-phases about 280 s after the earthquake origin time. Seismic source (especially for event # 4) was beneath the seafloor, in the south western margin of the Tagus Abyssal Plain (TAP), 470 km offshore Lisbon, and large part of the propagation path was characterized by the presence of a water layer across TAP.

The other events (# 1, 2, 5, 6 and 7), mainly located in the southeastern part of AGFZ, are not characterized by long duration. This fact can be explained by considering that waves travel along different paths – continental (# 1, 2, 5, 6 and 7) versus oceanic (# 3 and 4).

The 2002, Mw 5.9 Palermo earthquake, occurred 40 km offshore the northern coast of Sicily, represents a well documented case of efficient propagation of SRW at regional distances (Rovelli et al., 2004). Seismic source was beneath the seafloor at shallow depth and large part of the propagation path was characterized by the presence of a deep water layer across the Tyrrhenian Sea. The availability of very-broadband recordings of the September 6, 2002 earthquake at AQU station, in central Italy, at an epicentral distance of 440 km, allowed Rovelli et al. (2004) to investigate the dispersion of the SRW in the Tyrrhenian Sea.

Since recordings of event # 4 in the Atlantic ocean and the 2002 Palermo earthquake have similar oceanic propagation path, source-receiver distance, and source depth, we performed a frequency-time analysis to evaluate if they present common features in the period-group velocity window interested by SRW.

Figure 3 shows a comparison between the vertical and radial component signals filtered by applying a Butterworth band pass from 15 s to 4 s. Because of the different event size (Mw = 5.5 and Mw = 5.9 for the Atlantic ocean and the Tyrrhenian Sea earthquakes, respectively), amplitudes are different but the vertical and radial components of the two events have a similar long-duration coda. If these signals are compared in the period-group velocity domain we can retrieve, about SRW, the same features described in Rovelli et al. (2004). Figure 4 shows a comparison of the period-group velocity diagrams for the two vertical components shown in Fig. 3. Radial component diagrams are similar and do not add more detail to the analysis.

Group velocity measurements are obtained from source-to-receiver distances of 470 km and 440 km for PMST (Lisbon) and AQU (L'Aquila station), respectively. Dispersed surface waves arrive to seismic stations at a time that depends on the wave frequency. To estimate the group arrival time as a function of frequency, the original seismogram is filtered using narrow-band Gaussian filters, and the arrival time of the peak of the filtered signal envelope is used to estimate the group-delay time. Envelope amplitudes of AQU diagram (bottom panel of Fig. 4) at periods shorter than 4 s – 5 s are negligible in comparison with the large amplitudes shown at longer periods.

Large envelope amplitudes are found in the period range from 5 to 15 s and within the group velocity window from 1.3 to 2.5 km/s in both PMST and AQU

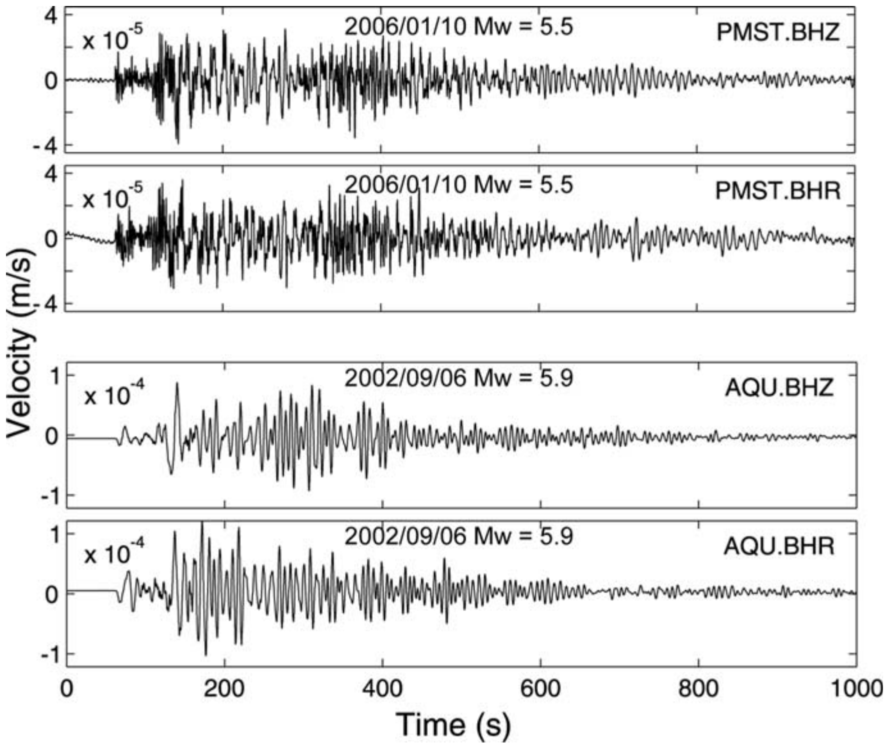

Fig. 3 (*Top panels*) vertical and radial component records of the PMST and (*bottom panels*) AQU stations, in Portugal and central Italy, respectively. Time series are filtered with a Butterworth band pass filter from 4 to 15 s. The source-to-receiver distance is 474 and 440 km, respectively

diagrams. Waves generated from scattering and multipathing effects are generally observed in this window. It is often difficult to interpret sonograms and detect a specific surface wave dispersion curve since different modes overlap in the same narrow band. Moreover, for mixed oceanic-continental paths, the coherency of surface waves can be destroyed at short periods by lateral inhomogeneities. However, in the sonograms of AQU, Rovelli et al. (2004) were able to distinguish and interpret the fundamental mode group velocity shape in the short-period band.

By modelling the 1-D wave propagation in the Tyrrhenian Sea Rovelli et al. (2004) observed that (1) synthetic seismograms generated for a 1D model of the Tyrrhenian basin reproduce the amplitude and duration trend of observations in the period band from 5 to 10 s and, (2) the displacement eigenfunctions of the fundamental mode at 4 and 6 sec indicate that the largest vertical and radial amplitudes are confined in the uppermost 4 km confirming that short-period crustal Rayleigh waves are coupled with acoustic water waves across the

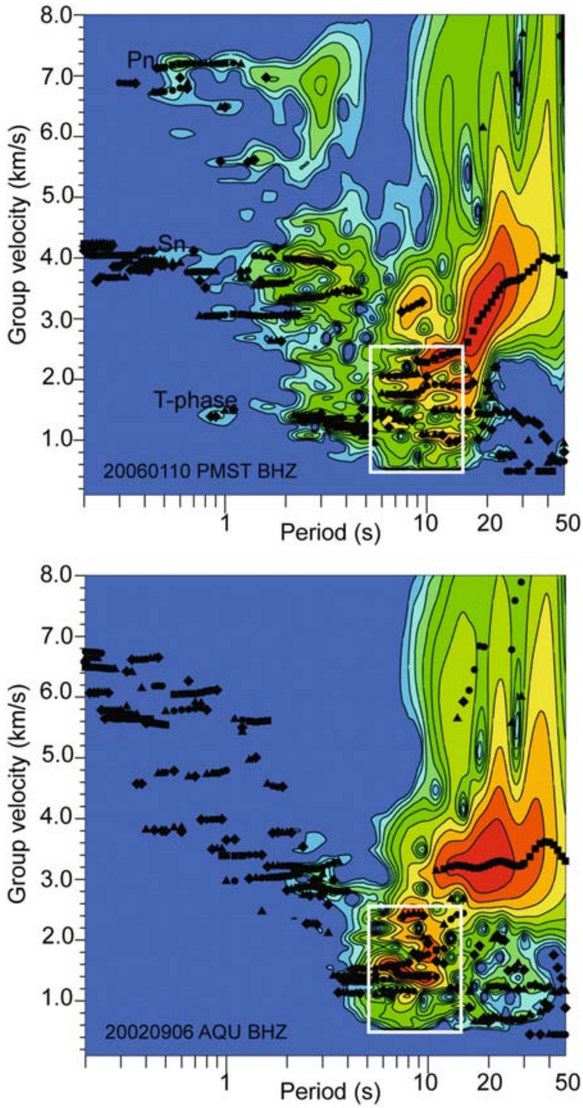

Fig. 4 Group velocity – period diagrams of two oceanic earthquakes. In the top panel the diagram of the 2006/01/10 Mw = 5.5 seismic event (# 4 in Table 1) recorded by the PMST station in Lisbon. In the bottom panel the diagram of the 2002/09/06 Mw = 5.9 seismic event (offshore Sicily) recorded by station AQU in central Italy. The source-to-receiver distance is 474 and 440 km, respectively. Color scale represents the spectral amplitude in dB as a function of group velocity and period (*red* = 100 dB, *yellow* = 60 dB). The *black* color symbols represent the maximum peaks

seafloor. Because of the similarity in the diagrams of Fig. 4 in the period band of interest, we assume that the same is true for the Atlantic Ocean earthquakes recorded by PMST (events # 3 and 4 in Table 1).

The other events we analysed (# 1, 2, 5, 6, 7) do not show period-group velocity diagrams consistent with those presented in Fig. 4. In Fig. 5 is shown the vertical component period-group velocity diagram of event # 2, that we considered representative of this subset. A strong Sn phase at periods shorter

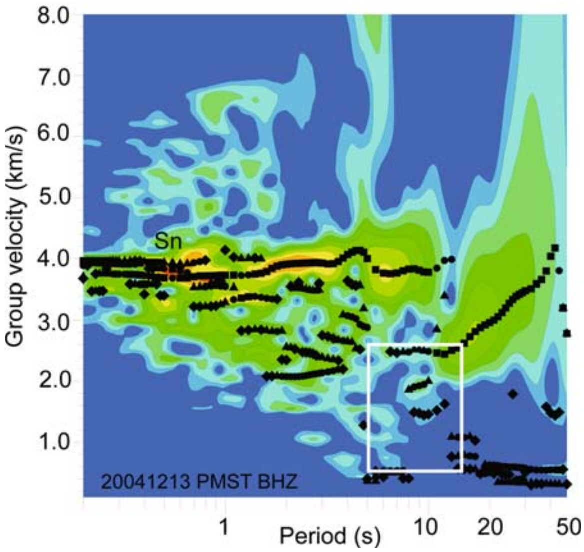

Fig. 5 Group velocity – period diagram of the 2004/12/23 Mw = 5.0 seismic event (# 2 in Table 1) recorded by the PMST station in Lisbon. The source-to-receiver distance is 285 km. Color scale represents the spectral amplitude in dB as a function of group velocity and period (*red* = 100 dB, *yellow* = 60 dB). The *black* color symbols represent the maximum peaks.

than 4 s and group velocity of about 4.0 km/s is observed. No SRW are evidenced at lowest group velocities.

3 The 1755, Mw 8.7 Lisbon Earthquake: Inferences and Preliminary Modeling

A role of SRW on increasing hazard at regional distance has never been considered so far. The amplitude of horizontal (radial) ground displacement recorded at PMST during the 2006/01/09 earthquake (Fig. 6) suggests that, up to magnitude 6, amplitudes are out of the range of engineering interest. Figure 6 also shows, in the inset, a prediction of the SRW amplitude at larger magnitudes. For the sake of simplicity, predictions are scaled according to a single-corner-frequency omega-squared source model; more complex, double-corner-frequency models would give slightly smaller values at intermediate periods (5–30 s). The results of Fig. 6, although based on the rough approximation of a point-source model, give an important indication: during large magnitude (> 8) earthquakes, SRW might reach amplitudes of engineering interest even hundreds of kilometres away from the epicenter. Moreover, the huge number of cycles is itself a reason of concerning, especially for large structures with narrow resonance peaks. The combination of even moderate amplitude with large duration could make the global effect devastating.

According to the observations shown before the Great 1755 Lisbon earthquake could have generated this type of waves: (i) its magnitude has been estimated as large as 8.7; (ii) the seismogenic zone and a large part of the

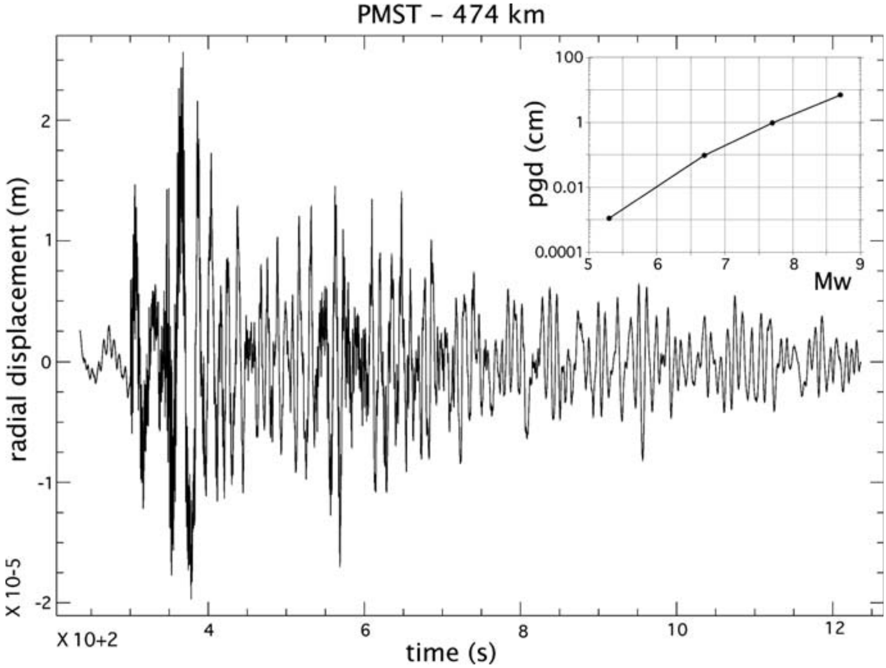

Fig. 6 Horizontal (radial) displacement recorded at station PMST during the 2006/01/09 earthquake, highpass filtered at 0.02 Hz. Note the long duration of the strongest motions. Predicted scaling of amplitude at larger magnitudes is shown in the inset. At Mw 8.7, peak-to-peak amplitude of horizontal ground motion is predicted to be as large as 7 cm

propagation paths are in the ocean, in a region where the average thickness of the water column is about 2 km; (iii) the wide damaged region and the level of destruction, especially on low-frequency structures, are typical of efficient propagation of surface waves at regional distances. A similar selective effect was also experienced in Mexico City, more than 300 km away from the 1985 Michoacan earthquake (Singh and Ordaz, 1993).

Historical chronicles report shaking durations of tens of minutes inland in southern Portugal, with two main zones of damage more than 200 km apart along the coast. The occurrence of multiple earthquakes has been invoked to justify the pattern of damage and the long duration of shaking but we believe that both of them could be also explained in terms of slowly propagating dispersed wave trains, as those shown in Fig. 3. The high percentage of large buildings that were destroyed in Lisbon (Chester, 2001) plays in favour of a leading role of SRW.

In this preliminary step, we have generated synthetic seismograms from 1D models where focal mechanism, source depth, and Vs and Qs of the seafloor are varied. Synthetics were generated through a wavenumber integration method (Herrmann, 2002). This method solves the 3-D full-wave equation in anelastic media with a vertically heterogeneous structure for a point source. The elastodynamic equation is written in cylindrical coordinates and then decomposed in the domain of angular frequencies and complex wavenumbers. This method is accurate and synthesizes realistic seismograms which include all wavefield phases.

For a preliminary analysis we use a simple 1D reference oceanic model. Vp/Vs and Qp/Qs ratios are fixed to 1.73 and 2, respectively. Even though a 1D structure seems too simplistic, we believe that it can be helpful in explaining and interpreting the gross features of the waves under investigation.

Without considering complex 3-D wave propagation (e.g. surface wave multipathing) wave propagation in 1-D models, including a water layer, is able to explain to some extent the observed amplitude and long coda. In Fig. 7 we show the match between synthetic and observed signals for the event # 4 (470 km from Lisbon) in the period band from 50 s to 5 s. Focal mechanism parameters, depth and scalar moment are taken from Harvard CMT catalogue (http:www.seismology.harvard.edu/CMTsearch.html).

The proposed epicenters of the Great 1755 Lisbon earthquake (Fig. 1) span over 600 km, a surprising uncertainty in view of a well-documented damage caused by this earthquake (Fonseca, 2005). For this study we assume a hypothetical source in the Atlantic Ocean, 300 km southwest of Lisbon, and a thrust faulting mechanism associated with the compressive structures of the Gorringe Bank (Buforn et al., 1988; see Fig. 1). This choice could be questionable. However, our parametric study shows that efficiency of excitation of SRW is not sensitive to variations of the focal mechanism.

Figure 8 summarizes the role of the other model parameters on the development of SRW. Curves in these figures are obtained by dividing the Fourier spectrum of the radial component of models with a water layer (1 and 2 km thick) by the Fourier spectrum of the radial component of the corresponding model without water. These spectral ratios quantify the effect of mode coupling along the seafloor on the ground motion at regional distances. In general, we observe a significant amplification in frequency bands of engineering interest depending on the choice of the model parameters.

Source depth and Qs are also varied. Source depth has a significant role on the generation of SRW: shallowest sources are much more efficient.

The largest effects are found when the water layer is 2 km thick (Fig. 8a) and source depth is very shallow (4 km, see Fig. 8b). Therefore, the excitation of SRW will be stronger during large crustal earthquakes, when the fault rupture reaches the surface.

According to Butler and Lomnitz (2002), a crucial role is played by Vs: when Vs of the seafloor tends to the acoustic speed in the water (Fig. 8a), the signal appears to be a composite of undispersed Rayleigh modes propagating along the ocean floor both in the sediments and in the water.

Finally, a variation of Qs from 500 to 250 decreases amplitudes by a factor of 2, approximately, in the frequency band from 1.5 to 2.5 Hz. However, it does not affect significantly the lower frequencies (Fig. 8b).

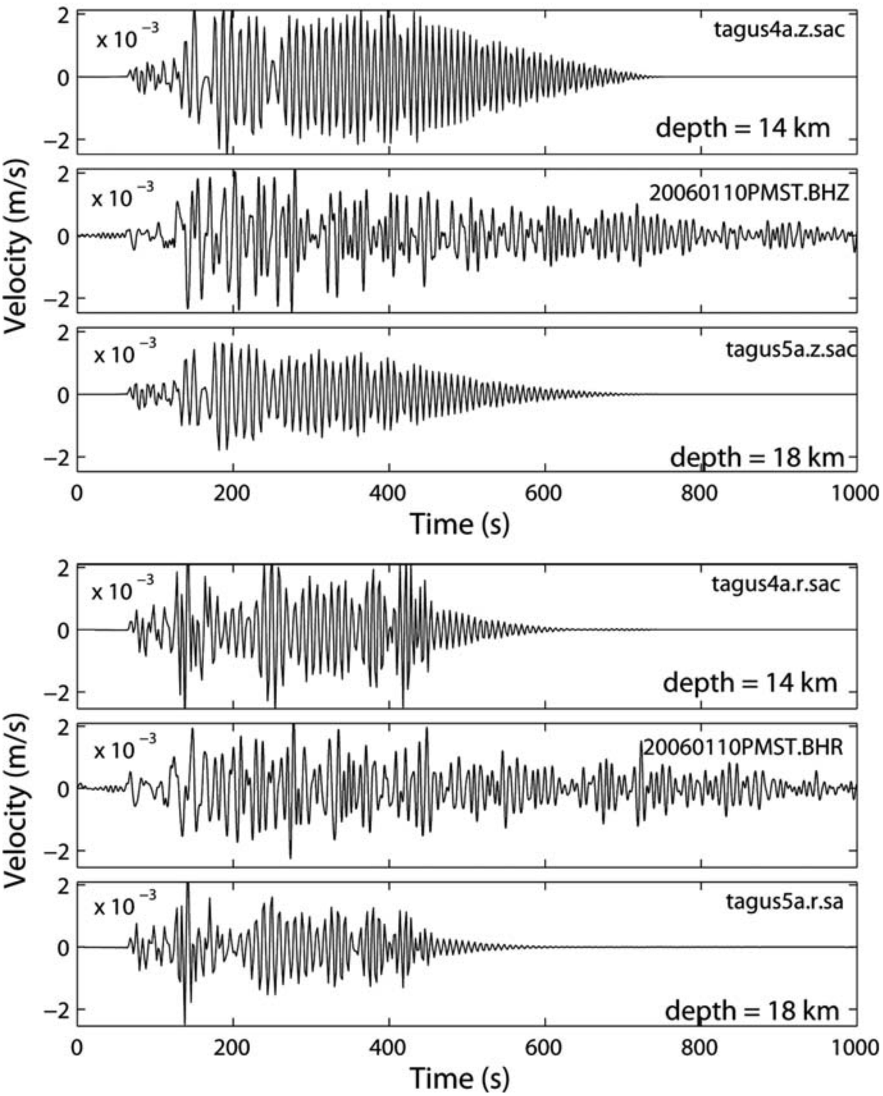

Fig. 7 (*Top panels*) vertical component and (*bottom panels*) radial component synthetic signals computed for a source depth of 14 km and 18 km (tagus4a and tagus5a, respectively) in comparison with the # 4 event (20060110PMST) of Table 1 . Waveforms are band pass filtered from 50 s to 5 s

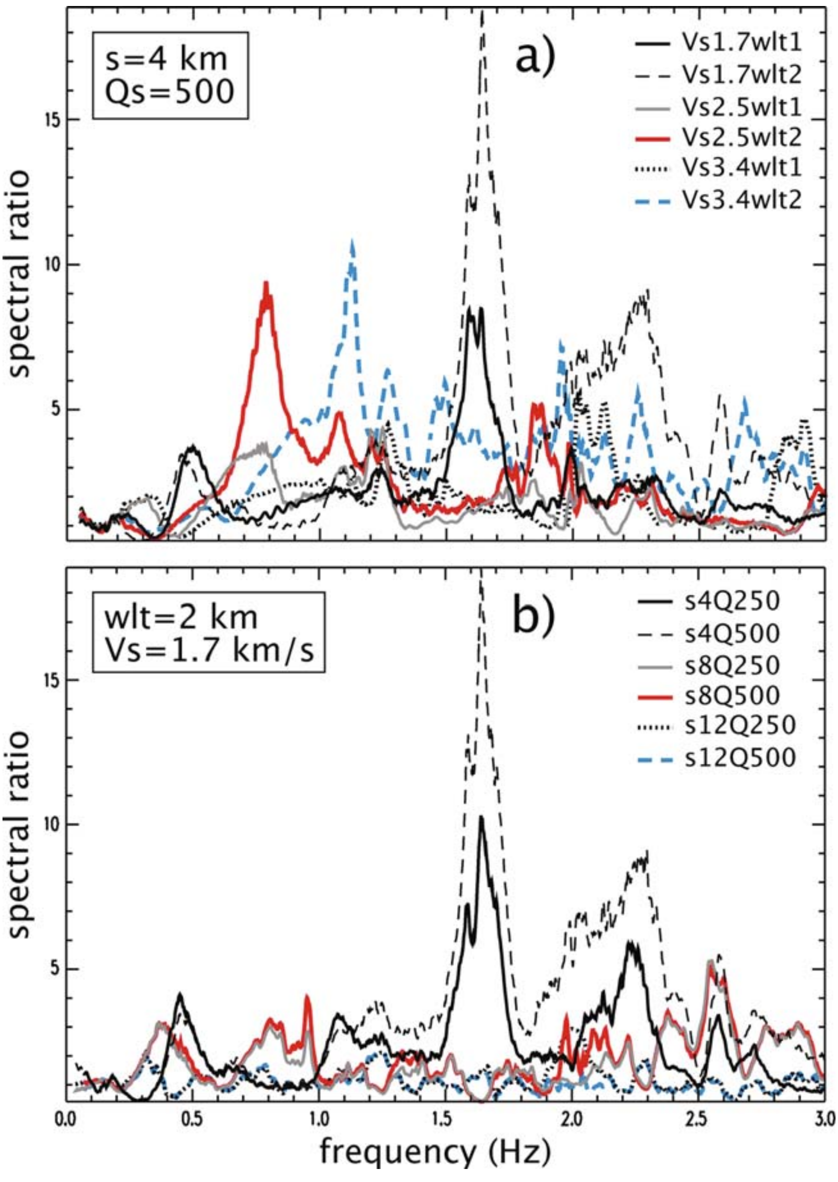

Fig. 8 Amplification of horizontal ground motion produced by the propagation of SRW in 1D models for an epicentral distance of 300 km. The reference of the spectral ratios is the Fourier spectrum of radial synthetics when no water layer is put in the model. (**a**) Source depth (s) and Qs are fixed at 4 km and 500, respectively; shear-wave velocity Vs and water depth (wlt) are varied in the modeling. The strongest effects occur when Vs is close to the sound speed in the water and the liquid layer is 2 km thick. (**b**) Water depth and Vs are fixed at 2 km and 1.7 km/s, respectively

4 Concluding Remarks

Observations in inland Portugal and Italy for shallow moderate-size earthquakes occurring beneath the seafloor indicate that, under specific conditions, SRW propagate efficiently at regional distances (> 300 km). Both seismograms and 1D numerical modeling yield duration of shaking of a ten minutes and amplitudes larger than those of models without column water. Timing of the SWR arrivals would explain many features of the peculiar damage pattern in south Portugal, including the wide region of destruction along the coast and the high level of damage on large-size structures in Lisbon (monumental buildings, churches, aqueducts, etc.). However, in terms of absolute amplitudes, point-source and 1D structures are oversimplifications of the reality not applicable to the 1755 Lisbon earthquake. More refined, extended-source and 3D models, based on the observations here analyzed, are in progress to achieve estimates of ground motion usable for engineering evaluations.

Acknowledgements We thank Bob Herrmann for providing his wavenumber integration code and Joao Fonseca and the Instituto Superior Tecnico of Lisbon for making available PMST seismic data. GMT package (Wessel and Smith, 1991) is used for drawing some pictures.

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Contribution to the Damage Interpretation During the 1755 Lisbon Earthquake

M. San-Payo, I. Moitinho de Almeida, P. Teves-Costa and Carlos S. Oliveira

1 Introduction

On the 250th anniversary of one of the largest earthquakes known, it is necessary to continue a series of studies, which have been developed in order to extract information contributing to not only a deeper understanding of the phenomena, but also to define an efficient prevention and action strategy in the case of large scale catastrophes.

Under those circumstances, a documental search on historical seismicity has been conducted, particularly related to the reports of damages caused in Lisbon by the November 1st, 1755 earthquake.

Using new Geographic Information Systems technologies, a GIS is being developed and assembled, with parish boundaries at the time of the earthquake, location of monuments, description of damage suffered, construction dates, and type of construction. As base information for the development of this Geographic Database, the research conducted by Pereira de Sousa (1909, 1928) and Oliveira (1982) constitutes the main source.

The result of the spatial analysis among the several levels of information present in the Database, as well as the comparison of that information with surface geology and soil frequencies may allow a better understanding of seismic behavior of that type of buildings.

2 Historic Framework

The magnitude and extent of the 1775 Earthquake led the Marquis of Pombal, among many other measures, to order an inquiry in the country, on the consequences of that event. In this inquiry, completed by the priests, the number of victims and ruins were assessed for each one of the Lisbon parishes. Some

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contemporaneous authors made an excellent effort to compile the damage data (Moreira de Mendonça, 1758; Castro 1762–1763).

It is mainly based on those reports that Pereira de Sousa (1909, 1928) develops a compilation of damage on monuments, performing, at the same time, a close correlation with the geological formations, over which they were built. Pereira de Sousa uses the Lisbon Map by Filipe Folque at the scale of 1:10.000 to mark the location of 150 monuments and present a damage classification. The monuments described are mainly Churches, Monasteries and Palaces. This author classifies the damages in 3 classes, having produced a damage distribution map (Fig. 1).

The analysis and study of this inquiry are not terminated and continue to be of great interest to the study of the seismic phenomena in Portugal and forecast of their consequences.

In the 1980's, Oliveira (1982) continues the research and systematization works of data relative to the damages caused on the buildings by the earthquake. Using an extensive bibliographic list (Portugal and Matos, 1974; Moreira de Mendonça, 1758) with a main focus on the work of Pereira de Sousa (1909, 1928), he produces a data file and presents a new damage classification. Traditional data organization and recording methods are used in that work, that is paper files and manual cartography over the same Filipe Folque cartographic base.

Oliveira proceeds to the construction of a database with approximately 400 files with the description of damage in several types of buildings, including Churches, Palaces, Monasteries, Hospitals, Fortresses, Towers and Bridges.

Monuments distributed by 43 parishes are also studied in this work, 39 of which being, at that time, integral part of the urban area and the other 4 corresponding to the parishes of the farms, which today are part of the Lisbon District. The type of information gathered, more oriented towards the description of main buildings, either for their cultural, religious and/or economical importance, doesn't contemplate descriptions of damages occurred on residential buildings.

A numerical identification code was assigned to each monument, and later associated additional information regarding its history, implantation site, type of structure and damage suffered during the earthquake. The buildings were located manually over Filipe Folque 1:10.000 cartographic base, and colored according with the degree of damage (Oliveira, 1982).

Through computing and digitalizing, the new GIS technology makes possible to exploit this large amount of information, providing an additional value to that work.

In this way the 1755 Earthquake Geographic Database (BDG1755), together with surface geology and soil frequency data, enables a better knowledge of the seismic phenomena in Lisbon. It is also intended to use this study to calibrate the model parameters used in earthquakes simulators of the Lisbon Civil Protection.

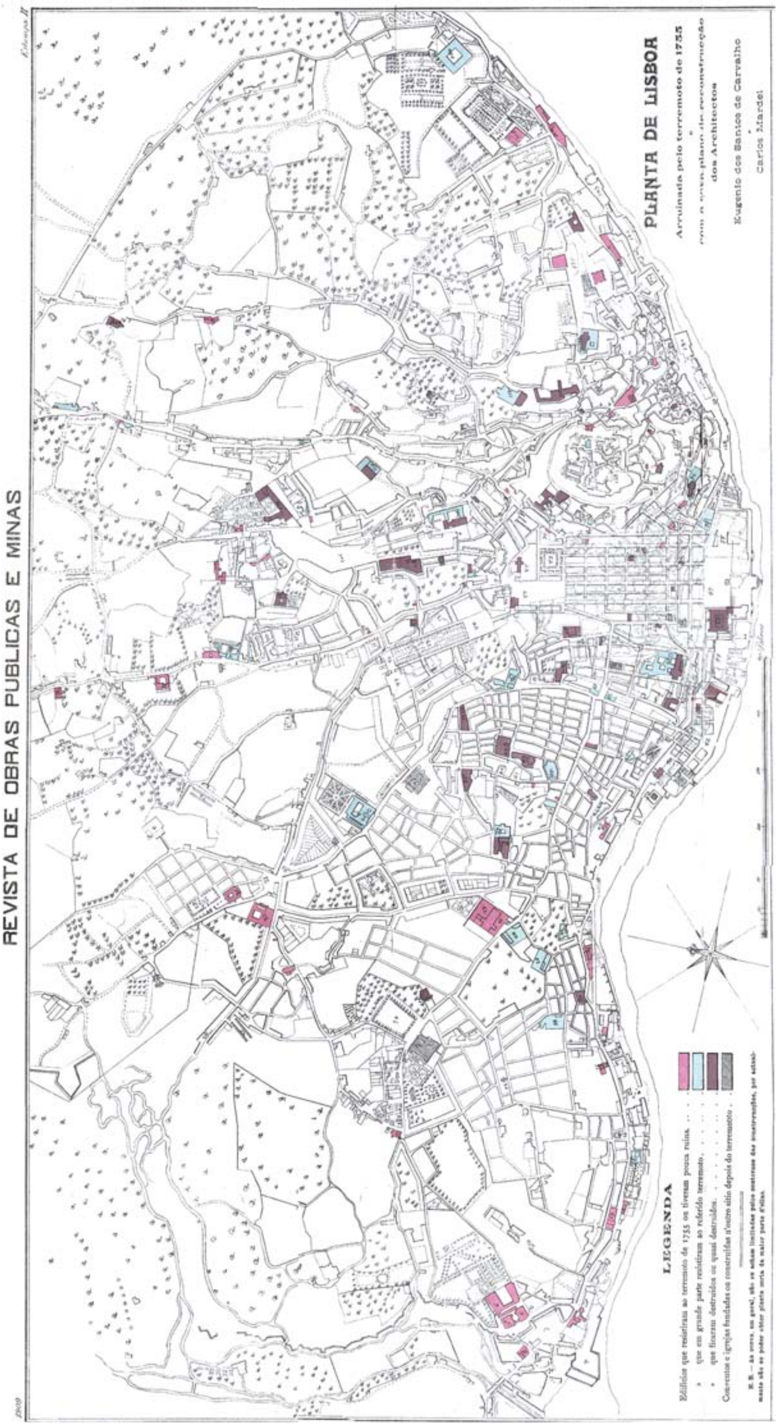

Fig. 1 Map of the damaged monuments in Lisbon (Pereira de Sousa, 1909)

3 Methodology

3.1 Research and Information Recording

The information researched for this initial assembling phase of the BDG1755 was found and obtained on the personal archives of Carlos Sousa Oliveira and Fernando Moitinho de Almeida, and also the Libraries and Archives of the Lisbon City Council (CML).

The bibliography and data gathered encompass a relatively large historic period (XVIII, XIX and XX centuries). The data set was grouped under two fundamental themes: Parishes and Monuments.

3.1.1 Parishes

Regarding the parish information at the time of the Earthquake, it was found that there were differences among the various sources in what concerns their designation and limits.

In fact, the city of Lisbon went through successive reforms during the period studied, which produced some obstacles to the task of reconstructing the historic limits existing at the time of the earthquake (Alves, 2004).

The descriptions and maps of the Lisbon parishes in accordance with the parochial remodeling of 1770 (Santana, 1975) was used to draw the parish limits. Note that, although those limits do not coincide with the 1755 limits, this document constitutes the best approximation available. An example of those maps is presented in Fig. 2.

Since it is necessary to know this data with better accuracy, it is intended to continue the research, namely at the National Archive (Torre do Tombo) and other historic archives, to confirm the parish limits at the time of the Earthquake.

The digitalization process entailed several precautions. As it is known, the city network suffered modifications during the past centuries, and the cartographic production methods at the time of the earthquake were very primitive and inaccurate.

For that reason, it was selected as base map for 1770 limits identification the 65 sheets of the Filipe Folque Topographic Map (1858) at the 1:1.000 scale, since the objects represented on those sheets, chronologically closer and prior to the strong urban expansion phase, allow a better approximation than the current CML digital cartography. In Fig. 3 it is shown the implantation of the 1770 limits, over the Filipe Folque map sheets.

The 65 sheets were digitalized and later geo-referenced to the Hayford Gauss – Datum 73 (HGDt73) Cartographic Projection System, to superimpose them on the urban digital cartography.

Then, by visual comparison between the 1770 map drawing and the mosaic built with the 65 geo-referenced images, the parish limits were digitalized.

Fig. 2 Map of Nossa Senhora da Encarnação parish in 1770 (Santana, 1975)

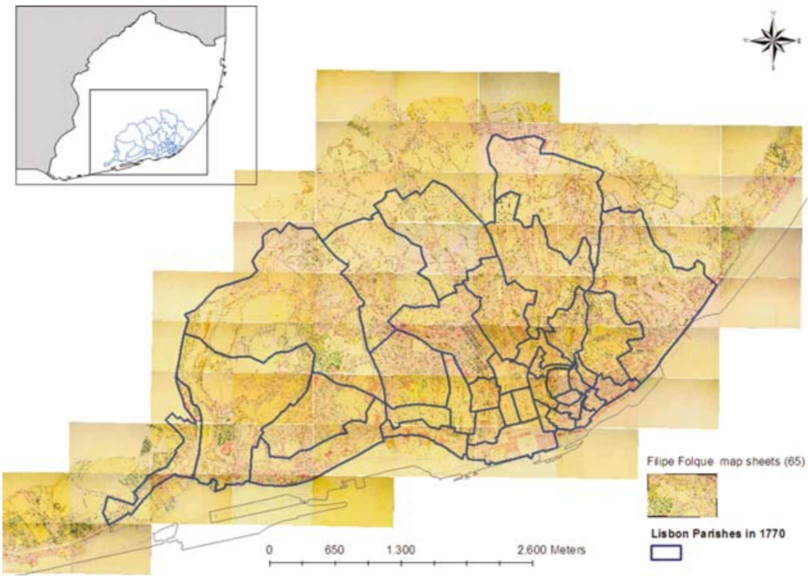

Fig. 3 Parish limits in 1770 over the 1:1.000 Filipe Folque (1858) map sheets mosaic

In this way, an approximation was obtained of what the parish limits in 1770 would have been. Despite of the errors, which naturally occurred, a reasonable approximation has been obtained of the limits at that time.

3.1.2 Monuments

To digitalize the monuments with records of damage caused by the 1755 earthquake, two data sources were used: the Pereira de Sousa (1909) publication and data compiled by Oliveira (1982).

As reference digital cartographic base, we used the Lisbon street centerlines and polygons of the Municipal Heritage Inventory Map. This map contains, in addition to the monuments classified by the Portuguese Institute for the Architectonic Heritage (IPPAR), many others considered of patrimonial interest by the CML.

In order to locate some monuments not identified in the Municipal Heritage Inventory Map, because of their inexistence nowadays, other cartographic sources were used for instance Vieira da Silva (1950, 1987a,b).

The monuments were represented by polygons, since that is the format adopted by the Municipal Heritage Map, and because its behavior analysis could be better understood in the case of large footprint buildings and/or ones developed over different lithologies.

Based on the descriptions and all the historic and current cartographic documents previously referred, the several city monuments were localized and coded.

3.2 Production of the Geographic Database with the 1755 Earthquake Damage (BDG1755)

The production of the BDG1755 had an initial phase necessarily constituted by:

- evaluation and analysis of the available data;
- cartographic database identification of the historic base documents;
- choice of cartographic base to be used;
- definition of minimal cartographic unit to use for information gathering;
- selection of cartographic projection system to be used;
- definition of entities and respective attributes.

For acquisition and digitalization on the BDG1755 of the two fundamental entities obtained from scratch, that is, the Parishes of that time and the Monuments, it was selected and organized the geographic data sets (spatial and attributes) to be compiled in different layers or themes.

To introduce all this data into the geodatabase it was necessary to organize the data as showed in Table 1.

With regard to the acquisition of the spatial component of those two entities, the methodology and process have already been referred on the previous paragraph.

Table 1 Data colleted for each Parish and Monument

Themes/Layers	Atributtes
Parish	Number of people in 1758
	Number of buildings in 1755
	Number of monuments
	Damages
	Seism intensity (Pereira de Sousa)
Monument	Description/type of damage in 1755
	Date of construction
	Date of reconstruction
	Local geology
	Bibliographic references

For the digitalization of the attribute component, considering its extension and the format of the available archive, it was adopted the scanning of the approximately 400 paper files from Oliveira (1982) and next perform a character recognition process, to create digital documents with the information of each parish and monument.

Note that it was not possible to obtain all the attributes elements for all buildings considered.

After the input of all this data on BDG1775, several thematic maps were created, for example those showed in Figs. 4 and 5, presenting respectively the monuments with the damage degree and the monuments affected by the fire.

Fig. 4 Map of monuments damage degree in the downtown area

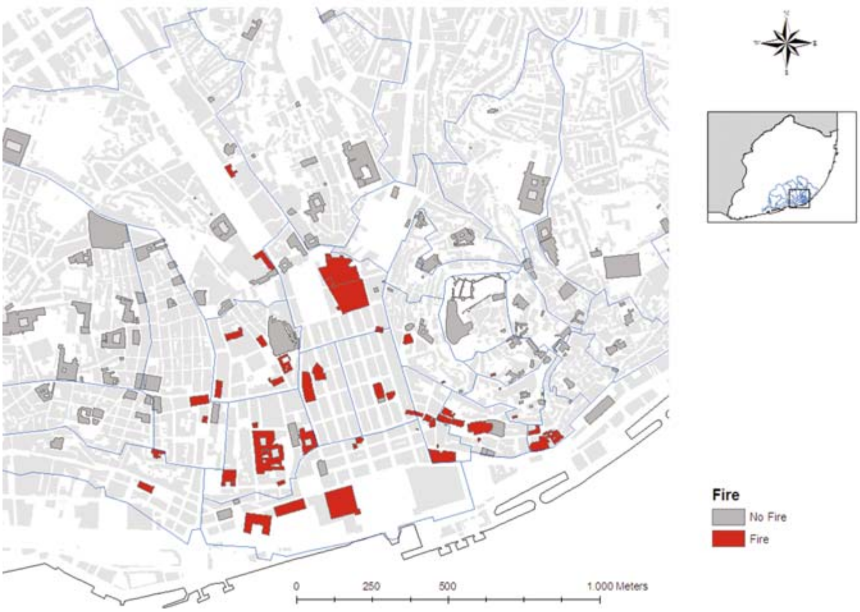

Fig. 5 Map of monuments affected by the fire in the downtown area

3.3 Characterization of Surface Geology and Site Effects

Since the objective is to know the global damage inflicted by the earthquake and understand the building behavior considering the site geology, the Lisbon Geological Map (Moitinho de Almeida, 1986) was integrated on the BDG1755. From the analysis of that map and taking into account the similar lithologies and age formation, as well as their physical properties (Teves-Costa et al., 2001) the following lithological groups were defined (Fig. 6):

Alluvia;
Miocene sands;
Miocene clays and limestones;
Rocks

Subsequently, the themes "type of monument damage" and "lithological groups" were overlayed. The statistical analysis of this data allowed investigating the correspondence between the damage type and the lithostratigraphy (Fig. 7).

It can be observed that the largest percentage of collapses occurred on the alluvia (class E damages), whereas the monuments erected on rock suffered

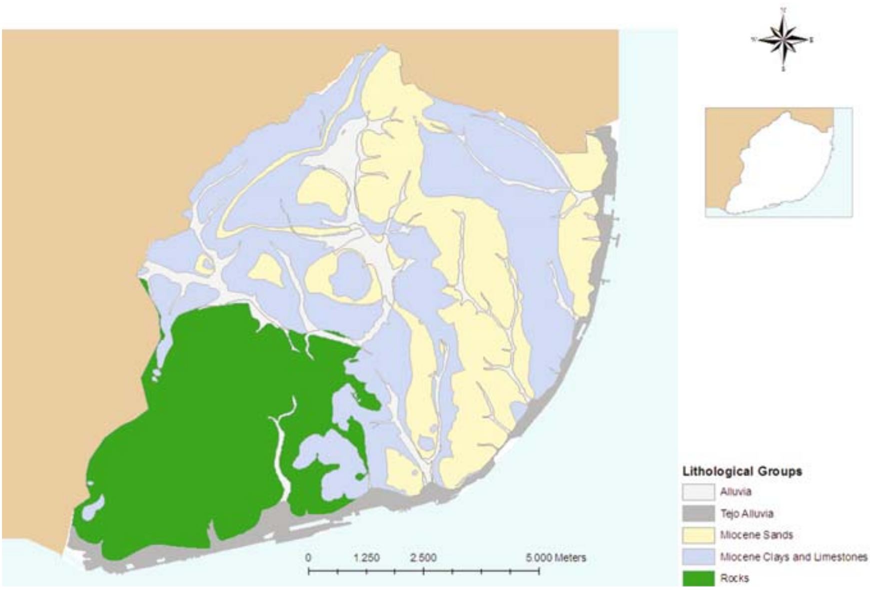

Fig. 6 Simplified geological map of Lisbon

only light and moderate damage (class A, B and C damage). On the intermediate formations (miocene sands and miocene clays and limestones), damage distribution is very similar, although a little over 50% of the monuments built on those formations have suffered severe damages and collapse. In Fig. 7 it is presented the number of monuments studied in each lithological group, to show the number of affected buildings and thus validate this statistical analysis.

After making the damage distribution analysis over the surface geology, we proceed to the analysis of predominant soil frequencies in order to determine the possible occurrence of resonance phenomena. As it is known, each building has its own natural resonance frequency; if a certain building is erected over a formation with a frequency close to its natural frequency, it can start resonating and will suffer very severe damage or collapse.

The predominant frequency map of Lisbon (Fig. 8) determined from the analysis and treatment of environmental vibration recordings (Teves-Costa et al., 1995) was used to read the soil vibration frequency distribution.

This map was geo-referenced to allow correlation between the values of soil frequency with other parameters already introduced in the database. The first analysis consisted on the comparison between surface geology and soil vibration frequency (Fig. 9).

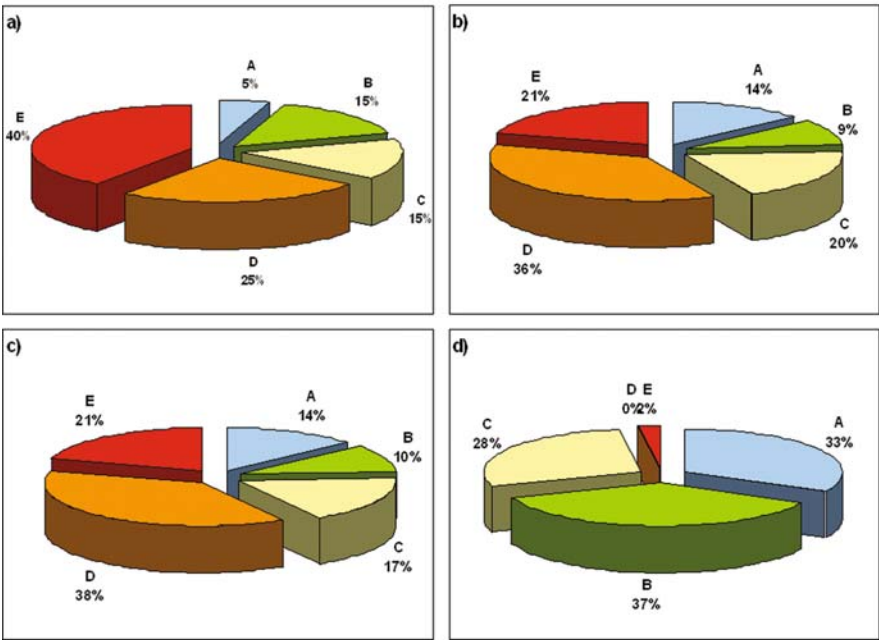

Fig. 7 Damage distribution by lithological group: (**a**) Alluvia (40 monuments); (**b**) Miocene sands (107 monuments); (**c**) Miocene clays and limestones (58 monuments); (**d**) Rocks (43 monuments)

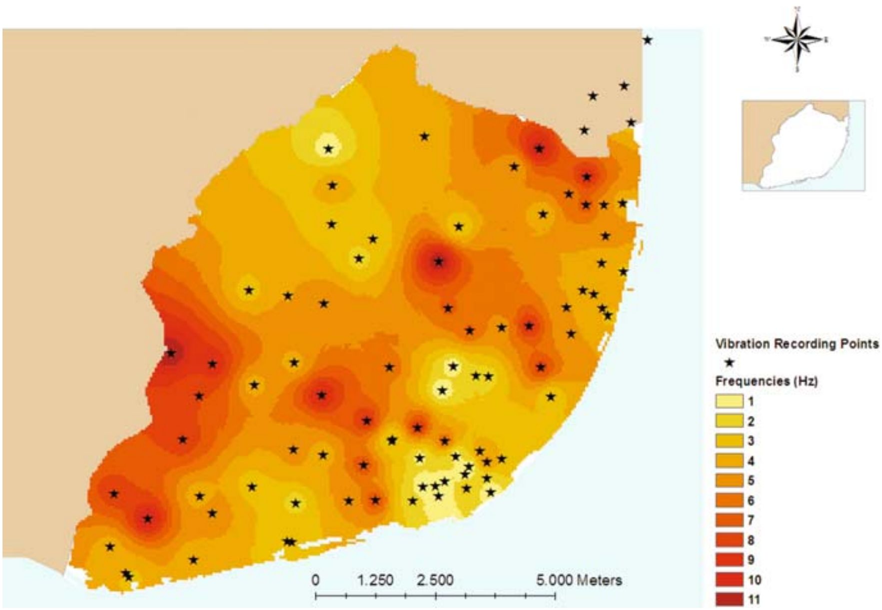

Fig. 8 Lisbon soil frequency map

4 Conclusions

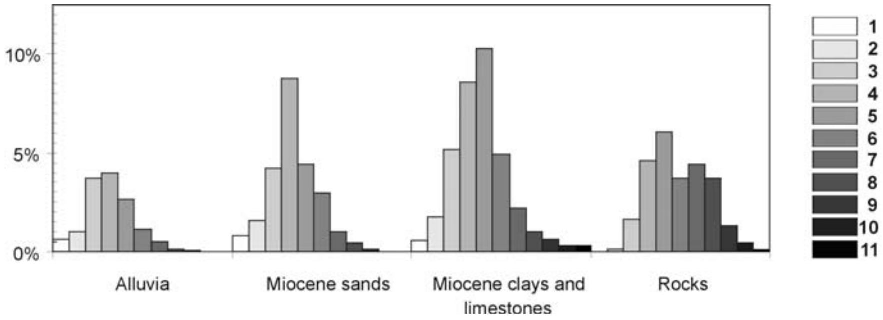

Fig. 9 Dominant frequencies distribution by lithological group

This work presents a first contribution for a new approach for the study of site effects observed in the 1755 Earthquake. Its continuation will allow better clarification of, not only soil seismic behavior, but also the behavior of existing buildings of that time.

The identification process of the monuments at the time of the earthquake on current cartography is not yet concluded. It is a time consuming process, requiring historic investigation and some field work, since many of the documented buildings no longer exist.

Once this process is concluded, we plan to continue data analysis and interpretation of the damages inflicted by the earthquake on the buildings, as a function of surface geology and topography.

It is also intended to revise the values of soil frequency and correlate them with the frequency of the different monuments.

Once the studies and analysis are concluded, their results will constitute a source of pertinent information to continue with the calibration of the model parameters used in earthquake simulators, developed in the CML Civil Protection Department, to estimate the damage distribution in the city.

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Part VI
How to Build Earthquake Resistant Buildings Under the Environmental Constrains

Caveats for Nonlinear Response Assessment of Shear Wall Structures

P. Gülkan

1 Introduction

Recent focus on the use of simplified procedures in performance-based earthquake engineering has led to comprehensive research resulting in improved techniques applicable for buildings with generally regular geometrical and structural features. These methods are intended to provide basically an adequate level of equivalent linearization applicable to these types of systems so that the calculated nonlinear deformations match results calculated for linear systems of varying complexity. It should be recalled that all linearization techniques are designed to minimize the error in this calculation. In the final analysis the performance of performance based methods has not been tested by nature, so the goodness of a given method is assessed against results calculated for another method or technique. In this article we examine the accuracy of a number of commonly used methods in calculating the response of a short-period structural mockup that had been previously tested on a shake table. Our blind predictions of its response were successful, so this has encouraged us to use many more naturally recorded ground motions to calculate its response with use of a number of currently used methods and compare this with a fully nonlinear analysis.

2 Antecedents

The International Atomic Energy Agency (IAEA), in cooperation with the EU Joint Research Center (JRC), has initiated research to investigate the safety implications of near-field earthquakes on nuclear facilities. This calls for a critical assessment of the validity of displacement-based procedures for stiff structures that are typical for nuclear sites. These investigations stem from the need to develop reliable guidelines for safety re-evaluation of existing nuclear

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structures. A coordinated research project (CRP) on safety significance of near fault earthquakes was launched by IAEA in 2002. The objective of this CRP has been to propose the most appropriate earthquake engineering practice to assess the seismic vulnerability of typical structures in nuclear facilities subjected to the effects of near-fault earthquakes. The research was crafted to use experimental data available from earlier investigations. A series of benchmark shaking table experiments had been carried out in the Saclay Nuclear Center in France in 1997. One particular specimen from that program, CAMUS1, has been re-studied in the CRP organized by IAEA. This specimen is a 1/3-scale model of a representative 5- story reinforced concrete building detailed according to current French practice (Combescure, 2002). It is considered a typical example for a stiff structure, but its reinforcement details are more typical of residential construction.

In the first phase of the investigations, a reliable and representative analytical model of the tested specimen was developed, based on the accurate duplication of physical conditions and loadings imposed during the laboratory tests. The experimentally measured results have been predicted analytically with convincing accuracy as presented in (Kazaz et al., 2005). This article is complementary to the first phase. It deals with the analytical assessment of the seismic response of the CAMUS1 structure under a suite of 55 ground motion records. The ground motion set selected for the study contains far- (FFE) and near-field (NFE) earthquake records on firm soil sites where nuclear power facilities are typically built. The near-fault records used in this study do not necessarily exhibit the forward directivity characteristic, i.e. a long-duration pulse of high velocity dominating the event. Only a few records we use contain such dominant velocity pulses.

The response of the structure calculated using nonlinear response history analyses is considered to be "exact," because the analytical model we have constructed has successfully predicted the observed response (including local strains, curvatures and shear forces or moments) of the specimen on the shaking table in the preceding stage. The structure is then re-analyzed using approximate static procedures. The results are examined to evaluate accuracy and validity of the approximate nonlinear static analysis procedures for similar types of structures. Most nonlinear response procedures are based on the dynamics of SDOF systems, so this exercise also provides an understanding of the degree of extrapolation that is acceptable in arriving at estimates of the response of MDOF systems.

3 Analytical and Experimental Model

The analytical model used in the nonlinear dynamic analyses is a realistic duplication of the CAMUS1 specimen used in the experimental program. In all of the following discussion, these results will be considered as "exact" for comparison with analytical results.

The experimental program consisted of testing a 1/3-scale representative component of a 5-story reinforced concrete shear wall building on shaking table at Commissariat a l'Energie Atomique (CEA) in the Saclay Nuclear Center. The specimen, named CAMUS1, had a total mass of 36 tons with additional masses attached to it. The walls had no openings, and were linked by square slabs (1.7 × 1.7 m). A heavily reinforced concrete footing allowed anchorage to the shaking table. The total height of the model was 5.10 m. They had a width of 1.7 m and thickness of 6 cm. The specimen had a measured fundamental natural frequency of 7.24 Hz. The dimensions and the mass distribution of the specimen are shown in Fig. 1. The experimental study provided the measured response quantities of the model shear walls subjected to different input seismic motions. The walls were loaded in their own plane. The input motions were representative of near fault as well as of far fault ground motions selected for the purpose of comparing any effects such different ground motions may produce on the structure.

A finite element model of the tested shear walls as shown in Fig. 2 was created. The actual material properties and boundary conditions in the experiment were implemented in the model to reflect the required aspects of the test specimen. The finite element model of the test specimen has a computed fundamental frequency of 7.28 Hz. The same loading sequence was applied to the model.

The experimentally measured and numerically computed response quantities including top story displacement, base shear, bending moment at the base, top

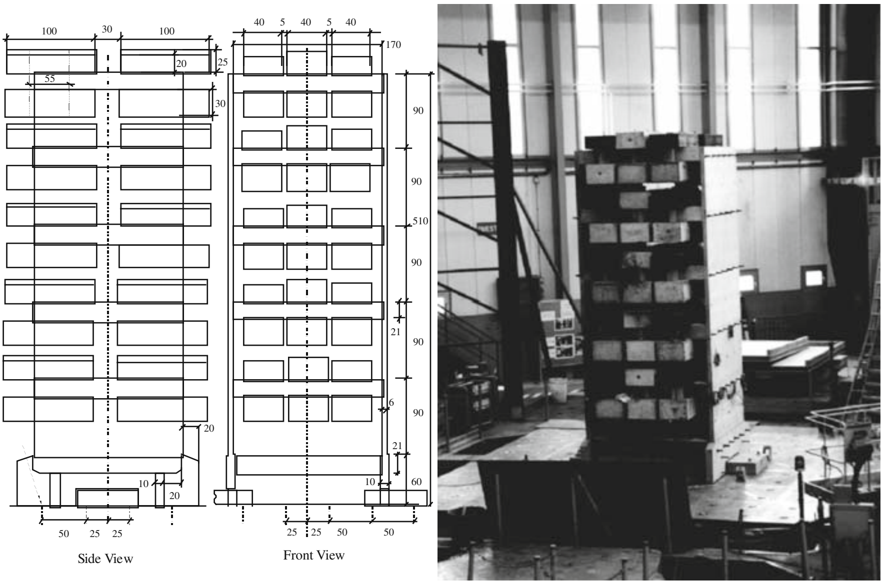

Fig. 1 CAMUS specimen

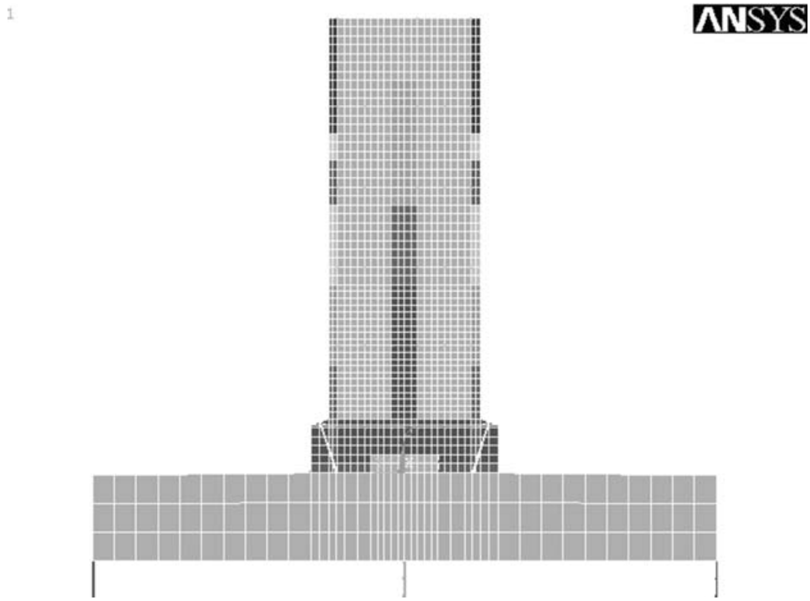

Fig. 2 Analytical model (ANSYS Engineering Analysis System, 2002)

story horizontal accelerations and the local results such as strains were found to be in very good agreement. The results, given in detail elsewhere (Kazaz et al., 2005), clearly indicate that the analytical model developed here is able to display the inelastic response of the tested specimen quite satisfactorily. The same model has been employed in this study to perform further analyses for a suite of 55 ground motions that are described next.

The selected ground motion set consists of 55 records obtained from 20 earthquakes of which 31 are near-field seismograms. The database was intended to cover both NFE and FFE records. These ground motion records were classified according to their site-to-source distance based on the recommendations given by (Martinez-Pereira and Bommer, 1998). The records used in the analyses were intended to reflect the characteristics of firm site ground motions, i.e., wave forms rich in high frequency content and effective in the short period range on a narrow period interval (0.1–0.4 s). High frequency components of ground motions tend to attenuate rapidly with distance and hence are not observed at stations located even a few tens of kilometers from the fault rupture. Some records were generated by scaling the original ground motions. Since the model is a 1/3 scale of a real structure, the ground motions used in the analyses were also scaled in the time axis by a factor of $1/\sqrt{3}$. Ground motions named as Run1 through Run4 are signals that were used in the shaking table experiments. Run1 is a synthetic ground motion and Run2 is the 1957 San Francisco Earthquake recorded in Golden Gate Park. Run3 and Run4 were obtained by scaling Run1 and Run2, respectively. The rest are natural records.

4 Analytical Results

The selected ground motions were applied to the model and various nonlinear response parameters from time history analyses were determined. Correlation of typical strong motion parameters with the calculated response quantities of the structure was investigated first. For the selected ground motion database, as evidenced from Fig. 3a,b, no clear trend was observed between the structural deformations and the strong ground motion parameters such as PGA and PGV for either type of ground motion. Nevertheless, since the largest induced force is related to PGA for stiff structures, it correlates better than PGV with top displacement. Another commonly used ground motion intensity measure is the spectral acceleration at the fundamental period of the structure, Sa (T1). The spectral acceleration reveals a loose correlation for the top floor displacement and the base shear as shown in Fig. 3c,d, the dispersion being smaller for base shear. In Fig. 3c,d the yield displacement and base shear of the structure are shown in the respective square. The dispersion is more significant as the

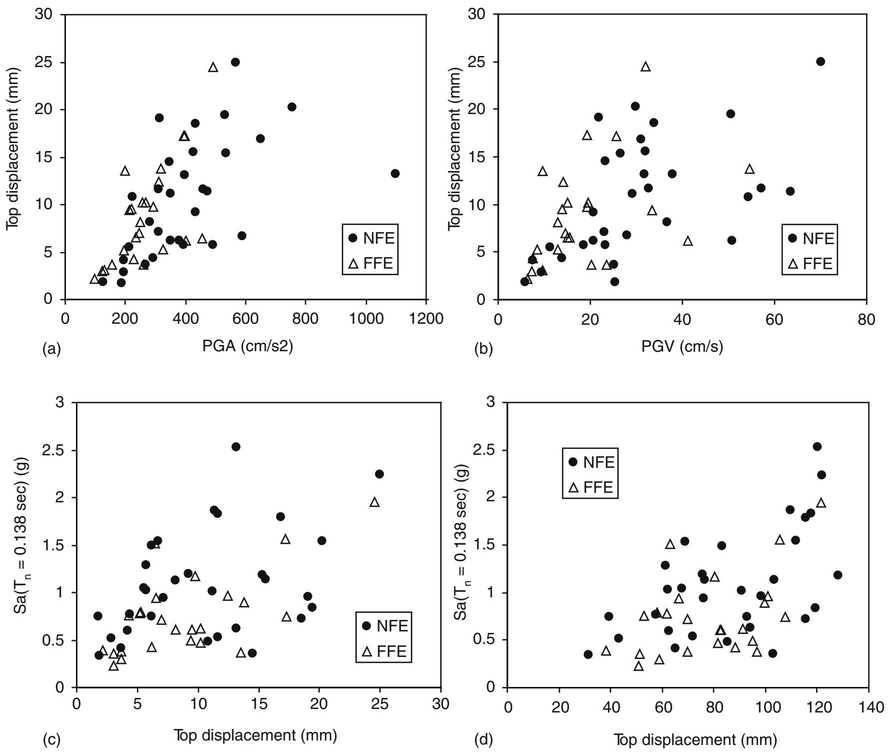

Fig. 3 Correlation of top-level displacement with (**a**) PGA, and (**b**) PGV, and Correlation of response parameters with Sa (Tn), (**c**) Max. Roof Displacement, (**d**) Max. Base Shear

structure responds in the inelastic range due to the effect of strength and stiffness degradation.

4.1 Pushover Analysis

Full nonlinear time history analysis for a multi-storey building is the principal tool that provides the most intimate insight about its response to earthquake excitations. A complete time history analysis is quite demanding and computationally expensive as compared to static analysis. As an alternative, nonlinear static pushover analyses are carried out by applying lateral forces at the mass locations of the structural system, assuming that they will account for the distribution of inertia forces acting at the story levels during the dynamic excitation of the structure. This procedure can provide considerable insight for the nonlinear behavior of the structure although it conceals unresolved uncertainties and approximations. The load patterns that are typically used in the pushover analyses are calculated by utilizing modal analyses of the structure. The first mode shape or a combination of modes is used as the representation of the dynamic loading. In the elastic range of the dynamic excitation, results obtained with these prescribed load patterns agree with the exact solution in many cases.

The CAMUS structure was analyzed next using the pushover analysis procedure to compare with the "exact" results. Three distinct vertical distributions of lateral loads were applied. In the first loading shape called modal push pattern, a vertical distribution of lateral forces proportional to the shape of the fundamental mode in the plane of the shear wall was used. Next, a triangular lateral load pattern representing the contribution of each story mass to the inertia force relative to the sum of inertia forces was utilized. Lastly, a uniform distribution consisting of lateral forces at each level proportional to the total mass at these levels was used. The pushover curves of these three different loadings are plotted in Fig. 4 which also displays the results of nonlinear dynamic analyses from the ground motions contained in the dataset.

As seen in Fig. 4 for structures dominantly responding in the fundamental mode, the modal load pattern seems to serve as the lower bound for the seismically induced base shear, and the uniform load pattern acts as an upper bound for the same parameter. It was observed that for the ground motion records that exhibit long acceleration pulses where the duration of the pulse is larger than the natural period of the structure, a higher inelastic displacement demand is induced on the structure (Anderson and Bertero, 1987). Furthermore we observed that acceleration spikes of high amplitude and short duration induce high base shear demand on the structure. The points that are above the upper enveloping curve in Fig. 4 belong to ground motions supporting our observation: Run2, NS component, Ito-Oki EW component, Tabas and Friuli NS component.

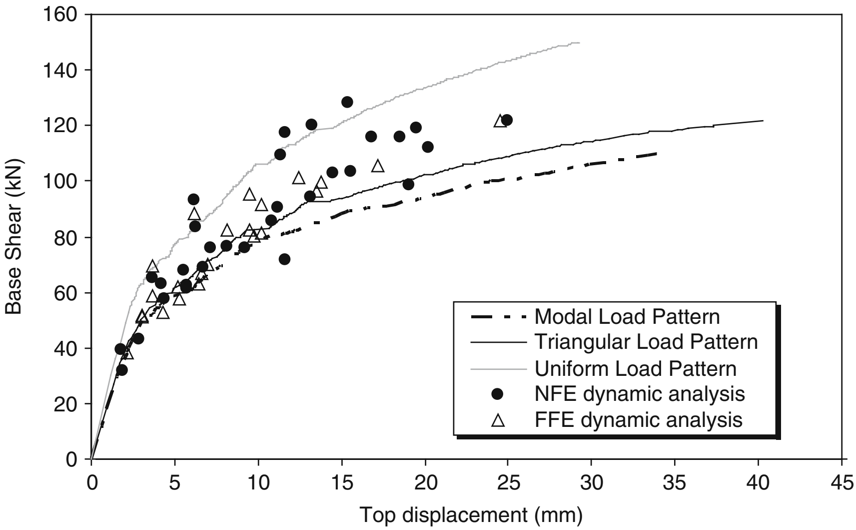

Fig. 4 Pushover curves compared with dynamic analysis results

4.2 Linear Time Integration Analyses

Linear time history analyses were carried out to investigate their accuracy in estimating the inelastic deformation demands. Time history results for roof displacement of the linear model are plotted against the nonlinear time history results in Fig. 5.

The results presented in Fig. 5 confirm that as the nonlinearity and ductility demand increases in the system the ability of linear analysis in predicting the inelastic top displacement diminishes. For stiff and short period systems, such as considered here, if the period of the structure is much shorter than the predominant period of the input record, then the equal displacement rule does not hold. In fact, the nonlinear analyses of the test structure showed that, the inelastic model produced greater deformations than did the corresponding elastic model except in a few cases, which is consistent with the generally accepted wisdom that derives from single degree of freedom analyses. This follows from the fact that when a system with a short initial period yields, its period elongates and shifts closer toward the predominant period of the ground motion. The exceptions are those cases where the spectral acceleration ordinates corresponding to undamaged (elastic) state are much higher than the values corresponding to softened state, leading to higher deformations even if the structure behaves elastically. This observation is valid for moderate degree nonlinearity and is strongly dependent on the base shear yield strength of the structure.

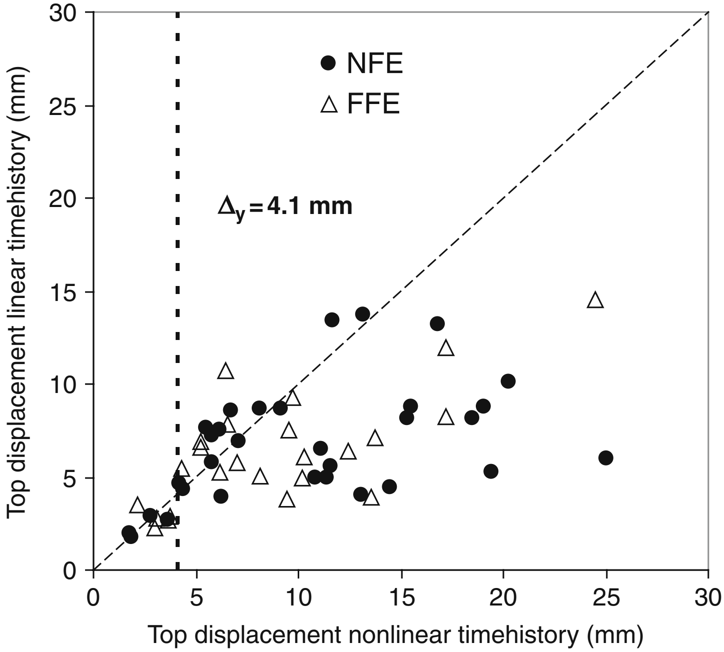

Fig. 5 Linear vs nonlinear top displacement time history analysis

5 Displacement Based Procedures

The motivation for research dealing with the development of simplified procedures that are used to estimate the inelastic displacement demand of structures has essentially been to find an alternative approach as substitute to nonlinear response history analyses that involve analytical complexity and computational expense. These simplified procedures generally rely on the reduction of MDOF systems to equivalent SDOF representations. The two most commonly used procedures of this nature are the Capacity Spectrum Method of ATC-40 (1996), and the Displacement Coefficient Method contained in FEMA 356 (2000). These procedures have been evaluated by many researchers, highlighting their weaknesses as well as their adequacy. Sometimes conflicting findings have led to uncertain and incompatible conclusions on their range of validity. Unlike many previous investigations that either deal with SDOF systems (Miranda, 2001; Miranda et al., 2002; Chopra et al., 2003) or generic MDOF systems(Chintanapakdee and Chopra, 2003), this study evaluates the accuracy of these procedures for the CAMUS1 model that has been studied experimentally and analytically. It is based on nonlinear response history analyses under a comprehensive suite of ground motions. In addition, the nonlinear analyses of the equivalent SDOF systems and a proposed modification to the CSM have been carried out to test their success in matching the exact results.

5.1 SDOF Analyses

The inelastic response (generally the roof displacement with which damage may be associated) of a MDOF system can be estimated from the corresponding equivalent SDOF system in varying degrees of accuracy depending on the particular ground motion used in the analysis and the structural properties of the MDOF system. There is a divergence between the ductility demands imposed on multi-storey buildings and SDOF systems. Two particular parameters give rise to the differences between the "input" and "output" ductility demands (deformations) of SDOF and MDOF systems. These are higher mode contributions and inter-story drift demands (local response behavior of MDOF system) (Chopra, 2000). These two characteristics cannot be incorporated directly into the structural characteristic of a SDOF system, where they are subsumed in a single bilinear force-deformation relation. So in cases where the contribution of these two parameters to the structural response is limited or negligible a good estimation of global deformation demand of a MDOF system can be obtained, otherwise the contribution of these effects must be taken into account with certain correction coefficients.

A nonlinear SDOF system with bilinear force-deformation relation with some post elastic-stiffness can be described completely with the following parameters; T (elastic period), ξ (damping), either of m (mass) or k (stiffness of the system in the elastic range), $\eta = f_y/W$ (yield base shear coefficient), and α (post elastic stiffness coefficient). Yield base shear coefficient or base yield strength (f_y), given in Equation (1) is determined in the design stage for a desired ductility level under a postulated ground motion effect that defines the constant ductility response spectrum.

$$f_y = \frac{A_y}{g} W \tag{1}$$

To determine the response of the CAMUS structure by employing equivalent SDOF systems, a representative SDOF model of the MDOF structure was obtained. The accuracy of this approximate procedure depends strongly on how well various structural aspects of the MDOF are represented by the corresponding SDOF system. Examination of the pushover curves obtained for the three aforementioned lateral load patterns revealed that the triangular load pattern provides the best representation of the dynamic behavior for all ground motions except those with high amplitude acceleration spikes, so it has been used for further analyses. It is also important to note that the change in the initial slope of pushover curve due to bilinearization also requires a change in the natural period of the system that is modified with $T_e = T_n.(K_i/K_e)^{0.5}$.

The following steps were implemented to convert bilinear MDOF system's capacity curve to that of SDOF counterpart.

(1) In constructing a relation between the MDOF system and its equivalent SDOF system, a widely accepted procedure that is based on the fundamental mode properties of the MDOF system was used. The horizontal axis of the load deformation curve describing the global roof displacement was divided by the factor PF1 which is the modal participation factor for the first mode obtained by assuming that the mode shape is normalized to unity at the top of the structure, Equation (2).

$$\mathrm{PF}_1 = \frac{\Phi_1^T.M.1}{\Phi_1^T.M.\Phi_1} \tag{2}$$

(2) We note that, to keep the effective period (T_e) and damping ratio (ξ) of the corresponding SDOF system the same as the fundamental mode properties of the multi-storey structure, we can either use the total weight of the MDOF structure (W), and derive a new elastic stiffness (K_e*) by using Equation (3), or by keeping the elastic stiffness (K_e) of the bilinear curve constant we can calculate a modified weight for structure (W*). Regardless of which weight and elastic stiffness pair is used, they must give the same elastic period calculated by Equation (3).

$$T = 2\pi\sqrt{\frac{W/g}{K}} \tag{3}$$

(3) By using the initial stiffness (K_e*) of the pushover curve, the yield base shear value [$fy = (\Delta_y/PF1).K_e$*] for SDOF system is calculated.
(4) Lastly, $\alpha.K_e$* will define the post elastic strain hardening stiffness of the SDOF system. In Table 1 he force-deformation parameters used for SDOF analysis are given for both cases.

The effective period of multi story structure was calculated as 0.145 s due to bilinearization (the calculated natural period of the MDOF system was 0.138 s).

Table 1 Bilinear structural characteristics of MDOF system and SDOF counterpart

Properties	MDOF system	SDOF system (Stiffness unchanged)	SDOF system (Mass unchanged)
T (secs)	0.145	0.145	0.145
ξ	0.02	0.02	0.02
$k_{initial}$ (kN/m)	18090	18090	31920
k_{post}(kN/m)	1312	1312	2314
α	0.0725	0.0725	0.0725
$(\Delta_{roof})_y$	4.10	3.0	3.0
V_{by} (kN)	74.2	54.4	95.76
W (tons)	17	9.634	17
V_{by}/W	0.436	0.576	0.574

Modal participation factor, PF1 was calculated as 1.364 assuming the deflected shape as triangular. The effective mass coefficient resulting from the triangular load assumption was calculated as 0.818, leading to the amount of mass mobilized in dynamic action as 0.818 W.

The roof displacements calculated using the equivalent SDOF models having bilinear hysteretic behavior is plotted against the exact roof displacements obtained from nonlinear time history analysis of MDOF structure as shown in Fig. 6. For motions that impose large inelastic displacements the equivalent SDOF model underestimates the displacements regardless of the type of ground motion. The response of elastic SDOF system gave better results than the inelastic one in many of the cases.

Although the multi-storey structure was assumed to respond predominantly in the first mode, in the later stages of nonlinearity the results of SDOF model deviated from the exact displacements, causing significant underestimates of global displacement demand. This can be attributed to the influence of local response as opposed to the global response. Upon yielding at any level, significant ductility demands were imposed on particular sections due to inelastic excursions. It should be noted that this observation contravenes the behavior of frame structures that have reduced roof displacement due to local concentration of inelasticity, such as at a soft story. The inelastic displacement demands computed using the equivalent SDOF systems tend to underestimate the global roof displacement of the structure (Fig. 6b). A similar trend was observed in linear response history analysis of the MDOF structure (Fig. 5).

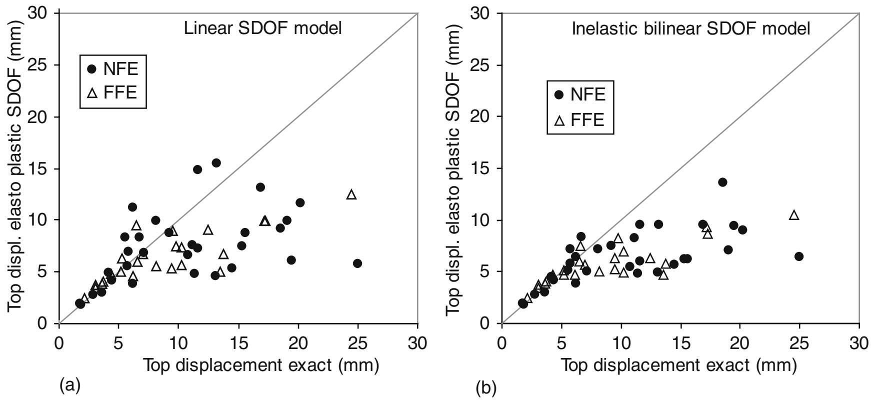

Fig. 6 Comparison of "exact" non-linear and SDOF model results, (**a**) Linear model (**b**) Inelastic bilinear model

5.2 Capacity Spectrum Method (CSM)

The capacity spectrum method initially characterizes seismic demand using a reduced elastic response spectrum. This spectrum is plotted in ADRS format which allows the demand spectrum to be "overlaid" on the capacity spectrum for the building. The intersection of the demand and capacity, if located in the linear range of the capacity, would define the actual displacement for the structure; however this is not normally the case as most of analyses include some inelastic nonlinear behavior (ATC 40, 1996).

The Nonlinear Static Procedures in ATC-40 is based on the Capacity Spectrum Method originally developed by Freeman et al. (1975) that uses equivalent linearization. In equivalent linear methods, the inelastic deformation demand of a nonlinear system is approximated by the elastic response of an equivalent elastic SDOF system that has a smaller stiffness and larger damping than the inelastic system.

To locate the point where demand and the capacity are equal, a point on the capacity curve close to the elastic displacement is selected as an initial estimate. Using the spectral acceleration and displacement defined by this point, reduction factors to apply to the 5 percent elastic response spectra to account for the hysteretic energy dissipation, or effective damping, associated with the specific point are calculated. The relationship between effective damping (ξ_{eff}) and the displacement ductility ratio (μ) adopted by ATC-40 procedure is given in Equation (4) based on the post elastic stiffness ratio (α) and the hysteretic behavior type factor (κ). If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point that point is considered as the solution. If the intersection is not reasonably close to the initial point, then a new point is assumed and the process repeated until a solution is reached. This is the performance point where the capacity of the structure matches the demand for the specific earthquake.

$$\xi_{eff} = 0.05 + \kappa \frac{2(\mu - 1)(1 - \alpha)}{\pi\mu(1 + \alpha\mu - \alpha)} \tag{4}$$

The procedure outlined above was applied to the model structure employed in order to compute its inelastic displacement demands for the set of ground motions considered in this study. The results were distinctively investigated for NFE and FFE records and are compared with the exact values in Fig. 7a. Although no clear evidence of superiority of one set of results on the other was observed, a relatively better correlation for NFE earthquakes is notable.

A clear outcome is that the Capacity Spectrum Method significantly underestimates the inelastic displacements for both NFE and FFE records when the ductility demand is high. The principal reason for this outcome is the unrealistic reduction in the demand, i.e. reduced elastic response spectrum, owing to exaggerated damping values. The estimated damping value to take into account the inelastic behavior in the system is well above twenty percent in many of the

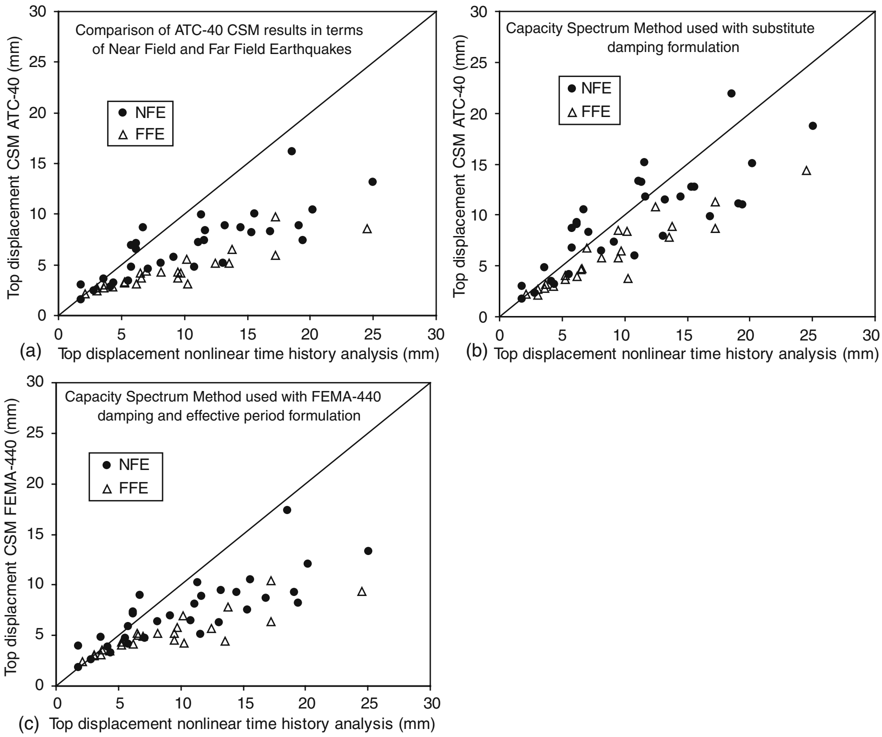

Fig. 7 Comparison of CSM results with different effective period and damping: (**a**) Comparison of top displacement obtained by ATC-40 CSM with the "exact" ones; (**b**) Capacity Spectrum Method with substitute damping; (**c**) Capacity Spectrum Method with FEMA-440 damping and period formulation

cases. The equal energy principle used in effective damping calculation for CSM can be faulted on two points:

- Derivation of equivalent damping that accounts only for the hysteretic cycle of maximum deformation, neglecting other, smaller amplitude yield excursions leads to high damping ratios as it does not include the reduction in stiffness and strength. The level of maximum deformation cannot alone account for the state of damage occurring in the structure, because the number of excursions and reversals would be parameters for the increased effective damping.
- Nonlinear time history analysis results revealed that the maximum displacement occurs after a few reversals of motion. We cannot attribute the same damage state for the structure prior to maximum excursion and following it, especially for near fault earthquakes where commonly a single pulse causes the one-sided maximum displacement excursion. Reduction of the force on the basis of maximum deformation will in turn reduce the driving forces that cause the maximum deformation.

A possible remedy to overcome this problem would be to use other methods that are based on more realistic assumptions for computing the equivalent viscous damping. The substitute damping (ξ_S) given in Equation (5) was proposed by Gülkan and Sözen (1974) to determine the equivalent viscous damping that considers approximately the influence of inelastic excursions. Here, μ is the ductility factor that describes the ratio of maximum displacement excursion to the displacement at the yield.

$$\xi_s = (1 + 10(1 - 1/\sqrt{\mu}))/50 \tag{5}$$

The underlying principle in Equation (5) is based on the idea that the response of reinforced concrete structures to strong earthquake motions is controlled by two basic phenomena: reduction in stiffness and increase in energy dissipation capacity. Furthermore, the maximum dynamic response of reinforced concrete structures, represented by SDOF systems, can be approximated by linear response analysis using a reduced stiffness and a substitute damping. Substitute damping represents the increase in energy dissipation capacity through the use of Equation (6).

$$\xi_S \left[2m\,\omega_o \int_0^t (\dot{u})^2 dt \right] = -\int_0^t m\,\ddot{u}_g\,\dot{u}\ dt \tag{6}$$

Equation (6) is based on the assumption that the energy input from a ground motion is entirely dissipated by an imaginary viscous damper which has a damping ratio equal to substitute damping. In this equation, ξ_S is the substitute damping ratio, m is the mass, t is the total duration of response, is the ground acceleration, and is the velocity of the mass. ω_o is equal to the natural frequency of the system with reduced stiffness (Gülkan and Sözen calculated ω_o as the square root of the ratio of maximum absolute acceleration to maximum absolute displacement). We note that Equation (6) takes into account all inelastic excursions, not only that with the largest amplitude in calculating the equivalent damping coefficient. Equation (5) was incorporated into the algorithm of CSM procedure as described in ATC-40 instead of the equivalent viscous damping to compute the inelastic displacements demands.

The results obtained by substitute damping were superior to those obtained by equivalent damping as shown in Fig. 7b. This finding emphasizes the significance of the accuracy in calculating the damping used in the CSM. It has been observed that the approximate nonlinear static procedures give better results with near field records of small-to-moderate magnitude earthquakes than the far field ones when the strong motion duration is short and the excitation imposes a few yield excursions on the structure due to pulse like motion. This is so because, if we assume the nonlinear time history of a particular structure to be the combination of responses of different equivalent linear systems that characterize the change of structural stiffness upon yielding and increase in damping because of sustained damage, a structure responding to a pulse-like near-field record of a moderate magnitude earthquake will be more likely to be represented with a unique equivalent linear system. A more realistic representation of the equivalent damping would then improve the results further.

A recent document, FEMA 440 (2004), devoted to evaluating existing approximate displacement-based procedures to address their drawbacks proposes a procedure that uses the effective period (T_{eff}) and equivalent viscous damping (ξ_{eff}) expressions given in Equations (7) and (8), respectively, to obtain improved results when applying the CSM procedure. In these equations, the constants A, B, C, D, G, H, I and J depend on the hysteretic model and the post elastic slope of the capacity curve. For the shear wall structure employed here the coefficients are: A = 4.61, B = -0.95, C = 10.9, D = 1.6, G = 0.12, H = -0,02, I = 0.17 and J = 0.12, respectively.

For $\mu < 4.0$:

$$\xi_{eff} = A(\mu - 1)^2 + B(\mu - 1)^3 + \beta_0 \tag{7a}$$

$$T_{eff} = \left[G(\mu - 1)^2 + H(\mu - 1)^3 + 1\right] T_0 \tag{7b}$$

For $4.0 \leq \mu \leq 6.5$:

$$\beta_{eff} = C + D(\mu - 1) + \beta_0 \tag{8a}$$

$$T_{eff} = [I + J(\mu - 1) + 1]\, T_0 \tag{8b}$$

The results of the improved procedure are compared with the exact inelastic displacement demands in Fig. 7c. An immediate observation is that the tendency to underestimate the displacements as the inelasticity increases in the system observed for the conventional CSM procedure is also true for the new procedure.

The findings of our analyses presented here are in conflict with the observations made by other researchers as presented in FEMA 440. Our results are valid for the structure employed here, a stiff multi-story structure with a fundamental period of 0.145 s, and seems to suggest that CSM in ATC-40, a widely used approximate nonlinear analysis procedure, underestimates the displacement demands. The evaluation of results presented in FEMA 440 and elsewhere (Akkar and Miranda, 2005), however, indicates the opposite trend that for structures with fundamental periods smaller than 0.5 s the CSM of ATC-40 overestimates the results by a large margin. There might be two main reasons for this inconsistency. The first is that we use an actual stiff structure whereas the FEMA-440 evaluations are based on the SDOF analyses. This, however, seems not to be the case because our SDOF analyses show quite good

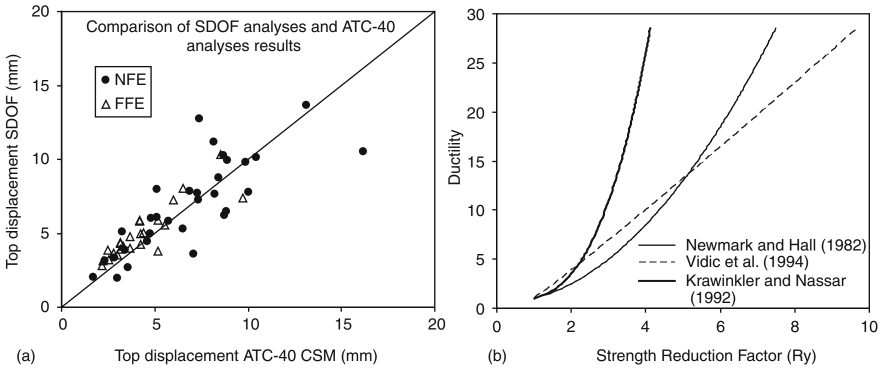

Fig. 8 (**a**) Comparison of SDOF model analyses and ATC-40 Capacity Spectrum Method, (**b**) Relationship for strength reduction factor (R) and ductility ratio (μ) for T = 0.14 s

agreement with the CSM estimates as shown in Fig. 8a. Thus the main reason appears to stem from the solutions of SDOF systems that represent unrealistically stiff structures. The SDOF systems employed in other research programs display unrealistic ductility ratios, as can be observed in Fig. 8b that presents commonly used relationships for strength reduction factor (R), ductility demand (μ) for a given period (T). At small periods representing stiff structures, the ductility demand of the system increases drastically reaching values in the order of tens which could not be attained by real structures. This indicates that for stiff structures SDOF-based evaluations of approximate procedures can be misleading because of the very small displacements involved.

Its simplicity in application notwithstanding, CSM as defined in ATC 40 or FEMA 440 has other drawbacks. The most significant of these shortcomings that our analyses have identified are listed as follows:

- In the conversion of force-displacement curve to the acceleration-displacement response spectra (ADRS format) the same modal participation factor (PF1) and modal mass coefficient (α_1), calculated with elastic properties, are used even in the nonlinear range. Both PF_1 and α_1 decrease as the system digresses into the inelastic range.
- Excessive damping values different from the customary value of 5 percent cause significant modifications in the shape of the response spectrum. This may change the definition of the spectral regions, masking other features of the ground motion (Akkar and Gülkan, 2000). Exaggerated damping values cause great underestimation in the earthquake response spectra demand and this finally results in grossly underestimated target displacements. Upon yielding, for the target displacements very close to yield point unrealistically high effective damping values may be calculated. This represents a great shift in the viscous damping with only a small increment in the inelastic displacement of the structure.

5.3 Displacement Coefficient Method (DCM) in FEMA 356

A simpler procedure, the Displacement Coefficient Method, is proposed in FEMA 273/356 (2000) to predict the inelastic displacement demand using the building's capacity curve and the elastic site-specific response spectrum. The displacement demand is calculated using Equation (9) that takes into account various characteristics of the structure and the ground motion through different adjustment coefficients.

$$\delta_t = C_0\, C_1\, C_2\, C_3\, Sa \frac{T_e^2}{4\pi^2} \mathrm{g} \tag{9}$$

The coefficient C_0 relates the top floor displacement of the structure to the displacement of an equivalent single degree of freedom system (SDOF). C_1 modifies the elastic displacement to obtain the corresponding inelastic displacement. The coefficient C_2 depends on the structural system and varies with the hysteretic behavior. The increase in the displacement demand due to P-Δ effect is taken into account through the coefficient C_3. This approximate procedure is applied to a system that has bilinear capacity curve so the original curves are needed to be idealized which would have an effective fundamental period (T_e).

The DCM was applied to the model structure here to determine its approximate inelastic displacement demand under the ground motion set considered. The "exact" roof displacements compared with results from the DCM are given in Fig. 9. It is observed from the figure that the estimates of roof displacement are improved in comparison to the results of the CSM. The better prediction capacity can be attributed to the coefficients (C_1, C_2 and C_3), that all amplify the response and take into account effects arising from nonlinearity utilized in DCM. However, the dispersion is quite significant, especially for NFE records. The equivalent SDOF solutions presented in Fig. 6b reveal that the exact inelastic displacement demands are underestimated which is inconsistent with the findings of other research, such as Miranda and Ruiz-Garcia (2002), and FEMA 440 (2004). The main reason for the discrepancy arises from the differences in results between the elastic perfectly plastic hysteretic models and bilinear models. The structure we have used has significant post elastic yield stiffness that reduces the inelastic displacement significantly, especially at large displacement ductility demand as compared to elastic perfectly plastic models that are mostly employed by earlier research.

The characteristics of the ground motion set employed as well as the structure considered here exercise significant influence on the observed discrepancies. The wall type structures are too stiff to display large inelastic deformations leading to small ductility ratios and insignificant strength reduction values. Therefore, the inelastic displacement ratios (C_1) based on SDOF solutions

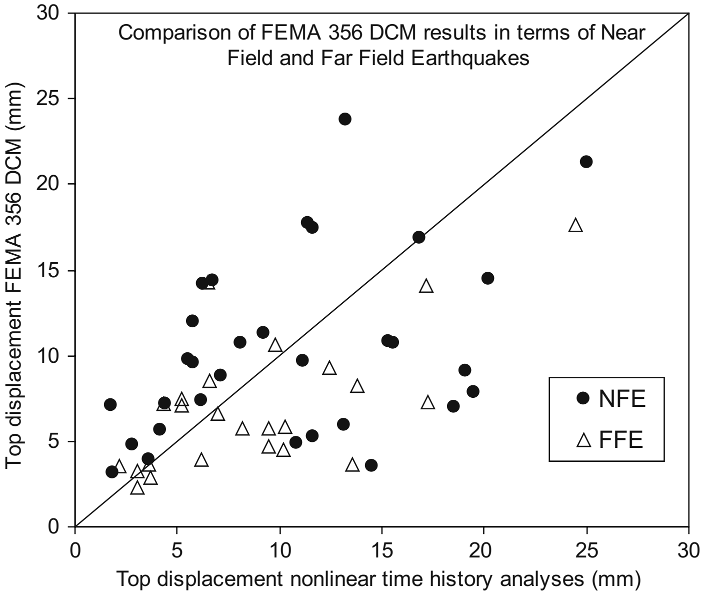

Fig. 9 Top displacements obtained from DCM for NF and FF earthquakes

appear not to be applicable to the structures with high yield strength capacity similar to that employed here.

The accuracy of DCM depends strongly on the system and the ground motion because the coefficients in Equation 5.9 are derived from the analyses of SDOF systems that themselves are not uniformly capable of representing adequately the behavior of the MDOF system considered in this investigation as discussed in the preceding sections.

6 Discussion

The results derived for this particular short-period structural assembly must be carefully interpreted to judge their general applicability. There exists a complex interaction between the types of ground motions used and the response these would generate in the analytical model of a particular test specimen. Our analytical model did not respond exclusively in the inelastic range for all input records, although for most trials it did, so the ductility demand varied from less than 1–6. The base motions fell into two groups defined as near- and far-field. It is still instructive to take stock of the performance of all the currently developed approximate approaches toward estimating the top-level translation of the model by comparing them with what would be considered to be “exact” displacement that follows from a general nonlinear analysis. We have been preoccupied with the global displacement because this is the quantity that best correlates with damage potential and performance evaluation. Force quantities such as base shear or overturning moment are supportive of conclusions that are drawn from global displacements, but their interpretation is less straightforward.

Table 2 the numerical expression of visual information contained in Figs. 5, 7b, 8 and 9 that consistently have been arranged to differentiate between near- and far-field records.

In a number of trials the elastic yield displacement limit of 4.1 mm was not transcended, so, not surprisingly, linear analysis results match experiments best. The vision provided by the other approaches is reflected better by the deviation than with the mean alone. Entries into this table are the top displacement calculated according to the corresponding approximate method divided by the exact number. We note that refinements in the CSM have indeed improved its accuracy, but for this set of records it is still on the unsafe side. The scatter in DCM is greatest among all of the methods examined here, and it is the only approach for which the average estimate ratio is greater than unity. The substitute damping ratio used in conjunction with the CSM is the next best performing method. Its prediction falls some 13 percent too low, but has a smaller standard deviation. Surprisingly, a linear approximation shows better average estimation capacity than both of the CSM formulations, but its scatter is large.

Table 3 similarly arranged for base shear estimates. From here we note that FEMA 440 represents a significant improvement over ATC 40, and the displacement overestimates in FEMA273/356 translate into even larger base shear force overestimates. Interestingly, for the MDOF system considered here, the ratio of the elastic to “exact” inelastic forces is relatively stable at an average value of 1.6. This may be interpreted as the R value for T = 0.14 s.

Table 2 Comparison of displacements

		Elastic range			Inelastic range			Overall		
		FFF	NFE	All	FFE	NFE	All	FFE	NFE	All
Linear MDOF (Fig. 6)	Mean	0.97	0.98	0.98	0.81	0.74	0.77	0.84	0.78	0.81
	SD	0.39	0.15	0.28	0.38	0.58	0.50	0.37	0.53	0.46
SDOF (Fig. 6b)	Mean	1.09	0.97	1.03	0.59	0.72	0.66	0.69	0.76	0.73
	SD	0.22	0.19	0.19	0.18	0.47	0.38	0.18	0.44	0.35
CSM ATC40 (Fig.7a)	Mean	0.85	1.01	0.93	0.50	0.70	0.61	0.57	0.75	0.67
	St. SD	0.09	0.39	0.27	0.11	0.44	0.34	0.11	0.43	0.33
CSM substitute damp. (Fig. 7c)	Mean	0.86	1.12	0.99	0.69	0.96	0.85	0.72	0.99	0.87
	SD	0.12	0.39	0.27	0.14	0.54	0.42	0.13	0.51	0.39
CSM FEMA 440 (Fig. 7c)	Mean	0.99	1.27	1.13	0.60	0.73	0.68	0.68	0.82	0.76
	SD	0.10	0.54	0.37	0.16	0.43	0.34	0.15	0.44	0.34
FEMA 273/356 (Fig. 9)	Mean	1.05	1.97	1.51	0.90	1.12	1.02	0.93	1.26	1.11
	SD	0.37	1.16	0.81	0.50	0.74	0.64	0.46	0.80	0.67
Number of ground motions		5	5	10	19	26	45	24	31	55

Table 3 Comparison of base shear forces

		Elastic range			Inelastic range			Overall		
		FFE	NFE	All	FFE	NFE	All	FFE	NFE	All
Linear MDOF	Mean	1.62	1.33	1.48	1.61	1.58	1.59	1.61	1.54	1.57
	St. Deviation	0.56	0.45	0.48	0.65	0.64	0.63	0.62	0.60	0.61
SDOF	Mean	1.28	1.13	1.21	0.93	0.86	0.89	1.01	0.90	0.95
	St. Deviation	0.10	0.17	0.13	0.18	0.40	0.32	0.17	0.37	0.30
CSM ATC40	Mean	0.95	1.08	1.01	0.74	0.79	0.77	0.78	0.84	0.81
	St. Deviation	0.07	0.15	0.11	0.18	0.30	0.25	0.16	0.28	0.23
CSM substitute damp.	Mean	0.99	1.14	1.07	0.82	0.90	0.87	0.86	0.94	0.90
	St. Deviation	0.10	0.18	0.14	0.11	0.34	0.26	0.10	0.31	0.24
CSM FEMA 440	Mean	1.15	1.29	1.22	0.93	0.92	0.92	0.97	0.98	0.98
	St. Deviation	0.07	0.26	0.18	0.17	0.39	0.31	0.15	0.36	0.29
FEMA 273/356	Mean	1.66	2.44	2.05	1.58	2.09	1.87	1.59	2.14	1.90
	St. Deviation	0.64	0.74	0.65	0.74	0.83	0.78	0.70	0.81	0.76
Number of ground motions		5	5	10	19	26	45	24	31	55

7 Conclusions

The investigation described in this article has demonstrated the analytical power of computational structural mechanics as applied to the detailed assessment of the dynamic response of a test mock-up to strong base motions. It has also served as a reminder that approximate simplified methods that have been developed and refined over the last few decades enable the analyst with remarkable ability to predict the likely limits of displacement response, and by its extrapolation, levels of damage that structural assemblies are likely to experience under a prescribed earthquake. Unlike previous research focusing on SDOF analyses, a comprehensive ground motion data set was used to evaluate the response of an analytical model that is a highly reliable representation of a particular experimental structure. The results obtained from nonlinear time history analyses indicated that stiff structures that are typically used for nuclear facilities respond to both near and far field records in firm soils in similar manner. The top floor displacement that is very commonly used in displacement based performance engineering as a design as well as performance parameter was similar for the same level of far-field and near-field earthquakes.

The equivalent SDOF systems are not adequate in representing the actual performance especially in the region of significant nonlinearity even for systems where elastic response is dominated by the first mode. The procedures such as DCM in FEMA 273 that use coefficients derived from the SDOF analyses must be re-examined before extrapolating their applicability in general.

Among the available approximate procedures implemented here DCM of FEMA-273 yielded the most satisfactory results. The worst predictions were obtained from the CSM in ATC-40, the major reason being the over estimation of the viscous damping leading to severely underestimated displacements. A significant improvement leading to more accurate predictions of the response was achieved when the substitute damping was incorporated into the CSM, although it appears that even smaller damping should be invoked for better prediction.

The results based on SDOF analyses might be misleading for stiff structures that are incapable of exhibiting large inelastic deformations. As other research has also confirmed the SDOF results imply unrealistically large ductility ratios for these structures, but large ductility demand does not translate into large damping ratios.

It has been observed that for stiff structures used for nuclear facilities none of the available approximate displacement based procedures is in its current form appropriate for an assessment of performance. For design purposes where the ground motion is represented by a design spectrum, the DCM of FEMA 273 and CSM in ATC-40 in conjunction with the substitute damping may be used if only the mean response is des.

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Rapid Probabilistic Assessment of Structural Systems in Earthquake Regions

A.S. Elnashai and S.-H. Jeong

1 Introduction

The use of fragility curves, defined as a relationships between ground shaking intensity and the probability of reaching or exceeding a certain response level, is in increasing demand both for the pre-earthquake disaster planning and post-earthquake recovery and retrofitting programs. This is due to the difficulties associated with analyzing individual structures and the importance of obtaining a global view of anticipated damage or effects of intervention, before or after an earthquake, respectively. Apart from the regional loss assessment application of fragility curves, they are useful in the probabilistic assessment of damage to individual structures taking into account material and input motion randomness.

Existing fragility curves can be classified into the four generic groups of empirical, judgmental, analytical and hybrid according to whether the damage data used in their generation derives mainly from the observed post-earthquake surveys, expert opinion, analytical simulations or combinations of these, respectively. Empirical fragility curves are constructed based on statistics of real damages from past earthquakes. Detailed review on this type of curves is given in Rossetto and Elnashai (2003). Judgment-based fragility assessment, e.g., ATC-13 (ATC 1985) and HAZUS (NIBS 1995), resort to information from expert opinion. Hybrid methods attempt to compensate for the scarcity of observational data, subjectivity of judgmental data and modeling deficiencies of analytical procedures by combining data from the different sources. Examples of the latter method are the seismic risk assessment methods in ATC-13 (ATC 1985) and ATC-40 (ATC 1996). The fragility curves are derived primarily based on expert opinion and also incorporates limited observational data.

Analytical fragility curves (e.g., Singhal and Kiremidjian 1996, Chryssanthopoulos et al. 2000, Erberik and Elnashai 2004) adopt damage distributions simulated from the analyses of structural models under increasing earthquake

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Geotechnical, Geological, and Earthquake Engineering 7,
DOI 10.1007/978-1-4020-8609-0_21,

Table 1 Comparison of fragility assessment methods

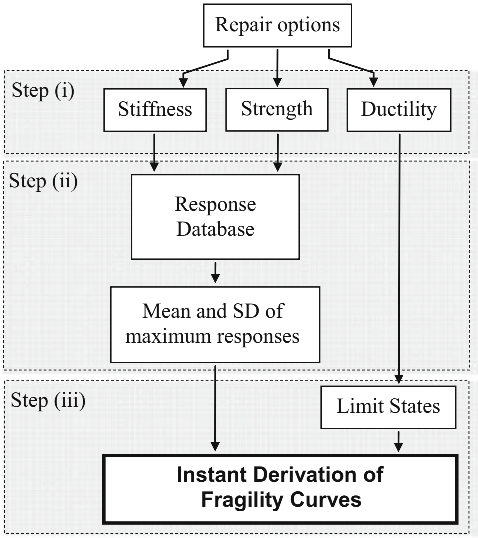
Methods	Empirical	Judgmental	Hybrid	Simple analytical model	*Detailed analysis procedure*
Effort	*increasing time and computation effort* →				
Precision	*accuracy of the assessment* →				
Application	*building stock*				*individual building*

loads as their statistical basis. Analyses can result in a reduced bias and increased reliability of the vulnerability estimate for different structures compared to post-earthquake observation or expert opinion. However, their application is generally limited by the time and computation effort. Adopting simplified analytical models reduces computation effort and allows conducting more simulations of wide variations of structure. Various methods of fragility assessment differ in the required resources and precision of the assessment results, as shown in Table 1 (Lang 2002). Therefore, the choice of a method should be made considering the tradeoff between effort and precision. The aim of the method proposed in this paper is to provide a tool for the construction of fragility curves with a reasonable precision level using a super-efficient and simple procedure. In order to overcome the large uncertainty of observational data and subjectivity in damage estimation, analytical method is employed.

2 Overview of the Proposed Methodology

Fragility curves may be derived analytically by simulations. Even for a limited number of random variables and for modest ranges of variation, the simulation effort is very considerable, reaching several hundreds of thousands of analyses. Every time the structure is replaced or even modified, the repetition of the simulation is required. It is hereafter proposed to parameterize the problem in such a manner that a generic set of fragility curves will be derived. The parameters influencing the shape of the functions are related to (i) stiffness, closely related to serviceability limit state, (ii) strength, closely related to damage control limit states and (iii) ductility, closely related to collapse prevention. By using the latter parameters with a response database which is a collection of pre-run inelastic response analyses of structures with a wide range of response parameters, the fragility curves are directly obtained without the need for simulation. This feature, allows consideration of various structural configurations in the decision making of earthquake mitigation strategies, by reducing the time and effort in the derivation of fragility curves.

Fig. 1 Overall procedure of the parameterized fragility curves



As represented in Fig. 1, the parameterized fragility curves is derived through three main steps: (i) determination of response parameters of the structure, (ii) response estimation using the Response Database of which the detail is presented in the following section and (iii) construction of fragility curves with various limit states (L.S.).

It is noteworthy that response estimation (Step (ii)) can be performed without considering the effect of ductility supply. This is due to the fact that once the yield point is determined by stiffness and strength ultimate displacement capacity does not affect the shape of the capacity curve which, in this study, determines the response of a structure. Since the ultimate displacement is defined as a multiplication of ductility and yield displacement, the effect of ductility is implemented in determining the collapse limit state to derive fragility curves. Additional details on the proposed method are given elsewhere (Jeong and Elnashai 2007).

3 Response Parameters

In earthquake engineering, inelastic static procedures (referred to in Applied Technology Council documents as nonlinear static procedure – NSP) have been widely used for the response estimation of structures due to their simplicity and efficiency. Modern seismic design and assessment guidance documents, such as ATC-40 (ATC 1996) and FEMA 273 (FEMA 1997), incorporate NSPs for estimating the peak displacements of multistory buildings. In the latter methods, the peak displacement estimate is based on a first mode analogy of the building in conjunction with the equivalent single degree of freedom (ESDOF)

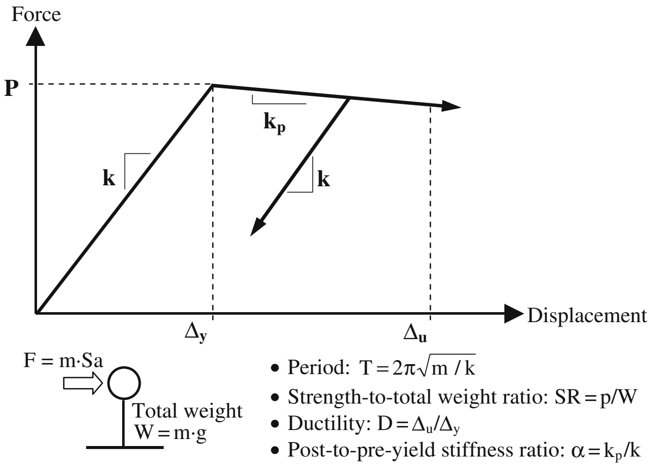


Fig. 2 Bilinear force-displacement relationship and response parameters

system. The ESDOF system can be used to estimate the response of multi-degree of freedom (MDOF) systems when response is predominantly in a single mode. Details of the ESDOF are given in references (Saiidi and Sozen 1981, Fajfar and Fischinger 1988, Aschheim and Black 2000).

The NSP with SDOF simplification has been considered as a suitable method to estimate the maximum responses of regular structures. The approach proposed in this paper also employs the latter method. The primary curve of the force-displacement relationship adopted in this study is represented in Fig. 2. The stiffness of the unloading path is assumed to be the same as the yield stiffness (k). The bilinear curve in Fig. 2 can be defined by three parameters: period (T), strength ratio (SR) and post-to-pre-yield stiffness ratio (α). The latter parameters do not entail any unit conversion in various applications. Strength ratio (SR) is defined as the ratio of lateral strength (P) to the total weight (W) of the structure. If the post-to-pre-yield stiffness ratio (α) is set to 0, an elastic-perfectly-plastic (EPP) relationship ensues.

The stiffness of a system is represented by the period T and the strength is determined by SR. Ductility is calculated by dividing the ultimate displacement (Δ_u) by the yield displacement (Δ_y). The definition of ultimate displacement is determined by the analyst according to the type of the structure under consideration and does not affect the proposed approach.

4 Concept and Structure of the Response Database

In the proposed method, the fragility curve is constructed using parameterized structural response characteristics (stiffness, strength and ductility) and the Response Database (RD). The structural response parameters are defined for the single-degree of freedom (SDOF) system that is equivalent to the complex structure. The Response Database is obtained from pre-run dynamic analysis results for a range of structural response parameters. Simulation is therefore no longer needed for a newly defined structural system. The proposed methodology has conceptual analogy with earthquake response spectra, because it (i) utilizes simplified structural models (SDOF system), (ii) obtains maximum value of response history and (iii) constructs curves which replace dynamic response history analyses. The reliability of the response estimate is at hand and reflecting it onto the obtained fragility curves is part of the procedure.

The database is designed to store information on maximum responses of a wide range of structures as statistical parameters. This enables the analyst to construct fragility curves by dealing with only two statistical parameters (mean and standard deviation) instead of massive data from a group of dynamic response history analyses. This feature renders the construction of fragility curves much easier than the conventional methods. Concurrently, the fidelity of information is maintained because the cumulative normal or log-normal distribution that is used to represent the probability distribution of maximum response depends on only the mean and standard deviation of the response variable. The structure of the Response Database is represented in Fig. 8.

For a structure with known response parameters, a group of maximum responses are obtained from dynamic response history analyses with a series of ground motions. Based on the latter responses, a set of mean and standard deviation is calculated to be the basic element of the database. This process is repeated for a range of earthquake intensities and structural response parameters to construct the Response Matrix of a specific earthquake scenario. The dimension of the response matrix can be reduced by representing the mean and standard deviation as functions of earthquake intensities. After this step, the response matrix contains coefficients of regression functions that represent the relationships between earthquake intensity and statistical parameters (mean and standard deviation) of maximum displacement demand. Finally, the Response Database is constructed by collecting the response matrices for various earthquake scenarios and structural idealization types such as bilinear and tri-linear simplifications.

5 Fast Demand Estimation Using Response Database

The Response Database is a collection of pre-analyzed responses for a wide range of structures. Therefore, the response estimation of a structure entails only retrieving the value from the latter database corresponding to the given response parameters and the earthquake scenario. Thus, the fragility analysis can be instantly carried out.

For the reference derivation in this paper, a set of artificial ground motions is used. The latter ground motions are synthesized to simulate an earthquake event for lowland soil profile in Memphis, TN, USA and entitled 'Scenario #3' among three scenarios generated as a part of the Mid-America Earthquake (MAE) Center research project HD-1 (Hazard Definition project 1). Scenario #3 consists of ten records simulating an earthquake event of magnitude (M_w) 5.5 and a focal depth of 20 km with 84 percentile level (one standard deviation above the mean value) from the prediction model. Details of the records are given elsewhere (Romero and Rix 2002).

In order to obtain mean and standard deviation of maximum responses using the database, the analyst needs to determine the structural idealization type and response parameters. As an illustrative example, a generic structure with the elastic-perfectly-plastic (EPP) force-displacement relationship (Fig. 3) is studied. The period of the structure is 0.8 s. and its strength ratio (SR) varies: SR $=0.1$, 0.2 and 0.3.

As indicated in the structure of the response database (Fig. 8), mean and standard deviation of maximum displacement responses from a series of inelastic dynamic response history analyses are collected and organized in the response matrix. Each cell of the latter matrix consists of six coefficients of a sixth order polynomial regression function with intercept at zero (0, 0). This function represents mean or standard deviation of the maximum displacements as a function of earthquake intensity, as shown in Equation (1).

$$y = a_1 \cdot x^6 + a_2 \cdot x^5 + a_3 \cdot x^4 + a_4 \cdot x^3 + a_5 \cdot x^2 + a_6 \cdot x \tag{1}$$

Where x is earthquake intensity and y is mean or standard deviation of the response quantity. A linear interpolation will be used for a structure with response parameters that do not match the predetermined values of SR and period in the response database. A set of mean (μ) and standard deviation (σ)

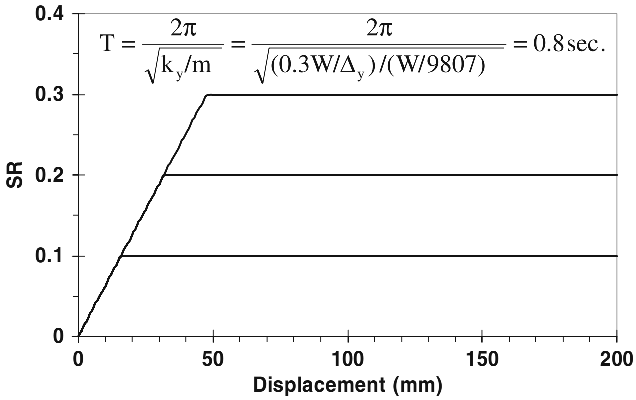
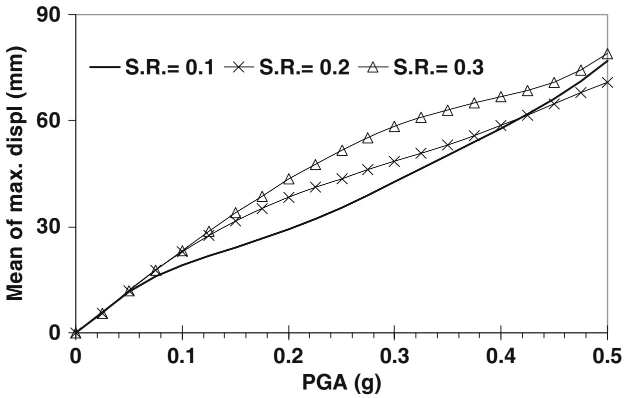
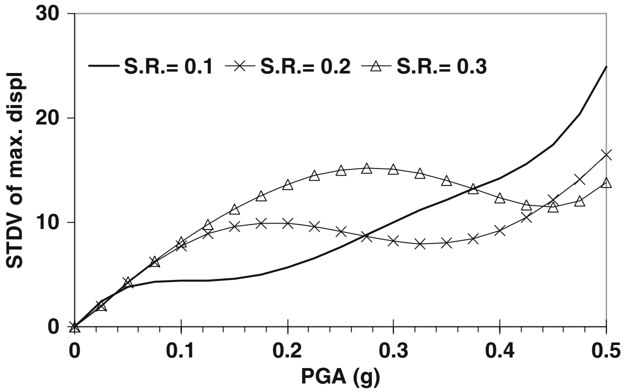


Fig. 3 Force-displacement relationship of the example structure

Fig. 4 Mean and standard deviation of maximum displacement of the example structure as functions of earthquake intensity. (**a**) Mean (μ) of maximum displacement (**b**) Standard deviation (σ) of maximum displacement



(a)



(b)

for the example structure with various strengths are shown in Fig. 4. Mean and standard deviation (SD) of maximum responses are expressed as 6th order polynomial functions of earthquake intensities (Equation 1). The coefficients of the latter functions are obtained from the response database.

6 Reference Derivation of Fragility Curves

In this paper, maximum responses are assumed to follow a lognormal distribution. The probability of reaching or exceeding a limit state (LS) at a given earthquake intensity (s) can be expressed as follows:

$$P\,(LS/s) = P\,[(\Delta_{LS} \leq \Delta_{\max})/s] = 1 - F(\Delta_{LS})|_{\,s} \tag{2}$$

where, $F(\Delta_{LS})|s$ is a cumulative probability of obtaining the maximum response ($\Delta_{\max}$) between 0 and Δ_{LS} when the earthquake intensity is s. Δ_{LS} is a threshold response quantity for a limit state. The fragility curve for a limit state is obtained by plotting the probability in Equation (2) over a range of earthquake intensities.

The cumulative probability of lognormal distribution $F(\Delta_{LS})$ in Equation (3) can be calculated by

$$F(\Delta_{LS}) = \Phi\left(\frac{\ln \Delta_{LS} - \lambda}{\xi}\right)$$

$$= \int_0^{\Delta_{LS}} \frac{1}{\sqrt{2\pi}\xi y} \exp\left[-\frac{1}{2}\left(\frac{\ln\ y - \lambda}{\xi}\right)^2\right] dy \tag{3}$$

where, λ and ξ are mean and standard deviation of $\ln(\Delta_{\max})$, respectively. They can be expressed in terms of median (x_m) and coefficient of variation (δ), as shown in Equations (4) and (5).

$$\lambda = \ln x_m = \ln\left(\mu/\sqrt{1+\delta^2}\right) \tag{4}$$

$$\xi = \sqrt{\ln(1+\delta^2)} = \sqrt{\ln[1+(\sigma/\mu)^2]} \tag{5}$$

In which, μ and σ are mean and standard deviation of maximum response ($\Delta_{\max}$), respectively.

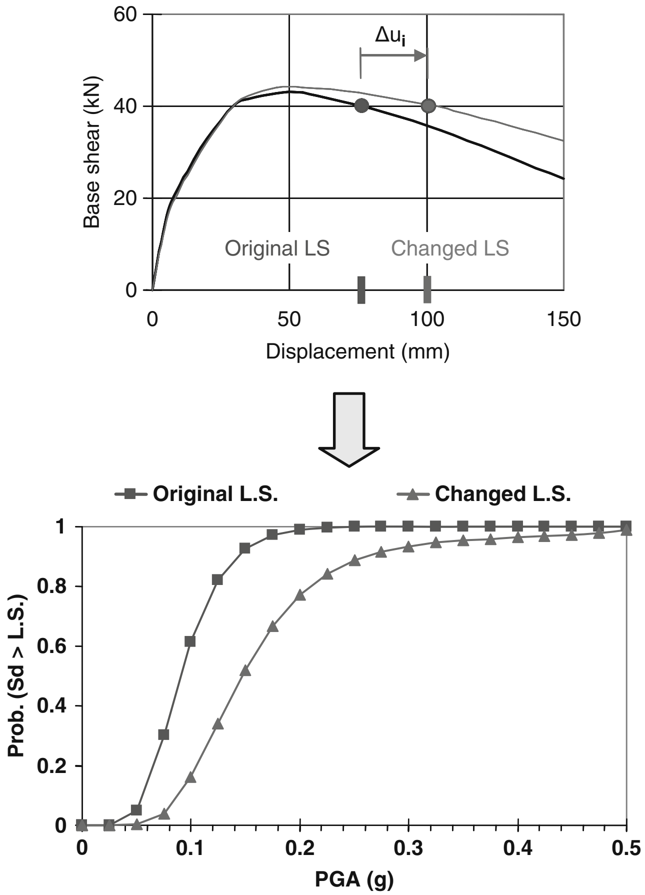
Uncertainties can be implemented in the calculation of limit state probability as below (Equation 6):

$$P[(\Delta_{LS} \leq \Delta_{\max})/s] = 1 - \Phi\left(\frac{\ln \Delta_{LS} - \lambda}{\sqrt{\beta_1^2 + \beta_2^2 + \cdots + \beta_n^2}}\right) \tag{6}$$

where β_i (i = 1, 2, $\cdots$, n) represent various uncertainties. Sources of uncertainties can be categorized as (i) inherent randomness (aleatory uncertainty source) and (ii) errors of ignorance and simplification (epistemic uncertainty source). While variation due to the former can be quantified by standard deviation of response variables, estimating the latter effect is complex and affected by various sources such as modeling errors, measurement errors and statistical errors (Geysken et al. 1993).

While the effects of stiffness and strength changes on the fragility curves are considered when using the Response Database, the variation of ductility supply can be implemented into the calculation procedure of conditional probability by changing the limit states (Δ_{LS}), as shown in Fig. 5.

Fig. 5 Effect of ductility change on fragility curves



Sample derivation of fragility curves corresponding to various displacement limit states, for the example structures shown in Fig. 3 (elastic-perfectly-plastic, T = 0.8 sec. and SR = 0.1; 0.2;0.3), are presented in Fig. 6

Recent studies (e.g., Wen et al. 2004, Pinto et al. 2004, Kwon and Elnashai 2005) have shown that the effect of randomness in capacities on the variability of global response is overshadowed by that of variability in the ground motion. The latter observation indicates that fragility curves are less affected by the capacity randomness than input motion randomness, and thus can be derived considering only the randomness in ground motions. In this paper, earthquake strong ground motion is considered as the only random variable. The proposed method is, however, capable of dealing with cases of capacity randomness by performing analyses for structures with a range of response parameters.

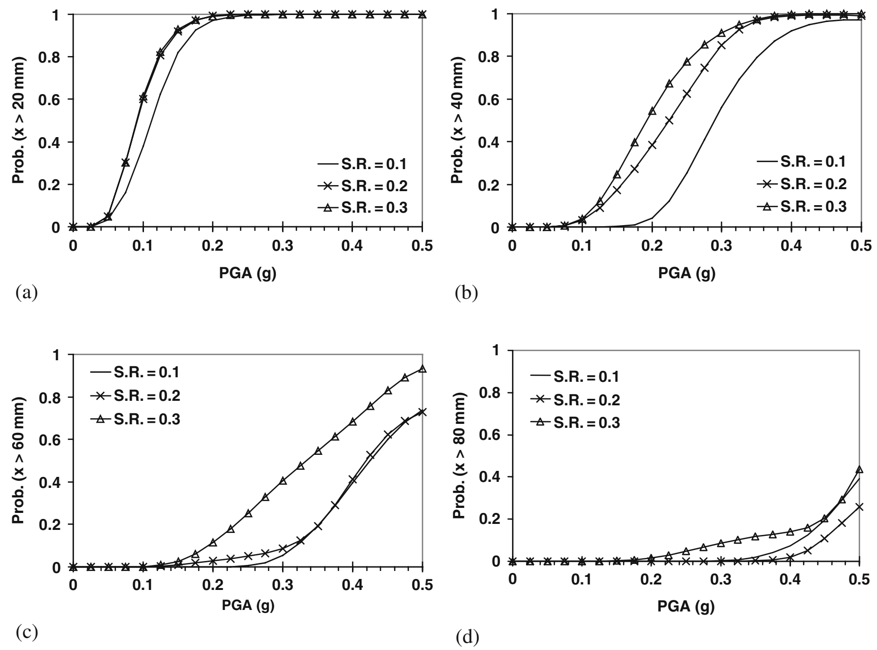


Fig. 6 Vulnerability functions for various limit states (**a**) Limit state = 20 mm (**b**) Limit state = 40 mm (**c**) Limit state = 60 mm (**d**) Limit state = 80 mm

7 Comparison and Verification

The estimates based on the Response Database are compared with those by the Capacity Spectrum Method (CSM), one of the most widely used NSPs in earthquake engineering. The comparison is performed by deriving fragility curves for a simple structure with EPP force displacement relationship (T = 0.8 s and SR = 0.13) and 5% viscous damping ratio. Response variation due to input motion randomness is considered as aleatory uncertainty. For epistemic uncertainty, the coefficient of variation (COV) due to modelling errors is assumed to be 0.3 (Wen et al. 2004).

In the parameterized fragility method (PFM), the response estimation is performed by the response database, as described in the proceeding section. The analyst can instantly obtain mean (μ) and standard deviation (σ) of maximum displacement as functions of earthquake intensity, provided that the response parameters (period and strength ratio) are prescribed and the response database is ready to be utilized.

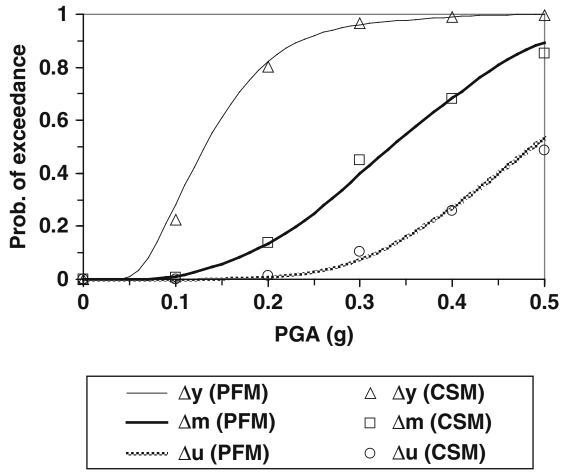
In the CSM, the maximum displacement is estimated by locating the intersection point of the capacity curve and demand spectra. The latter are inelastic spectra in the acceleration-displacement (or composite) response spectra (ADRS) format. Inelastic spectra represent the modified response of a SDOF system according to the level of inelasticity. In this study, constant-ductility spectra by inelastic dynamic analysis are employed for the comparison because the latter method gives exact estimation of maximum inelastic responses of a SDOF system. Appropriate ductility values are determined by an iterative method where the ductility demand of the intersection point is equal to the corresponding ductility value of the constant-ductility spectra. This method was proposed by Chopra and Goel (1999) to improve the accuracy of the ATC-40 procedures. In order to obtain mean and standard deviation of maximum displacements by CSM, the approach proposed in Shinozuka et al. (2000) is employed. In the latter study, mean and mean $\pm\sigma$ (standard deviation) of maximum displacements are obtained from the CSM with mean and mean $\pm\sigma$ demand spectra, respectively. The latter spectra are developed by plotting the mean and mean $\pm\sigma$ of spectral acceleration and displacement in the ADRS. The latter procedure requires the calculation of mean and standard deviation of spectral accelerations and displacements for a set of earthquake records with a given earthquake intensity. The inelastic demand spectrum is developed by plotting the constant-ductility spectra in the ADRS format and represents modified responses due to inelasticity. The above procedure should be repeated for a range of earthquake intensities to develop fragility curves.

Fragility curves are derived for limit states of (i) yield displacement (Δ_y), (ii) intermediate damage displacement (Δ_m) and (iii) ultimate displacement (Δ_u). In the reference derivations, the ductility supply of the example structure is assumed to be 3 ($\Delta_u/\Delta_y = 3$). The intermediate damage displacement is defined as the middle point between the yield and ultimate displacements; $\Delta_y + (\Delta_u - \Delta_y)/2$. The definition of the limit states is not an integral part of the proposed method, but can be determined by the analyst to appropriately represent the relationship between response of a structure and its damage states. In general, limit states are determined based on information on the structure such as structural type, loading and boundary conditions, detailing and failure mode.

The fragility curves of the example structure (EPP; T = 0.8 s, SR = 0.13 and damping ratio = 5%) derived by the PFM and CSM are compared in Fig. 7. The agreement between the probability estimations by the PFM and the CSM is very good. This verifies that the PFM can be an efficient substitute for the conventional NSPs in deriving fragility curves. The former is much faster than the latter, yet provides the same level of accuracy.

The parameterized fragility method (PFM) is an efficient tool for deriving probabilistic fragility relationships. For the analyst, the procedure is simple and fast; input response variables of a structure and obtain the fragility curves instantly. An application software developed by the authors is represented in Fig. 9. By virtue of its instantaneous nature, the proposed method is especially useful for the practical application of analytical fragility curves to the planning of seismic rehabilitation and repair, or regional earthquake

Fig. 7 Fragility curves by the PFC and the CSM



mitigation where fast estimation of limit state probabilities for a large number of structural configurations and different mitigation measures is essential.

Since the proposed method adopts an ESDOF system to represent responses of a structure, the accuracy of response estimation may not be adequate for structures with significant mixed-mode effects. This is a common limitation in all conventional methods employing ESDOF simplification. However, the latter methods are considered to be accurate enough for many practical situations and the use of the simplified methods has been increasing. For instance, the CSM is one of the most widely used method for seismic assessment and various other NSPs are implemented in seismic guidelines such as ATC-14 and FEMA-273.

8 Conclusions

Derivation of probabilistically-based vulnerability relationships for a single type of structure requires many analyses, up to tens or even hundreds of thousands, especially when a large number of random variables are required. These will then have to be re-derived for different structural configurations as well as for different repair and/or strengthening effects. In this paper, a novel analytical fragility assessment framework based on parameterizing the response by its fundamental quantities of stiffness, strength and ductility, is proposed. Therefore, with predetermined stiffness, strength and a set of earthquake records, the framework provides mean value and their associated standard deviation of inelastic response quantities of the corresponding structure without the need for simulation. The effect of ductility variation is included in the limit state definition in the calculation of cumulative conditional probability. Thus, once the user defines stiffness, strength and ductility of a structural system, a set of probabilistic fragility curves is instantly available, the accuracy of which is the same as closed-form ESDOF-based inelastic simulation approaches.

The implications of the success of the developed approach are wide-ranging. For cases of selection between different retrofitting options, the parameterized fragility curves approach gives rapid estimates of probabilities of various damage levels being inflicted onto the structures under consideration, given only the stiffness, strength and ductility for each alternative retrofitting scheme. Additionally, the presented fragility assessment methodology enables the analyst to practically investigate the vulnerability of large number of structural configurations instantly without simulation. Therefore, this method blends very well with Consequence-Based Risk Management (CRM) approach of the Mid-America Earthquake Center, where the fragility assessment for generic structural systems in a large region is sought. This approach is being implemented into the Mid-America Earthquake Center seismic loss assessment system MAEviz.

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Appendix

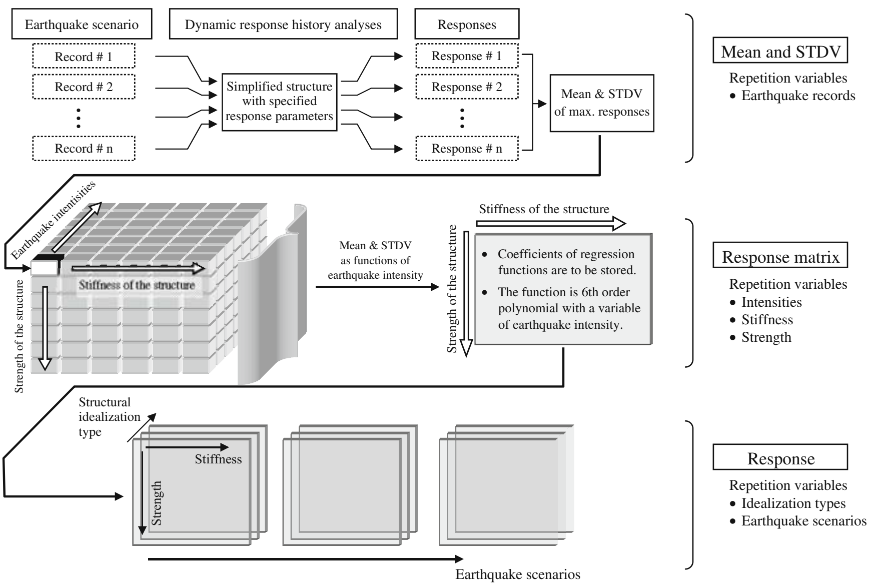

Fig. 8 Structure of the response database

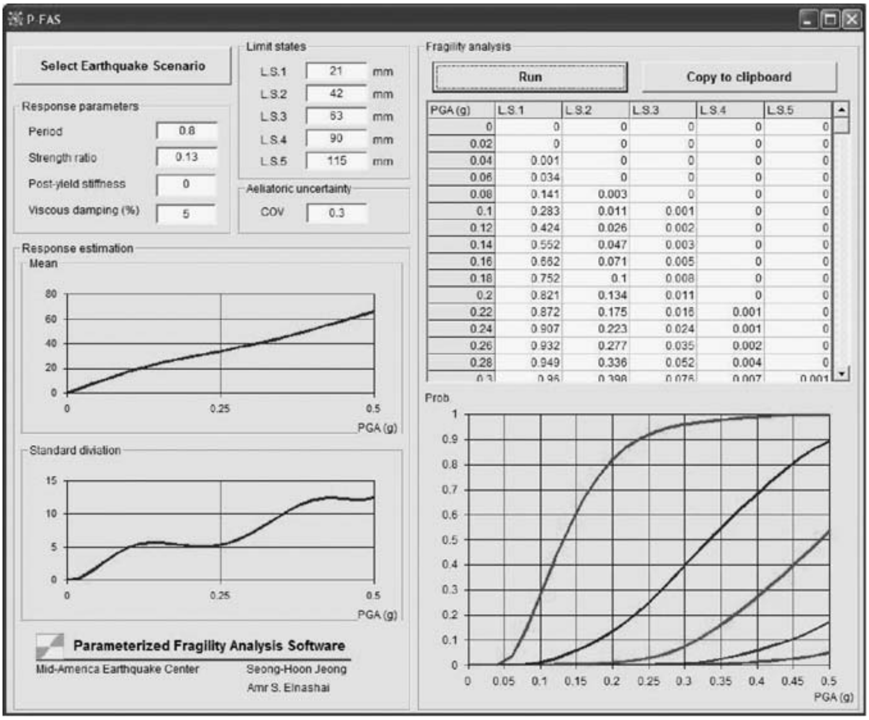

Fig. 9 Software for parameterized fragility analysis*

*The Parameterized Fragility Analysis Software is implemented into the MAEViz which is the interactive visualization tool developed by the Mid-America Earthquake Center for earthquake risk assessment across regions.

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Development of European Shaking Tables

R.T. Severn

1 Introduction

In the early 1950's, Southern European countries of Italy, Spain and Portugal were making significant progress in the design and construction of large concrete dams, particularly arch dams. The names of Carlo Semenza, designer of the ill-fated Vajont dam in the Italian Dolomites, and Lagina Serafim of the Cabril dam in Portugal, come easily to mind. British Consulting Engineers were someway behind, but in 1957a British firm -Sir Alexander Gibb and Partners, became involved in the Tang-y-Soleyman dam in Iran, for which the design included earthquake loading, possibly for the first time. Earthquake engineering was a neglected subject in the UK at that time, requiring Gibb to request the recently created LNEC Lisbon, to act as consultants. . It is clear from their 1961 report to Gibb that LNEC also had no experience of such research, and was required to create its own experimental expertise in order to carry out this task, as the following quotation indicates.

> *The tests made it necessary to install new testing equipment, to prepare special setups and to develop measuring techniques not yet used. Several difficulties were met, not only in performing the tests, but also in what concerns the practical application of the theory on which the study of random vibration is based. These being pioneer tests it is easy to understand that their accuracy is not completely satisfactory but they are considered nevertheless, to furnish practical results of much interest, and to allow a more clear understanding of the phenomena involved in earthquake action.*

From the LNEC report, I have selected two illustrations. Figure 1 is the result of their study of model scaling laws, which I believe to have been original in at least one respect -that of the Cauchy number -with the conclusion that with the reservoir empty it should be the same for model and prototype, but if the reservoir water plays a part, and its restoring force is taken to be elastic, the Cauchy number must be satisfied for the water as well, the consequence of

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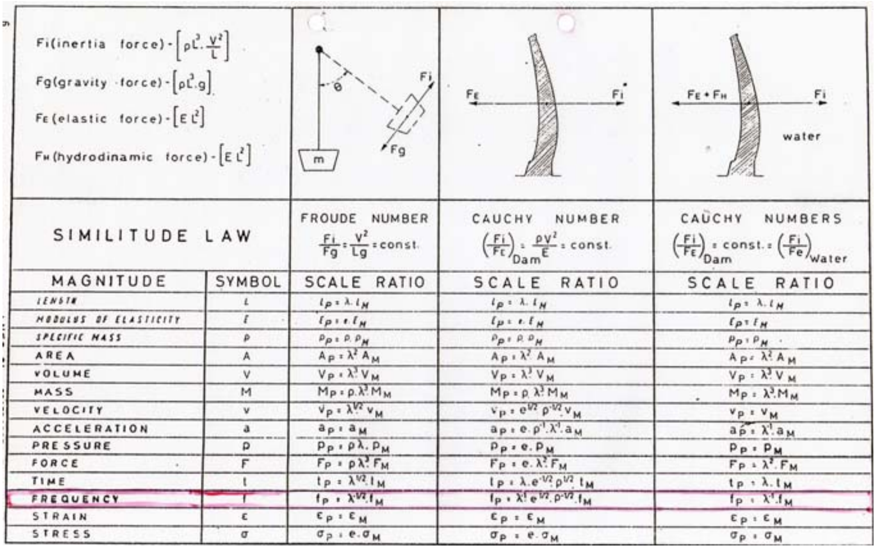

SIMILITUDE LAW		FROUDE NUMBER $\frac{Fi}{Fg} = \frac{V^2}{Lg} = \text{const.}$	CAUCHY NUMBER $\left(\frac{Fi}{FE}\right)_{Dam} = \frac{\rho V^2}{E} = \text{const.}$	CAUCHY NUMBERS $\left(\frac{Fi}{FE}\right)_{Dam} = \text{const.} = \left(\frac{Fi}{Fe}\right)_{Water}$
MAGNITUDE	SYMBOL	SCALE RATIO	SCALE RATIO	SCALE RATIO
LENGTH	l	$l_P = \lambda . l_M$	$l_P = \lambda . l_M$	$l_P = \lambda . l_M$
MODULUS OF ELASTICITY	E	$E_P = e . E_M$	$E_P = e . E_M$	$E_P = E_M$
SPECIFIC MASS	ρ	$\rho_P = \rho . \rho_M$	$\rho_P = \rho . \rho_M$	$\rho_P = \rho_M$
AREA	A	$A_P = \lambda^2 A_M$	$A_P = \lambda^2 A_M$	$A_P = \lambda^2 A_M$
VOLUME	V	$V_P = \lambda^3 V_M$	$V_P = \lambda^3 V_M$	$V_P = \lambda^3 V_M$
MASS	M	$M_P = \rho . \lambda^3 . M_M$	$M_P = \rho . \lambda^3 M_M$	$M_P = \lambda^3 . M_M$
VELOCITY	v	$v_P = \lambda^{1/2} v_M$	$v_P = e^{1/2} \rho^{-1/2} . v_M$	$v_P = v_M$
ACCELERATION	a	$a_P = a_M$	$a_P = e . \rho^{-1} . \lambda^{-1} . a_M$	$a_P = \lambda^{-1} . a_M$
PRESSURE	p	$p_P = \rho \lambda . p_M$	$p_P = e . p_M$	$p_P = p_M$
FORCE	F	$F_P = \rho \lambda^3 . F_M$	$F_P = e . \lambda^2 . F_M$	$F_P = \lambda^2 . F_M$
TIME	t	$t_P = \lambda^{1/2} . t_M$	$t_P = \lambda . e^{-1/2} . \rho^{1/2} . t_M$	$t_P = \lambda . t_M$
FREQUENCY	f	$f_P = \lambda^{-1/2} . f_M$	$f_P = \lambda^{-1} e^{1/2} . \rho^{-1/2} . f_M$	$f_P = \lambda^{-1} . f_M$
STRAIN	ε	$\varepsilon_P = \varepsilon_M$	$\varepsilon_P = \varepsilon_M$	$\varepsilon_P = \varepsilon_M$
STRESS	σ	$\sigma_P = e . \sigma_M$	$\sigma_P = e . \sigma_M$	$\sigma_P = \sigma_M$

Fig. 1 Similitude Laws developed by LNEC Lisbon for dam testing (1957)

which is that the model must be constructed of the same material as the prototype. Figure 2 is a photograph of an actual test

One important consequence of this LNEC study was that the UK Institution of Civil Engineers set up a Committee, composed of equal numbers of

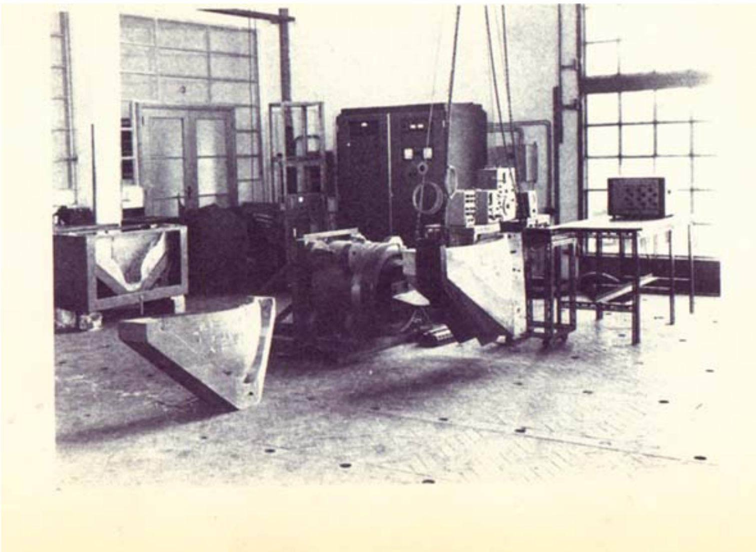

Fig. 2 LNEC Lisbon: dynamic test in progress

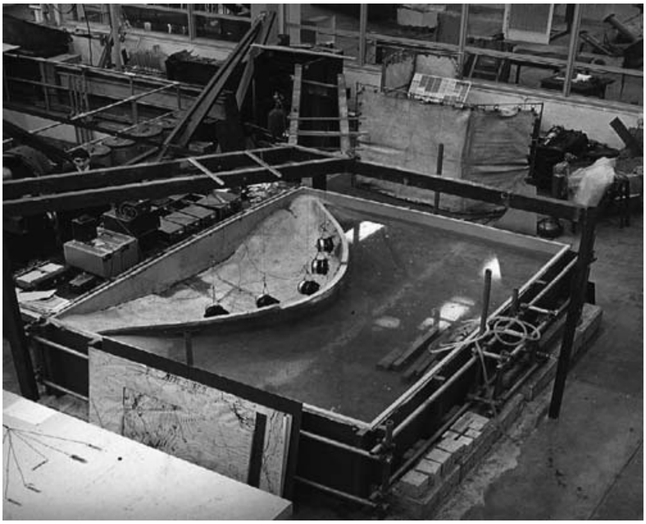

Fig. 3 Multi-point dynamic tests on a 1:200 model of the Hendrik Verwoerd arch dam

Consulting Engineers and academic researchers, to study the problem of the design of arch dams, at a time when digital computers were becoming available. From the Committee's report in 1968 (Dungar and Severn, 1968), it is clear that not only had the use of thick shell and 3D finite element methods been developed for the first time and used for static analysis, but that these computer-based methods had been employed also for dynamic effects caused by earthquakes, including the added-mass of the reservoir water.

However, experimental methods lagged behind, being restricted to the determination of natural frequencies and mode shapes of single-curvature perspex models for comparison with those obtained analytically. A few years later it became possible to make use of advances made in vibration testing by the aerospace industry, where a multi-vibrator electro-dynamic system had been developed for design studies on Concorde. Because the exciters were electronically driven, it was possible to develop a control system that guaranteed the precision of frequency and phase between the different exciters -a requirement for the production of pure modes of vibration -theoretically up to the number of exciters used. This system was adapted for dynamic studies on a model of the Hendrik Verwoerd Dam in S.Africa, to a design of Sir Alexander Gibb and Partners (Fig. 3) (Back et al., 1969). Referring to Fig. 3, analysis of the effects of the reservoir water had shown (Selby and Severn, 1972; Greeves and Severn, 1991) that vibration reflection from the rigid model walls was not significant if they extended upstream for 3H, where H is the height of the model dam.

2 Early Developments

I do not know when the first shaking table or reaction wall was established in Europe, but it is likely to have been sometime in the late 60's or early 70's. Shaking tables would have been uniaxial and supported on an oil base with guides (as at ISMES), or constrained against all but uniaxial motion by bearings, producing unknown and unwanted forces. Those that were driven by electrodynamic oscillators (as at Bristol) had a frequency range of 10–1000 Hz., but were difficult to control at the lower end of this range. This period did, however, see advances in the broad subject of control of mechanical systems and the development of hydraulic actuators for various purposes, one of which was the operation of shaking tables and reaction walls, so that by the early 1980's, tables capable of motion in all six degrees-of-freedom (6DOF) were in existence at Julich in Germany and at ISMES, Athens and Bristol, having plan area dimensions varying between 9 and 20 m^2 and a frequency range of 0–100 Hz. At CEA Saclay in France, a 25 m^2, 3DOFtable was being designed around the same time, and a little later LNEC Lisbon chose to build a 3DOFtable of in-house design, in which the rotations were to be removed by massive torque tubes. In the UK, Germany and France, these developments were driven to a large extent by testing needs for the nuclear power industry. Hydraulic actuators were also being used for the reaction wall type of test but computer speeds were not sufficiently fast to provide for much more than a few points of excitation.

In 1990, the European Commission's (EC) Large Installations Plan (LIP) established working groups in many branches of science and engineering to determine what large infrastructures existed, and whether there was need for augmentation at European level. In the course of its work, the LIP in earthquake engineering visited the World's major laboratories before concluding that there were sufficient existing shaking tables for likely European needs, but that no state-of-the-art reaction wall facility existed in any Member State. An important rider was later added to the first of these conclusions to the effect that if a co-ordinated European programme in earthquake engineering was envisaged, the actual performance capabilities of the existing tables should be assessed. The second conclusion resulted in the creation of the reaction wall at JRC Ispra, the subsequent second conclusion that we recall in particular today, because John Donea not only seized the opportunity to promote the building of the missing reaction wall here at JRC Ispra, but he had, in his personal make-up, the ability to mobilise effectively the talents of the many colleagues required to bring such a major project to completion.

3 The Study of European Shaking Table Characteristics

The Athens and Bristol laboratories were members of the LIP group, and in 1992 received a contract from the EC to begin the study of European shaking table performance. The common test piece of Fig. 4 was agreed, and identical

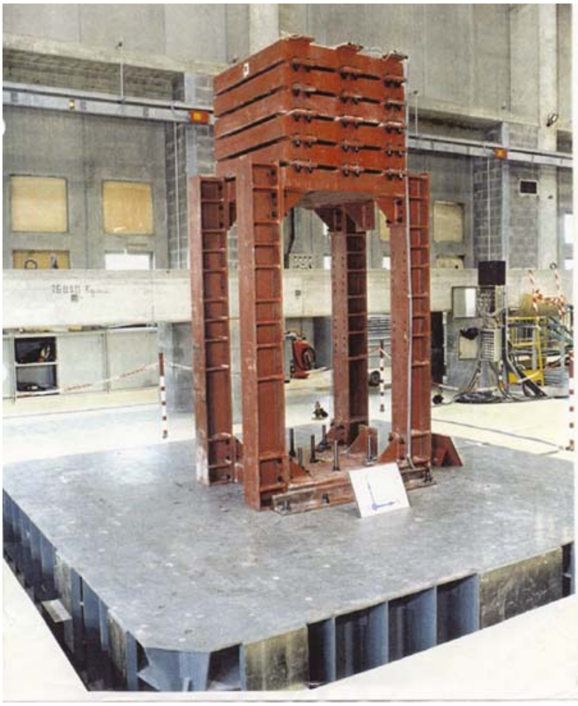

Fig. 4 Bristol/Athens Testpiece for shaking table performance studies

versions built at Bristol for each laboratory. Its design was predicated on the need to test each table to the limits of its capacity, particularly its ability to generate and control the three rotational degrees of freedom. This was made possible by vertical positioning of a number of 1T steel plates so as to vary the centre of mass in the vertical direction and hence the overturning moments. Before the actual programme could be started, Athens and Bristol were joined by LNEC Lisbon and ISMES Bergamo. After studies lasting three years, the following deficiencies were found, in varying degrees, in all four tables (Crewe and Severn, 2001).

- For testpieces that remained elastic, a satisfactory match between desired and achieved input could be obtained, but the success of the matching process depended on skilled operators;
- If the measured accelerations of the table were used for matching purposes, its displacement was not controllable;
- Small spurious motions of the table had a very strong influence on the response but were difficult to remove;
- All tables were controlled out-of-real-time and the control became unstable if the properties of the testpiece changed.

The common background to these deficiencies was the use of conventional fixed-gain control algorithms. These are based on a linear model of both the shaking table itself and the testpiece upon it, the parameters of the latter being fixed for the duration of the test. Although the influence of the testpiece can be partially removed by fine-tuning of the linear controller, this cannot deal with the non-linear effects and is limited by the expertise of the operator.

Fortunately, it was found that the general field of control engineering had itself been forced to address similar problems in non-linear robotics, and had developed *adaptive* control algorithms in which the control strategy adapts to the changing characteristics of the robot, in particular to its non-linear behaviour. A major contributor to this adaptive control development was the *Minimal Control Synthesis (MCS)* algorithm, invented at Bristol University by Prof. D Stoten (Stoten and Neild, 2003). Six laboratories (now including CEA Saclay and JRC Ispra) contributed to the removal of the four shortcomings listed above, and, having removed them, were successful in obtaining from the EC a series of research contracts in the following areas, all of which utilized the very significant achievements of *real-time* control, which in practice means an accurate reproduction of the time-scale of the actual event being replicated.

- The use of shaking tables to study non-linear behaviour of testpieces;
- Substructuring on shaking tables;
- Multiple support input;
- The effect on testpiece response of spurious input motions;
- Continuous pseudodynamic testing at reaction walls.

The importance of the first of these is that modern seismic design procedures utilize the energy absorbing qualities of construction materials when they behave non-linearly. One such method, referred to as "pushover" analysis, utilizes one or more modal shapes in the seismic design of buildings and has received much theoretical development; however, experimental studies have been lacking owing to the inability of shaking tables to exercise control for non-linear behaviour. The second is of great importance because a sometime disadvantage of shaking table testing is that a small-scale model must be used in which important details cannot be replicated properly. In the technique of substructuring, the more important part of the system can be modeled at full or large scale on the table whilst the remainder is modeled numerically in a computer. The interface between the two parts is still a subject of international research, and a potential major application is the topic of structure-foundation interaction. Hitherto, it has been very difficult to represent the foundation itself on the shaking table, partly because its properties are non-linear. But with substructuring, the various theories could be modeled numerically. For the third point of non-linear behaviour above, it is known that very large structures (e.g. bridges, dams and pipelines) receive different ground inputs. To replicate this properly requires different shaking tables to be precisely controlled in real-time with different input signals. Regarding point four above, for the seismic qualification of industrial products, it is important to be aware of the tolerances that can be allowed in the testing procedure.

The fifth of the above topics indicates that JRC Ispra had become a major contributor to the research programme, emphasizing the essential complementary contributions of the two types of experimental infrastructure. At this time, alongside the deficiencies associated with shaking tables, the major deficiency at reaction walls was the large interval between successive stages of the process, which made it difficult to cope with strain-rate effects. By a combination of improved algorithms, faster computing speeds and the use of MCS, this deficiency has been removed.

4 The Shaking Table Control Problem

Having discussed what had been achieved by European laboratories, let us now look at what the problems were (Stoten and Gomez, 2001). The concentration is on shaking tables but, as we have already seen, what has been achieved for them opened up new research areas for reaction walls. I am also conscious that this talk is a personal odyssey, and that my JRC colleagues are more fitted than Ito talk about reaction walls.

Introduction: Shaking tables are of value because they are experimental devices replicating the true nature of the earthquake input. This they do by applying the ground motion to the base of the structure, thereby inducing inertia forces in every element of mass. It is these forces which generate the response displacements and stresses.

The ground motion itself will have components in all 6DOF, although only the time-histories of the three translations are usually measured and often only one of these is specified for testing purposes. It might be considered therefore that a table capable of controlled motion in one axis would be satisfactory, but this begs the question of how motion in the remaining five is to be removed. Early tables were of this single-axis variety, with physical restraints which introduce a pattern of unknown forces into the table and which modify the intended single-axis motion. It can be argued that for research purposes this is not important -the investigator measures the motion input into the testpiece and uses this for comparison with a theoretical analysis. But in many instances shaking tables are used to certify industrial products for practical use against well-defined input motions. These can be recorded time-histories of acceleration or displacement, artificial time-histories which have been obtained from specified response spectra, precise sinusoidal motion of the sine-sweep or sine-dwell variety, and random noise. All theses types of input are used for specific purposes in which no corruption of the input can be allowed.

The motion of tables is produced by servo-hydraulic actuators, usually 8 in number, and the fact that this is larger than the 6 DOF gives rise to the shaking table control problem discussed below. Before doing so, it is noted that so-called Stewart platforms for vibration testing can be produced, in which the 6 DOF are controlled by only 6 actuators, but these have to be arranged and oriented in a special manner. Such platforms were devised for aerospace and mechanical engineering applications, in which the rotations of the testpiece are possibly more important than the translations. For seismic testing the most used input is that of translation in one, or sometimes two or three directions, with rotations being small, and in most cases unwanted. It follows that for efficient utilization of available actuator forces, the actuators must be oriented in these translational directions, but arranged so that rotations can be either produced, or easily controlled.

Shaking Table Control: Consider first a table constrained to uniaxial motion (Fig. 5) where displacement is used as the control parameter. The specified input, the command signal, is applied to the servo-valve controlling the flow of oil in a single actuator, but is corrupted by forces associated with the electrical/ mechanical components of the table, so that the displacement signal recorded at the base ofthe testpiece, when compared to the command signal at the summing junction, contains a measurable error. This error is then used iteratively to modify the input signal until the desired command signal arrives at the testpiece. In essence, this is the shaking table control problem, but there are two issues which need to be addressed – measurements at the testpiece and the time taken for the control calculations.

Measurements at the Testpiece. The parameters to be measured at the testpiece could be acceleration, and/or displacement, for comparison with corresponding values input to the servo-valve. Figure 6 shows a single-axis control in which velocity as well as displacement and acceleration are used, although the velocity is not measured but is determined by integration of acceleration. Acceleration might be considered the better parameter to measure at the testpiece because recorded time-histories of earthquakes are normally those of acceleration. However, accelerometers used for shaking table measurements lose their accuracy at frequencies lower than about 10 Hz, whereas displacement transducers lose accuracy above this value. The issue therefore revolves around the frequencies at which the table is required to operate, and this in turn depends on the scale of the testpiece, together with the fact that the recorded

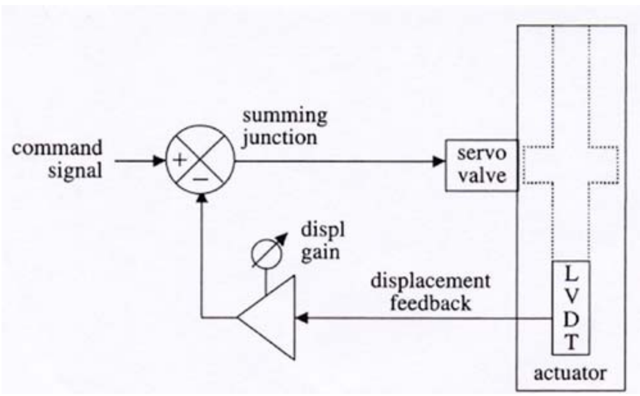

Fig. 5 Uni-axial control using displacement feedback

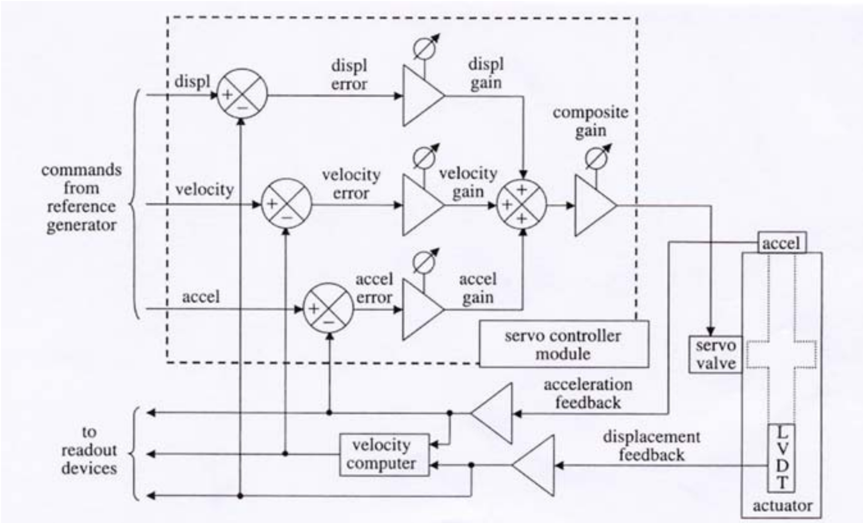

Fig. 6 Uni-axial control using displacement velocity and acceleration feedback signals

earthquake signals contain little energy above about 8 Hz. If the testpiece is at a scale of1/N of the prototype, the requirements of dynamic similitude require that the frequency content of the input be factored by N, thereby increasing the frequency content of the input signal, and making it likely that accelerometer measurements would be appropriate as a basis for the control process. But if the testpiece is at large or full scale, displacement measurements would be more suitable. Clearly, the ideal solution would be a control system which uses testpiece displacements below some critical frequency, and accelerations above this. There is, however, another important issue; it is that if the displacement of the table is known, the acceleration is uniquely defined, but the converse is not true. Thus, if acceleration is the control parameter, it is possible for unknown drifts of the table to occur, possibly beyond the displacements limits of the table. The solution to these issues uses a composite displacement, made up of a direct displacement below a critical frequency and a displacement derived by twice-integrated acceleration above this frequency. Suitable filters ensure that drift errors are removed.

Time Taken for the Control Calculations: A finite time interval is required for the measurement of the corrupted signal at the testpiece, and for the calculation of the error used to modify the initial input. To comply with signal processing theory, the rate of sampling at the testpiece should be at least twice the maximum frequency to be measured, and since tables up to around25 m^2 can be controlled in the 0–100 Hz range, this frequency is more than 200 Hz; that is, a sample should be taken every 0.05 s or less. But when European research on these issues began in 1995, the calculation of the error took far longer than this, and the control system was therefore said to be “out-of-real-time”.

Furthermore, the iterative removal of the error will only be stable if the testpiece maintains its initial physical state. If the testpiece begins to fail œ a not uncommon purpose in earthquake engineering testing œ an unstable control situation occurs. In practice, this has meant that testing must proceed cautiously from the beginning by a series of steps using factored input so as to avoid non-linear behaviour and damage to what could be a unique or expensive testpiece. Such a constraint on shaking table testing was a serious drawback, because modern economic design requires progressive planned failure of non-critical components, making use of non-linear material properties, but avoiding collapse.

Multi-axis Input: having discussed the problems associated with input from a single actuator, now turn to multi-axis input. If the number of permissible axes of table movement (i.e. DOF) is equal to the number of actuators, the control problem simply requires a repetition for each actuator of the arrangement shown in Fig. 6. But additional problems arise when the number of actuators -probably 8-is greater than the 6 DOF. In practical terms, this means that only certain configurations of the actuators are possible, and unless all actuators are moved to a possible configuration, some energy will be wasted in distorting the table itself. Resolution of these difficulties requires a study of the kinematics of the table in order to show that it is necessary to be able to compute the 8 actuator lengths for a given position of the table; this is referred to as the "inverse kinematic problem". Similarly, an algorithm is required which will perform the converse problem of calculating the position of the table from knowledge of the length of each of the 8 actuators; this is referred to as the "direct kinematic problem". The inverse problem is capable of straightforward real-time solution, which is fortunate, because a control signal is applied to each actuator to produce the required 6DOF displacement of the table. The solution of the direct kinematic problem is mathematically more difficult, and although it is not strictly required in the control process, it is useful in obtaining checks on the performance of the control system.

Implicit in the foregoing discussion is the assumption that control of the table is based on displacement, but the same is true of acceleration, although the measurement of the 6 components of acceleration is not so straightforward.

It is seen that the control of a multi-axis shaking table is based upon its inverse kinematic model. It is accurate if the model is error-free, which, in turn, depends on the accuracy of the measured geometry of the total system œ table and actuators. Because there is a limit on the resolution of available surveying instruments for this purpose, a technique has been introduced for correcting the measured geometry based upon the use of the redundancy of the system.

5 Postscript

This paper describes recent shaking table developments which have brought European facilities up to, and in some cases beyond, corresponding facilities in the USA and Japan. In the former, the \$80 million NEES (Network for

Fig. 7 NEES Three 2DOFShaking Tables at the University of Nevada (Reno)

Earthquake Engineering Simulation) Programme has provided new and enhanced experimental facilities for studying the same variety of topics as are being pursued in Europe. For example, Fig. 7 shows the new facility at the University of Nevada (Reno), consisting of three 2DOF shaking tables for the study of multiple-support input. In Japan a 1200T capacity table has been constructed, and both here and in the USA, our own MCS control algorithm has generated mutual interest and collaboration.

Acknowledgements The author expresses his sincere thanks to colleagues in different European laboratories who have collaborated with him in producing the research results recorded here, establishing a European Research Area in the experimental aspects of earthquake engineering. However, without the support of the European Commission's Access to infrastructure programme, none of this would have happened; Marco Mala carne and Panayotis Moschopoulos are given special thanks.

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The Seismic Behavior of Reinforced Concrete Structural Walls: Experiments and Modeling

P. Kotronis, J. Mazars, X.H. Nguyen, N. Ile, J.M. Reynouard, P. Bisch and A. Coin

1 Introduction

Many European buildings are situated in seismic regions of low or moderate seismicity. Among these, a large part is not designed under parasismic regulations. Within this context the evaluation of the vulnerability of existing structures is an important issue. In the framework of the European Community Ecoleader programme, a seismic research project has been performed around shaking table tests on mock-ups representing parts of reinforced concrete buildings, the structure of which is based on structural walls. The program concerns two mock-ups: a Slovenian one and a French one (Fig. 1), the tests being performed at the laboratory LNEC in Lisbon.

This work is related to the analysis of the response of the French mock-up. The structure is characteristic of a typical building met in France designed according to the European regulation EC8-1 with the French appendix. It is composed of two parallel walls linked with a perpendicular one that has openings. All the walls are designed for the seismic level prescribed for a typical seismic region in France.

Two orthogonal directions of loading have been considered, X (parallel to the main walls) and Y (parallel to the wall connecting the main ones). Natural accelerograms at different levels have been used (PGA = from 0.3 g to 0.85 g for direction X and from 0.14 g to 0.50 g for direction Y) and various data have been collected from the different tests (strain on reinforcements, displacements, accelerations...). In order to follow the evolution of the stiffness, the apparent mode has been measured after each test.

Two kinds of modeling are performed hereafter: a simplified one using multifiber beams and a refined one, based on a 3D finite element description of the mock-up.

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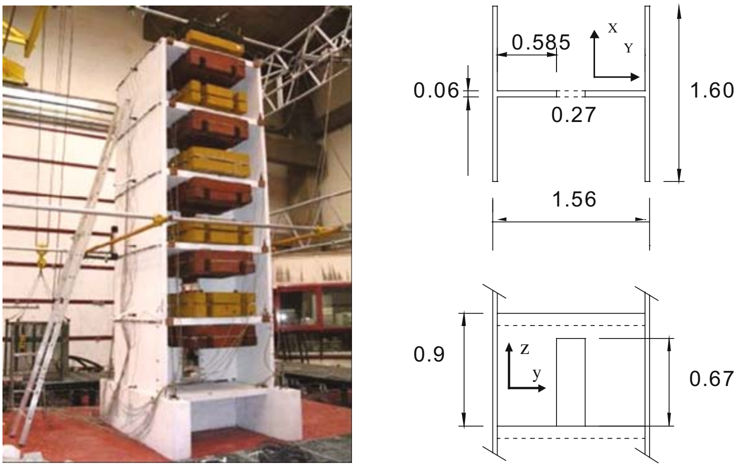


Fig. 1 Geometrical data of the French mock up (m)

Constitutive laws are based on damage mechanics and plasticity to describe cracking of concrete and the plastic behavior of steel. In order to reproduce correctly the behavior of the structure the stiffness of the shaking table have to be introduced in the models.

It will be shown that both models are able to describe the global behavior of the structure and qualitatively the distribution of damage. This conclusion confirms the results of previous works done on structural walls in France (Bisch and Coin 2007). For more detailed information on this test see (Bisch and Coin 2005), (Mazars et al. 2005), (Nguyen 2006).

2 Outline of the Test

Two kinds of artificially generated earthquake motions independent to each other, were applied in the X and Y directions. The sequences of tests are given in Table 1.

Table 1 Sequences of the tests

Tests	In plane (direct. X)	Out of plane (direct. Y)
T0	0.3 g	–
T1	–	0.14 g
T2	0.24 g	0.13 g
T3	0.45 g	0.27 g
T4	0.55 g	0.30 g
T5	0.74 g	0.36 g
T6	0.85 g	0.50 g

The main damages appeared at the base of the walls and one rebar has buckled (but not broken) for a signal closed to the design level (X PGA = 0.45 g – Y PGA = 0.27 g). For higher levels damage increased and at 0.85 g some rebars broke at the base of the walls where a large fracture appeared.

3 3D Model

To predict the inelastic seismic response with sufficient accuracy, due care has been given to create a detailed model of the specimen, taking into account the necessary geometric characteristics, construction details and boundary conditions. An example of the 3D finite element mesh used in the analyses is reported in Fig. 2. Due to the direction of the applied loading, in-plane as well as out-of-plane behavior of the walls need to be analyzed. A discrete modeling is adopted to represent the reinforcement through the use of two-node truss-bar elements. The structure is assumed fully restrained at all nodes along the base of the shear wall. During previous tests on CAMUS specimens (Bisch and Coin 2007), it was observed that the specimen oscillation have induced vertical and rocking displacements on the shacking table, leading to significant reductions of the

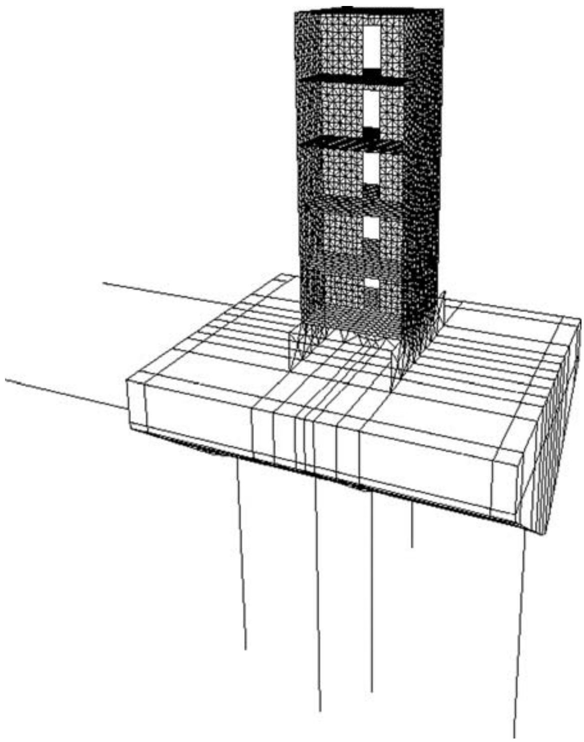


Fig. 2 3D finite element mesh of the specimen

corresponding natural frequencies. Therefore, the shaking table itself in terms of mass and its external supports in terms of stiffness have to be included into the numerical model. Perfect bond is assumed to exist between concrete and reinforcement. The possibility of non-linear material behavior is specified for all wall concrete and reinforcing bar-elements, while the behavior of the foundation and bracing system is considered as elastic. Assuming a 1% critical damping factor for the first and second vibration mode, the damping parameters a and b are calculated and used subsequently to form the Rayleigh damping matrix [C] = a[M] + b[K], M and K being the mass and stiffness matrix.

The concrete model used in analysis (Merabet and Reynouard 1999) adopts the concept of a smeared crack approach with a possible double cracking only at 90°. It is based upon the plasticity theory for uncracked concrete with isotropic hardening and associated flow rule. Two distinct criteria describe the failure surface: Nadai in compression and bi-compression and Rankine in tension. Hardening is isotropic and an associated flow rule is used. When the ultimate surface is reached in tension, a crack is created perpendicularly to the principal direction of maximum tensile stress, and its orientation is considered as fixed subsequently. Each direction is then processed independently by a cyclic uniaxial law (Fig. 3), and the stress tensor in the local co-ordinate system defined by the direction of the cracks is completed by the shear stress, elastically calculated with a reduced shear modulus to account for the effect of interface shear transfer: The model has been described in detail and verified elsewhere (Ile and Reynouard 2000), (Ile et al. 2002).

For steel, the cyclic behavior is described by the formulation proposed by Giuffré and Pinto and implemented by (Menegoto and Pinto 1973).

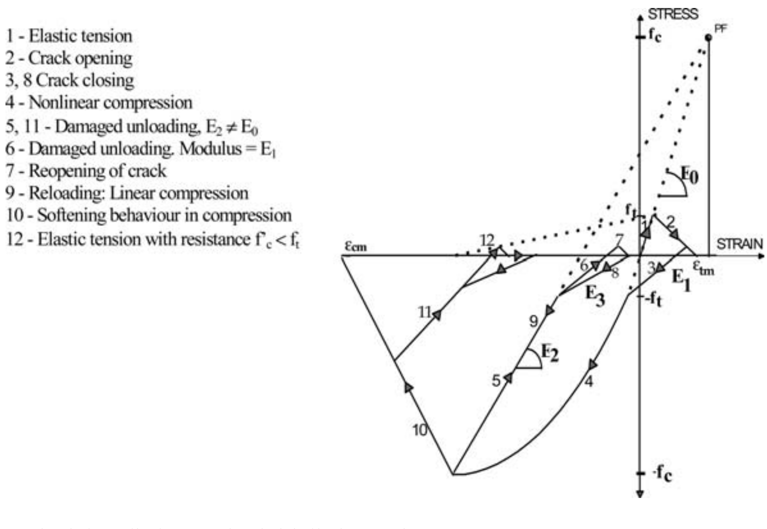


Fig. 3 Uniaxial cyclic law: point initially in tension

4 Simplified Model

Non-linear dynamic analysis of civil engineering structures requires large scale calculations. The necessity to perform parametrical studies led us to adopt a simplified approach in order to reduce the computational cost. The structure is modeled with beam elements to reduce the number of degrees of freedom of the problem. Using an Euler Bernoulli formulation the shear deformations are not modeled so we can use 1D versions of the non linear constitutive laws in the fibers (torsion is also kept linear). The multifiber beam used is the one implemented in the finite element code Aster (Ghavamian et al. 2002).

The finite element mesh is presented in Fig. 4. The additional masses and the weight load of each floor are concentrated at each storey. The Rayleigh damping coefficients have been adjusted to ensure a value of 1% on the two first modes.

The reinforcement steel is modeled with an isotropic cinematic hardening law. Constitutive model for concrete under cyclic loading ought to take into account some observed phenomena, such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and inelastic strains concomitant to damage. To simulate this behavior we use a damage model with two scalar damage variables one for damage in tension and one for damage in compression (La Borderie 1991). Unilateral effect and stiffness recovery (damage deactivation) are also included. Inelastic strains are taken into account thanks to an isotropic tensor (Fig. 5).

5 Main Results

The modal analysis has been performed using both models in order to insure that the boundary conditions and the distribution of the masses are well represented. The stiffnesses of the springs below the shaking table are identified to feet the first eingenmodes measured on the virgin structure before the test.

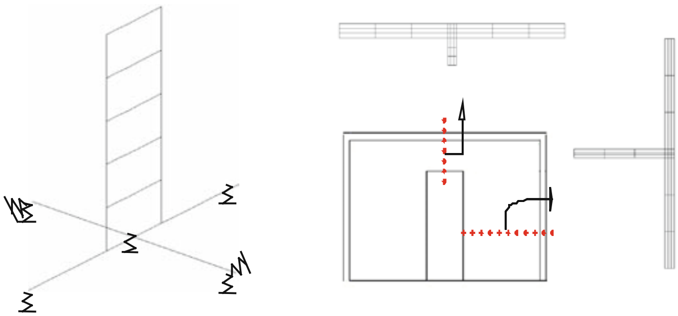


Fig. 4 Simplified finite element mesh of the specimen

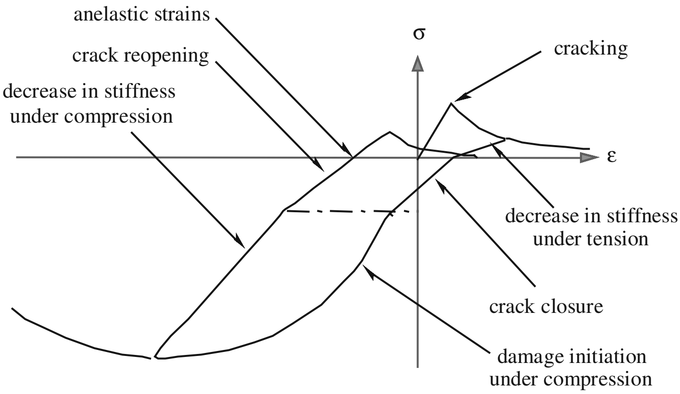
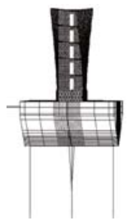


Fig. 5 1D cyclic response of the La Borderie model

Table 2 gives a comparison of the two approaches with the experimental results for the first three natural modes.

For the *3D model (refined model)* numerical analyses have been performed using the CAST3M finite element code. All the seismic signals applied to the specimen were considered in chronological order. The first comparisons presented in Figs. 6 and 7 concern the relative horizontal displacements in X and Y directions corresponding to the T5 (0.74 g) input motion, which caused significant damage to the specimen. The calculated response is generally under-

Table 2 Modal analysis

Model	In plane (direction X)	Out of plane (direction Y)	Torsion
Fiber model	 4.54 Hz	 7.0 Hz	11.0 Hz
3D model	 4.5 Hz	 7.06 Hz	 9.9 Hz
Experiment	4.5 Hz	7.13 Hz	not known

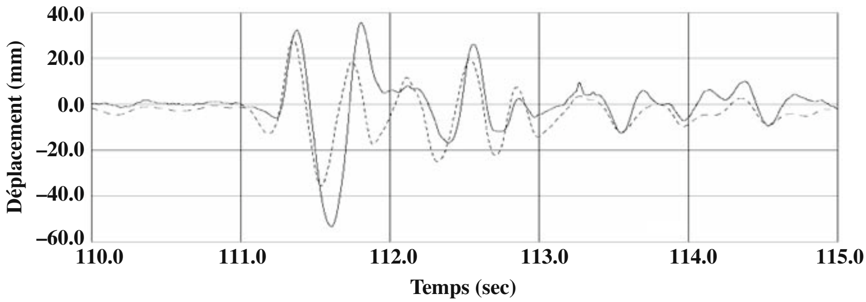


Fig. 6 Refined model, comparison between calculated and measured horizontal relative top – X displacement for T5 motion (0.74 g)

estimated in both directions and the correlation between analysis and experiment seems to be better for the X direction as compared to the Y direction. This may be due to the fact that the axial stiffness of the vertical rods supporting the shaking table may evolve during seismic response. It is difficult to take into account this aspect, when the variation of the axial stiffness of the rods is not known in advance. However, even if the numerical results do not match exactly the experimental ones, they give the opportunity to highlight some important characteristics of the structural behavior as described hereafter.

Local results as obtained from the dynamic analysis are presented in Fig. 8. This figure depicts the damage distribution corresponding to the maximum top displacement attained in the X direction for the T5 applied motion. The analysis results indicate that more damage is to be expected in the X walls as compared to the Y wall. They also show large compressive strains at one end of the X wall, indicating that concrete may fail in this location due to excessive strains. Actually, this seems to be in reasonable agreement with what was experimentally observed (Fig. 9): the wall extremities were heavily damaged in compression and steel bars buckled and have broken after that at this location.

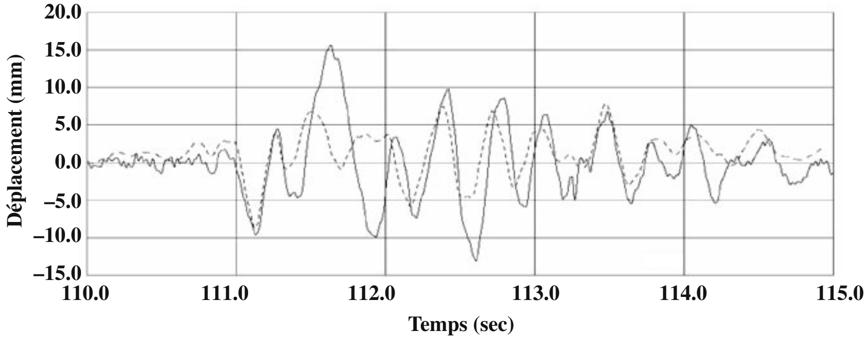


Fig. 7 Refined model, comparison between calculated and measured horizontal relative top – Y displacement for T5 motion (0.74 g)

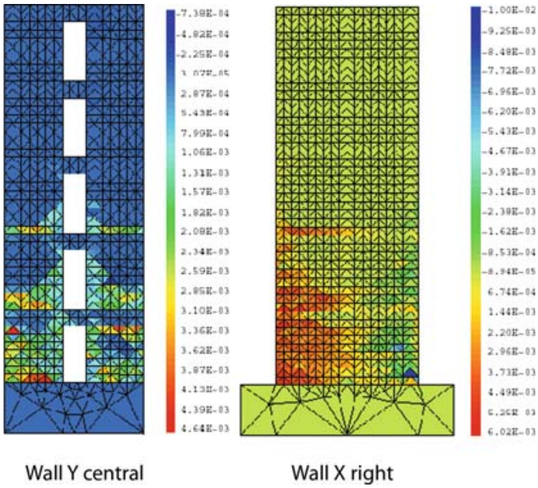


Fig. 8 Refined model, vertical concrete strain contours for T5 motion

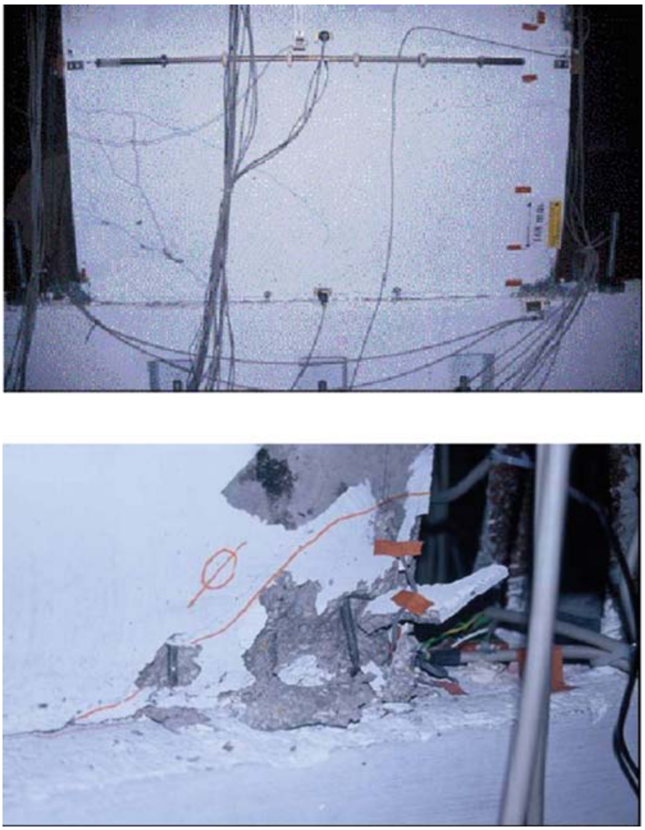


Fig. 9 Experimental damage pattern for T5 motion

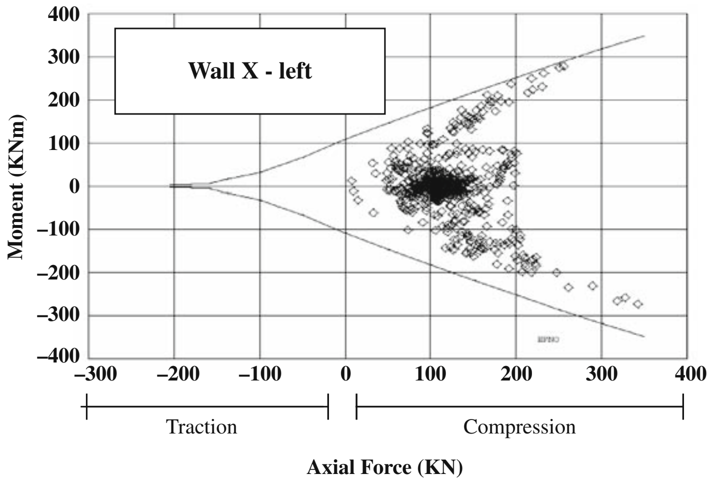


Fig. 10 Refined model, bending moment – axial force interaction diagrams and variation of bending moment and axial force at the base of the 1st storey (T5 motion)

Figure 10 presents numerical results in terms of bending moment – axial force interaction diagrams at the base of the 1st storey together with the variation of axial load and moment. This confirms the observed behavior and failure mode, since limit states tend to be obtained with high axial force values.

The numerical results of the *simplified model* are presented in Figs. 11 and 12. The time history of the calculated roof displacement is compared with the corresponding measured displacement for the T5 sequence. Simulation predicts satisfactory the maximum displacement for both sequences and there is no significant shifting between the curves.

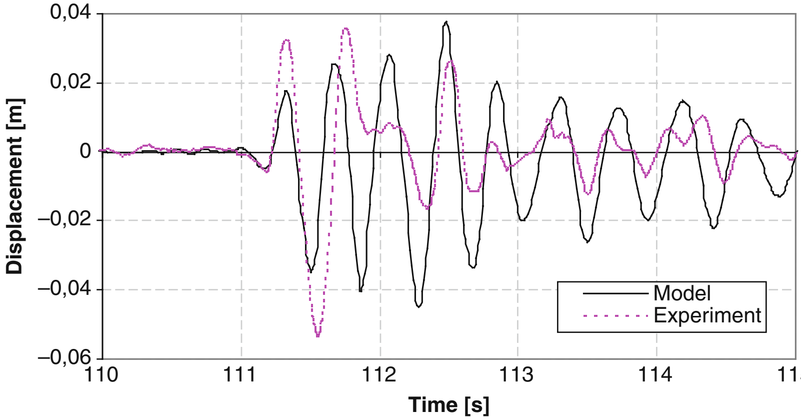


Fig. 11 Simplified model, comparison between calculated and measured horizontal relative top-X displacement for T5 motion (0.74 g)

The damage variable vary normally between 0 (non damaged section) and 1.0 (completely damaged section). By filtering their values between 0.95 and 1.0 we omit the micro-cracks and we have an image of the bigger cracks of the model. Figure 13 presents the damage pattern due to tension at the end of the calculation for the T5, where we can see that damage is concentrated at the base.

The evolution of the top displacement for T6 is presented in Figs. 14 and 15. Comparison of the distribution of damage and strains in the steels (Fig. 16) with the actual position of cracks (Figs. 17 and 18) shows again that the model is able to reproduce qualitatively the local behavior observed experimentally.

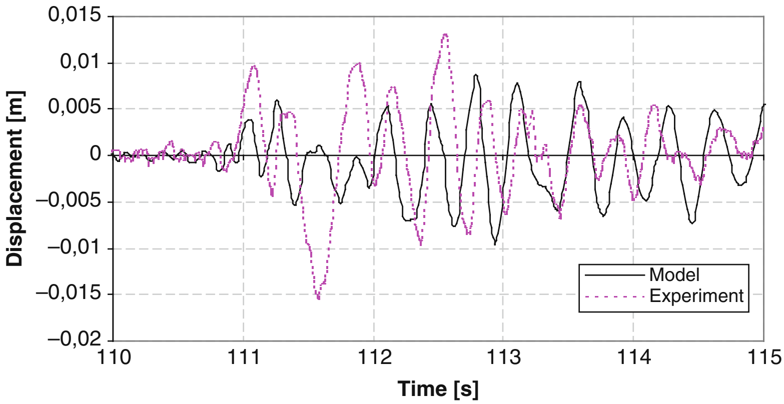


Fig. 12 Simplified model, comparison between calculated and measured horizontal relative top-Y displacement for T5 motion (0.74 g)

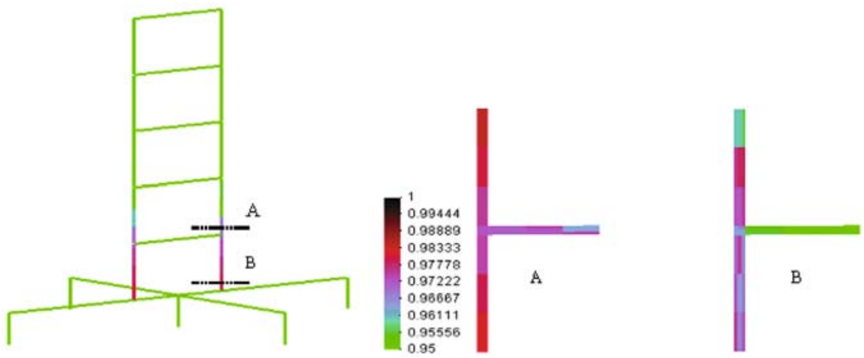


Fig. 13 Simplified model, state of damage for T5 motion

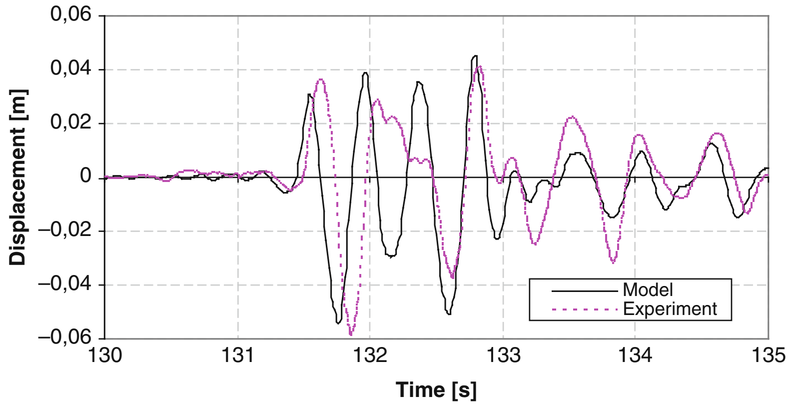


Fig. 14 Simplified model, comparison between calculated and measured horizontal relative top-X displacement for T6 motion (0.85 g)

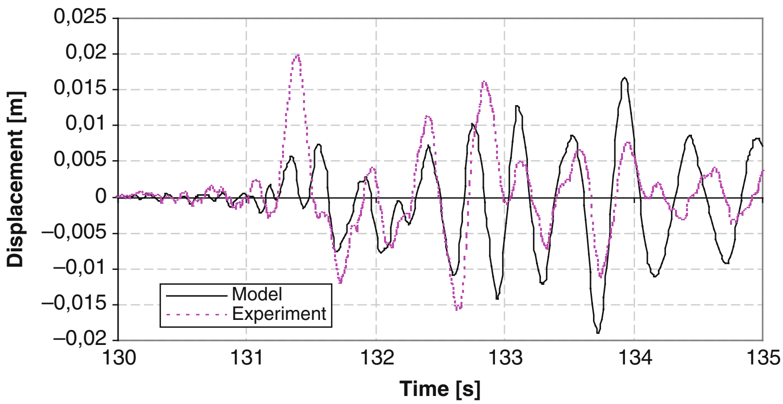


Fig. 15 Simplified model, comparison between calculated and measured horizontal relative top – Y displacement for T6 motion (0.85 g)

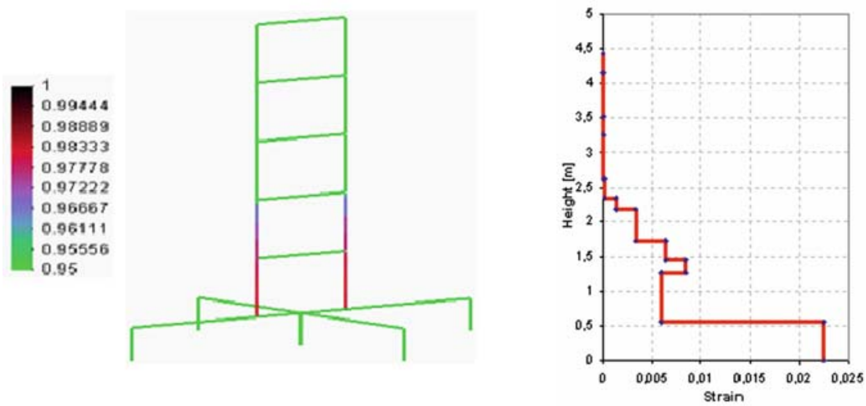


Fig. 16 Simplified model, state of damage in the structure and strain of reinforcement at the base of the wall X right for T6 motion

Main damages are again located at the base of the specimen. Furthermore, as shown in Fig. 16, the simplified model provides strains exceeding 2%, a limit corresponding to the ultimate strain of the reinforcement used in the experiment.

Fig. 17 Experimental damage pattern for T6 motion

Fig. 18 Broken steel at the base of the wall X right for T6 motion

6 Conclusions

As demonstrated by the results presented in the paper both models were able to reproduce with good approximation the global response of the structure and qualitatively the distribution of damage. A refined 3D model allows obtaining detailed information about the behavior of the specimen under complex loading conditions whereas a simplified approach helps reducing computational cost.

The works highlights once again the importance of the axial force variation and its influence on the failure mode to be expected in the case of lightly reinforced walls. Based on the results obtained in this study, it appears now possible to use these models to investigate numerically the behavior of a wider variety of configurations that is practically impossible to study experimentally.

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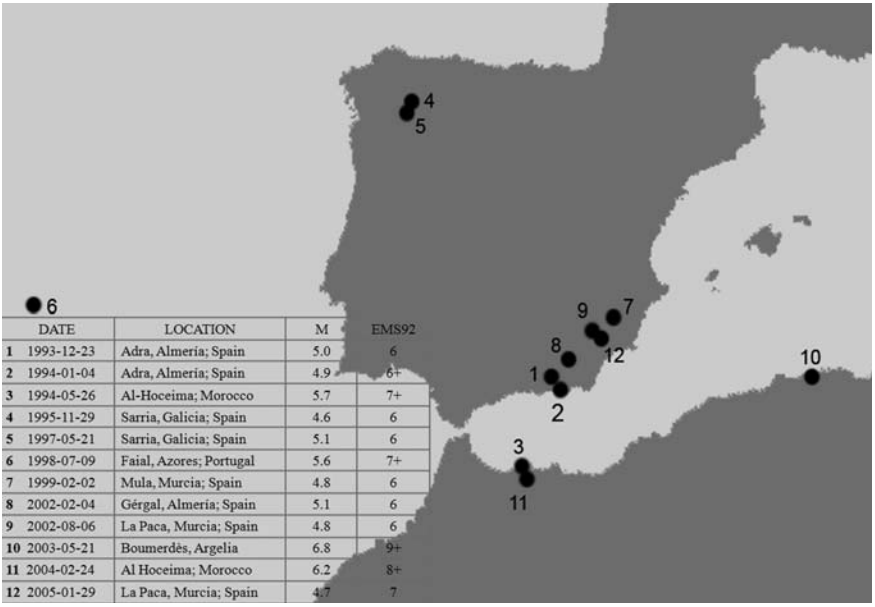
Building Performance During Recent Earthquakes in the Iberian Peninsula and Surrounding Regions

P. Murphy Corella

1 Introduction

This work presents a summary of field studies conducted by the author of earthquakes in Iberia and surrounding regions including the western Maghreb and the Azores islands from 1994 to 2005. During this period at least twelve



	DATE	LOCATION	M	EMS92
1	1993-12-23	Adra, Almería; Spain	5.0	6
2	1994-01-04	Adra, Almería; Spain	4.9	6+
3	1994-05-26	Al-Hoceima; Morocco	5.7	7+
4	1995-11-29	Sarria, Galicia; Spain	4.6	6
5	1997-05-21	Sarria, Galicia; Spain	5.1	6
6	1998-07-09	Faial, Azores; Portugal	5.6	7+
7	1999-02-02	Mula, Murcia; Spain	4.8	6
8	2002-02-04	Gérgal, Almería; Spain	5.1	6
9	2002-08-06	La Paca, Murcia; Spain	4.8	6
10	2003-05-21	Boumerdès, Argelia	6.8	9+
11	2004-02-24	Al Hoceima; Morocco	6.2	8+
12	2005-01-29	La Paca, Murcia; Spain	4.7	7

Fig. 1 Earthquakes considered for this work in Iberia and surrounding areas

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events with magnitudes ranging between 4.7 and 6.8 have caused damage to buildings during this time, including two large scale catastrophic events in Algeria and Morocco. A summary of the main events happening during this period and their locations are included below in Fig. 1.

2 Building Types Considered in this Work

2.1 Traditional or Vernacular Structures

Undressed fieldstone set into a lime mortar forms the basis of the traditional building stock in the Western Mediterranean, with numerous variations, all originating from the Roman bond Opus Incertum. Unlike Northern Europe, wood is scarce and an expensive material in the Mediterranean basin and thus limited to roof and floor systems, or large span beams and trusses in more significant buildings. With the exception of some areas in the Atlantic seaboard of Iberia, almost all buildings are rendered externally in a whitewash or mortar render. Roof systems include curved ceramic tiles over wooden joists, with slate or other materials used in specific regions. In the drier areas of Iberia and over much of the Maghreb, roofs are flat and often used as terraces.

Brick is a common material found in townhouses of Iberia Figs. 2–9, generally indicative of higher quality constructions. Brick courses alternating with areas of stone are also common in historical buildings and originate from the Roman Opus Mixtum bond. In towns, brick forms the basis of the three to five story townhouses dating from the 18th century onwards and forms an important part of the building stock.

Fig. 2 Traditional fieldstone construction exposed in a damaged building and a typical contemporary RC frame construction for housing in Spain

2.2 Engineered Structures

Reinforced concrete is the main engineered structure type in contemporary construction in the region allowing for the continued use of masonry trades in the form of brickwork for internal partitions and facades and stonemasonry and plasterwork for internal finishes. Reinforced Concrete (RC) structures became widespread in Iberia from the 1950s onwards concrete is the prevailing form of construction for most building types. RC construction in Iberia has almost completely replaced load-bearing masonry even in small-scale constructions like detached housing. There is a leapfrogging effect in many rural areas where engineered structures coexist with an aged vulnerable building stock. An RC frame, with a continuously cast structure provides a clear improvement for seismic performance and is considered at least vulnerability C in the EMS scale. Values D, E and F represent increasing seismic performance depending on the design earthquake and other factors like the existence of quality control procedures that guarantee a correct execution during construction.

2.3 Earthquake Resistant Design (ERD)

ERD began to be applied in Spain for ordinary construction from 1968 onwards with significant revisions in 1994 and more recently in 2002. In the 2002 revision, ordinary buildings in seismic areas >0.08 g have to comply with the earthquake codes and are best defined from having a moderate level of ERD as defined by the EMS authors, and therefore assigned vulnerability D. In areas >0.16 g, buildings are expected to meet a high level of earthquake resistant design and are best defined as vulnerability E. In the RC construction culture of the region, seismic codes are generally met by either the insertion of RC shear walls or with moment resistant frames, with a preference for the latter system, probably because it has the least impact in the plan of a building.

2.4 Building Codes

Any study of damage to buildings should be set against the codes which where current during construction. In the case of Spain, this analysis is enlightening. The first building codes and hazard map originate from 1962, and the national codes have been revised in 1968, 1974, 1994, and more recently in 2002. The hazard map of 1974 was in effect until 1996 when the 1994 code came into full effect. It is interesting to compare the 1974 and 2004 maps, especially in terms of the areas where code compliance is mandatory, since the 1974 code was only applicable to normal construction in a very small portion of the country. This area was greatly extended in the 1994 and 2002 revisions. The 1968 map however, is remarkable for its longevity, being in effect for 22 years during a period of massive urbanization in Spain, resulting in a consolidated building stock of RC frames with no particular provision of seismic loading in the same areas where the code is mandatory today.

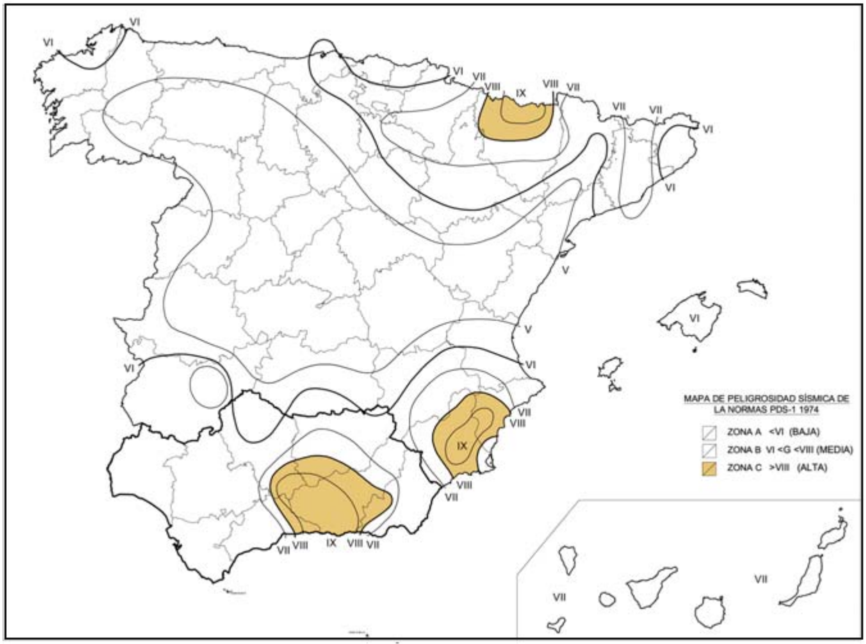

Fig. 3 1974 hazard map from the Spanish building codes in effect until 1996, shows the geographical areas where the code was mandatory for RC structures of normal importance

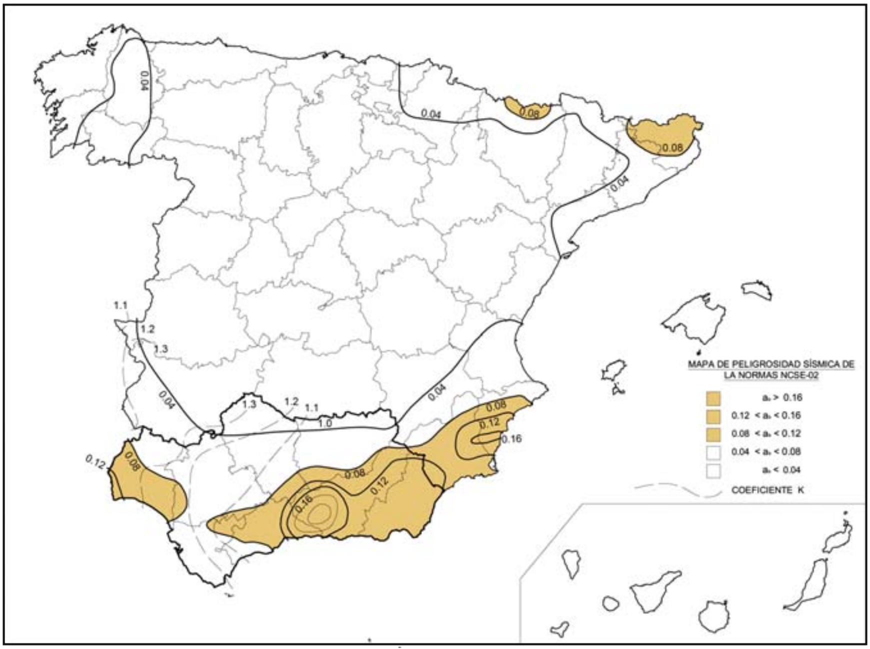

Fig. 4 2002 hazard map from the Spanish building codes, currently in effect, shows the geographical areas where the code is mandatory for RC structures of normal importance

3 Damage to Masonry Structures

3.1 Shear Damage

Masonry walls exhibit high stiffness in a direction parallel to the plane of the wall but are likely to crack on account of their brittleness and lack of ductility. In the area under study the majority of masonry structures present is either fieldstone or brick structures. In fieldstone structures particularly,

Fig. 5 Shear damage in fieldstone masonry construction in Cedros, Faial and Zarcilla, Murcia, (1998 and 2002 earthquakes) and model of X-crack generation in a masonry wall

Fig. 6 Severe shear damage to the Punta Ribeirinha lighthouse in Faial Island (brick) and to a fieldstone masonry house in Tazaghine, Morocco. (1998 and 2004 earthquakes)

shear damage is observed even in moderate loading, resulting in shear failure with diagonal cracking. Characteristic damage includes X-shaped shear cracks caused by the reversal of loads as the building shunts back and forth. Masonry piers between large openings are particularly vulnerable due to the increased shear loading they experience. In fieldstone masonry construction with particularly weak bonds, shear stress may cause crack swarms leaving a crackling effect on walls, as was widely observed in the Azores Islands.

3.2 Out of Plane Drift

Masonry walls are subject to out of plane drift due to the inertial forces generated by their high mass. This is particularly so in fieldstone buildings where a very high mass coincides with poor coupling between walls due to the nature of fieldstone construction. Generally a heavier wall will detach itself first from a shorter stiffer wall on account of its larger mass. This has become a common and typical diagnosis for damage of grade 2 to buildings in Spain.

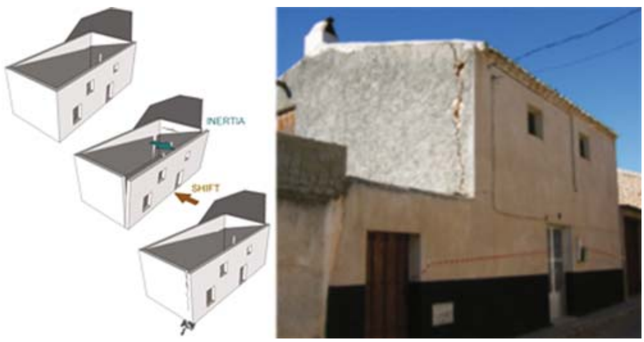

Fig. 7 Out of plane drift in a fieldstone masonry construction in Murcia, Spain. (2005 earthquake)

3.3 Gable Walls

Gable walls are extremely vulnerable to drift and out of plane collapse because they are largely unsupported infill walls unrestrained by the roof structures. Failed gable walls are a common diagnosis for damage of grade 3.

Fig. 8 Gable wall failure in brick and concrete block construction in Murcia, (1999 and 2005 earthquakes)

3.4 Corner Damage

This is a very common type of damage and often related to the loss of connection between bearing walls described earlier, as load reversal in perpendicular walls has a crushing effect on corners. Failed corners leave walls dangerously unrestrained and very vulnerable to out of plane collapse. Failed corners are a typical diagnostic for grade 3.

Fig. 9 Moderate and advanced corner damage to brick and adobe construction in Murcia and Al-Hoceima. (1999 and 2005 earthquakes)

3.5 Bearing Wall Failure

Bearing wall collapse is a diagnostic for damage of grade 4, and often follows wall drift failures. In practice there are often difficulties in the field in distinguishing between grade 4 and 5, as further collapse may be triggered by lack of redundancy and not realistically reflect a linear increase in motion severity.

4 Damage to RC Structures

RC structures have an improved resistance to earthquake loading regardless of any additional seismic loading considerations on account of their increased strength and the nature of the material, resulting in a continuously formed structure and meriting a vulnerability grade C in the EMS scale. During the 60s and 70s, confidence in RC structures was such that they were exempt from code compliance for normal construction in most of the country, under the assumption that the inherent characteristics of the material guaranteed loading stability under moderate seismic conditions. This vision has changed now, but the longevity of the 1968 hazard map has resulted in a large building stock of vintage RC buildings with considerable limitations for earthquake loading as has been evidenced on the field.

4.1 Brittle Failure

It is clear now that RC structures with no built-in ductility for earthquake loading are vulnerable to fragile behaviour and brittle failure of critical areas like beam to column or beam to slab connections. This type of damage was widespread during the large magnitude Al-Hoceima earthquake of 2004 but fragile damage has also been observed in more moderate earthquakes in Spain in buildings built with no seismic resistance provisions until 1996, as illustrated in Figs. 10 and 11.

Fig. 10 Bearing wall collapse in Espalafatos, Faial Island and Murcia. (1998 and 2005 earthquakes)

Fig. 11 *Left*: Severe damage to a RC portal frame in Imzouren during the 2004 Al-Hoceima earthquake. (grade 4) *Top*. Moderate fragile damage to a column and beam connection in Adra during the 1994 earthquake. (grade 3) *Bottom*: Moderate damage to a column and beam connection in Zarcilla during the 2005 earthquake. (grade 2)

4.2 Soft Storey Effects

Varying stiffness between floors in RC structures may focus seismic load forces onto individual elements and trigger fragile failure. This may be caused by geometric variations, (same column sections but varying column lengths between floors) or by mechanical reasons. (Accidental stiffening caused by non-structural elements) These factors often concur on ground floors of

Fig. 12 Soft story damage with severely deformed ground floor and intact upper floors in Imzuren. (2004 Al-Hoceima earthquake)

Fig. 13 Moderate soft story damage in Murcia. Note the stiff brick upper floors over the diaphanous garage. (1999 earthquake)

apartment buildings where the upper floors, stiffened by non-structural distribution walls lie over open-plan ground units designed for commercial space, garages, or other uses, typically with a much higher ceiling height often calls for mixed use buildings with commercial or tertiary activities on the ground floors at street levels and housing in the floors above. This situation encourages a soft storey performance of the ground floor due to the higher floor heights required for commercial use. The resulting difference in relative stiffness between the ground floor and the rest of the building produces the so-called soft storey effect. This weakness is particularly widespread because its occurrence has remained unchecked for large parts of Spain until the code renewal of 1994. Figures 12, 13 and 14 show different degrees of soft storey failures including the total collapse of the ground floor which was observed in at least twenty buildings in the 2004 Al-Hoceima earthquake.

Fig. 14 Building collapse motivated by a soft storey effect in a RC apartment building in Al Hoceima during the 2004 earthquake. Compare this process to Figs. 12 and 13

4.3 Captured Columns

Confinement caused by non-structural elements such as brick infill panels may alter the mechanical properties of a column, resulting in a condition known as a captured or short column, effectively reducing its active length. Again, this effect has been largely unchecked in Spain and is still common today on account of the popularity of stiff infill panels.

Fig. 15 Short column effect; damage to a column caused by interference of masonry panels in an apartment building in Imzuren. (2004 earthquake)

Fig. 16 Short column effect caused by geometric irregularities in Murcia. (2005 earthquake)

4.4 Non-structural Elements in RC Construction

Most non-structural elements in RC construction in Spain such as partition or closure walls are built in brick or masonry with rendered finishes. This solution enjoys a long tradition in the regional construction culture on account of its excellent hygienic and climatic properties, but poses specific challenges for earthquake resistance. Problems arise with the incompatibility between the stiffness of brick and masonry and the relative flexibility and deformability of RC frames. There is a tendency therefore, for masonry panels to interfere with the performance of the RC frame, in particular in attracting and supporting seismic loads before the frame has a chance to deform. Current codes limit this problem partly by restricting the amount of ductility –and therefore deformability- of the frame if masonry infill panels are foreseen in design. Field evidence however, confirms a continuous conflict between masonry and frame interactions. The damage observed in Figs. 15 and 16 are typical cases.

Fig. 17 Damage to masonry infill panels caused by interference between structural and non-structural elements. (Lorca 2005 earthquake)

Fig. 18 Concrete block gable wall failure in a steel framed industrial building in Murcia, Spain. Brittle claddings over flexible steel structures are incompatible for seismic conditions. (1999 earthquake)

5 Case Study: Damage to the Civic Centre of La Paca During the 2005 Lorca Earthquake (Figs. 17–23)

A typical performance of a RC frame building in the region can be illustrated with the damage observed to the civic centre of La Paca which suffered extensive non-structural damage during the 2005 earthquake. The building considered here is an L-shaped two-storey RC frame with brick infill panels for internal and external walls. The building dates from 1992 and as such was executed under the 1974 building code, which did not require seismic considerations for buildings of normal importance although the same location today would require code application.

The building suffered severe non-structural damage to many internal and external infill panels, which were sheared in large X-shaped cracks. There were also full internal partition failures, showering seating areas with rubble. No injuries were sustained as the earthquake happened a few minutes before the building opened to the public.

Fig. 19 Views of damage to external and internal infill panels in the Civic Centre of La Paca during the 2005 Lorca earthquake

In geometric terms, analysis of the building plan reveals an L-shaped building with no structural joints, resulting in an obvious eccentricity between the centre of rigidity and centre of mass. In mechanical terms, the building has no primary stiffening elements and depends solely on the coupling effect between columns and floor slabs. If we consider that all columns were round, thus exhibiting a reduced leverage effect as compared to square columns of the same sectional area, the building suffers from a clear lack of stiffness and a tendency to torsional effects on account of its uncorrected geometric asymmetry.

The building was modelled for its 3D analysis and subject to a torsional motion around its approximate centre of rigidity, resulting in large displacements on the ends of its perpendicular bars that closely match the locations where damage was recorded in the field.

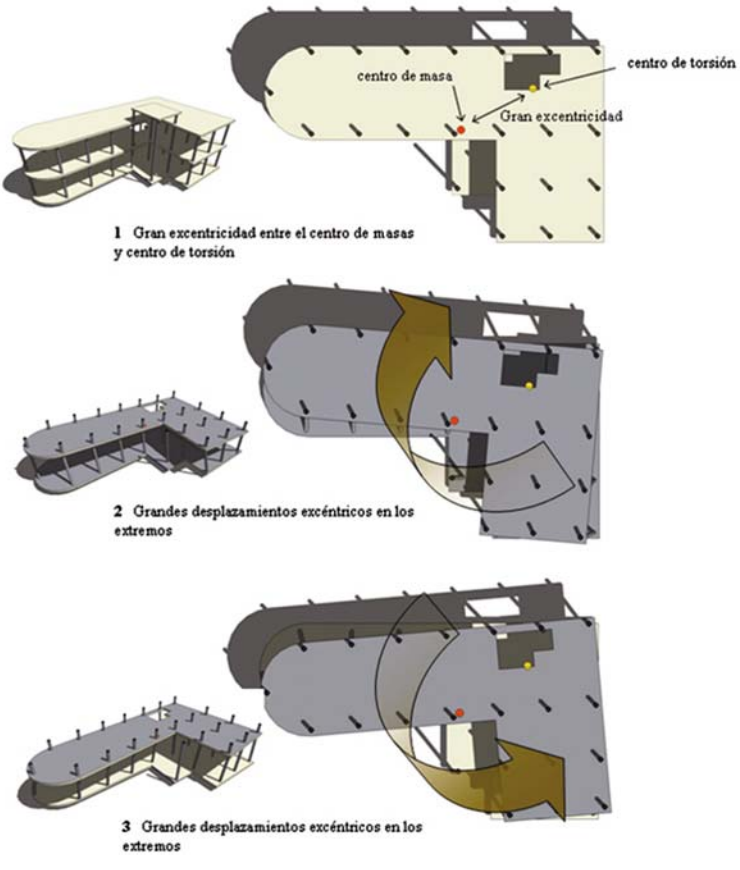

Fig. 20 Analysis of the existing building revealing poor rigidity and an eccentric floor plan

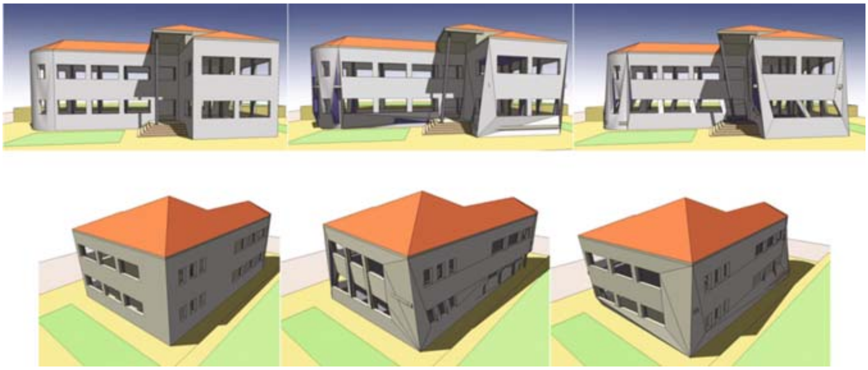

Fig. 21 Torsion deformation applied to the model seen from the front (*top*) and back. (*Bottom*)

In particular, damage was found to coincide with the impact locations of the first floor slab, which punctured sections of the infill wall out, and in the severe shear damage to the first floor end of the bars, which would support the idea of damage exacerbated by the torsional motion of the building.

The building was then subject to an analysis of the design requirements of the current codes, which would require both bars to be made independent from each other to reduce the torsional effect, as well as generally stiffening up the building to reduce its drift for use with masonry infill panels.

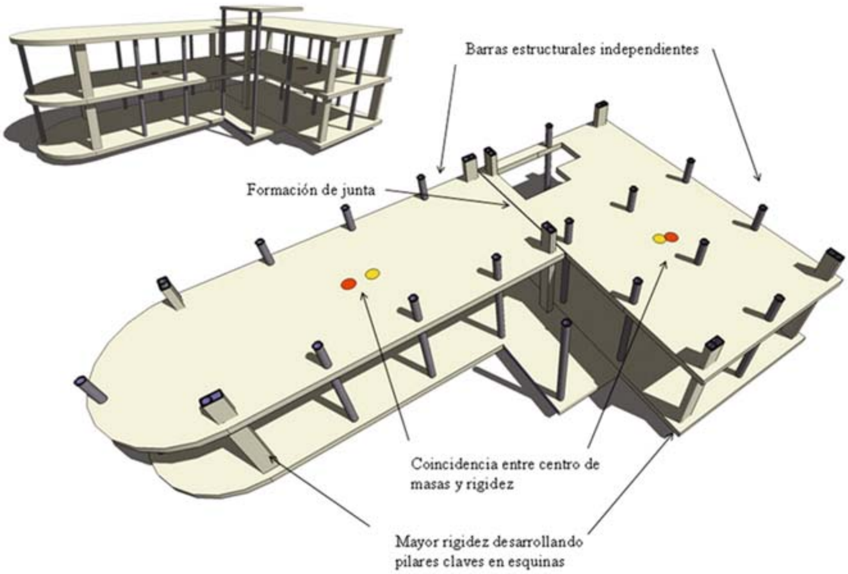

Fig. 22 Idealised structural solution if the building were to comply with the current codes

6 Conclusions

Flexible RC frames with stiff brittle brick infill walls are widespread in Iberia and exhibit severe non-structural damage even in moderate earthquakes. The common Spanish block of flats with commercial use on the ground floor has a

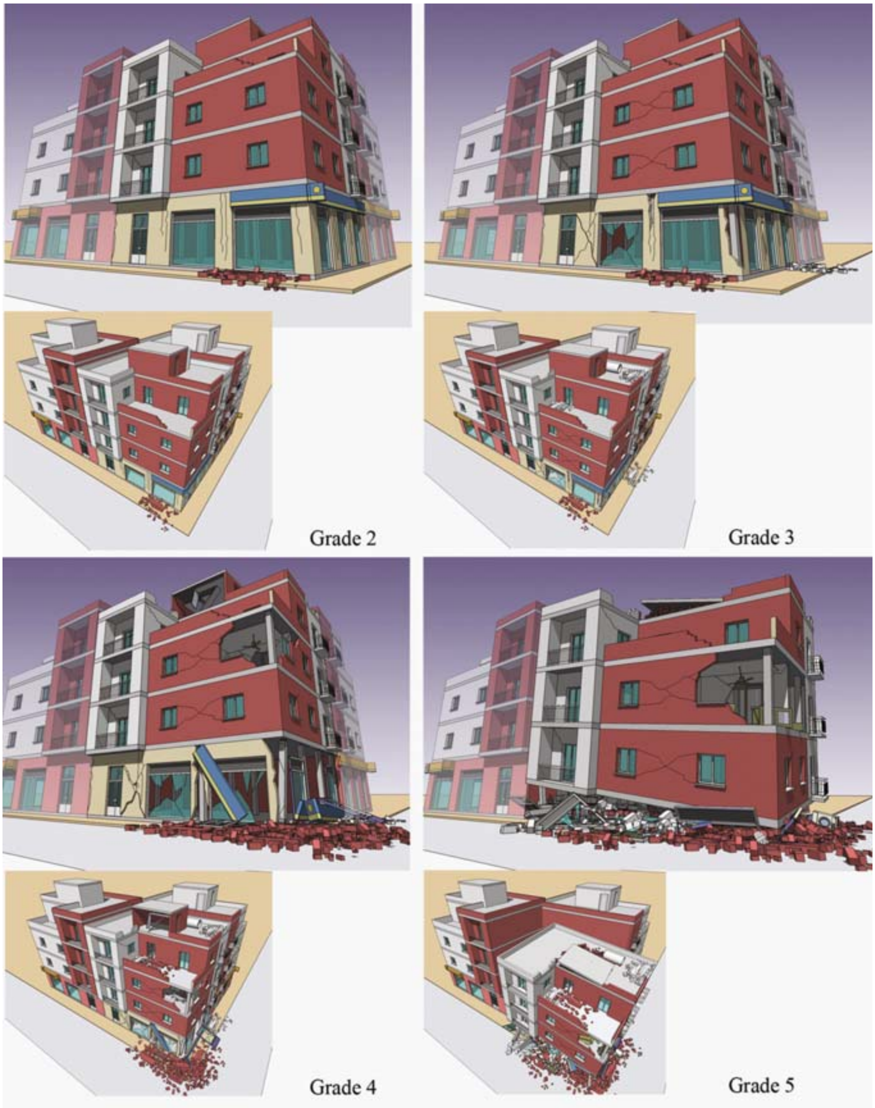

Fig. 23 Damage grades 2–5 corresponding to the EMS 98 scale applied to a typical RC framed building in Spain

large soft-storey potential evidenced in the field with earthquakes that have happened in the last 10 years. The Al Hoceima earthquake of 2002 resulted in many ground floor collapses of buildings of a similar typology to those found in Iberia and suggests damage of this type is to be expected in Spain under conditions of severe ground motion, particularly for the building stock previous to the 1994 code revision. On the other hand, the commonness of this building type and its associated soft story risks should be the object of specific attention during future revisions of the NCSR-02 Spanish seismic code. Meanwhile, continued damage to non-structural (masonry) elements in RC construction is to be expected until specific solutions are spelled out regarding their interaction. Because of how deeply masonry construction is embedded in the regional construction culture, the code should be more specific about the stiffness of RC frames to accommodate widespread use of brick and masonry partitioning, which is likely to continue in region. In this aspect, the use of stiff RC shear walls in buildings with brittle masonry elements that are likely to become loaded with frame deformation seems logical and may be one solution to be recommended and supported from the pages of the codes.

Finally, a set of drawings have been developed to summarise damage grades observed in RC frames in the region during the last ten years in terms of the EMS scale, in order to contextualise the scale to this region.

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Part VII
New Approaches to the Seismogenesis on the 1755 Earthquake

Seismotectonics of the Azores-Tunisia Region

E. Buforn

1 Introduction

The Lisbon earthquake of 1755 has been considered by many authors as the largest shock ever occurring in Europe, with an estimated epicentral intensity of XI–XII and Mw = 8.5. Several locations for the epicenter and tectonic sources have been proposed in order to explain its origin (for example: Machado, 1966; Buforn et al., 1988; Jhonston, 1996; Baptista, 1998; Zitellini et al., 2001; Gracia et al., 2003; Vilanova et al., 2003; Gutsher, 2004). These different hypotheses for the origin of the 1755 event reflect the tectonic complexity of the Azores-Tunisia seismic active region where it took place. With this term we denominate the region which extends from the Azores Islands to the Strait of Gibraltar in the Atlantic and continues through south of Iberia and to the north of Africa as far as Tunisia. It corresponds to the westernmost part of the plate boundary between Eurasia and Africa, with a transition from an oceanic plate boundary in the western part to a continental boundary in the eastern part.

The region has different characteristics of tectonic, seismicity and the focal mechanisms. The region has been object of numerous studies from the early ones by Udías and López Arroyo (1972), McKenzie (1972), Udías et al. (1976), up the more recent studies done by Jimenez-Munt et al. (2001) and Buforn et al. (2004). This paper drafts the main seismotectonic characteristics of the region.

2 Seismicity

From a seismological point of view, the Azores-Tunisia region may be divided from west to east into three large areas: Azores, Central and Ibero-Maghrebian (Fig. 1). In the Azores area, from 29°E to 24°W (Fig. 1a), earthquakes occur at

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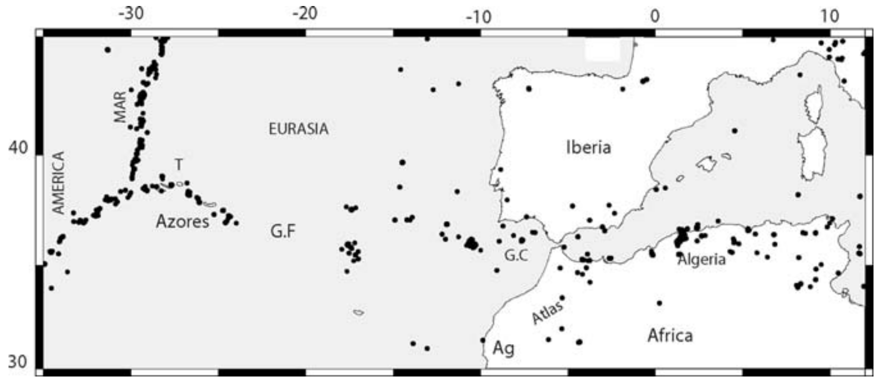


Fig. 1a Distribution of epicenters of shallow earthquakes (h < 40 km) with magnitude greater than 5.0 for the period 1931–2000. MAR = Mid Atlantic Ridge. G.F. = Gloria Fault. T = Terceira Island. G.C. = Gulf of Cádiz. Ag = Agadir

shallow depth and with moderate magnitudes (less than 7.0). In this area the most important seismic activity is concentrated along the Mid Atlantic Ridge, in a N-S direction to the triple point where the America, Eurasia and Africa plates merge. From this triple point earthquakes are located along the Azores islands, following a W-E direction to Terceira Island (27°W) and from there in a SE direction (Borges, 2003). The largest earthquakes occurring in this area during the instrumental period are the events of May 8, 1939 and January 1, 1980, both with Ms = 7.1 (Buforn et al., 1988). Seismicity stops at the Gloria

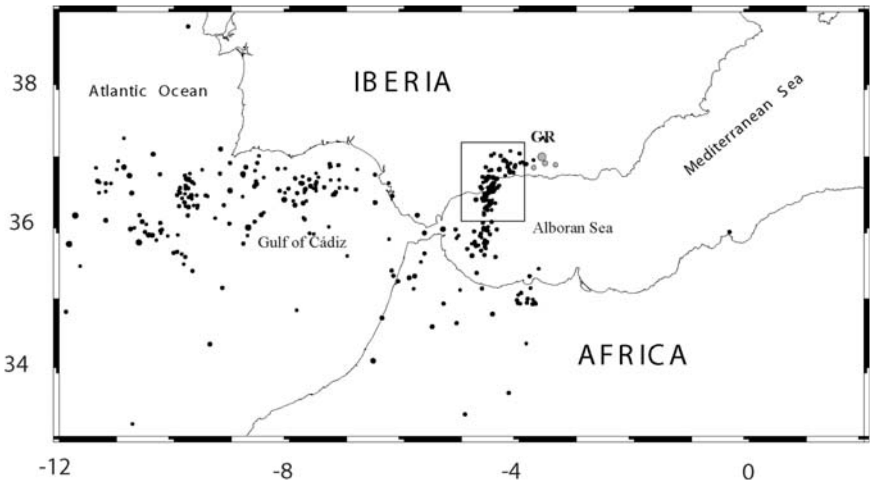


Fig. 1b Distribution of epicenters of intermediate depth (40 < h < 150 km, *black circles*) and deep earthquakes (h ≈ 650 km, *grey circles*). GR = Granada

fault (from 24°W to 19°W), seismically inactive at present. From historical seismicity we know that shocks with maximum intensity of IX (MSK) have occurred in 1522, 1757 and 1926 in the Azores Islands, causing severe damage (Borges, 2003) on Terceira, Faial and San Miguel islands (Fig. 2a).

The central area of the region (from 19°W to 12°W, Fig. 1a) is characterized by the occurrence of major shallow earthquakes with magnitude greater than 8.0 (November 15, 1941). The epicenters follow a W-E direction marking very clearly the plate boundary between Eurasia and Africa. A second line of epicenters follows a NW-SE direction, from 19°W to Agadir, at the Moroccan Atlantic coast; on this secondary line the large shock of 26 May 1975, M = 7.9, was located. Buforn et al. (1988) have proposed a triangular subplate limited by this line, the main W-E plate boundary and the NE-SW seismicity associated with the Atlas range (Fig. 1a).

The Ibero-Maghrebian area region, the third part of the region (Fig. 1a), extends from 12°W longitude to Tunisia and includes the seismic active areas of Gulf of Cadiz, south of Iberia, Alborán Sea and northern Africa (Morocco and Algeria). The situation in this area is more complex than in the other two. On the Gulf of Cadiz and Algeria the epicenters are distributed on a narrow E-W band that marks the trend of the plate boundary. The seismicity is characterized by the occurrence of large shallow earthquakes, larger in the Gulf of Cadiz with

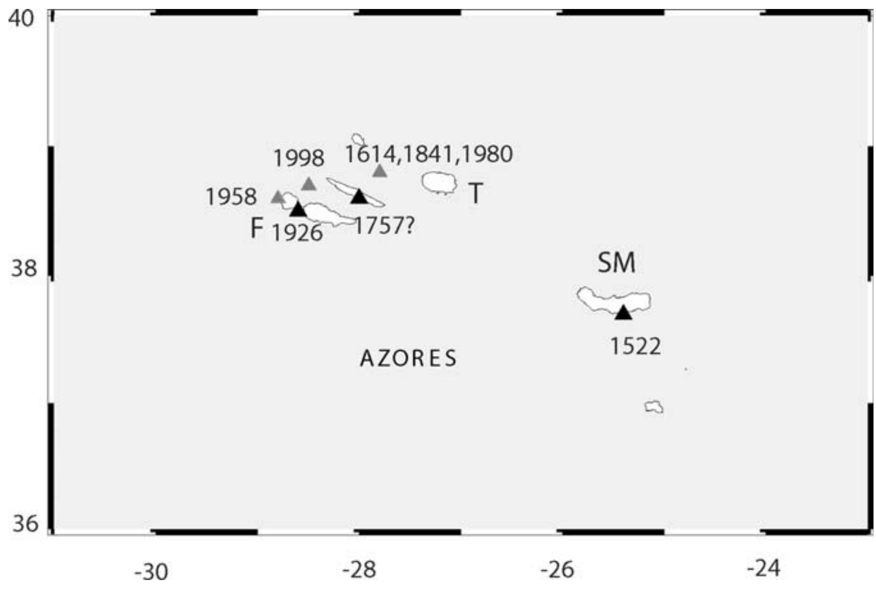


Fig. 2a Historical seismicity for Azores (area A) taken from Borges (2003). Earthquakes with maximum intensity VIII (*grey*) and IX (*black*) are represented (symbols are proportional to intensity). F: Faial island, T: Terceira island, SM: San Miguel island

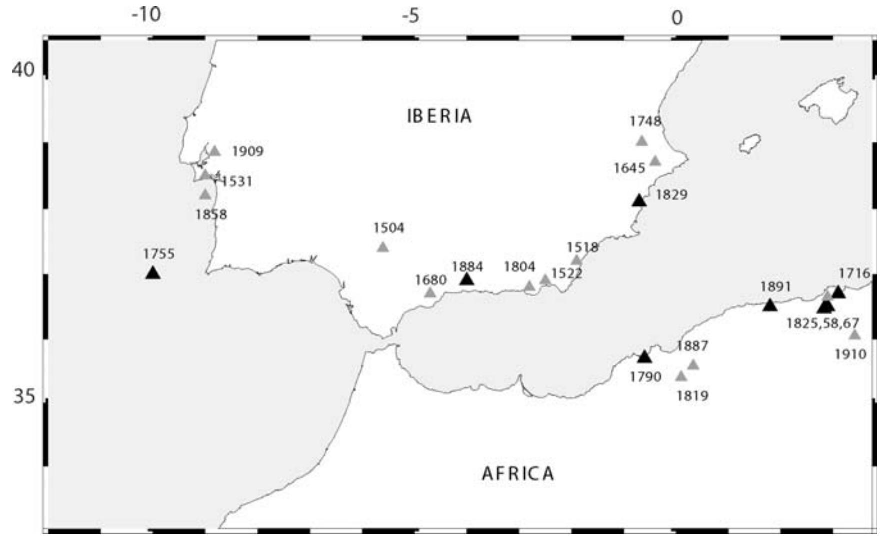


Fig. 2b Historical seismicity for the Ibero-Magrhebian region (area C). In *black* earthquakes with maximum intensity X, in *grey* maximum intensity IX

maximum magnitudes about 8 (28 February 1969, Ms = 8.0) and about 7 on Algeria (10 October 1980, Ms = 7.1; 21 May 2003, Ms = 7.2). In southern Iberia, Alboran Sea and northern Africa, magnitude of earthquakes decreases with the occurrence of only moderate magnitude earthquakes (less than 5.5) with the exception of the Al Hoceima earthquakes (northern Morocco, 26 May 1994, Mw = 5.8 and 24 February 2004, Mw = 6.2). An important characteristics of the seismicity of this area is the occurrence of earthquakes at intermediate depth ($40 < h < 150$ km, Fig. 1b), with a low magnitude, less than 4.5. The epicenters are distributed in an E-W direction in the Gulf of Cadiz and in a N-S direction in the western part of Alboran Sea following a narrow band (width 50 km) along 4.5° W longitude (Buforn et al., 1991, 2004). This seismicity stops at a depth of 150 km. A striking feature of this area is the occurrence of a nest of very deep earthquakes (depth 650 km) to the south of Granada, the largest in 1954 (M = 7.0).

Historical seismicity (Fig. 2b) show the occurrence of large earthquakes (Io $\geq$ IX MSK) in the Ibero-Maghrebian area mainly concentrated on three zones: The first goes from the west of San Vicente Cape, where the 1755 Lisbon earthquake is located, to the Lower Tajo Valley near Lisboa (1858, 1531 and 1909 events); the second located along the southern (1680, 1884, 1804, 1522 and 1518 events) and the eastern coast of Iberia (1829, 1645 and 1748 events) and the third along the northeast of Algeria, with two main areas near Oran (1790, 1819 and 1887 events) and El Asnam (1891, 1825, 1858, 1867, 1716 and 1910 events). In northern Morocco there is no information of large historical shocks. The Al Hoceima region, which has been active since 1994, was seismically quiet in the past.

3 Focal Mechanisms

Figure 3 shows a selection of focal mechanisms for shallow earthquakes with magnitude equal to or greater than 5.0; parameters are listed in Table 1. Normal faulting is predominant along the Mid Atlantic Ridge (events 1 to 6) with horizontal extension in an E-W direction, normal to the ridge. From the triple point to the Gloria fault a change of focal mechanisms can be observed. Strike-slip faulting (events B01, 13 and 14) with horizontal extension in an E-W direction and compression in the N-S direction has been obtained for earthquakes from the triple point to Terceira Island and normal faulting (events 18 and 19) with N-S extension for earthquakes from the Terceira island to Gloria fault. The existence of two reverse mechanisms (events 16 and 17) is difficult to explain.

At the central part, at the Atlantic Ocean, between Azores and Gibraltar, earthquakes mechanism shows a clear strike-slip motion (events 21–26). Most mechanisms have a vertical plane oriented in an E-W direction with right-lateral motion in agreement with the orientation and motion of the plate boundary. The large May 26, 1975 (event 20) event located at the secondary seismicity line, shows strike-slip motion similar to that of the main alignment.

In the Ibero-Maghrebian area thrusting mechanism solutions have been obtained for the large earthquakes in the Gulf of Cádiz (events 28, 29 and 31) and northern Algeria (events 00, 56, 31A, 34, 48, B03, 49, 25A, 1A and 24A). In the Gulf of Cadiz one plane is near the vertical and is oriented in an E-W direction in agreement with the plate boundary. In Algeria most solutions have

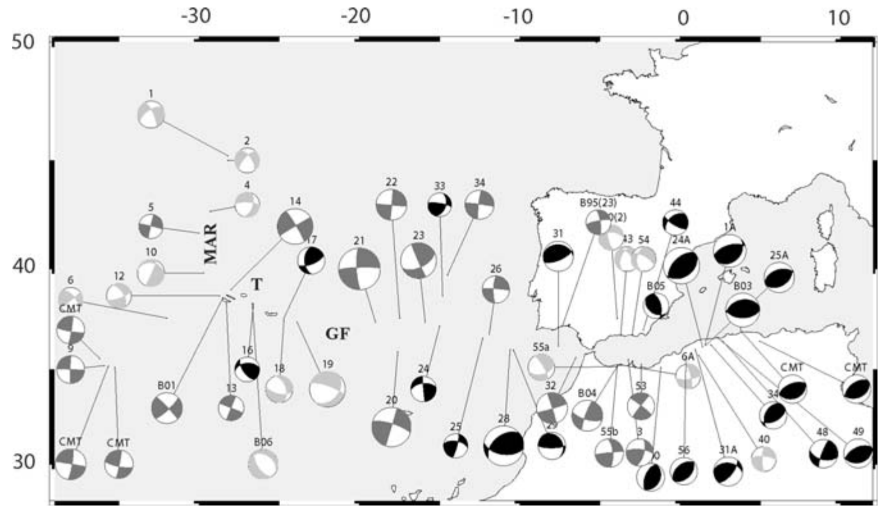
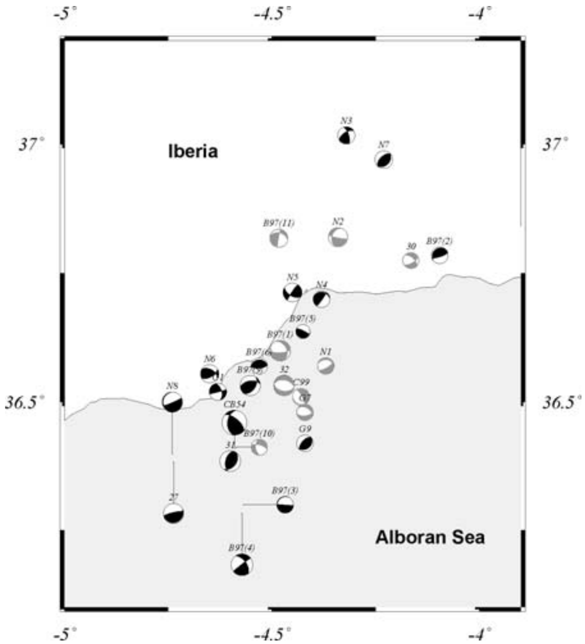


Fig. 3a Focal mechanisms for shallow earthquakes (h < 40 km) and mb ≥ 5.0. In *black* thrusting mechanims, in *dark grey* strike-slip and in grey normal faulting. Size is proportional to magnitude. Numbers correspond to Table 1. GF: Gloria Fault

Fig. 3b Focal mechanisms for intermediate depth earthquakes (40 < h < 150 km) for the area show as a square in Fig. 1b. Thrusting mechanims in *black* and normal faulting in *grey*. Numbers correspond to Table 2



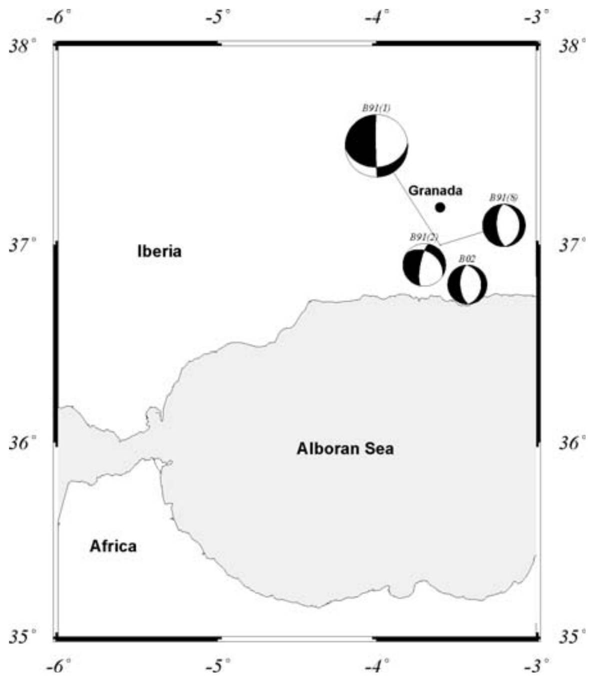


Fig. 3c Focal mechanism for very deep earthquakes (h ≈ 650 km). Numbers correspond to Table 3

Table 1 Shallow earthquakes with magnitude equal to or greater than 5.0

Date	Lat.N.	Long.E	Depth	M	ϕ	δ	λ	NF	Ref
200531	37.4	−15.9	15	7.1	64	59	4	23	3
080539	37.40	−23.90	15	7.1	41	35	−154	19	3
251141	37.40	−19.00	15	8.4	177	79	−6	21	3
190551	37.58	−3.93	30	5.1	169	69	−35	B00(2)	5
090954	36.28	1.57	10	6.5	253	61	104	1A	1
230859	35.51	−3.23	20	5.5	276	70	153	3	3
051260	35.60	−6.50	15	6.2	73	86	−178	1	3
150364	36.20	−7.60	12	6.1	276	24	117	31	3
170564	35.2	−35.9	20	5.6	272	89	3	9	3
110764	41.7	−29.9	15	5.0	193	88	−4	5	3
180964	39.8	−29.7	20	5.5	283	26	−12	10	3
290665	36.6	−12.3	15	5.0	15	80	32	25	3
290965	45.2	−28.2	15	5.4	237	63	−17	1	3
040766	37.50	−24.70	10	5.4	341	49	−42	18	3
050766	37.60	−24.70	18	5.1	179	48	30	17	3
170468	35.24	−3.73	22	5.0	83	70	−162	6A	1
200468	38.30	−26.60	15	5.0	264	45	39	16	3
280269	36.10	−10.60	22	8.0	231	47	54	38	3
050569	36.00	−10.40	29	5.5	324	24	142	29	3
060969	36.90	−11.90	35	5.4	182	75	–5	26	3
301270	37.20	−15.00	15	5.1	265	54	1	24	3
130572	45.00	−28.20	15	5.0	99	80	−35	2	3
230572	37.60	−32.00	15	5.0	320	60	4	6	3
111273	38.70	−28.70	15	5.0	329	58	−10	12	3
231173	38.40	−28.30	15	5.1	292	89	−0	13	3
080375	38.70	−14.80	15	5.0	20	46	16	33	3
260575	35.90	−17.60	15	7.9	287	76	180	20	3
110979	42.70	−29.30	10	5.0	273	72	−40	4	3
010180	38.80	−27.80	10	7.1	239	88	−5	14	3
101080	36.16	1.39	5	7.3	225	54	83	24A	1
101080	36.24	1.59	10	6.1	58	43	81	25A	1
071280	36.02	0.94	5	5.8	277	40	140	31A	1
010281	36.27	1.90	11	5.5	210	43	64	34A	1
240183	39.70	−14.50	15	5.8	187	83	−4	34	3
171083	37.60	−17.50	4	6.0	95	88	−178	22	3
240684	36.80	−3.70	5	5.0	201	48	−46	43	1
130984	37.00	−2.30	9	5.1	121	73	156	44	1
311088	36.44	2.33	13	5.7	103	55	167	48	1
291089	36.61	2.33	31	5.8	242	55	87	49	1
201289	37.30	−7.30	23	5.0	351	77	10	B95(23)	2
230593	35.27	−2.42	6	5.4	308	86	4	53	1
231293	36.77	−2.99	8	4.9	300	70	−130	54	1
260594	35.14	−3.92	7	5.3	330	77	45	55a	1
260594	35.14	−3.92	8	5.7	355	79	2	55b	1
180894	35.60	0.36	4	5.7	58	45	95	56	1
270697	38.33	−26.68	7	5.9	301	35	−111	B02	–

Table 1 (continued)

Date	Lat.N.	Long.E	Depth	M	ϕ	δ	λ	NF	Ref
090798	38.65	−28.63	8	6.6	156	88	0	B01	–
221299	35.26	−1.45	6	5.6	25	31	92	B00	4
020299	38.10	−1.50	1	5.1	125	39	56	B05	6
210503	37.02	3.77	6	6.8	250	40	80	B03	7
270503	36.45	4.96	15	5.7	64	38	98	CMT	CMT
040703	36.94	3.72	15	5.7	70	31	92	CMT	CMT
240204	35.14	−4.00	6	6.2	305	88	179	B04	8

ϕ = azimuth
δ = plunge
λ = rake
NF = number on Fig. 3a
Ref = Reference
1. Bezzeghoud and Buforn (1999); 2. Buforn et al. (1995); 3. Buforn et al. (1988); 4. Yelles-Chaouche et al. (2004); 5. Buforn et al. (2004); 6. Buforn et al. (2005); 7. Bezzeghoud personal communication; 8. Buforn et al. (in preparation);

fault planes dipping about 45° and trending in NE-SW direction. At the central part of the Ibero-Maghrebian area (Betics, Alboran Sea and Rif) there is a change in focal mechanisms to strike-slip motion on northern Morocco (events 55b, 3, 54 and B04) and normal fault solutions in south Iberia (events 43, 54 and B00). These solutions correspond to earthquakes with magnitude less than 6.0, with the exception of the 2004 Al Hoceima earthquake (event B04).

Focal mechanisms for a selection (digital data and magnitude greater or equal to 3.5) of intermediate depth earthquakes ($40 < h < 150$ km) are shown in Fig. 3b and Table 2. Most epicenters are offshore, with focal mechanisms showing vertical compression for the earthquakes located east of 4.5°W longitude and vertical extension for shocks west of the 4.5°W longitude. However, it is important to remember the low magnitude of these earthquakes, less than 5.0. For very deep earthquakes focal mechanisms obtained (Fig. 3c and Table 3, the 1954 event, B91(1) was magnitude 7.0) show a vertical plane oriented in N-S direction and the pressure axes dipping 45° to the east.

4 Discussion

The stress regime in a region can be represented by the total moment tensor (TMT) obtained from focal mechanism of shallow earthquakes during a certain period of time. The TMTs for Azores, Central and Ibero-Maghrebian areas have been used to estimate the stress regime of each area. The TMT representation has an advantage versus the Frolich diagrams, where all earthquakes have the same weight independent of their magnitude, and, consequently it is a better way to quantify the stress regime. The total moment tensor is defined as the sum of the moment tensors calculated from individual solution for an area and is given by (Buforn et al., 2004)

Table 2 Intermediate earthquakes (40 < h < 150 km) of magnitudes greater to or equal to 3.5

Date	Lat.N.	Long.E	Depth	M	ϕ	δ	λ	NF	Ref
130586	36.60	−4.48	90	4.3	87	74	−123	B97(1)	1
270387	36.79	−4.10	79	3.5	69	72	76	B97(2)	1
300588	36.52	−4.63	80	3.6	75	88	35	G1	3
281188	36.30	−4.57	100	3.5	93	88	−85	B97(3)	1
121288	36.28	−4.57	95	4.5	232	87	146	B97(4)	1
190789	36.64	−4−43	95	3.0	296	79	94	B97(5)	1
060290	36.57	−4.53	68	3.4	270	23	96	B97(6)	1
130490	35.61	−4.82	89	3.9	263	53	45	B97(8)	1
020590	36.53	−4.55	95	4.2	36	49	57	B97(9)	1
181190	36.41	−4.59	85	3.4	175	51	−30	B97(10)	1
250891	36.82	−4.48	58	3.8	286	39	−173	B97(11)	1
140392	36.51	−4.43	64	3.6	118	14	−123	C99	2
030992	36.48	−4.42	86	3.5	298	41	−61	G7	3
091193	36.42	−4−42	70	3.5	223	60	86	G9	3
010194	36.57	−4.37	68	3.5	60	71	−103	N1	4
170395	36.82	−4.34	56	4.0	100	85	−56	N2	4
181195	37.02	−4.32	52	3.6	238	59	154	N3	4
281195	36.70	−4.38	68	3.5	35	84	76	N4	4
220696	36.71	−4.45	68	3.9	120	58	171	N5	4
271296	36.56	−4.65	59	3.8	60	60	49	N6	4
180397	36.96	−4.23	56	3.7	43	34	87	N7	4
200897	36.40	−4.65	68	4.2	67	86	−63	N8	4
040702	36.78	−4.17	63	3.4	311	58	−34	30	5
240802	36.40	−4.60	69	4.2	37	41	107	31	5
211102	36.53	−4−47	92	4.3	97	45	−94	32	5

1. Buforn et al. (1997); 2. Coca (1999); 3. Morales et al. (1999); 4. Buforn et al. (2004); 5. Fresno (2004)

$$M_{ij}^{total} = \sum_{k=1}^{N} M_0^k m_{ij}^k \tag{1}$$

where N is the number of earthquakes, M_0 the scalar seismic moment of each event and m_{ij} the seismic moment tensor components. From this equation it is obvious that larger earthquakes in an area with high values of M_0 will control the stress regime. The amount of non-DC (CLVD) component in the TMT can

Table 3 Very deep earthquakes

Date	Lat.N.	Long.E	Depth	M	ϕ	δ	λ	NF	Ref
210354	37.00	−3.70	640	7.0	179	88	−122	B91(1)	1
300173	36.90	−3.70	660	4.8	191	74	−56	B91(2)	1
080390	37.00	−3.60	637	4.8	177	62	−91	B91(8)	1
310793	36.80	−3.43	663	4.4	177	60	−91	B02	2

1. Buforn et al. (1991); 2. Buforn et al. (2004)

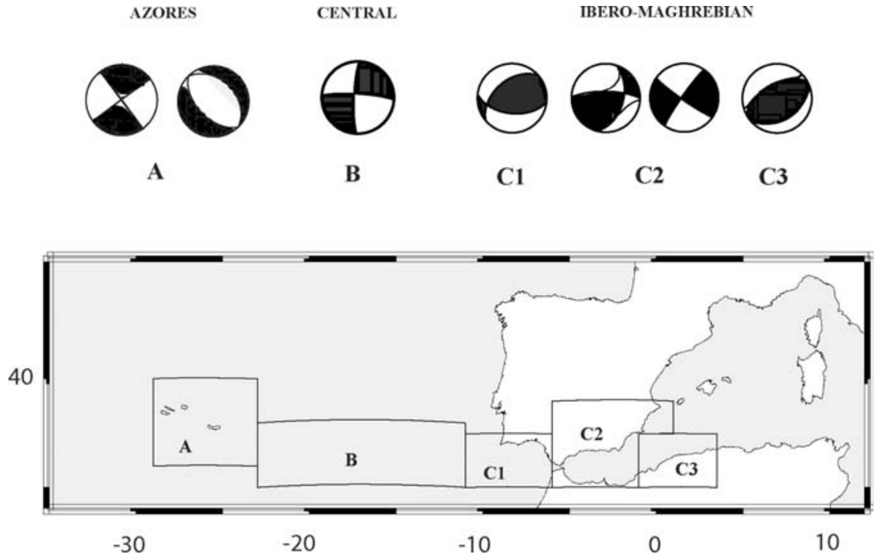


Fig. 4 Total seismic moment tensor (*on top*) for the three studied areas (*on bottom*)

be considered as a measure of the regularity of the stress regime in the area; the lower the non-DC component the greater the regularity. From Table 1 we have estimated the total moment tensor for the three area with different characteristics, namely Azores (A), Central, (B) and Ibero-Maghrebian (C) (Fig. 4).

For the Azores area (A), Borges (2003) has obtained two different stress regime: left-lateral strike-slip faulting in the western part, where the seismicity follows a W-E direction and normal faulting in the eastern part. In each part horizontal compression acting in the N-S and NE-SW directions, respectively, was obtained. The amount of non-DC (CLVD) in the TMT is 8% and 12% respectively, which confirms that the TMT's plotted in Fig. 4 are representative of the faulting in the Azores area. For the Central area (B), TMT corresponding to right-lateral strike-slip faulting has been obtained, with 2% of non-DC component. The TMT plotted in Fig. 4 corresponds practically to the focal mechanisms of the large earthquakes of 1941 and 1975 (events 20 and 21 in Fig. 1) which control the stress regime at the Central area. This stress regime corresponds to horizontal NE-SW extension and NW-SE compression.

For the Ibero-Maghrebian area the situation is more complex. Buforn et al. (2004) have divided the area into three zones (C1, C2 and C3 in Fig. 4). For zones C1 (Gulf of Cádiz) and C3 (Algeria) the TMT are similar. They correspond to reverse faulting, with horizontal compression in NNW-SSE direction. TMT plotted in Fig. 4, with non-DC component of 0.5% and 6% for C1 and C3 respectively, correspond to the large 1969 and 1980 events, which control the stress regime in both zones. In the central zone (C2) two different TMT have been obtained. For southern Spain the TMT corresponds to reverse faulting with 28% of non-DC component and for the Alboran Sea and northern Morocco the TMT corresponds to strike-slip faulting with 1% of non-DC component. The strike-slip faulting corresponds practically to the focal mechanism of the Al Hoceima 2004 event, the largest occurred in C2 zone in the last 50 years. The two solutions obtained for zone C2 agree with a horizontal compression in NNW-SSE direction, similar to those obtained for C1 and C3.

From seismicity and focal mechanisms it is possible also to estimate the moment rate and average slip velocities for each area (Buforn et al. 2004). For the Azores area values obtained by Borges (2003) have been used. For the Central and Ibero-Magrhebian areas shallow earthquakes ($m_b > 5$) occurred during the period 1900–2004 have been used. The average slip velocity was estimated from moment rate according to the expression

$$\Delta\dot{\bar{u}} = \frac{\dot{M}_0}{\mu S} \tag{2}$$

where μ is the rigidity coefficient (3×10^4 MPa) and S is the area of a vertical fault with 10 km width and length as show in Fig. 4. The values obtained are shown in Table 4. The most active area is the Central (B), where large earthquakes ($M > 8.0$) occur resulting in an average slip velocity of 20 mm/yr. In the Gulf of Cádiz (C), where the 1969 shock ($M = 8.1$) is located the slip velocity is 5.5 mm/yr, in agreement with the 5 mm/y predicted by the plate motion model NUVEL-1a (DeMets et al., 1990). A similar value has been estimated for the Algeria zone (3.7 mm/yr), greater than the value estimated by Buforn et al. (2004), due to the occurrence of the Boumerdes ($Mw = 7.1$) earthquake which has been taken in account in the present study. The average slip velocities estimated by Borges (2003) for Azores 7.2 mm/yr and 2.7 mm/yr show the different seismic activity in the two parts of the area, with the occurrence of large earthquakes in the part nearer to the Mid Atlantic Ridge. The lower value of average slip velocity corresponds to the central part of the Ibero-Maghrebian area (C2), 1.4 mm/yr. This value is higher than the value estimated by Buforn et al. (2004), 0.6 mm/yr), due to the occurrence of the Al Hoceima 2004 earthquake ($Mw = 6.2$). The occurrence of the 2004 Al Hoceima earthquake may be interpreted as the beginning of a period of reactivation for this region, which may compensate the low moment rate and slip velocity of the 20th century (Table 4).

Table 4 Moment rates and average slip velocity in the studied areas

Area	Moment rate (Nm/y)	Average slip velocity (mm/y)
Azores I*	4.1×10^{17}	7.2
II*	2.1×10^{17}	2.6
Central	8.3×10^{18}	20.0
Ibero-Maghrebian C1	9.2×10^{17}	5.5
C2	3.4×10^{16}	1.4
C3	5.0×10^{17}	3.7

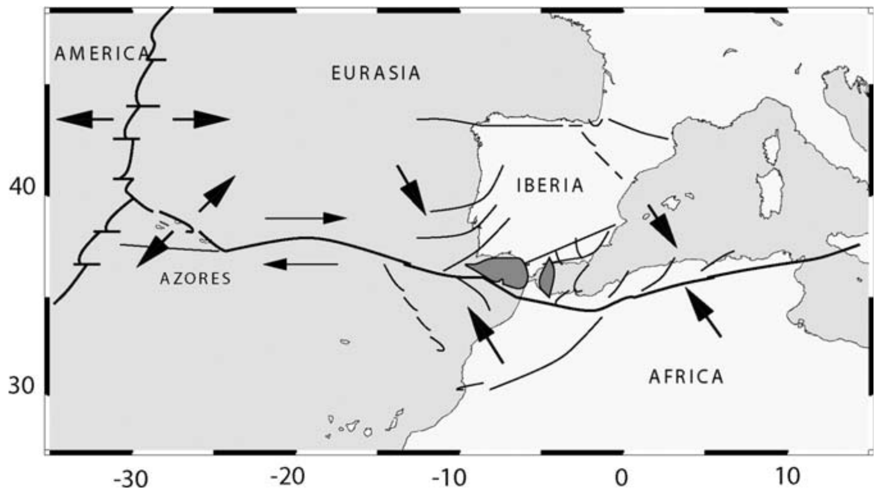


Fig. 5 Seismotectonic framework for Azores-Tunisia region. *Arrows* indicate the direction of the regional stresses obtained from focal mechanisms of shallow earthquakes. The *grey* area at south Iberia indicates the intermediate depth seismic activity

Finally, Fig. 5 shows a simplified seismotectonic framework obtained from seismicity and focal mechanisms. Along the Mid Atlantic Ridge the stress pattern corresponds to horizontal extension normal to the ridge. This situation continues from the triple point to the Gloria Fault along the Azores islands. In this area a change in faulting may be observed from left-lateral strike-slip to dip-slip normal motion. Moderate values of average slip velocity indicate the occurrence of earthquakes with maximum magnitudes about 7.0. In the central area, motion changes to right-lateral strike-slip, along an E-W trending fault. The large average slip velocity obtained indicates the occurrence of large earthquakes, with magnitudes larger than 8.0. In the east area, in the Gulf of Cadiz and Algeria, we have obtained reverse faulting motion with NNW-SSE horizontal compression. In the center of this area, south Iberia, Alboran Sea and Rif, the situation is more complex with two different faulting motions: reverse and strike-slip motion, but in both cases horizontal compression on NNW-SSE direction is present. However, the lower values obtained for the average slip velocity in the center of the Ibero-Maghrebian area may be an indication of a quiet seismic period and, consequently, the stress pattern obtained may not corresponds to the regional pattern. The plate boundary, which is very well defined in the Azores and Central areas by seismicity, changes to a wider deformation area in the Ibero-Maghrebian area, and extends through south Iberia, the Alboran Sea and northern Morocco. It reappears well defined again in northern Algeria and continues eastward to Tunisia. As a summary, we can conclude about the complex nature of the plate boundary behaviour in the Azores-Tunisia region, with extension in the Mid Atlantic Ridge and the Azores

Islands, transform motion in the Central part and horizontal compression in the Ibero-Maghrebian area. The occurrence of intermediate depth earthquakes at the Ibero-Maghrebian is a further indication of the complex situation at this region. In the Gulf of Cádiz this intermediate depth activity may be explained in terms of a collision of the Eurasia and Africa plates along a E-W line in agreement with the distribution of epicenters in this area. More difficult to explain is the existence of a N-S very narrow band of intermediate depth earthquakes east of Gibraltar, which is not connected with the deep earthquakes. The very deep seismic activity continues to be difficult to explain and may be related with an anomalous high velocity region (Blanco and Spakman, 1993) extending from 200 to 700 km. Only a very small part of this anomalous region, located at south Granada at a depth of 650 km is active. The intermediate and deep earthquakes may be related to subduction processes, more recent for the intermediate depth shocks and older for the very deep activity (Buforn et al., 2004).

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The 1755 Lisbon Earthquake: A Review and the Proposal for a Tsunami Early Warning System in the Gulf of Cadiz

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1 Introduction

The present paper reviews the proposed sources for the 1/11/1755 Lisbon earthquake in the context of the geodynamics of Southwest Iberia, the region where it was generated. We will also refer briefly the implications for seismic hazard both at global and regional scale.

Because the 1755 earthquake occurred before the foundation of instrumental seismology, the source location and mechanism can be inferred only on the basis of the Historical Seismicity through interpretation of records of direct witnesses of the earthquake and tsunami (Pereira de Sousa 1919–1932), through the geological record left by the tsunami in the SW coast of Portugal (Andrade 1992) and through the evidences of downslope mass movement triggered by the earthquake as recorded in the sedimentary column of the surrounding Abyssal Plains (Lebreiro et al. 1997). The isoseismal distribution derived by the description of the damages suggested a seismic moment magnitude of 8.7 and source area located SW of Lisbon (Richter 1958, Johnston 1996, Buforn et al. 1988, 2004).

The West-Iberia continental margin is more active in terms of instrumental and historical seismicity than usual for passive Atlantic-type margins. Epicentres distribution shows that Iberia is characterized by an inner core with increased rigidity and stability with respect to its margins.

Reconstruction of the present day stress field shows deviations of the maximum compressive stress trajectories from the core of Iberia, where they are oriented NNW-SSE, to its western margin, where they become WNW-ESE oriented (Ribeiro et al. 1996, Ribeiro 2002). Along the N-S trending West Iberia Margin the orientation of the main compression also changes from approximately NW-SE in the north to WNW-ESE in the south. It was firstly argued by Ribeiro and Cabral (1986, 1987) and Ribeiro et al. (1996) that the West Iberia Margin initiated the process of transition from passive margin to active margin

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in Pliocene times. According to Ribeiro (2002) the estimated convergence in W-Iberia is slow (1-2 mm/year) and generated the earthquake and tsunami recorded in historical seismicity, with large return periods for the highest magnitude events.

2 Sources of the 1755 Earthquake and Tsunami

2.1 The Gorringe Bank

The high magnitude of the 1755 earthquake requires a large rupture area and, or a large mean displacement for this tectonic earthquake. Until mid 1990 most specialists proposed a source area in the Gorringe Bank. This major morpho-tectonic feature of the Eurasia-Africa plate boundary lies at the easternmost end of the Gloria Fault, the central segment of the Azores-Gibraltar Fault Zone. It exposes upper mantle rocks of oceanic lithosphere overthrusted onto the Tagus Abyssal Plain along a NW directed thrust of Miocene age. The kinematics of this plate boundary, according to the NUVEL1-A model (DeMets et al. 1990) is, from W to E, transtensional with dextral transform in the oblique rift of the Azores, pure dextral transform in the Gloria Fault and NW-SE oriented convergence to the E of the Tore-Madeira Rise, where the Gorringe Bank is located.

The location of the source area of the 1755 earthquake in the Gorringe Bank raises some problems. The macroseismic intensity, IX, as derived from the historical records, is the same in Lisbon, 260 km to the NE of the Bank, and in Lagos (Algarve), 200 km ENE of the Bank. This is in disagreement with the seismic intensities distribution of the $M_S = 7.9$, 28/02/1969 earthquake located less than 100 km to the SE of the Gorringe Bank, because the intensities were VII in Lagos and VI in Lisbon. This led some specialists, even before 1980, to propose a complex rupture with a branch directed toward Lisbon, even in the absence of marine data that could support it (Machado 1966).

On the other hand, seismic reflection profiles across the Gorringe Bank, in situ observations with the submersible NAUTILE in 1996 and improved monitoring of instrumental seismicity in the last decade (Terrinha 1998, Zitellini et al. 2001, Carrilho et al. 2004, Zitellini et al. 2004) showed that the thrust on the NW flank of the Bank strongly diminished its activity, probably since the Upper Miocene ($\approx$6 MY). These results stimulated the search for alternative solutions for the source area location of 1755 Lisbon Earthquake (Fig. 1).

2.2 The Marquês de Pombal Thrust (SW Iberia Margin)

Marine geosciences campaigns carried out in the nineties identified a 60 km long NNE-SSW easterly dipping major thrust structure located close to the ocean-

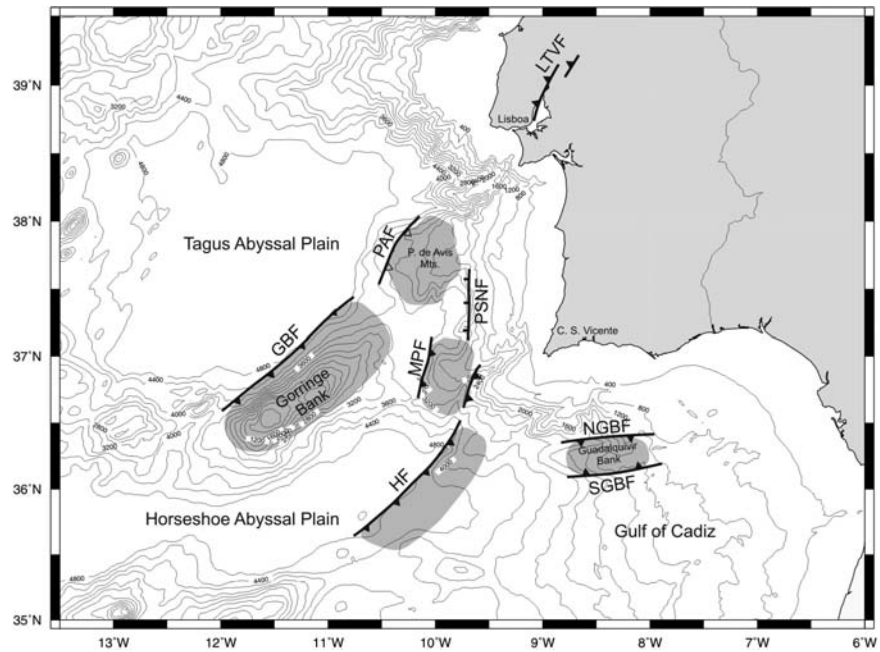

Fig. 1 Major seismogenic zones in the SW Iberia Margin. GBF – Gorringe Bank Fault; PAF – Príncipes de Avis Fault; MPF – Marquês de Pombal Fault; HF – Horseshoe Fault; NGBF – Northern Guadalquivir Bank Fault; Southern Guadalquivir Bank Fault; PSNF – Pereira de Sousa Normal Fault; LTVF – Lower Tagus Valley Fault. Modified after Zitellini et al. (2004)

continent transition (Zitellini et al. 1999, 2001, 2004). Here the continental margin overthrusts the ocean basin and creates an escarpment with a sea bottom elevation of 1.1 km in a domain located 100 km to the SW of Cape S. Vicente. The thrust fault could be imaged in a multi-channel seismic line until a depth of approximately 20 km where it merges into a deep décollement where is present a cluster of hypocentres. It was named *Marquês de Pombal Thrust* (Zitellini et al. 2001) and it was considered the most probable source area for the 1755 earthquake and tsunami, confirming earlier proposals for this source location (Ribeiro and Cabral 1986, 1987, Cabral and Ribeiro 1989, Ribeiro 1994, Cabral 1995, Ribeiro et al. 1996, Buforn et al. 1988). In addition, already since 1996 (Baptista 1996, Baptista et al. 1998a, 1998b) showed that the hydrodynamic modeling of the 1755 tsunami ruled out the Gorringe Bank as the source area because the amplitudes and arrival times to Portugal, Spain and Morocco did not fit the historical records. These authors, in absence of available geological constraint, proposed "ad-hoc" source areas for the 1755 earthquake that were much closer to the shore and partially overlapping with the Marquês de Pombal Thrust.

The detailed mapping of the Marquês de Pombal Thrust showed that associated to the 60 km long fault scarp are present a submarine landslide and a turbidite system (Gràcia et al. 2003, Terrinha et al. 2003). Nevertheless the estimated moment magnitude of 8.7 requires additional rupture areas. Large earthquakes are frequently generated by complex ruptures due to spacio-temporal integration of sub events that can justify the high magnitude. Additional source areas to the Marquês de Pombal Thrust have been proposed to explain this area deficit:

a) Horseshoe Thrust Fault: it is situated to the SW of the Cape S. Vicente canyon reverse fault, in the SE flank of the Horseshoe Abyssal Plain. It dips to SE and just to the West of the fault trace is the epicenter of the M_S 7.9 event of 28/02/1969, the highest instrumental seismicity event in the Iberia-Morocco-Atlantic domain (Ribeiro 2002, Zitellini et al. 2004).
b) Pereira de Sousa – Montanha do Príncipe de Avis Fault System: the Pereira de Sousa Fault is a N-S Mesozoic normal fault with downthrow of the W block. It was uplifted and rotated by ongoing compression without reactivation because it is not suitably oriented relative to the present stress field; below it is the E dipping blind thrust of Príncipe de Avis seamount. It is a possible source are for the $M_w \sim 7$ earthquake of 12/11/1858, offshore Sines and Setúbal. This thrust is probably a splay of the major deep décollement under the Marquês de Pombal – Horseshoe thrust system.
c) Guadalquivir - Bank Alignment: it is an E-W pop-up 50 km long and 30 km wide. It is the source of instrumental seismicity, suggesting active uplift of an inherited paleo relief of late Mesozoic age (Zitellini et al. 2004). Tsunami modeling is compatible with this additional source area (Baptista et al. 2003) for the 1755 earthquake but other solutions can also fit the large uncertainties in the historical data.

The geometrical parameters for the three structures discussed above are presented in Table 1.

Among the different possibilities we favor the Marquês de Pombal – Horseshoe Thrust or the Marquês de Pombal – Guadalquivir Bank couples as solutions for additional area/slip for the 1755 event. The Marquês de Pombal – Horseshoe Thrust solution seems more realistic because the two faults have coherent orientation and almost geometric continuity facilitating the strain/displacement transfer

Table 1 1755 Source Parameters (Mw = 8.7)

	L (km)	α	H (km)	w (km)	Ar (km^2)	D (m)	Δe
MP	60	30°	60	120	7 200	10	1.67×10^{-4}
SV	50	45°	100	140	7 050	10	2.0×10^{-4}
HS	175	45°	100	140	24 537	10	5.7×10^{-5}

MP – Marquês de Pombal Thrust; SV – São Vicente Canyon Fault; HS – Horseshoe Fault
L – Fault length; α – Fault dip; H – Fault height; w – Fault width; Ar – Fault area; D – Fault displacement; Δe – Strain (Δe = D/L)

between them. The Marquês de Pombal – Guadalquivir Bank alternative, although it fits the historical data well, it requires almost instantaneous stress transfer between two distinct structures with different orientations and opposite polarity of thrusting.

The Pereira de Sousa – Príncipe de Avis Seamount fault zone requires a discontinuity of rupture in the basal décollement, which is more compatible with another model of stress transfer that we will refer below.

2.3 The Gulf of Cadiz Accretionary Wedge and the Horseshoe Fault

Gutscher et al. (2002) used seismic tomography to show a 700 km subducted slab under the Gibraltar arc and proposed this subduction zone as the source area for the 1755 event. According to these authors, active subduction from W to E of Iberia under Africa formed the Gulf of Cadiz accretionary wedge and generates deep earthquakes, such as the magnitude 7.1, 1954 Granada event with hypocenter at more than 600 km depth.

A link between the Gulf of Cadiz source area and the 1755 event is doubtful in two aspects. First the macroseismic intensity distribution for the 1755 event appears incompatible with the Gulf of Cadiz source area because the highest intensities are in the SW corner of Iberia, in W Algarve, the SW coast and Lisbon area and not in E Algarve and northwest Morrocco, as should be if the epicentre was in the central part of the Gulf of Cadiz. Moreover it appears that the subduction beneath the Gulf of Cadiz accretionary prism is not active any more. Even if there is geometrical continuity of the slab on the basis of seismic tomography, there is absence of seismic continuity/activity along the slab. On the other hand, the present tectonic regime of the Alboran Sea is not back-arc extension, as should be expected, but dextral strike-slip on ENE-WSW faults that induces sinistral strike-slip on NNE-SSW faults; as testified by the last destructive earthquake as recently as 24/02/2004, with magnitude 6.3 at Al-Hoceima, Morocco, causing 600 deaths. Although the geodynamic setting of recent magmatism in the Alboran domain is controversial, we observe the absence of subduction related magmatism for the last 5 MY. This suggests that the "roll-back" of the downgoing slab and back-arc basin development has stopped in the end of the Miocene. Also, the stacked thrusts of the imbricate wedge are covered by a barely deformed packed of sediments whose age ranges from uppermost Miocene through Holocene, showing that if not absolutely inactive, this subduction zone dramatically diminished its activity.

Another interesting aspect revealed by the seismic tomography published by Gutscher et al. (2002) is a high velocity discontinuity underneath the surface rupture of the NE-SW easterly dipping Horseshoe thrust Fault. The Horseshoe Fault exceeds 100 km in length and when considered together with the Marquês de Pombal fault they approximate 200 km. This fault also correlates with the main cluster of epicentres in the Gulf of Cadiz – Horseshoe Abyssal Plain, west of longitude 8° 30'W and was possibly the source of the 28/02/1969 magnitude 7.9 earthquake.

3 Relaxation and Reloading of the 1755 Seismogenic System

To generate an 8.7 magnitude earthquake in a convergent fault zone, with mean inclination around 45°, at the slip rate of 1–2 mm/year it is necessary to accumulate stress during 1000–2000 years, as a function of stress build up and aseismic deformation. During the earthquake there is a sudden stress drop that modifies the stress field in an area 1–2 times the size of the coseismic rupture area; the modification of the static stresses generates aftershocks around the rupture area and short term delayed ruptures further away, in a self triggering process.

A second triggering process consists in the coupling of the upper lithospheric layer, the elastic schizosphere, with the less viscous plastosphere, the lower lithospheric layer, dragged from below by the low viscosity astenosphere. The sudden movement of the schizosphere during the earthquake decreases the resistance at the base of the schizosphere and accelerates the movement of the plastosphere right beneath and the astenosphere further down. Theoretically, this process can trigger earthquakes over distances of thousands of kilometers at a time scale of years or decades as a function of the rheology and kinematics of the different layers.

One should ask if these processes operated after the high magnitude 1755 event. We can exclude triggering by dynamic stresses because there is no historical record of subsequent seismic activity in fault systems that are at large distances from the epicenter area of the 1755 event.

The first process has been proposed for triggering an onshore rupture of magnitude 7 in the Lower Tagus Fault Zone (Vilanova et al. 2003). According to these authors, a segment of this fault, near Lisbon, was near rupture and the static stress changes, induced by the main shock, even quite small, were enough to trigger this rupture. This model is based on the assumption of a return period of moderate M$\sim$7 earthquakes on the order of 200 years in this fault segment, which has thus generated the earthquake sequence of 1344, 1531, 1755 and 1909.

This model was criticized because it requires a slip-rate on this segment of the reverse Lower Tagus Fault Zone of approximately 4 mm/year (Matias et al. 2005). The resulting convergence rate should be larger than the present movement between the Iberia Plate and the Atlantic to the West, which was estimated at 1–2 mm/year (Ribeiro 2002). The distribution of macroseismic intensities in Lisbon, with IX-X MMI in the alluvia of the Tagus river valley and tributaries in downtown Lisbon ("Baixa Pombalina") can be better explained by site effects and seismic rays focusing in the Lower Tagus Sedimentary Basin to the East of

Lisbon. The earthquakes attributed to the 200 years interval are thus generated in different structures with longer return periods.

The second process, coupling of the schizosphere to the plastosphere-asthenosphere could be responsible for the migration of the seismic activity towards the North along the Portuguese West continental margin since 1755, with generation of the 11/11/1858, $M \sim 7$ event offshore Sines/Setúbal, and the 23/04/1909, M_W 6 Benavente earthquake (Teves-Costa et al. 1999), by rupture in the Lower Tagus Valley Fault Zone (Cabral et al. 2003). Although the spatio-temporal scale of this migration, of 300 km in 150 years, is compatible with the rate of convergence in the W-Iberia margin, we are aware that this process must be modeled in further detail.

After the tectonic stress was released during the 1755 earthquake the system has been progressively reloaded at the centurial scale by the steady movement of the plates involved in the seismogenic process. The elastic component reaction to stress accumulation can be measured by GPS, which estimates the instantaneous velocity of the system; the shortening component at geologic long term is estimated by tectonic methods and expresses the inelastic-viscoelastic component of the convergent zone by aseismic and seismic displacement during various seismic cycles (Liu et al. 2000). So we must look at GPS in the Ibero-Atlantic-Maghreb domain for constraining the seismic cycles and monitor the on-going plates interaction.

A synthesis of the available GPS data for the last decade (Nocquet and Calais 2004) shows that Africa is moving towards the WNW with respect to Iberia, whilst the NUVEL1-A kinematic model indicated a NW-wards displacement based on the a dataset that integrated information for the last 3 MY period (DeMets et al. 1990); the first model also states that the present velocity is 30–60% slower than the second model. This means that the Nubia-Iberia collision changed from frontal to oblique, increasing the dextral strike-slip component of this transpressive regime.

The S. Fernando station, near Cadiz, moves at 2.4 ± 1.1 mm/year to the West relative to stable Europe (Nocquet and Calais 2004; Fernandes 2004). This velocity vector is sub-perpendicular to the Marquês de Pombal – Horseshoe Fault System, meaning it is being reloaded for the next large earthquake with return period larger than 500 or even 1000 years; an earthquake and tsunami comparable to the 1755 event occurred near Cadiz in 216–218 BC, on the basis of paleoseismological evidence (Luque et al. 2001). The 28/02/1969 $M = 7.9$ event (Fukao 1973) can be explained as a compressive earthquake in the outer-rise below the Horseshoe thrust, showing that the African segment of this thrust is accumulating stress more rapidly than the Iberian segment of the Marquês de Pombal thrust. There is also geological and geophysical evidence pointing in the same direction as the geodetic methods: E-W dextral faults detected in a multi-beam campaign in 2004 (Mendes-Victor, Matias and MATESPRO team, oral information and work in progress) cut across the thrust front of the oriented NE-SW Horseshoe Fault.

These are the Solid Earth geosciences data that must be used in the assessment of seismic hazard in the area under survey. In Ribeiro (2002) Iberia behaves as a buffer plate spinning in clockwise sense by dextral drag of Eurasia relative to Africa. This can be tested by longer time-series in GPS observations covering the 3 plates involved. This affects the seismic hazard evaluation for the Iberia-Maghreb-Atlantic area; the uncertainties are large because the geodynamic evolution is unsteady and the seismotectonic activity is scattered in a large area, with transition from interplate to intraplate seismic events.

4 Geodynamics and Seismic Hazard: From Regional to Global

A comparison between source parameters of the 26/12/2004 tsunamigenic event of Sumatra with the 1755 Lisbon earthquake (Ribeiro et al. 2006) suggests that the 1755 earthquake generated a tsunami because it was a high stress drop shallow event and not because it was a slow rupture event. We infer that the Lisbon event occurred by a fast rupture process in a convergent setting, instead of the mature subduction zone as in Sumatra. This is an additional reason to consider that the source area of the 1755 event is the Marquês de Pombal thrust system and not the subduction related to the Gulf of Cadiz accretionary prism emplacement.

We conclude that the earthquake and tsunami hazard is significant at a global scale; if we restrict the generation area to the SW Iberia-NW Africa seismotectonic province, the return period for en event such as the 1755 is very large, in the order of 1000–2000 years. But a smaller event of magnitude 8, such as the 1969 event must have a much shorter return period, in the order of 200 years and represents still a high potential for generating a considerable tsunami, depending on its source parameters. Furthermore, if our identification of the source area for the 1755 earthquake is correct, then the Eurasia-Africa plate boundary did not rupture completely by this event and major earthquakes can still be generated to the East, in the Gulf of Cadiz, and to the West, in the Gorringe-Gloria fault system.

Tsunami alert systems must be considered a first priority in the Mediterranean sea, the Indian and the Atlantic oceans, and this concerns us all not only as Europeans but as citizens of the World. Let us hope that the democratic societies of the XXI century are more efficient than the enlightened despotic societies of the XVIII century in improving safety standards for their citizens.

5 Tsunami Early-Warning Proposal

The amount of knowledge on tsunami events that was implemented around the North Atlantic and the Mediterranean areas, thanks to dedicated projects performed, since the last fifteen years partly financed by the EU, justify the consideration and opportunity to design a Tsunami Early Warning System (TEWS).

Interdisciplinary studies were developed over large areas by teams of scientists, including, historians, sociologists, seismologists, engineers, geographers and geologists. It was possible to identify seismic, volcanic and landslides as sources of tsunamis on different zones, where hazard assessments were evaluated. The modelling of tsunami propagation was developed in order to allow the simulation of the run up along coastal areas and the mapping of inundation on those areas.

Tsunami early warning was the object of some experiments concerning the regions of SW Atlantic Iberia, the Western Mediterranean (Italy and France) and the Tsunami prone areas of Greece.

It is time to raise the question of setting the organization of an operational network able to assure at almost real-time (RT) the tsunami early warning, covering the regions above mentioned. This goal might be achieved taking advantage of the on going experiment settled in the Gulf of Cadiz on behalf the EU project NEAREST (http://nearest.bo.ismar.cnr.it/) for the detection of tsunami waves. An abyssal station has been installed 100 km South of Cape San Vicente in August 2007, at 3200 m water depth and the length of the experiment will be one year. The main characteristic of the abyssal station regards the instrumental equipment specifically designed for low energy consumption, an indispensable requirement to work continuously for one year. The scientific payload consists on geophysical (seismometer, hydrophone, gravity meter) and oceanographic instrumentation and it is also equipped with a new sensor designed: the tsunami meter. This instrument was realised to work close to the tsunami generation area and to send automatic alert messages. It works with a double control of seismic and pressure signals and also considering the possible ground movement and the relative acceleration of the sensors installed. In fact the study of correlation between possible seafloor movement and water column perturbation represents the key for the unsolved problem about earthquake and tsunami generation. Another important feature is that the instrument parameters can be changed during the mission through a reconfiguration performed by the shore station.

Even if more intensive research on the related tsunami matters has to developed, the organization of the operational network under the responsibility of the participant countries and in the framework of the EU has the duty not only to install an Early Warning System but also the duty to develop the prevention regulation and emergency plans to mitigate the tsunami impact.

The coordinating body and the Central Headquarters (TEW) should be connected to the Regional Tsunami Early Warning Centre (RTEWC) responsible for each one of the 3 areas: North Atlantic, Western Mediterranean and Eastern Mediterranean (Fig. 2). In all the Regional Centers, the 7 days 24 hours duty would be assured by national hosting institutions, where dedicated specialists should be able to propose the validation of the warning by the

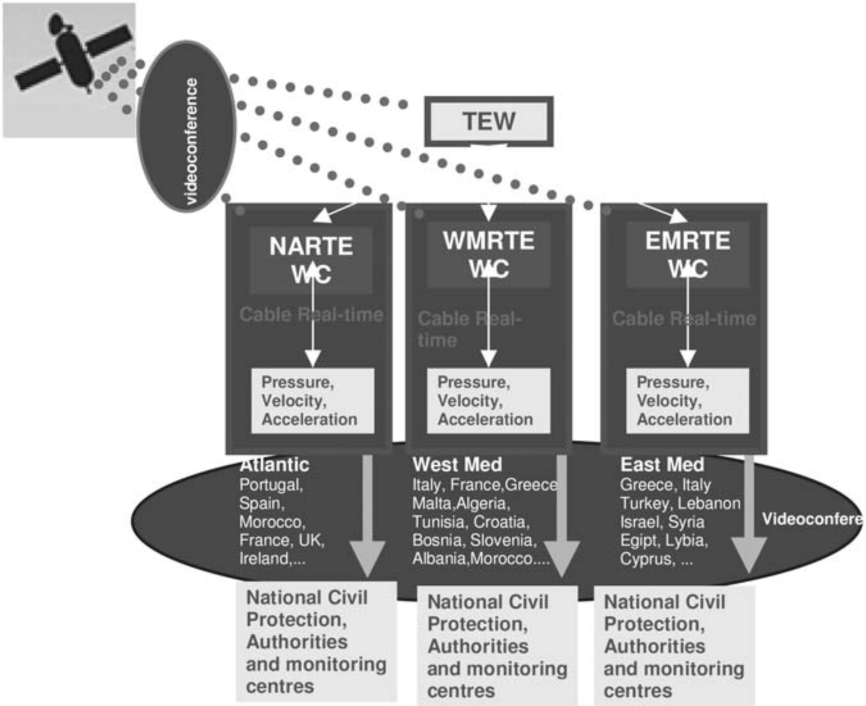

Fig. 2 Conceptual model for the Tsunami Early Warning System proposed for the Mediterranean and NE Atlantic areas

Headquarters Center, in almost real-time (better 1 min), thanks to direct real-time communication.

The Regional Operational Centers would assure the fast flow of observational data coming from the sensors (seismic and pressure) used in adequate regional network deployed to assure the monitoring of region.

The concept of Operational Center must be similar for all the network structure but the Regional Centers would be in direct connection (RT) with Civil Protection Authority and with the Seismic Monitoring Regional Center or Volcano Monitoring Center, in order to guarantee the almost real-time warning, avoiding false alarms.

The following steps have to be envisaged for the establishment of this network, under the framework of the EU, requesting appropriate funds for the development:

1. Identification of the source areas and elaboration of genetic scenarios.
2. Selection and installation of seismic and pressure sensors, GPS, pressure gauges, etc.

3. Real-Time (RT) communications capacity to be installed in the network, including RT videoconference linkage with Civil Protection and Seismic Centers.
4. Analysis in RT of detected phenomena, using automatic procedures to be developed and implemented, when existing.
5. Elaboration of Alert concept.

The countries integrating the TEW network will take the appropriate measures to facilitate the housing of headquarters and the RTEWC buildings, locate on the three regions. The RTEWC communication systems have to be supported by the countries integrating the region.

Satellite or any other very fast communication linkage and the automatic local processing systems, will be designed through a specific project developed under the EU as soon as possible.

Three committees will assist the TEW:

1. Science and Technology
2. Processing, Information and Sensitivity Analysis
3. Finance

The chairpersons of these committees will integrate an Executive Board of Directors.

The General Governing Council of the TEW, where all the countries integrated in the system should be represented, should elect the President of the Board.

6 Conclusion

The investigation already performed in the Gulf of Cadiz and western Iberia margin have identified a small number of large tectonic structures that are able to generate very large earthquakes and tsunami. There is not a single structure able to generate the great Lisbon earthquake 1st November 1755 and the most probable source for this event involves some interaction between neighboring faults. Very large tsunamigenic earthquakes may occur along the Iberia-Africa plate boundary with a 1000–2000 years return period, on the same structure. However we should consider that the hazard for earthquakes and tsunami in the Gulf of Cadiz area is very high today since the 1755 event did not rupture completely the plate boundary and un-ruptured structures may lie to the East and West of the Marquês de Pombal and Horseshoe fault system. It is thus essential that a Tsunami Early Warning System be developed for this area that should take into consideration the very short time before the first waves hit the coasts of Portugal. This system would benefit Portugal, Spain and Morocco and also all the countries bordering the Atlantic that recorded the 1st November 1755 tsunami.

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Evaluation of the 1755 Earthquake Source Using Tsunami Modeling

M.A. Baptista and J.M. Miranda

1 Introduction

The Lisbon earthquake of November 1st, 1755, is probably the best-documented historical event, with an estimated magnitude of 8.5–9.0. The associated tsunami ravaged the coast of SW Portugal and the Gulf of Cadiz, with run-up heights reported to have reached 5–15 m and even the West Indies (Lander and Lockridge 1989, Lander et al. 2002, Mader 2001).

Several authors investigated the source of the Lisbon earthquake, using either macroseismic data (Martinez-Solares 1979, Levret 1991), average tsunami amplitudes (Abe 1979), or scale comparisons with the 28 February 1969 event (Johnston 1996); all these studies were based on the assumption that the 1755 earthquake source was located south of the Gorringe Bank, in the Horseshoe Abyssal Plain (cf. Fig. 1), close to the 1969 earthquake and tsunami source (Gjevik et al. 1997), and most probably related with the bank build up (Fukao 1972).

A different approach was considered by Baptista (1998) and Baptista et al. (1998a,b) throughout the systematic study of the historical records of the 1755 tsunami wave heights observed along the Iberian and Morocco coasts. (Baptista et al. 1998b), based on tsunami hydrodynamic modelling, concluded for a different source position, located, closer to the SW Portuguese continental margin. The approach took by Baptista et al. (1998b) was not to deduce “the source” from tsunami data, but to deduce the constrain given by tsunami data on the location of the source.

Independently, Zitellini et al. (1999), based on the outcome of a regional MCS survey performed in 1992 (AR92 lines, Fig. 1), identified a very large active, compressive, tectonic structure located 100 km offshore SW Cape S. Vicente which was proposed as a good candidate for the generation of the 1775 event. This location was compatible with Baptista et al. (1998a,b) numerical modelling. An alternative solution was proposed by Gutscher et al. (2002)

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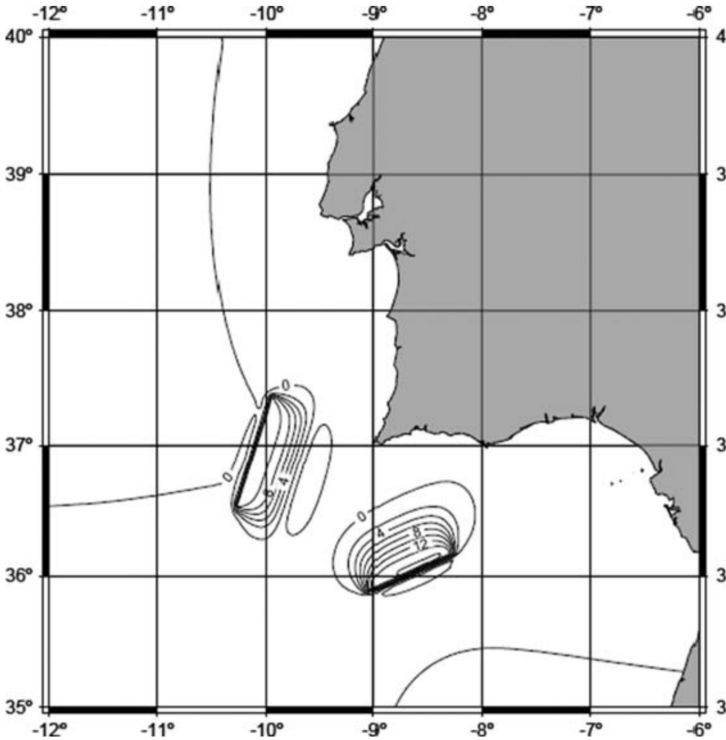

Fig. 1 Initial displacement for Marquês de Pombal fault

based upon seismic images of the crustal structure in the Gulf of Cadiz and tomographic images indicate an active accretionary wedge, overlying an eastward dipping basement and connected to a steep, east dipping slab of cold, oceanic lithosphere beneath Gibraltar. Gutscher use's the geometry of the shallow east dipping fault plane of the Gibraltar subduction as determined in a parallel study (Thiebot and Gutscher, in press) and calculate the ensuing seismic moment, as well as model synthetic tsunami waves generated by rupture along such a fault plane. Finally, we must refer the approach of Vilanova et al. (2003) which considered that an event triggered in the Lower Tagus Valley was the source of most of the damage observed close to Lisbon, and even of some "tsunami like" phenomena described in Oeiras and along the Estuary.

2 Historic Data

The evaluation of observed wave heights and travel times along the Iberian coast, Morocco, Madeira archipelago (Table 1) and United Kingdom (south coast) was obtained by Baptista et al. (1998a,b) through detailed study of coeval

Table 1 1755 tsunami travel time data

Model	Run-up (m)	Travel-time (min)
Oporto	1	–
Figueira	–	45 ± 10
Oeiras	>6	25 ± 10
S Vicente	>10	16 ± 7
Huelva	–	50 ± 10
Cadiz	15	78 ± 15
Porto Santo	–	60 ± 15
Madeira	4	90 ± 15
Safi	>6	30 ± 4

Portuguese, Spanish and English reports. Most significant results for tsunami modelling are summarized in Baptista et al. (1998b). In the following table we show a set of relevant data along Portugal, Spain and Morocco, as presented in Baptista et al. (1998b).

For tsunami propagation purposes we need to compute the initial sea disturbance. Tsunami propagation was computed using Swan model (Mader 2004). For each of the sources, compatibility between tsunami modelling and macroseismic analysis was analysed, taking into consideration the limitation of historical information.

3 Marques de Pombal

The Marquis de Pombal/Guadalquivir source was first proposed by Zitellini et al. (1999, 2001), based on the outcome of a regional MCS survey performed in 1992, identified a very large active, compressive, tectonic structure located 100 km offshore SW Cape São Vicente which was proposed as a good candidate for the generation of the 1775 event. This localization was compatible with Baptista et al. (1998b) numerical modeling. Hydrodynamic modeling using the Marquês de Pombal thrust fault (MPTF) segment as a single source and 20 m slip along the fault plane (Baptista et al. 2003) showed that the synthetic wave heights are underestimated overall Iberia shore, Madeira Islands and Morocco. Given the fact that an average displacement of 20 m must be seen as an upper limit (Wells and Coppersmith 1994) and that the seismic moment of the proposed source seems too small to justify the observed macroseismic field, new attempts were made to locate other "candidate-segments". Some alternatives were obtained during the BIGSETS MCS survey. Beside Gorringe Bank and Marquês de Pombal, the presence of other active compressive tectonic structures of regional significance: Horseshoe Fault (HS), Guadalquivir Bank (GB) and a large hill of tectonic origin located at 37.7°N 10°W can be considered as candidates for the large rupture area that took place in the 1-11-1755 earthquake.

One of the composite models, coherent with the tectonic and seismostratigraphic data available is displayed in Fig. 1. Its shape is similar to a "pop up", where the western segment corresponds to the Marques de Pombal fault. The size of the proposed tectonic source is constrained by MCS reflection profiles. Fault parameters of the two source segments are the following: MPTF segment: 105 km long, 55 km wide, dip angle 24°, strike 21.7°, slip 20 m; GB segment: 96 km long, 55 km wide, dip 45°, strike 70°, slip 20 m. The slip used, is comparable with average values deduced by Wells and Coppersmith (1994) and Johnston (1996).

Tsunami modeling, including also new information from the UK and the Caribbean is presented in Baptista et al. (2003). The analysis of the synthetic mareograms obtained from tsunami modeling showed a good agreement between the model travel-times and the observations for the most of the localities of SW Iberia, Madeira and Porto Santo, with the relevant exception of the wave height in Cadiz and the arrival time in Safi. The proposed source produce lower wave height and shorter arrival time in UK than observed and very low wave heights in Caribbean area.

4 Gibraltar Slab

A different proposal was made by Gutscher et al. (2002) that emphasized the difficulty of Zittelini source to justify the seismic moment of the 1755 earthquake, for a reasonable set of fault parameters (e.g. co-seismic displacement, rigidity, recurrence), given the slow, NW-SE relative convergence between Africa and Iberia at about 4 mm/a (Argus et al. 1989, Fernandes et al. 2003). Also, seismic images of the crustal structure in the Gulf of Cadiz and tomographic images indicate an active accretionary wedge, overlying an eastward dipping basement and connected to a steep, east dipping slab of cold, oceanic lithosphere beneath Gibraltar (Gutscher et al. 2002) (Fig. 2).

The principal fault plane is represented by a series of rectangular sub-planes extending from 6.5 km depth eastwards to a maximum depth of 24 km. The dip increases progressively from 2.5°, to 5° to 7.5°. The geometry of the seismogenic zone is obtained from deep crustal studies and can be represented by an east dipping fault plane with mean dimensions of 180 km (N-S) × 210 km (E-W). For 10 m of co-seismic slip an $Mw = 8.64$ event results and for 20 m of slip an $Mw = 8.8$ earthquake is generated. Thus, for convergence rates of about 1 cm/yr (for the slab relative displacement), an event of a very large magnitude can be generated every 1000–2000 years.

Modeled tsunami travel times show good agreement for the Cape St. Vincent (6 min difference) and Madeira stations (5–15 min difference). The modeled arrivals for Huelva and Vila Real are 7 min later and 8 min earlier, respectively, than the reported 45 min. The modeled arrival time for Cadiz is about 40 min too early (36 min vs. 78 min). This is a consequence of

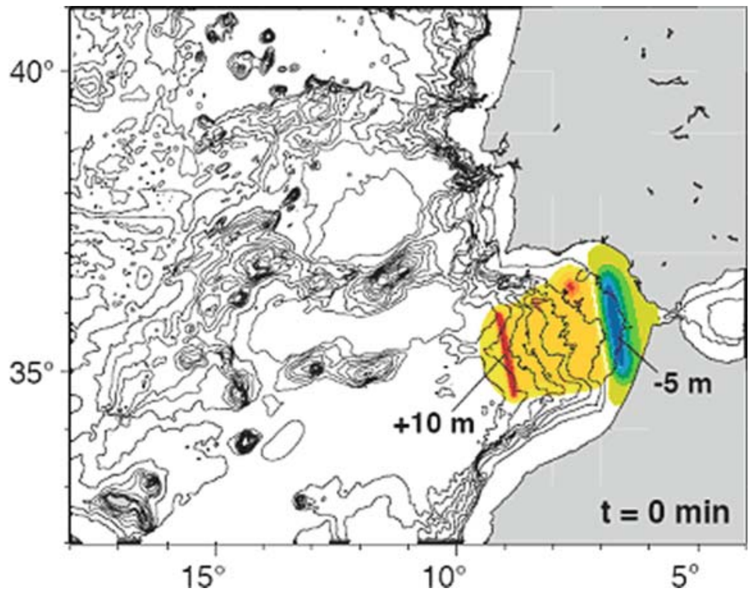

Fig. 2 Initial displacement for Gibraltar Slab source

the eastward extent of the deep fault plane, where the down-dip limit was taken on the basis of thermal modeling of forearc thermal structure (Thiebot and Gutscher, in press). A shorter "deep" fault plane would yield a larger travel time to Cadiz and provide a better travel time fit, while slightly reducing the total seismic moment. Arrival times for the Portuguese west coast and Safi, are about 25–40 min later than historical reports. This may be due to a combination of several possible causes: substantial delays due to poorly constrained shallow shelf bathymetry, inaccuracy of historical reports or multiple tsunami sources. Modeled tsunami amplitudes are greatest in the Gulf of Cadiz - Cape St. Vincent region, in agreement with historical observations. However, most modeled amplitudes are significantly less than historical reports (30–50% of the reported amplitudes for the 20 m fault splay source). An even greater discrepancy exists for Safi (Morocco) where only a 1.2 m high wave is modeled but a 6 m wave was reported. For many stations some of this discrepancy may be due to run-up effects, which have been estimated to locally increase tsunami amplitudes by 30% (Baptista et al. 1998b). But for Safi another explanation must be sought. Given the systematically late arrivals and low amplitudes for the Portuguese west coast stations, it appears that the subduction source proposed here, cannot alone reproduce the observed tsunami for the entire region and that a contribution from an additional source further to the NW may be necessary (Gutscher et al., in press).

5 Lower Tagus Source

Vilanova et al. (2003) infer the existence of a contradiction between the tsunami travel times observed in several points at and near Lisbon and its presumed origin from an offshore source. Their basic proposal is that Oeiras observation (25 min) reports the LR's tsunami while the Lisbon (Marvila, 90 min) and Porto Novo (or A-dos-Cunhados, 75 min) would refer to the offshore-originated tsunami.

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Pereira de Sousa discusses: "o cura de A. Dos Cunhados informa que esse intervalo foi de 5 quartos. Devia ter sido mais tempo do que na Cruz Quebrada mas a diferença não podia ser tão grande" (Sousa, p. 849). Sousa (1932) already detected this problem and his conclusion was to disregard the A-dos-Cunhados description. As regards the travel time observed at Porto Novo, this report was considered unreliable by Baptista (1998) and Baptista et al. (1998a,b). Historical data concerning Figueira da Foz (north of Porto Novo, 45 min) are well constrained by more than one answer to the Inquiry of Marques de Pombal, while for Porto Novo only a single report was found, that is the transcription made by Sousa (1932) from parish Archives. Consider for example the following document (N° 638) from ANTT (Arquivo Nacional da Torre do Tombo), relative to Buarcos, close to Figueira da Foz "acabados que foram na Terra os Terremotos, no referido dia primeiro de Novembro, passado discurso de meia hora, com pouca diferença, que veio a ser pelas 10 horas e um quarto do dia, começou este a levantar tais montes de água, que lá foram combater com os de terra, o que meteu tanto susto aos habitantes desta Vila, que quase assentaram consigo que se tinham escapado aos Terremotos da terra, dos do mar não escapariam [...] subiram fora da ordem natural 44 palmos, pouco mais ou menos, encheu por 3 ou 4 vezes [...]. In the same location the earthquake arrival time is evaluated as "às nove horas e meia da manhã". So the tsunami travel time in Buarcos is clearly established as 45 min, coherent with an early arrival to the Lisbon harbour.

6 Conclusions

Historical tsunami have inherent limitations due to the uncertainties in arrival times (only when the earthquake is also perceived we can have a good guess, not biased by local time systems), and also by the difficulty in the wave height evaluation, often confused with run-up highs. However, the data set available for the 1755 event is large and geographically representative, and so it gives a good constrain, particularly if combined with macroseismic information. Geodetic data must also be considered when recurrence is sought, because the low convergence rate between Eurasia and Nubia puts strong limitations on the set of possible source areas.

Finally, there is no substitute for tectonic research, particularly because the source dimensions must be very large, and the morphological signature must be relevant.

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A Finite–Fault Modeling of the 1755 Lisbon Earthquake Sources

A. Carvalho, A. Campos Costa and Carlos S. Oliveira

1 Introduction

In Portugal, being located at a moderate/low seismicity intraplate area, insufficient accelerograms have been recorded to satisfactory undertake any regional empirical study. The number of accelerograms is not only small but also refers to low-magnitude earthquakes located in only some parts of its entire seismic area. For that reason, most prediction techniques of ground motion in Portugal have been based on international empirical laws and not on regional data to quantify the characteristics of ground motions. However, differences in the regional geology can led to variations in ground motions characteristics and the use of empirical laws of other regions is questionable and may not be appropriate for Portugal. As prediction cannot be based on empirical analyses, theoretical models must be used as the basis for the predictions of strong motion in Portugal. The development of stochastic based ground motion synthesis associated to a seismological finite-fault modeling is a worldwide approach that can be used for representation of future large magnitude earthquakes occurring in Portugal, allowing the reproducing of specific source effects like directivity and asperities distribution.

The strong ground motion prediction based on finite-fault simulation requires the identification of the fault (strike, dip, length and width), source kinematics parameters (stress drop, velocity of rupture, slip distribution), regional crustal properties (geometrical spreading, anelastic structure, amplification and attenuation upper crust parameters) and the determination of amplification effects due to the local site geology.

The model parameters calibration has been obtained with a dataset that includes horizontal components of ground acceleration records (at rock sites) obtained by the Portuguese digital accelerometer network. Validation and comparison are entirely in terms of 5% damped pseudo absolute response spectra for acceleration.

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The demonstrated agreement between model and data for low to moderate events in Portugal provides strong grounds for accepting the stochastic-process model predictions for this type of events and to use it as the basis for characterization of stronger earthquakes considering a finite fault rupture modeled as a sum of a number of point sources distributed spatially and temporally.

Being so, the calibrated model is used to simulate ground motion acceleration for the 1755 Lisbon earthquake, considering different source models proposed in literature. For each possible source model several simulations were performed varying nucleation points. The soil effect was taken into account considering a nonlinear behaviour of the stratified geotechnical sites conditions. The simulated intensity is compared with the observed intensity values.

2 Numerical Approach

The non-stationary stochastic finite fault simulation method, herein called RSSIM (Carvalho et al. 2004, 2007) has been implemented starting from the classic simulation code FINSIM (Beresnev and Atkinson 1998), widely employed in the seismological literature for simulation of the ground motion from both moderate and high magnitude earthquakes. However, it differs from the classic FINSIM as it avoids the computation of acceleration time series representing the contribution of each sub-fault, but synthesizes the ground motion due to the entire fault from the Power Spectral Density Function (PSDF) radiated by each sub-fault, using the random vibration theory and the extreme values statistics (ex. Vanmarcke 1976, Boore and Joyner 1984, Boore 2003).

Like FINSIM, the RSSIM method assumes that the fault plane is a rectangle, subdivided into an appropriate number of sub-faults, which are modelled as point sources characterized by an ω^2 spectrum.

The method starts estimating, for each subfault, the amplitude of the acceleration Fourier spectrum, $A(\omega, R)$:

$$A(\omega, R) = \omega^2 \cdot C \cdot S(\omega) \cdot G(R) \cdot An(\omega, R) \cdot P(\omega) \cdot F_Z(\omega) \tag{1}$$

where C is a scaling factor including the free surface amplification factor, the radiation pattern of shear waves and the energy partition into the two horizontal components, $S(\omega)$ is the amplitude displacement source spectrum, $G(R)$ is the geometric spreading factor, $An(\omega,R)$ is the anelastic path attenuation factor, $P(\omega)$ accounts for the upper crust attenuation and $F_Z(\omega)$ is a crustal amplification function. The functional form of all these factors and the respective physical meaning can be found elsewhere (e.g. Boore 2003, Carvalho et al. 2004, Ferrer and Sánchez-Carratalá 2004) and in the modeling parameters section of this text.

Taking into account the Fourier amplitude spectrum $A(\omega, R)$ and a given source duration, T_s, the one-sided power spectral density function (PSDF) of acceleration can be derived by means of:

$$Sa(\omega) = \frac{1}{\pi}\frac{|A(\omega, R)|^2}{T_s} \tag{2}$$

The PSDF of the response of the oscillator with a circular frequency ω_n and a damping ratio ζ assuming a stationary process is calculated as:

$$S_x(\omega, \omega_n, \xi) = Sa(\omega)|H_x(\omega, \omega_n, \zeta)|^2 \tag{3}$$

in which $H_x(\omega, \omega_n, \zeta)$ is the complex displacement frequency response function relative to the input absolute base acceleration and given by

$$H_x(\omega, \omega_n, \zeta) = \frac{\omega_n^2}{-\omega_n^2 + 2i\zeta\omega_n\omega + \omega^2} \tag{4}$$

To cope with the non stationary of the intensity of the response of the oscillator, an intensity time-modulating response function, θ, is specified directly in a way that the evolutionary response moment of order k is obtained by the modulating function and the stationary response moment of order k as:

$$\lambda_k(t, \omega, \omega_n, \zeta) = \theta^2(t, \omega_n, \zeta) \cdot \lambda_k(\omega, \omega_n, \zeta) \tag{5}$$

in which

$$\lambda_k(\omega, \omega_n, \zeta) = \int_0^\infty \omega^k S_x(\omega, \omega_n, \zeta)d\omega \tag{6}$$

and θ, the response modulating function, is obviously dependent of frequency and damping of the one degree of freedom system and can be found in Duarte (1978):

$$\theta(t, \omega_n, \zeta) = \begin{cases} 0 & t \le t_1 \\ \sqrt{1 - e^{-2\cdot\omega_n\cdot\zeta\cdot(t-t_1)}} & t_1 < t \le t_2 \, . \\ \sqrt{e^{-2\cdot\omega_n\cdot\zeta\cdot(t-t_2)} - e^{-2\cdot\omega_n\cdot\zeta\cdot(t-t_1)}} & t > t_2 \end{cases} \tag{7}$$

where t_1 represents the starting time of the strong motion part of the accelerogram and t_2 gives the starting time of the decay part being $t_2 = T + t_1$, T the duration of the strong motion.

For a finite source, subdivided into N sub-faults, and considering that stochastic process associated to all the N subfaults are independent, the final evolutionary finite–fault response moment, λ_k^T, can be estimated as the sum of all subfault response moments. Therefore:

$$\lambda_k^T(t,\omega,\omega_n,\zeta)=\sum_{j=1}^{N}\theta_j^2(t,\omega_n,\zeta)\cdot\lambda_{k_j}(\omega,\omega_n,\zeta) \tag{8}$$

Considering the extreme values statistics and taking T as the duration of the earthquake, the displacement non-stationary response spectrum, RS, which synthesizes the integration over the fault of all the delayed sub-sources can be estimated (Vanmarcke 1976):

$$RS(T,\omega,\omega_n,\zeta)=\left(\sqrt{2\cdot\ln(2\cdot f_0\cdot T)}+\frac{0.577216}{\sqrt{2\cdot\ln(2\cdot f_0\cdot T)}}\right)\cdot\sqrt{\lambda_0^T(T,\omega,\omega_n,\zeta)} \tag{9}$$

Where f_o is the central frequency of the response written as

$$f_0=\frac{1}{2\pi}\left[\frac{\lambda_2^T(T,\omega,\omega_n,\zeta)}{\lambda_0^T(T,\omega,\omega_n,\zeta)}\right]^{1/2} \tag{10}$$

Comparison of response spectra obtained with the non-stationary stochastic finite fault simulation method, RSSIM, and with classic FINSIM can be seen in Zonno et al. (2005) and Carvalho et al. (2007).

Once the non-stationary response spectra have been achieved, an equivalent stationary PSDF can be iteratively estimated, following the classical theory of stationary random process. This approach was adopted in an automatic seismic loss estimate methodology (LNECLoss – Sousa et al. 2004) that was developed at LNEC. This PSDF will be the input for the 1D analysis of soil effects.

In order to define scenarios at locations where different geotechnical conditions prevail, an adequate transfer function must be defined to cope with the filtering effect and dissipative nature of local soil propagation medium. The complex transfer function $H_\gamma(\omega|\vec{h},\rho,\vec{\varsigma},\vec{G})_i$ from rock outcrop acceleration to shear strain γ of each layer i is obtained, which allows to write the shear strain PSDF as function of the independent variable associated with the circular frequency ω:

$$S_\gamma(\omega)_i=H_\gamma(\omega|\vec{h},\rho,\vec{\varsigma},\vec{G})_i\cdot S_{ro}(\omega)\cdot H_\gamma^*(\omega|\vec{h},\rho,\vec{\varsigma},\vec{G})_i \tag{11}$$

in which $\vec{h},\vec{\rho},\vec{\varsigma}$ and $\vec{G}$ are parameters vectors of $n+1$ dimension, representing the height, density, viscous proportional damping ratio and the elasticity distortion modulus, of each stratified soil layer i, respectively. $H_\gamma(\cdot)_i$ and $H_\gamma^*(\cdot)_i$ are complex conjugate functions.

This last equation is only valid under the hypothesis of elastic linear behaviour of soil meaning that, parameters $(\vec{\varsigma},\vec{G})_i$ should be kept constant even if they depend on the value of shear strain γ. However, even for low amplitude input motion, soil tends to exhibit a non-linear behaviour. In this case an equivalent iterative linear secant stiffness approach was adopted as it is frequently used in other soil dynamics computer codes (SHAKE, FLUSH). For that purpose it will be necessary to have estimates of the peak shear strain vector $\vec{\gamma}_{i,j}^{peak}$ whose members i vary from 1 to n for each soil layer, and j stands for the interaction cycle.

Extreme Statistics determines the estimates of $\vec{\gamma}_{j}^{peak}$ based on the hypothesis that the *PSDFs* $S_\gamma(\omega)_{i,j}$ are a Gaussian zero mean random processes. In that case estimates of $\gamma_{i,j}^{peak}$ can be obtained by:

$$\gamma_{i,j}^{peak} = \sqrt{2\lambda 0_{i,j}\left[ln\left(\frac{T}{2\pi}\sqrt{\frac{\lambda 2_{i,j}}{\lambda 0_{i,j}}}\right) - ln(ln(2)\right]} \tag{12}$$

where the stationary moments of 1st and 2nd orders of the strain are defined as defined in Equation (19) but for the *PSDFs* $S_\gamma(\omega)_{i,j}$ defined in Equation (11).

Once convergence is attained, the transfer function $H_a(\omega)$ between rock outcrop acceleration and site specific acceleration is defined, and the PSDF of this acceleration can be computed as:

$$S_a(\omega) = S_{ro}(\omega) \cdot |H_a(\omega)|^2 \tag{13}$$

in which $|H_a(\omega)|^2$ represents the transfer function between outcrop acceleration and the absolute acceleration at the top of the 1D vertically S wave propagation column of stratified soil medium.

This approach allows a significant saving of calculation time, roughly a factor of twenty with respect to the usual approach based on the traditional methods, can be achieved.

Resuming, each seismic input is defined by the rock outcrop power spectral density function of acceleration and, given a stratified soil profile, the new PSDF is computed at the surface level using procedure described above.

3 Modeling Parameters

Finite-fault simulations require that the fault-plane geometry (length, width, strike, dip, number of subfaults considered and depth to the upper edge), the source parameters (seismic moment, slip distribution, stress drop, nucleation point, rupture velocity), the crustal properties of the region (geometrical spreading coefficient and anelastic attenuation) and the site-specific soil response information be previously specified.

Carvalho et al. (2005, 2007) estimated most of this parameters using previous information from several regional studies and a least square sense, comparing synthetic and observed response acceleration spectra, to obtain a set of parameters that best fit data. In the last months a progress has been made towards understanding the source and crustal characteristics, estimating source parameters based on source spectra of S waves of 12 earthquakes records (corresponding to a total of 25 records) and the upper crustal attenuation parameter from the high-frequency decay of the acceleration spectra, after correcting observed spectra from geometrical spreading and anelastic attenuation effect. Here we report the results of that study.

The dataset used includes horizontal components of 25 ground acceleration records obtained by the portuguese digital accelerographic network, on rock sites. The regional distribution of the earthquakes epicenters and magnitudes are illustrated in Fig. 1. Some of the events were recorded in more than one station.

As two physical mechanisms of earthquake generations exists in Portugal, namely events originated by the movement between the Eurasian and African plates (here in called interplates events) and events originated in faults inside the Eurasian plate (herein called intraplate events), calibration of model parameters was done separately with data corresponding to intraplate and interplates events.

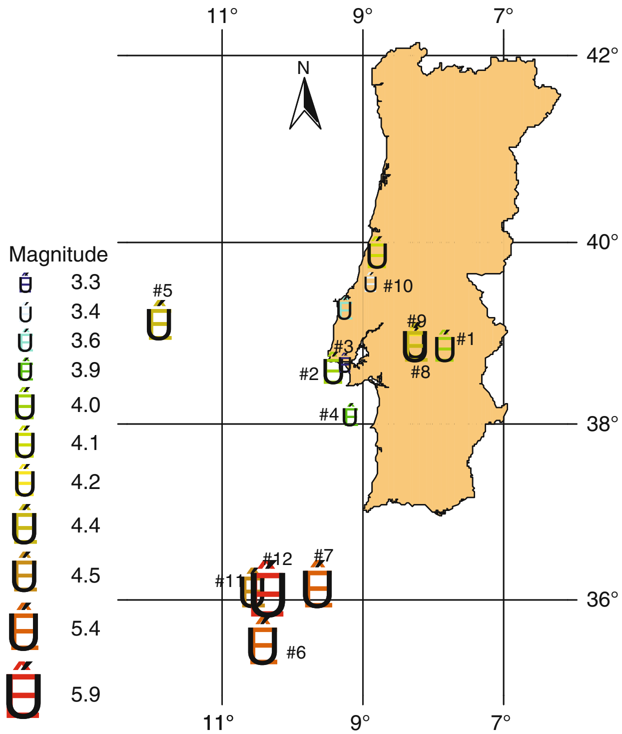


Fig. 1 Epicentral distribution of earthquakes of portuguese accelerographic data base

The horizontal components of the accelerations at each station were windowed, using a time window that included the main S-wave arrival and 90% of total energy, and then Fourier transformed to obtain acceleration and displacement-amplitude spectra.

3.1 The Observed Spectra

In RSSIM approach, the source term ($S(\omega)$ in Equation 1)is assumed as an omega-square (ω^2) model specified by (Brune 1970, 1971):

$$S(\omega) = \frac{M_o}{1 + (\omega/\omega_c)^2} \tag{14}$$

where M_o and ω_c stand for seismic moment and circular corner frequency respectively.

A visual inspection of the displacement-amplitude spectra of S-waves for the 12 events (see Fig. 2 for an example) leads to the conclusion that the spectral shapes for all events appear to be similar, showing a plateau at lower frequencies, a decrease of the spectral amplitudes as f^{-2} at higher frequencies and one corner frequency (the interception of both plateau and f^{-2} lines), as predicted by the ω^2 model. However this feature seems to begin to change as magnitude nears 5.4 and the spectral shape of the 6.0 earthquake has two corner frequencies, where the spectral envelope changes its general trend. This observational result implies that the shape of the source as described by the ω^2 model ceases to be valid as magnitude increases up to 5.5.

Another feature showed is that for frequencies greater than a certain frequency the spectral amplitudes decay more rapidly as f^{-2}. This decay is modeled as the high-cut filter $P(\omega)$ in Equation (1) and in this study is described as $P(\omega) = \exp(-\omega \cdot k/2)$(Anderson and Hough 1984), being k the attenuation upper crustal parameter.

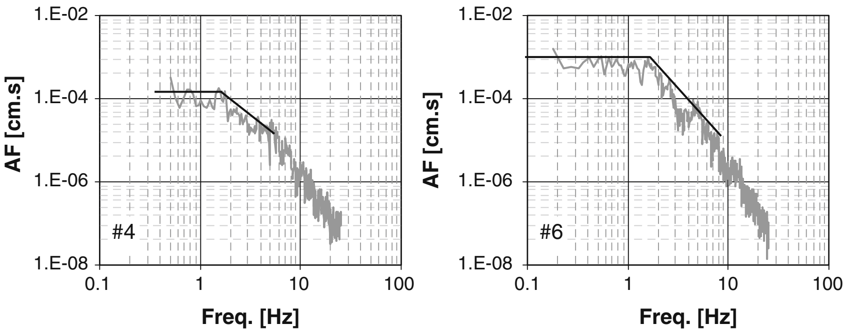


Fig. 2 Observed displacement-amplitude spectra of S-waves (for earthquakes #4 and #6, as in Fig. 1) and lines of ω^2 model

3.2 Path Parameters

We adopted the frequency-dependent quality factor, $Q(f) = 250f^{0.7}$ of Pujades et al. (1990) for the inelastic attenuation $An(\omega, R) = \exp(-\pi f R/Q(f)\beta)$ and the tri-piece-wise geometric attenuation function described by Atkinson and Boore (1995) defined by assuming a crustal seismogenic thickness of 31 km for the intraplate events (Jiménez-Munt et al. 2001) and 20 km for the interplate events (Dèzes and Ziegler 2001).

3.3 Upper Crustal Parameter

The term $P(\omega) = \exp(-\pi f \kappa)$ is a high cut filter to account for near-surface attenuation effects, which describes the observed rapid spectral decay at high frequencies. The k parameter is estimated by fitting the high-frequency decay of the acceleration spectrum with a straight line in a log-linear scale (Anderson and Hough 1984), as illustrated in Fig. 3. k was measured separately for the two horizontal components of all the 25 records of our database. A suitable frequency band (depending on quality of data, generally ranging from 5–10 Hz to 20–25 Hz) was chosen in order to perform reliable regressions. A value of $k = (0.032 \pm 0.005)$ s was inferred from the analysis of all records.

As regard to site-specific amplification, we expressed the factor $F_z(\omega)$ in Equation (1), as a function that takes into account of density and S wave velocity changes respect to an homogeneous propagation model:

$$F_z(\omega) = \frac{2}{1 + \left(\frac{\omega_{zi}}{\omega}\right)^2} \quad \omega > \omega_{zi} \tag{15}$$

We employed the impedance function with $f_{zi} = 0.5$ Hz, for interplate events, accordingly to Carvalho et al. (2007).

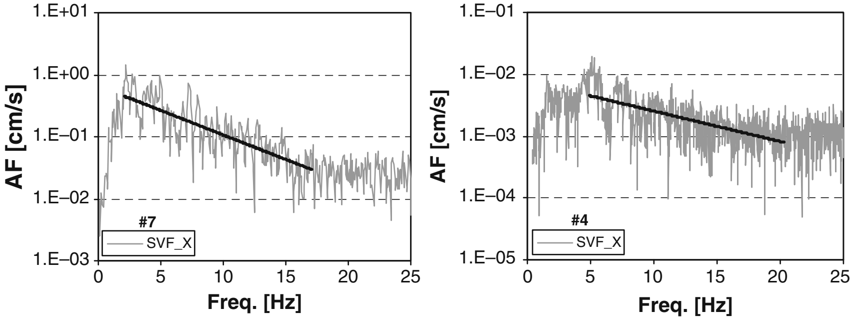


Fig. 3 Example of k estimations from the amplitude Fourier spectra of acceleration. SVF_X: x horizontal component of the S.Vicente Fora (SVF) station

3.4 Source Parameter

Having a better understanding of attenuation, we can correct the observed spectrum back to the source, obtaining the source displacement-amplitude spectrum as:

$$S(f) = \frac{A(f,R)}{(2\pi f)^2 An(f,R) \cdot G(R) \cdot P(f)} = \frac{A(f,R)}{(2\pi f)^2} \cdot e^{\pi f \cdot k} \cdot R^{\alpha} \cdot e^{\pi \cdot R \cdot f/\beta Q(f)} \quad (16)$$

where $A(f,R)$ is the S-wave spectrum of ground acceleration observed, the two exponential terms are used to correct both upper crust and path and attenuation and the terms R^{α} corrects the amplitude decrease due to geometric spreading.

In the w^2 model, the high-frequency level of the source model is controlled by stress drop, $\Delta\sigma$, that is correlated with frequency corner frequency in terms of the relation

$$f_c = 4.91 \times 10^6 \beta \cdot (\Delta\sigma / M_0)^{1/3} \quad (17)$$

where $\Delta\sigma$ is in bars, β in km/sec and M_0 in dyne.cm.

As stress drop increases for an earthquake of a given moment, so does f_c and the high-frequency level of the source spectrum. Being so, the parameters that take values appropriate to each earthquake source spectrum are seismic moment and stress drop.

On the basis of w^2 model, the seismic moment and the corner frequency can be obtained by the horizontal and f^{-2} lines from the Source displacement-amplitude spectra. Rovelli et al. (1991) suggests that one should avoid visual inspection or the simultaneous inversion of the ground motion spectrum in terms of both seismic moment and corner frequency and computed the corner frequency by means of an objective technique using the expression of Andrews (1986):

$$f_c = \frac{1}{2\pi} \sqrt{\frac{\int_0^{\infty} V^2(f)df}{\int_0^{\infty} D^2(f)df}} \quad (18)$$

where V(f) and D(f) are the source velocity and displacement spectra. Knowing f_c and the seismic moment, the stress drop can be obtained using Equation (17).

Another technique is to estimate simultaneously seismic moment and stress drop using the theoretical function of the source spectrum (Equation 14) and fit it to the source spectra of S waves for an assumed seismic moment with a constant stress drop (example in Fig. 4). The seismic moment is obtained from the source spectral amplitude at low frequency while searching to obtain $\Delta\sigma$ at high frequencies.

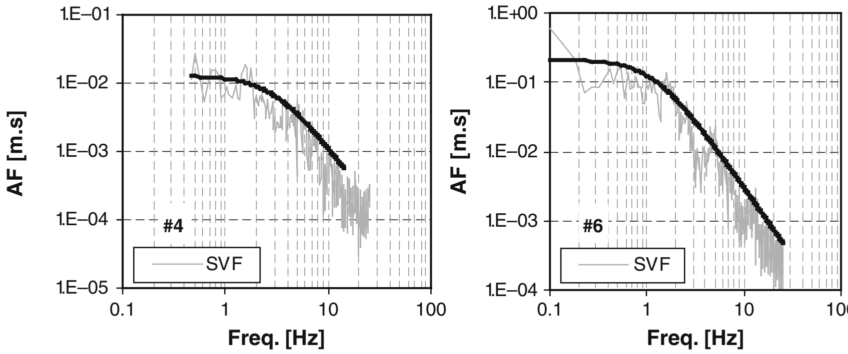


Fig. 4 Example of the source spectra of two events. The smooth curve represents the theoretical model of Eq. (14) that was fit to the envelope of the source spectrum

We have used the three techniques (visual inspection, Equation (18) and the fit of source theoretical function to the source S-wave spectra) to obtain source parameters for each earthquake.

For the set of earthquakes the average stress drop is 101 bars for intraplate events and 66 bars for interplates events.

3.5 Duration

The duration T of an earthquake signal at a hypocentral distance R can be represented as (Atkinson and Boore 1995)

$$T(R) = T_0 + bR \tag{19}$$

where T_0 is the source duration (here taken as $1/f_c$) and b is a coefficient controlling the increase of duration with distance. Carvalho et al. (2007) obtained $b = 0.1$ for intraplate events and $b = 0.015$ for interpolate events.

Table 1 Model parameters

	Intraplate	Interplate
Crustal thickness, D (*km*)	31	20
Quality factor, Q(f)	$250*f^{0.7}$	
Geometric attenuation	1/R (R $\leq$ 1.5D *km*) $1/R^0$ (1.5D *km*< R $\leq$ 2.5D *km*) $1/R^{0.5}$ (R > 2.5D *km*)	
Distance-dependent duration	0.1 R	0.015 R
k (*s*)	0.032	
Shear-wave velocity, β (*km/s*)	3.5	
Crustal density, ρ (g/cm^3)	2.8	
Stress drop, $\Delta\sigma$ (*bars*)	101	66
Amplification function $F_{zi}(\omega)$		$F_{zi}(\omega) = \dfrac{2}{1 + \left(\frac{\omega_{zi}}{\omega}\right)^2} \omega > \omega_{zi}$ $f_{zi} = 0.5H_z$

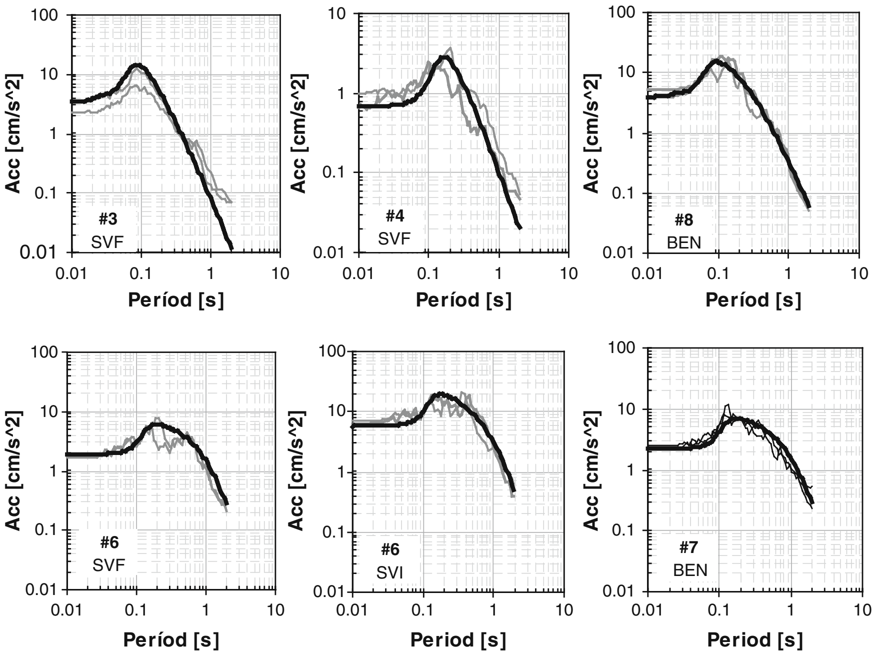


Fig. 5 Example of response spectra comparison between synthetic and observed data. Black lines are the simulated results, gray lines are both horizontal components of the recorded accelerogram. (simple line: component N-S; bold line: component E-W). Stations: SVF – S. Vicente Fora (Lisbon); BEN – Benavente (Benavente); SVI – S. Vicente (Algarve)

Results of model parameters are represented in Table 1 and Fig. 5 shows some examples of response spectra comparison between synthetic and observed. In general, the synthetics are in good agreement with observations for at least one of the recorded horizontal components.

4 1755 Lisbon Earthquake

4.1 Source Models

The 1755.11.01 earthquake, known as the 1755 Lisbon earthquake, generated the largest known tsunami in SW Europe and its magnitude has been estimated as Mw = 8.5–8.9 by several authors (e.g. Abe 1979, Moreira 1984). The exact location remains controversial, even though the earthquake epicentre is know to have been offshore. For the localization of the major seismogenic zones in the SW Iberia Margin, please refer to Fig. 1 of Ribeiro et al. (2008) (this volume).

The several isoseismal maps published led to the conclusion that the source location of this event was in the vicinity of Gorringe Bank (GBF) (Martinez

Solares et al. 1979, Levret 1991). This location was further supported by the occurrence of a tsunamigenic earthquake on 1969.02.28. Johnston (1996) suggested that a NE-SW trending fault, at the base of the NW flank of Gorringe Bank, could be responsible for the 1755 earthquake.

Baptista et al. (1998) performed a hydrodynamic modelling of the 1755 tsunami. Results from a backward ray tracing simulations suggest a tsunami source located quite close to the Portuguese coast. In order to precisely locate the 1755 seismogenic source, in 1998 the area between the Gorringe Bank and the Cape St. Vicente has been surveyed within the framework of the European BIGSETS projects (Big Sources of Earthquake and Tsunami in SW Iberia). One of the main results (Zitellini et al. 2001) was the characterization of the active tectonic structure located offshore Cape St. Vicente named as Marques Pombal Thrust fault (MPT) which, accordingly to the authors, could be the generator of the 1755 Lisbon earthquake.

Terrinha et al. (2003) pointed out that additional rupture areas have to be associated with the MPT system to generate such a destructive earthquake and that the 65 km long Pereira de Sousa Normal Fault (PSNF) located along the Portuguese margin, to the north of MPT, could constitute the northward prolongation of the MPT.

Baptista et al. (2003), based on new MCS (multi-channel seismic reflections surveys) data, presented a new reappraisal of the 1755 source and proposed a possible composite source including the Marques de Pombal Thrust fault and the Guadalquivir Bank (NGBF), which, accordingly to authors, will act as a possible southeastward extension of the rupture area related to the 1755 event.

Vilanova et al. (2003) propose that, although the main shock was offshore, the resulting static stress changes induced the rupture of the Lower Tagus Valley (LTVF), near Lisbon. They favour this model, rather that site effects causing high intensities in the Lisbon area, because not only the highest intensities show a negative correlation to soft soil but also because this local rupture can explain other phenomena described in the eyewitness accounts like an internal tsunami in the Tagus River, ground deformation affecting the course of the Tagus River, the spatial pattern of damaging aftershocks, duration of the event and the number of shocks felt.

Gutscher (2004) proposed an alternative area for the 1755 event, a shallow east dipping fault plane beneath the Gulf of Cadiz associated with subduction beneath Gibraltar.

Zitellini et al. (2004) and Ribeiro et al. (2006) proposed the Horseshoe Thrust Fault (HF) as an additional source area to the Marquês de Pombal Thrust. They favour this solution for additional area/slip for the 1755 event, given its orientation sub parallel to the MPT and the almost geometric continuity between both fault zones that facilitates the strain/displacement transfer between them.

Grandin et al. (2007), using a velocity model valid for frequencies up to 0,3 Hz and validated for the 1969.02.28 earthquake, proposed a regression to convert simulated values of Peak Ground Velocity (PGV) into Modified

Mercalli Intensity (MMI) in SW Iberia. They extrapolate the method and the empirical relationship PGV-IMM to the case of the 1755 earthquake, and tested three sources proposed by different authors, namely the Gorringe Bank solution, the Marques de Pombal – Pereira de Sousa fault system and the subduction in the Gulf of Cádiz. The authors concluded that rupture directivity plays a particularly important role and that a primary source located at Gorringe Bank is the most realistic hypothesis to fit the observed isoseismal pattern.

4.2 Modeling Geometry

Using the stochastic finite fault method explained above, we have tested the six different fault source geometries for the source of the 1755 earthquake proposed by the different authors, and mentioned in the previous section.

For all geometries, fault segments were divided into smaller subfaults, each one considered as a point source. The slip distribution is randomized, and for each fault model three nucleation points were considered, to take in account directivity studies. The geometry of each source model is presented in Table 2 and the fault source geometries and the different nucleation points considered are presented in the next figures. The geometry named GBF considers the model of Johnston (1996), MPT-PS considers the proposal of Terrinha et al. (2003), the model MPT-GqB considers the study of Baptista et al. (2003), model off-LVTF, considers besides the main shock offshore (Gorringe Bank or Marquês Pombal Fault), a second earthquake in Lower Tagus Valley Fault (LTVF) as proposed by Vilanova et al. (2003), the MPT – HF considers the Horseshoe Thrust Fault as an additional source area to the Marquês de Pombal Thrust Fault and the geometry model GC considers the proposal of Gutscher (2004). The numbers presented in each fault geometry represent the different nucleation points considered in simulations.

Table 2 Source parameters: fault geometry (strike, dip and dimensions) and magnitude considered

Source	Strike	Dip	(L × W) (km)	M
CBF	N60°E	40°	200 × 80	8.7
MPT	N20°E	24°	100 × 70	8.3
PSNF	N180°E	24°	100 × 120	8.45
MPT	N21,7°E	24°	100 × 55	8.3
GqB	N70°E	45°	96 × 55	8.3
HF	N60E	45°	175 × 140	8.3
MPT	N20°E	24°	100 × 70	
LVTF	N38°E	55°	40 × 28	6.7
		2,5°	162 × 68	8.4
GC	N11°W	5,0°	174 × 68	8.45
		7,5°	198 × 6.8	8.5

4.3 Surface Modeling

For each Portuguese council, the response spectrum derived from each source model, was transformed into Power Spectrum Density Function (PSDF), at the bedrock, using the classical theory of stationary random process already described. Site effects were, then, evaluated following the numerical approach explained elsewhere in this paper and using an available data base (herein DB1), containing information about soil classification for Portugal (classified as 0, 1 and 2 for rock, medium and soft soil respectively) and a data base (DB2) with stratified soil profile units for the Metropolitan Area of Lisbon (MAL). In DB2, each soil unit considers the thickness of shallow layers, shear waves velocity, density and plastic index. This data base was built in the framework of the project conducted by the Portuguese civil protection authority, for which it was conducted a geological – geotechnical survey that allowed the characterization of stratified soil profile units for MAL with a fair level of detail. Each classified soil type (0, 1 or 2, accordingly to DB1) for MAL correspond to a particular stratified soil profile accordingly to DB2 and we extrapolated this soil profile units for every Portuguese counties that had the same soil type classification.

5 Results and Discussion

For each fault geometry seismic action was computed at the bedrock level and surface. Results for each source model considered, for each nucleation point, are shown in Figs. 6 to 10. PGA_b and PGA_s means Peak Ground Acceleration at bedrock level and surface, respectively, PGV_s stands for Peak Ground Velocity at surface and IMM for the Modified Mercalli Intensity. Synthetic isoseismal maps were built using the regression of Grandin et al. (2007) that convert PGV into IMM in SW Iberia.

It is emphasized the primordial effects of rupture directions on strong ground motions, as also shown in Grandin et al. (2007), which remarks the importance of using methodologies that can account for directivity effects as already mentioned in Carvalho et al. (2007).

An important conclusion of these figures is that the soil effects play a particularly important role in the pattern of isoseismals. In fact, even when peak ground values maps show a radial pattern, the surface maps reproduce most features of the observed macroseismic field, for most of the source models.

Our results do not sustained a solution of a source located below Gorringe Bank, which produce too low values of IMM in south of Portugal. The composite models that considers the Marques the Pombal fault and a additional source area (MPT-GqB, MPT-PSNF, MPT-HF) seems good hypothesis, considering an upward rupture, towards Lisbon City, the Guadalquivir Bank source model reproducing higher intensities in Algarve. The subduction zone

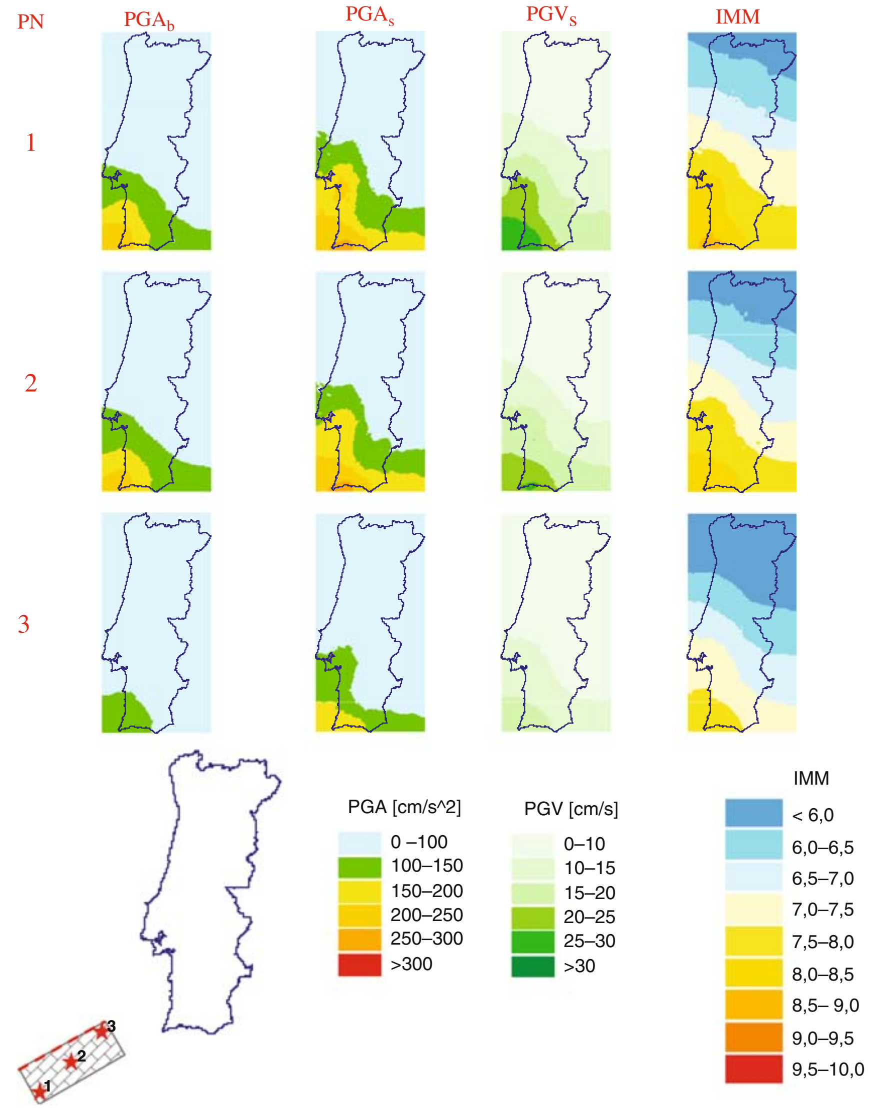


Fig. 6 Synthetic PGAb, PGAs, PGVs and isoseismal maps, for nucleation points 1–3, using the source model GBF

in Gulf of Cádiz fails to reproduce the isoseismal features of the 1755 earthquake in western Portugal.

Considering a triggered rupture in Lower Tagus Valley Fault, a M6.7 seems a too high magnitude for the inland earthquake (similar PGA values in Lisbon and Algarve) but a smaller magnitude, in order of M6.5 can give satisfactory PGA values in Lisbon. Nevertheless, PGV values in Lisbon are caused by the

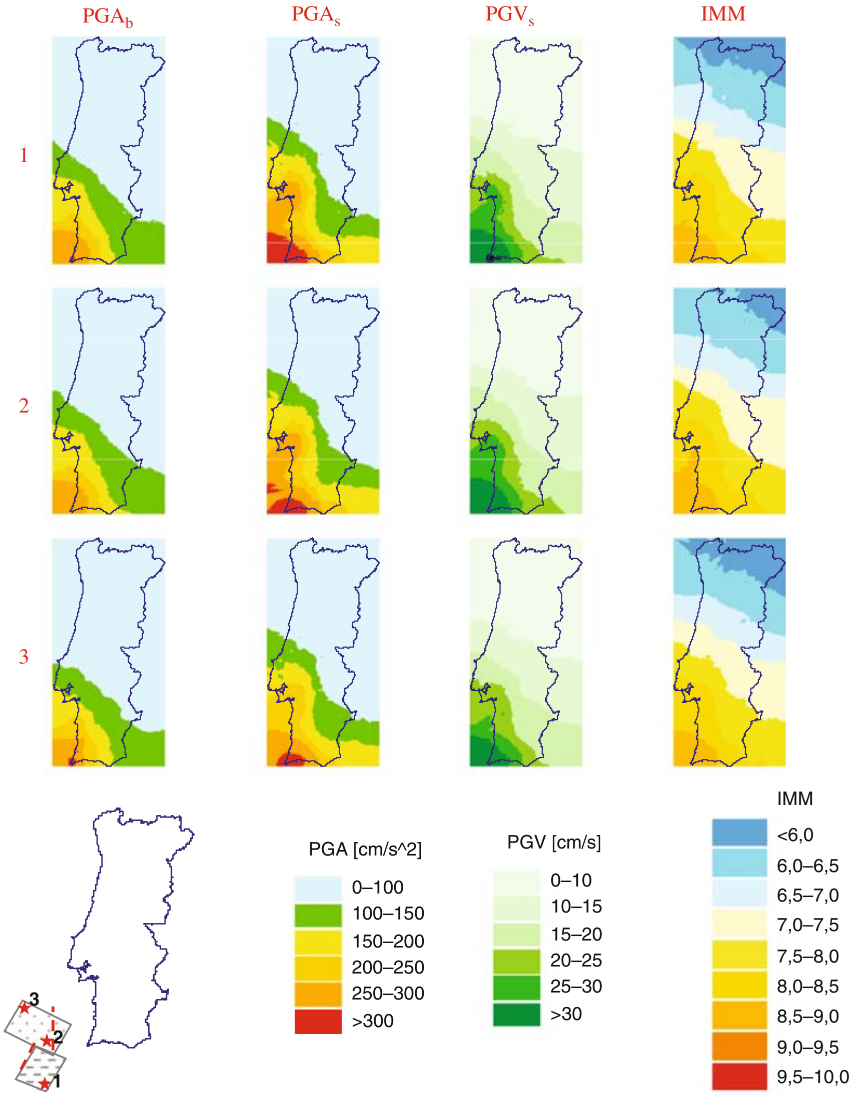


Fig. 7 Synthetic PGAb, PGAs, PGVs and isoseismal maps, for nucleation points 1–3, using the source model MPT-PSNF

offshore source and not by local source which means, considering PGV the ideal choice among the ground-motion parameters for damage direct relation, and as a consequence, that correlates best with intensity, that there is no need to look for a trigger solution to justify high intensities in Lisbon area.

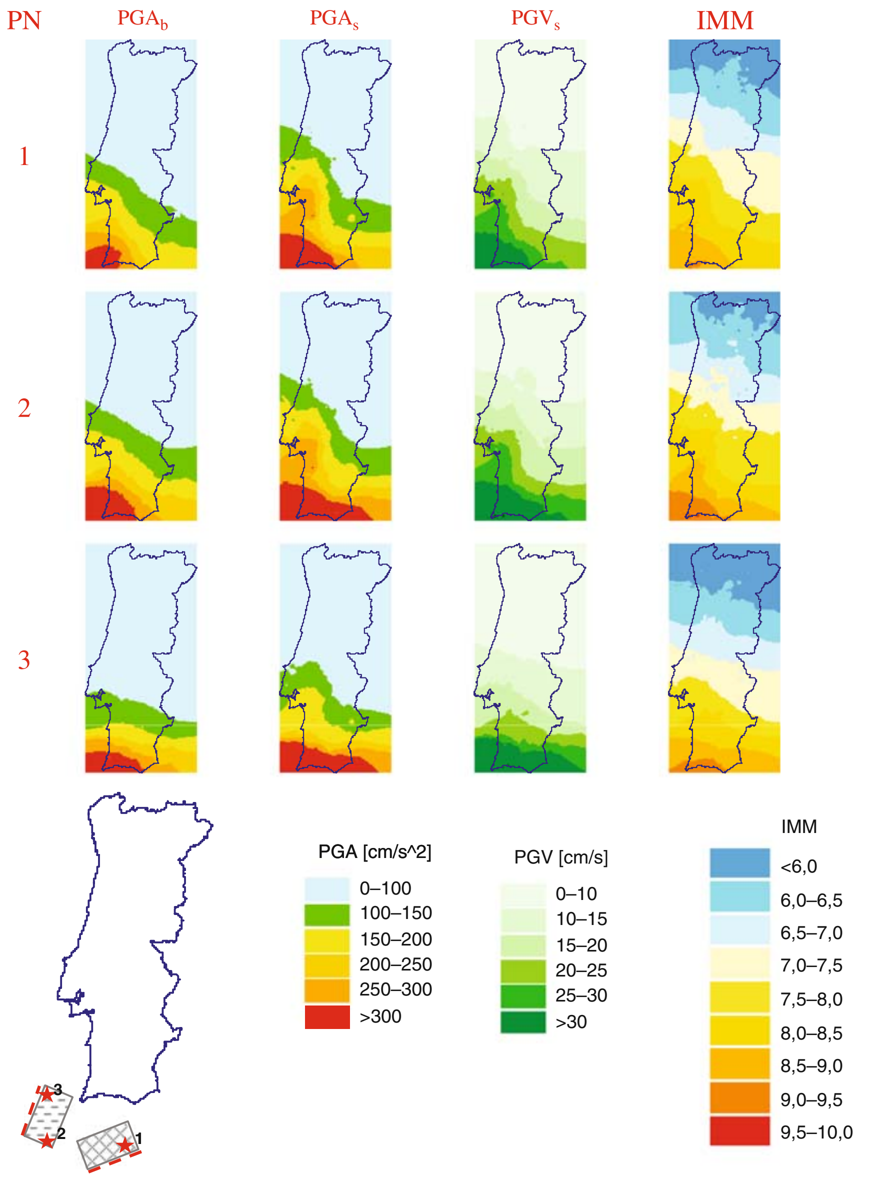
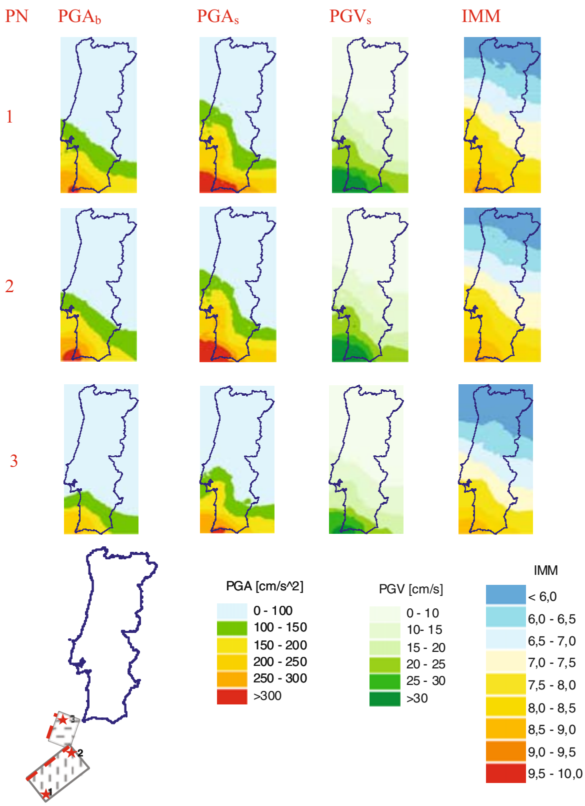
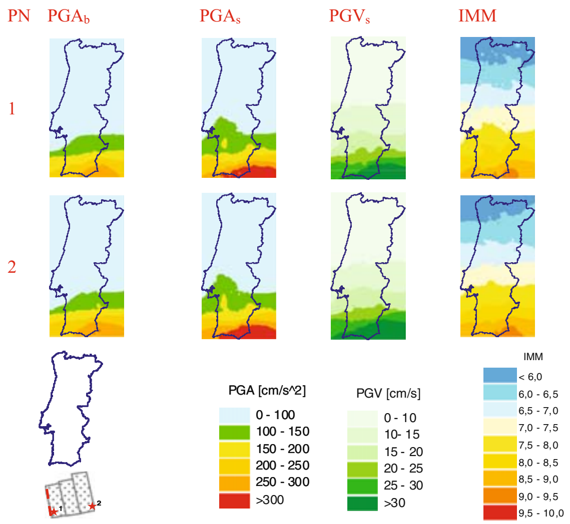


Fig. 8 Synthetic PGAb, PGAs, PGVs and isoseismal maps, for nucleation points 1–3, using the source model MPT-GqB

Fig. 9 Synthetic PGAb, PGAs, PGVs and isoseismal maps, for nucleation points 1–3, using the source model MPT-HF





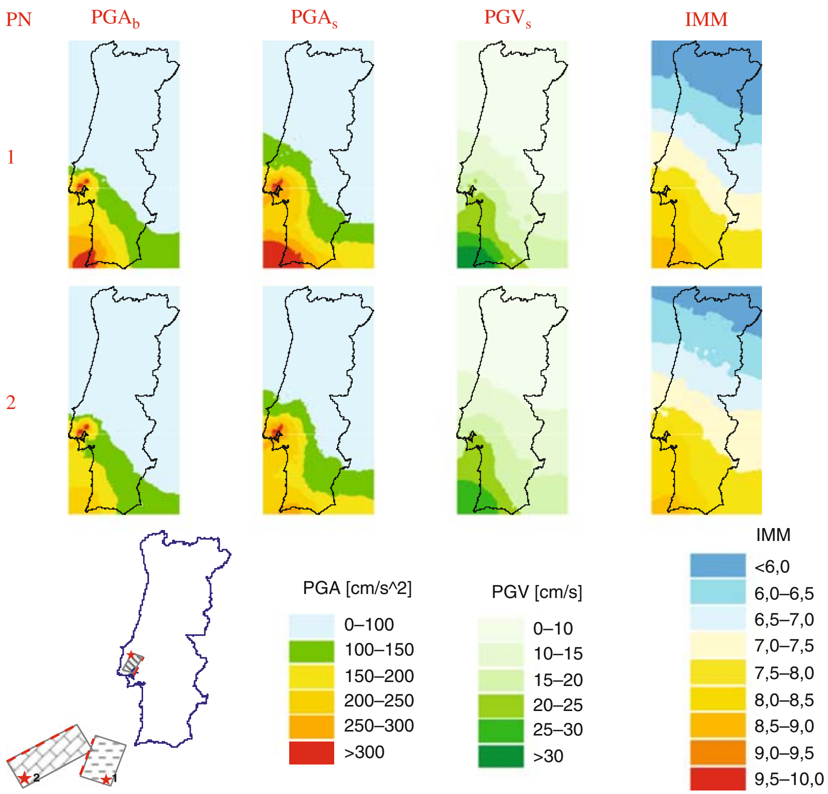


Fig. 10 Synthetic PGAb, PGAs, PGVs and isoseismal maps, for nucleation points 1 and 2, using UP: the source model GC; DOWN: the source model Off-LVT

6 Conclusions

A non – stationary stochastic method was applied to test four different models that have been taken as possible for the source of the 1755 earthquake. Source parameters of the model (corner frequency, stress drop) were estimated based on source spectra of S waves of 12 earthquakes records (corresponding to a total of 25 records) and the upper crustal attenuation parameter was estimated from the high-frequency decay of the acceleration spectra, after correcting observed spectra from geometrical spreading and anelastic attenuation effect.

Site effects were evaluated by means of an equivalent stochastic non-linear one-dimensional ground response analysis of stratified soil profile using the LNECloss system and using an available data base (herein DB1), containing information about soil classification for Portugal and a data base with stratified soil profile units for the Metropolitan Area of Lisbon (MAL).

Analyzing the most complete study performed here and its results we conclude that (i) the soil effects play a particularly important role in the pattern of isoseismals; (ii) the composite models that considers the Marques the Pombal fault and a additional source area (MPT-GqB, MPT-PSNF, MPT-HF) seems good hypothesis, considering an upward rupture, towards Lisbon City, the Guadalquivir Bank source model reproducing higher intensities in Algarve; (iii) The subduction zone in Gulf of Cádiz fails to reproduce the isoseismal features of the 1755 earthquake in western Portugal; (iv) The high intensities in Lisbon area, in Lusitania Basin and Lower Tagus valley Basin can be explained as site effects, together with a directivity effect (upward direction) from an offshore source, considering that PGV correlates best with intensity.

Other considerations, namely the possibility of each source to produce a tsunami, the time arrivals of tsunami, the geological evidences, the consequent damage and so on, are left to others.

Current methods of predicting ground motions for future earthquakes in Portugal will/should be based on an assumed seismological model of source and propagation processes.

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A Statistical Study of the Seismic Intensities of the 1755 Lisbon Earthquake

D.R. Brillinger and B.A. Bolt

1 Introduction

Substantial tragic effects result from great earthquakes – damage, deaths, tsunamis. Various groups, including seismologists, seismic engineers, government officials and insurers seek to quantify the effects in order to proceed with their work. The quantification methods employed include seismic intensity scales and damageability matrices particularly. Principal intensity scales employed are: the Modified Mercalli, (MMI), the Medvedev-Sponheuer-Karnik (MSK), and the European Macroseismic Scale. The scale values are typically denoted by roman numerals to reflect the fact that they are derived via verbal descriptions rather than some numerical physical measuring device. It is claimed that the MMI and MSK scales are similar, see e.g. (Sokolov et al. 1998).

Intensity scales are ordinal, that is the levels are qualitatively ordered and the level spacing does not matter. Adjacent categories can be merged. A probability approach is adopted and there has advantages. These include: one can examine scientific hypotheses formally, one can assess goodness of fit, one can compute and show uncertainty, one can compare alternate models, and there are often robust/resistant variants of general techniques. The broadly ranging subject matter of statistics becomes available. One is not meant to employ the intensity values using the rules of ordinary arithmetic. One purpose of this research is to examine the possibility of assessing formally if the data may be employed as if numerical-valued.

Isoseismals are often sketched on a map to indicate, generally, the seismic damage experienced. These isoseismals are meant to be contours of equal intensity, to bound areas within which the predominant intensity is the same. The lines prove useful to quantify the shaking pattern and to understand the damage. Traditionally isoseismal maps had been prepared by hand-drawing curves encompassing the observed intensities. The artist seeks to draw a curve

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Dedicated to Bruce Bolt, friend, colleague and collaborator.

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encircling, say all the *VIII* value locations, and if ignoring outlying *VIII*s. Professor Bolt once emphasized to this writer, (Brillinger 1993), a critical aspect of existing isoseismal maps, namely that they are conservative in two senses. First, the indicated intensity level at a location is the highest noted. Second, the isoseismals themselves are drawn as far out from the source as reasonable to include all locations with given intensity. However as (Reiter 1990) states, "... *drawing isoseismals can be a subjective process that may lead to different outcomes for different analyses*." and this provides a motivation for the present work.

(Perkins and Boatwright 1995) list some of the factors on which seismic intensities depend, namely, size of the earthquake, distance of the site from the earthquake source, the focusing of the earthquake energy and the regional and local geological effects. There is a falloff in severity of effect with distance from the source and substantial variability is inevitably present.

A prime objective of this work is to develop a statistical model involving intensities taking specific note of their ordinal character of the intensity scale data. It is anticipated that the model can be employed in probabilistic risk assessments. A principal assumption is that the dependence of the intensity on location is smooth. Related work was carried out for the 1989 Loma Prieta event in (Brillinger 1993), (Brillinger 1997), and (Brillinger et al. 2001), and for the 1994 Northridge event in (Brillinger 2003). Other researchers' papers include: (De Rubeis et al. 1992), (Pettenati et al. 1999), (Wald et al. 1999). The approach of this paper differs from that of the "other researchers" in that the ordinal nature of the MMI values is taken specific notice in an attempt to improve the results.

2 The Data

The concern is the Lisbon 1 November 1755 tragedy. It has been written about it as follows. This event has long held a place among the greatest in the modern world. It owes this distinction to the great destruction in Portugal, the deaths of over 60,000 people, an affected area of more than a million square miles, and the catastrophic sea wave. Actually, reports on the shaking indicate that there were three substantial, separate earthquakes within 3 hours. In the first, Lisbon was shrouded in thick dust, and the screams of the injured survivors added to the tragic scene. People in the city ran to any open space, particularly along the banks of the Tagus River. A British merchant ship was among the assembled shipping at the mouth of the Tagus. The captain described the first shock:

> I felt the ship have an uncommon motion, and could not help thinking she was aground, although sure of the depth of the water. As the motion increased, my amazement increased also, and as I was looking round to find out the meaning of the uncommon motion, I was immediately aquainted with the direful cause; when at the instant I looked toward the city, I beheld the tall and stately buildings tumbling down, with great cracks and noise.

Table 1 Observed MSK intensities and counts

II	*II+*	*III*	*III+*	*IV*	*IV+*	*V*	*V+*	*VI*	*VI+*	*VII*	*VII+*	*VIII*	*VIII+*	*IX*	*IX+*	*X*
5	5	11	27	128	114	104	48	111	59	84	27	29	17	13	11	17

Taken from (Bolt 2006, p. 1). That reference further goes on to say that one can speculate that the event was caused by a sudden thrust slip along the plate boundary running from the Mid-Atlantic Ridge near the Azores through the Strait of Gibralter. The earthquake's magnitude has been estimated as 8.7, its depth at 20–40 km and its epicentre at (–10.0, 36.5) a point about 90 km southwest of Sagres, the southwestern most point of Iberia. The data employed in the present work were provided by J. M. Miranda, who acknowledged Mezcua. There are 810 observations in Portugal and Spain.

There is discussion of seismic damage scales in (Bullen and Bolt 1985, pp. 433–437), and (Reiter 1990). A disadvantage resulting from the scales being concerned with damage is that damage can't occur at a given location if there is nothing there to be damaged. Other references using intensity values to understand the 1755 event include: (Mendes-Victor et al. 1999), (Baptista et al. 2003), (Martinez-Solares and Lopez-Arroyo 2004).

The counts of the numbers of the various MSK intensities recorded are provided in Table 1. There are intermediate levels, indicated by +, in the data set. Because such values are not part of the MSK scale they are not included in the analyses presented.

Figure 1 provides a histogram of the data values. One notices a lack of intensity+ values in some cases. For this reason, and because the official

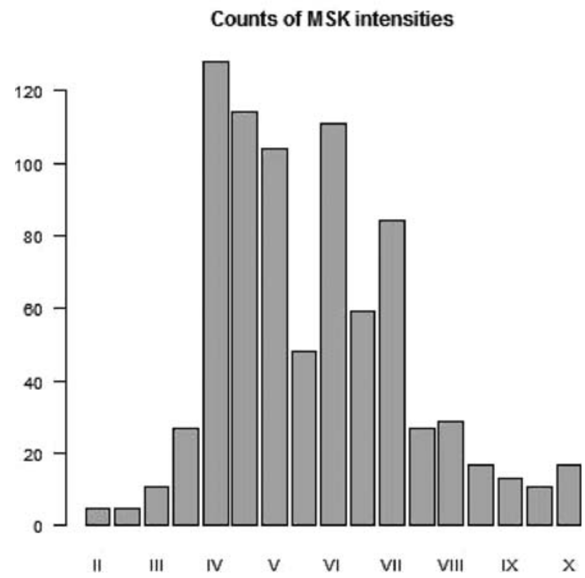


Fig. 1 Histogram of intensity values in the data set

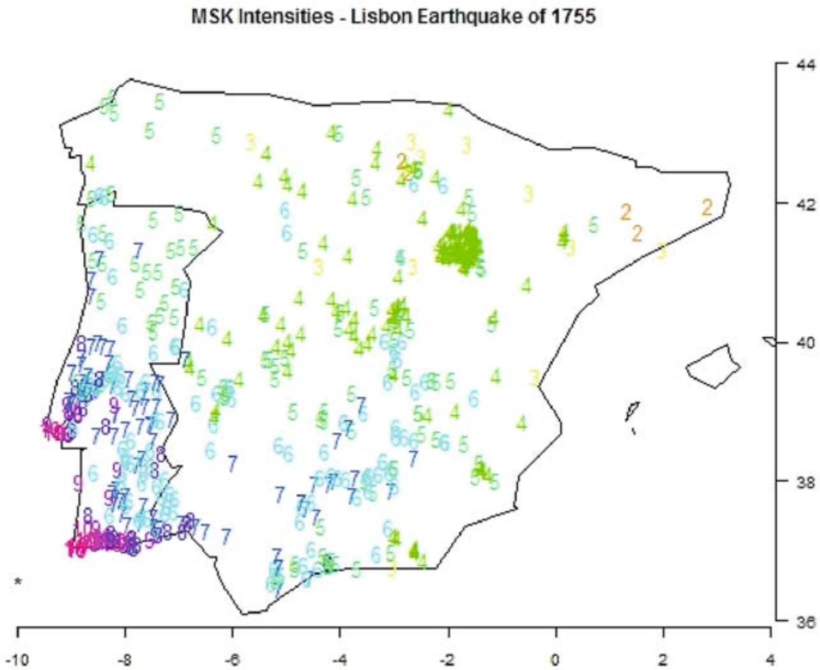


Fig. 2 Locations and recorded intensity values. The x- and y-axes are longitude and latitude respectively. The "*" is the estimated epicentre of the event

MSK scale is integer-valued only the integer-valued intensities are employed in the computations of the paper. One sees a modal value of *IV*. The histogram dips at *V*+ and *VI*+. They may be the result of statistical fluctuations or instead underassignment of the half values. The count at *X*, which stands out, is surely due to there being many damageable buildings close to the epicentre.

Figure 2 shows the locations of the measurements for the integer intensities. The clusters of values are associated with population centers. One sees a falloff from level *X* to level *II* as one moves north and east from the estimated epicentre denoted by "*" in the figure. It is located in the lower left corner and its coordinates have been taken from (Martinez-Solares and Lopez-Arroyo 2004). There is a lot of intermingling of different levels and overprinting.

3 The Statistical Methods Employed

A variety of statistical methods have been employed to develop the results of the paper. They are now described, in part. Statistical techniques have proven useful in addressing problems of insurance, risk management and seismic engineering, in particular those based on random process concepts. These include those of point processes for damage locations. A representation for a spatial point process is provided by

$$Y(x,y) = \sum\nolimits_j \delta(x - x_j, y - y_j) \tag{1}$$

with δ the Dirac delta functio. One for the so-called marked point process is

$$Y(x,y) = \sum\nolimits_j M_j \delta(x - x_j, y - y_j) \tag{2}$$

In the present case the marks, M_j, provide a measure of the severity of the event. The mark values are elements in the set {II, III, IV, . . ., X}. Both specific and general models have been developed for point and marked point processes and these processes are basic to probabilistic seismic risk assessment, (Ogata 1983), (Vere-Jones 1992), (Schoenberg and Bolt 2000).

For ordinal data the grouped continuous model, (McCullagh and Nelder 1989), (Agresti 1996) is effective. It involves, a latent (or state) random variable, ζ and cutpoints θ_j. It leads to representing intensity data values, Y, as

$$\begin{aligned} Y &= II \text{ if } \zeta \le \theta_{II} \\ &= j \text{ if } \theta_{j-1} < \zeta \le \theta_j \text{ if } j = II, III, \ldots, IX \\ &= X \text{ if } \theta_{IX} \le \zeta \end{aligned} \tag{3}$$

for $j = 1,\ldots,J$ with $\theta_0 = -\infty$, $\theta_0 = \infty$. The θ_j are to be increasing. There are J cells.

An important advantage of this model is that an explanatory variable **X** may be introduced directly by setting

$$\zeta = -\beta' X + \varepsilon \tag{4}$$

with β a coefficient to be determined from the data.

If one assumes that ε has an extreme value distribution, i.e. $\mathrm{Prob}\{\varepsilon < a\} = 1\text{-}\exp(\text{-}e^a\}$, then

$$\mathrm{Prob}\{Y = j|\mathbf{X}\} = \exp\{-\exp\{\theta_{j-1} + \beta' X\} - \exp\{-\exp\{\theta_j + \beta'\mathbf{X}\}\} \tag{5}$$

The use of an extreme value distribution may be motivated by the character of the situation. Its reasonableness may be checked empirically, see Fig. 3 below. For the model (5) the β's and the θ's may be obtained using functions in standard statistical programs. (In the work of this paper the statistical package R was employed, (Venables and Ripley 2002 and Wood 2006). To do so one represents the likelihood of the data as a product of binomial likelihoods, see page 170 in (McCullagh and Nelder 1989). In the computations reported a modified form of the R function logitreg of (Venables and Ripley 2002), the

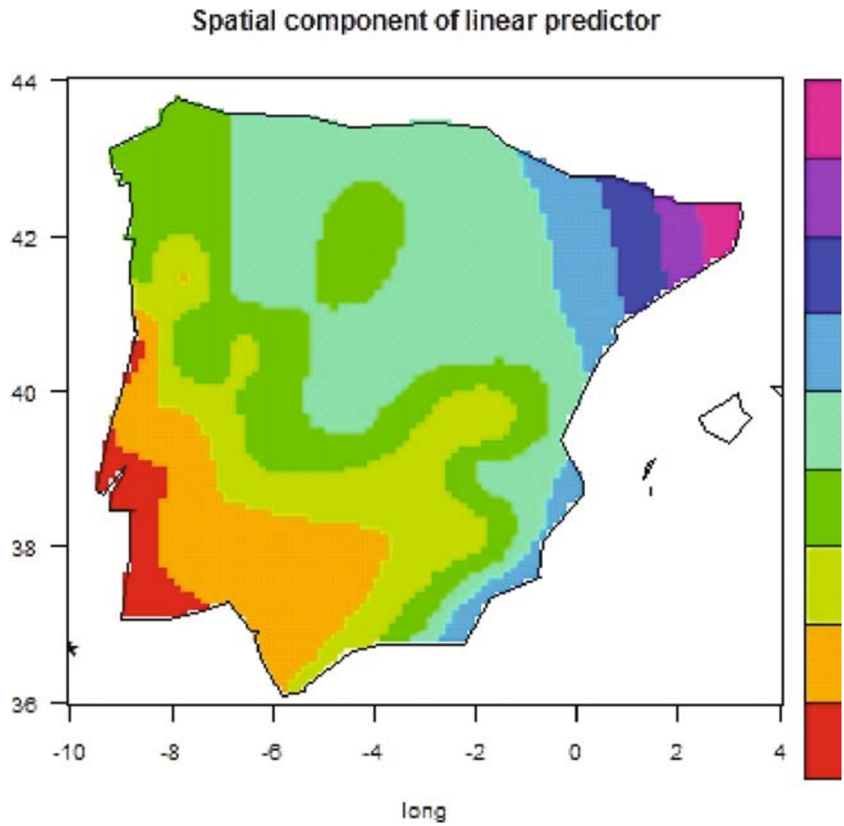


Fig. 3 The estimated spatial effect β(x,y) of the model (2.5.), (2.8.). The colors at the top of the legend correspond to larger values

clolog link and the binomial distribution were used. In this extreme value case an estimate of E{ζ|X} is provided by

$$-\,\mathrm{b}'\mathrm{X} + \gamma \tag{6}$$

where b is the estimate of β and γ is Euler's constant. In the spatial case at hand one takes

$$\beta'\mathrm{X} = \beta(\mathrm{x}, \mathrm{y}) \tag{7}$$

with j intensity, x longitude and y latitude and β(x,y) smooth. The function β(x,y) may be interpreted as a proxy for variables, such as geology, left out of the model. From expression (5) one has

$$\mathrm{Prob}\{\mathrm{Y}(\mathrm{x}, \mathrm{y}) \le \mathrm{j}|(\mathrm{x}, \mathrm{y})\} = 1 - \exp\{-\exp\{\theta_{\mathrm{j}} + \beta(\mathrm{x}, \mathrm{y})\}\} \tag{8}$$

One notes that this increases with θ and β.

In summary the statistical model to be employed is

$$\text{Prob}\{Y(x, y) = j|(x, y)\} = \pi_j(x, y|\theta, \beta) \tag{9}$$

with $\pi_j(x,y)$ of parametric form and given by expression (5) above. With the assumptions indicated the model forms a so-called generalized linear model (glm) (Wood 2006), and various inference procedures are available. In the results presented $\beta(x,y)$ will be approximated by a thin-plate spline.

Statistical concepts and techniques of R include ones for the estimation of parameters, for model validation, and for uncertainty computation. The estimation methods include maximum likelihood and parametric and nonparametric fitting. In fact there are several general methods that may be employed to evaluate the uncertainty associated with the estimates. These include: linearization, the jacknife and the bootstrap. Perhaps the easiest to employ here is the jackknife. It involves temporarily deleting data points in groups, computing the estimates for the remaining data points and then combining these values, (Mosteller and Tukey 1977).

4 Results

Consideration turns to fitting the grouped continuous model with the distribution (5), (8). The estimate used employs the thin-plate spline approximation to $\beta(x,y)$, namely

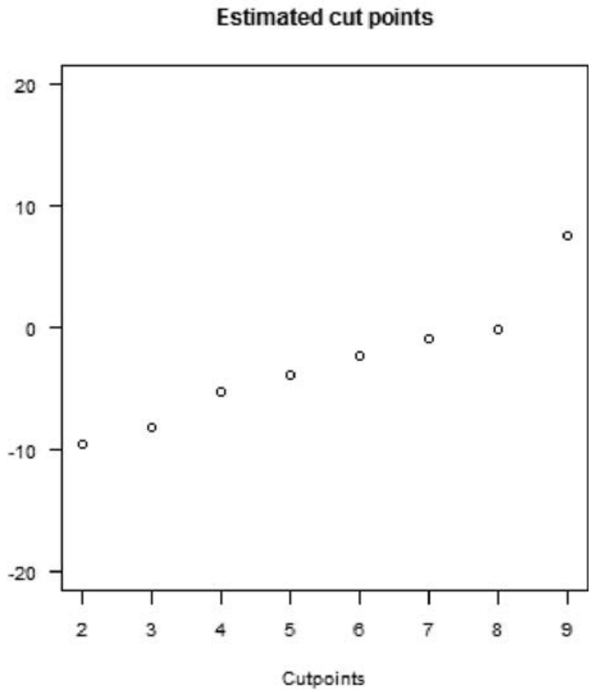
$$\sum_k \beta_k r_k{}^2 \log r_k \tag{10}$$

where for nodes (x_k, y_k) the variable $r_k{}^2 = (x-x_k)^2 + (y-y_k)^2$ and the β_k are parameters to be estimated. The (x_k, y_k) were taken to lie in a grid covering the peninsula. The expression (10) has the form $\beta'\mathbf{X}$ and maximum likelihood estimation may be employed.

Figure 4 is the estimate of the linear predictor component $\beta(x,y)$. Its interpretation is a background representing a smooth regional effect, in the presence of the cutpoint terms. The breakpoints for the color legend have been taken as uniformly spaced across the range of values of the estimate. One sees the estimate to tilt up from the lower left to the upper right. In interpretations of the result one needs to remember that the θ's are also in the model and that possibly there is an interaction between level and location.

Figure 5 provides the estimates of the θ_j of model (5), (8). Approximate ± 2 s.e. limits are indicated by the vertical lines. The estimated θ_j are seen to be increasing steadily, approximately linearly. The estimate of θ_{IX} is highly variable.

Fig. 4 The estimated θj. Approximate ±2 standard error limits have been added



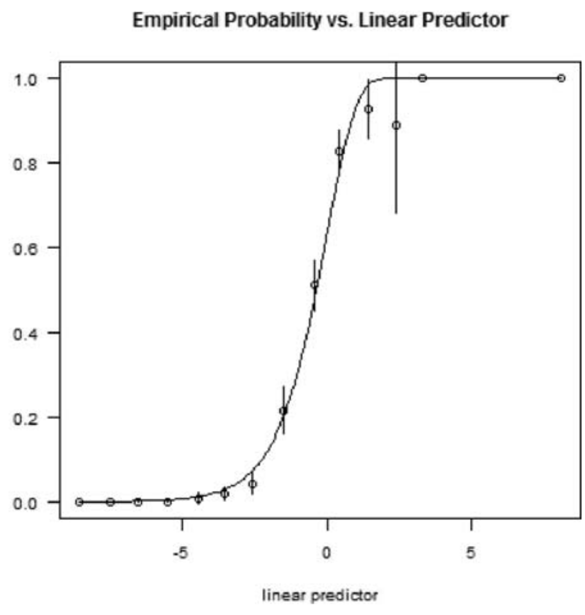


Fig. 5 Empirical Probabilities and the extreme value distribution function

Sometimes MMI values are treated as if they were numerical, for example being used as the dependent values in least squares analyses such as the model

$$Y = \beta(x, y) + \varepsilon \tag{11}$$

where Y is the intensity in arabic numerals, ε is noise and β(x,y) is smooth, instead of (5), (8).

It is necessary to assess the goodness of fit on any statistical model. Figure 6 provides the results of one study of the model (5). Having picked cells for the fitted linear predictor, $\theta_j + \beta(x,y)$, one plots the proportion of cases in a given cell versus the cell's midpoint. The continuous curve is the cumulative distribution function of the extreme value distribution. Also ±2 standard error limits have been added to the proportions. The fit seems reasonable for this method of assessment.

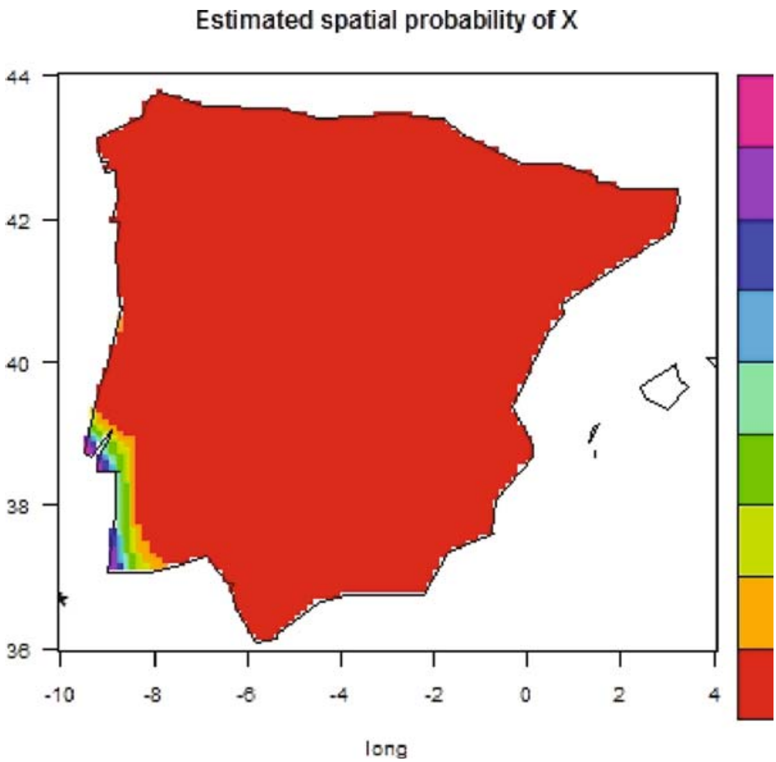


Fig. 6 Estimated probability for MSK intensity X as a function of location. The legend on the right goes from 0 at the bottom to 1 at the top

5 Uses of the Fitted Model

Once one has a specific stochastic model there are a variety of things that can be done.

For a given location one can now estimate the probabilities of the various MSK intensities occurring employing expression (5). Figures 6, 7, and 8 provide estimates for the particular cases of intensities *X*, *VII* and *II* respectively.

Unsurprisingly the estimate of the probability of intensity *X* is noteable only in the southwest corner of the figure, the region closest to the hypercenter.

Next is the figure for intensity *VII*. The noteable intensity *VII* probabilities are spread out in the southwest region of the map, but not at the southwest tip. This can be seen for the observed values in Fig. 5.

Lastly, Fig. 8 provides an estimate of Prob{I = *III* | (x,y)} is plotted. The values are near 1 in the northeast corner but not at the northeast tip. This is consistent with Fig. 4 only showing *II*s in that corner.

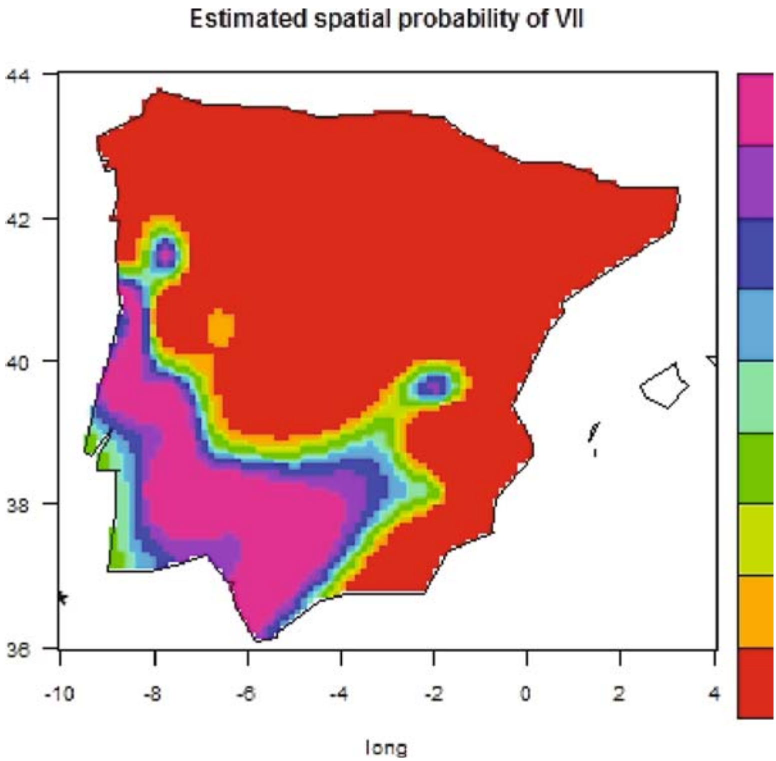


Fig. 7 Estimated probabilities for intensity VII

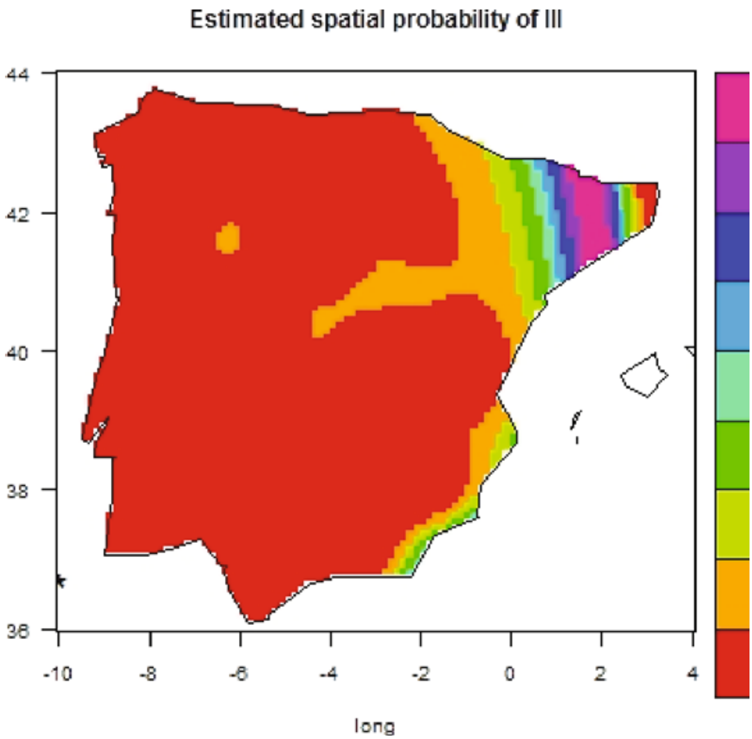


Fig. 8 Estimated probabilities for intensity II

There are other uses for fitted statistical models. Consider how the MSK intensity falls off with the distance of a location from the hypercenter of the event. Figure 9 is a scatter plot of intensity against the logarithm of the distance. One sees a general falloff in the intensity level as the distance increases with a great range in variability for any specific level.

As estimates of falloff in risk with distance from source are important in seismic engineering problems it is worth developing a specific model. An example of the development of estimates for ordinal data is provided in (Brillinger 1997), and (Brillinger 2003).

Sometimes it is more convenient to work with distance d than (x,y). (Bolt 2006). For the Lisbon 1755 data the following model will be considered,

$$\mathrm{Prob}\{Y = j|d\} = \exp\{-\exp\{\theta_{j-1} + \gamma(d)\}\} - \exp\{-\exp\{\theta_j \gamma(d)\}\} = \pi_j(d) \quad (12)$$

for $j = II, III, \ldots, X$ with d distance and γ, assumed smooth, expressed as a B-spline, (Venables and Ripley 2002).

Figure 10 provides the results for the 1755 event and the particular cases of intensities *X*, *VII*, and *III*. In the case of intensity *X* one sees concentration of probability in the region on the land closest to the event. The other

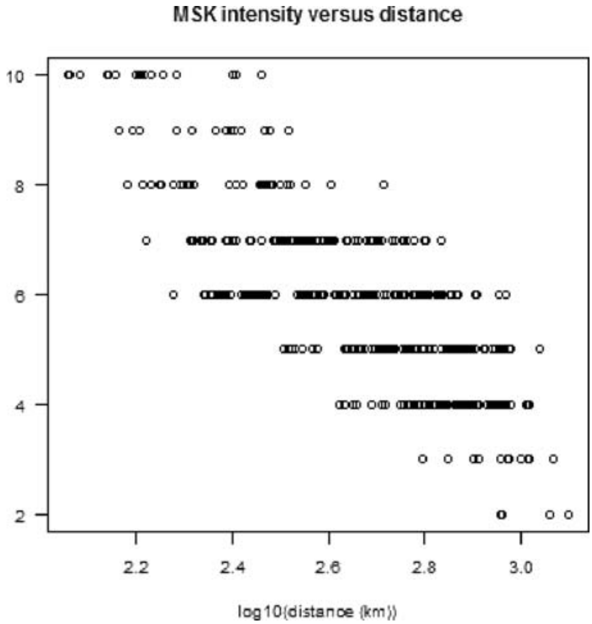


Fig. 9 Intensity values versus distance from the source

probabilities are largest at intermediate distances across the peninsula. This was apparent in Figure 6. The closest point on the land is approximately 114 km from the hypercenter. The intensity *VII* probabilities peak around

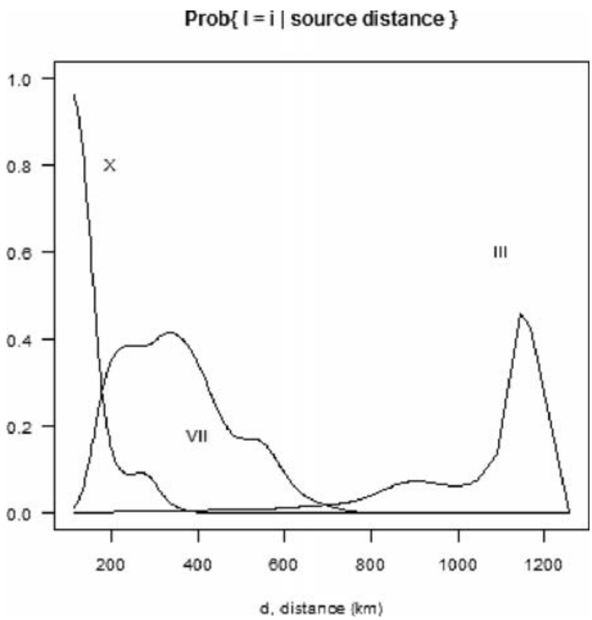


Fig. 10 Probability of a given intensity occurring as a function of distance from the estimated hypercenter for intensities III, VII and X

350 km and the intensity *III* curve quickly rises and then drops around 1100 km. All told the results are consistent with Fig. 5. At a given distance, the probabilities of intensities *II* through *X* sum to 1.

6 Other Derived Values

Having a statistical model one may compute statistical properties and displays for other quantities of interest. As an example consider the maximum acceleration. Empirical relations have been derived relating it to intensity.

As an example (Bolt 2006, Appendix C) provides some characteristic peak velocity and acceleration values as follows

The figures of Table 2 may be put together with the fitted probability model to obtain an estimated distribution for the maximum acceleration as a function of distance. Specifically, suppose intensities II and III are neglected. Then let a_j denote a characteristic value for the j-th row in the table, $j = J$, with *X* standing for *X-XII*. Let $Y = (Y_1,\ldots,Y_J)$ denote a multinomial variate with j-th probability, $\pi_j(d)$, at (9) above. The acceleration at distance d may be approximated by

$$A(d) = \sum\nolimits_j a_j Y_j \tag{13}$$

and distributional properties determined from those of the multinomial. For example the expected acceleration at distance d is approximated by

$$\sum\nolimits_j a_j \pi_j(d) \tag{14}$$

Another quantity is percent risk/damage or damageability matrix, (Munich Re 1991) providing loss ratios (Table 3) for three classes of buildings as a percent. (vulnerability)

Table 2 Peak velocities and accelerations associated with given MMIs

MMI	*IV*	*V*	*VI*	*VII*	*VIII*	*IX*	*X-XII*
vel (cm/sec)	1–2	2–5	5–8	8–12	20–30	45–66	>60
accel (g)	.015–.02	.03–.04	.06–.07	.10–.15	.25–.30	.50–.55	>.60

Table 3 Percentage losses. Taken taken from (Munich Re, 1991)

MMI	*VI*	*VII*	*VIII*	*IX*	*X*
residential	.4	1.7	6	17	42
commercial	.8	3	11	27	60
industrial	.1	.7	3	11	30

An expression like (12) above may be employed to evaluate probabilities associated with loss percentages.

7 Discussion and Conclusions

For many years isoseismals have been produced by hand. This paper presents an objective statistical approach to evaluating related quantities, specifically probabilities that a particular intensity value occurs at a particular location. As well as being less subjective it is computerized thus allowing rapid production of figures. The approach makes specific use of the information that the intensity scale is ordinal. These may be fed into estimations of quantities such as of loss ratios, occurring in later stages of risk analyses

The study has limitations. Bias may be mentioned. This would occur if for a case of damage the probability that it would go unrecorded depended on the location (x,y). (However if that probability could be estimated then the bias could be corrected for.) Next one can remark that the methods employed were based on assumed models. These may not hold. In particular the extreme value distribution had particular computational convenience, but others may prove useful. The smoothing methods involved tuning parameters, which need to be chosen. However the greatest limitation is not including other explanatory variables in the model. It was hoped to have site conditions, and geology and Professor Bolt was working on this when he died.

Acknowledgments Bruce Bolt was a very strong proponent of the use of statistical methods in seismology and seismic engineering. He introduced me to many of the important concepts of those fields. It is hard to find words of gratitude for his having shared his knowledge with me. This paper would have been so much better had he not died so suddenly.

Bob Wiegel, another Berkeley colleague, helped me to understand some of the research concerned with locating the source of the earthquake. J. M. Martinez-Solares, and J. M. Miranda provided the intensity data employed in the study. I thank them also.

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Part VIII
Global Response to Large Earthquakes

Eyewitness Reports of the 2004 Indian Ocean Tsunami from Sri Lanka, Thailand and Indonesia

Robin Spence, Jason Palmer and Regan Potangaroa

1 Introduction

The Indian Ocean Earthquake of 26.12.04 (Table 1) occurred at 00.58.53 GMT with its epicentre some 250 km off the west coast of Sumatra. Its magnitude (Mw) was 9.3, making it the second-largest instrumentally-recorded event in history. The event was triggered by a rupture in the Sunda Trench subduction zone over a length of about 1300 km causing a seabed uplift of at least 5 m over much of this length. Ground shaking from the earthquake was felt over a wide area, and was destructive in several towns of northern Sumatra (especially Banda Aceh), and in the Nicobar Islands. But it was the tsunami triggered by the seabed uplift which was most destructive, causing huge damage on the coasts of Sumatra and adjacent Indonesian islands, of Thailand, Sri Lanka and India, and affecting every other country bordering the Indian Ocean. In terms of loss of life the event was one of the worst of the last century: the Boxing Day 2004 earthquake and tsunami are now thought to have been responsible for around 230,000 deaths. In terms of economic loss and loss of livelihood it was also catastrophic, devastating fishing communities along the coast and also seriously impacting developing tourism (RMS, 2005).

Partly because of its international impact, and the very large number of European, North American and Japanese visitors who witnessed the event, there was a huge and sustained public and press attention to the event all over the world, which led to an unprecedented level of fund- raising both on national and individual levels, and to a sustained humanitarian action programme for emergency relief and long-term reconstruction which is still in progress, and will, in some areas, take many years to complete (UNDP, 2005). In addition to the humanitarian effort, there were numerous reconnaissance missions by specialist teams from scientific technical and insurance institutions across the world (EEFIT, 2006; EERI, 2006).

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Table 1 The 26.12.04 tsunami: summary data (from www.eeri.org)

Region and Country	Off the West Coast of Northern Sumatra
Moment Magnitude	9.3
Date	December 26, 2004
Time	00:58:53 (UTC)
Epicentre	3.267°N, 95.821°E
Fault source	Subduction zone between the India plate and the Burma microplate
Faulting mechanism	Megathrust faulting with vertical slip
Affected countries	Indonesia, Sri Lanka, India, Thailand, Somalia, Maldives, Malaysia, Myanmar, Tanzania, Seychelles, Bangladesh, Kenya, Singapore, Madagascar, Mauritius, South Africa, Mozambique, Australia, Antartica
Total casualties	283,100
Casualties by country	Indonesia (108,100 with 127,700 missing presumed dead), Sri Lanka (30,900), India (10,700), Thailand (5,300), Somalia (150), Maldives (82), Malaysia (68), Myanmar (59)
Displaced	1,126,900

The event has had a very significant impact on the awareness of the tsunami risk internationally, and much has been done since December 2004 to create new tsunami-warning systems and improve existing ones for tsunami-prone coasts worldwide, as well as to improve public awareness (EERI, 2006). The experience of 1755 shows that European coasts and especially the west and south coasts of Portugal are at risk of tsunamis resulting from any future large earthquake on the Marquis of Pombal fault (Baptista and Miranda, 2005). The recurrence of major events in southern Europe is so infrequent that public awareness and understanding of the phenomenon is very limited; and as in South Asia, a hugely increased population today lives, works, or takes holidays in the area, by comparison with the population at the time of the last event. Lessons from the 2004 South Asian event are therefore of vital importance to enable a proper disaster management response to be put in place to cope with the next European tsunami.

One of the unique aspects of the 2004 South Asian tsunami was the large number of UK-based eyewitnesses, (probably some 2000) and the authors considered that assembling a sample of their eyewitness reports could potentially add greatly to the damage information assembled by the post-event reconnaissance teams. Particular contributions from eyewitness reports are: accounts of the sequence of events, descriptions of the mechanisms of damage, accounts of individual behaviour and escape stories, and information on the levels of warning and awareness among visitors. Many proved to have moving stories, and some have also provided contemporary videos and photos which add greatly to the understanding of the hydrodynamics of the event in particular locations.

In an initial study 75 eyewitness reports, mainly from UK tourists, were obtained and collated. A summary and analysis of these reports is published elsewhere (Spence et al., 2007). A follow-up study, using the same questionnaire, was carried out in northern Sumatra, during June and July of 2005; the 388 respondents in this survey were largely local residents. The map in Fig. 1 shows the locations, by country, of these initial eyewitness reports.

This paper will summarise what has been learned from these surveys about the response of individuals who were caught in the disaster, but escaped; about the extent to which they received warnings or had prior knowledge about tsunamis; whether they felt the earthquake; how they experienced the tsunami; about their injuries; if they were in buildings, about the performance of the buildings; about their means of escape; and about their overall assessment of the experience. These are topics beyond what can be learnt from post-event reconnaissance reports, and to that extent the information here supplements those reports. The eyewitness reports also contribute to a historical archive of the event which will be of value to all future research in this field.

In this paper we first describe the survey (Section 2); we then summarise in separate sections the information from the eyewitness reports in Sri Lanka and Thailand (Section 3), and those in Indonesia (Section 4). Cross-comparisons and conclusions are presented in Sections 5 and 6.

2 The Survey

At the outset of the project we anticipated that eyewitnesses would be able to provide data about:

- The height, time sequence, succession of peaks and troughs of the incoming wave
- The speed and extent of wave run-up on land
- Maximum depths of inundation
- Physical damage to structures and likely cause
- Causes of death and nature and extent of injury to survivors
- Aspects of human behaviour in response to the tsunami impact

The survey questionnaire (Spence et al., 2007) introduced the research aims and sought informed consent. The lead questions were open-ended and ordered to give survivors the opportunity to tell their stories to an empathetic and active listener. This meant preparing interviewers to focus on the human story, before asking specific questions about the tsunami and the buildings.

The UK interviewing team identified, contacted and interviewed or examined written descriptions from 87 people who witnessed the tsunami first hand, representing about 400 individuals who were members of their party or family with them at the time. About 40 of the eyewitnesses were interviewed by telephone or in person (with the remainder coming from reliable written sources or self-reporting, some via the Internet).

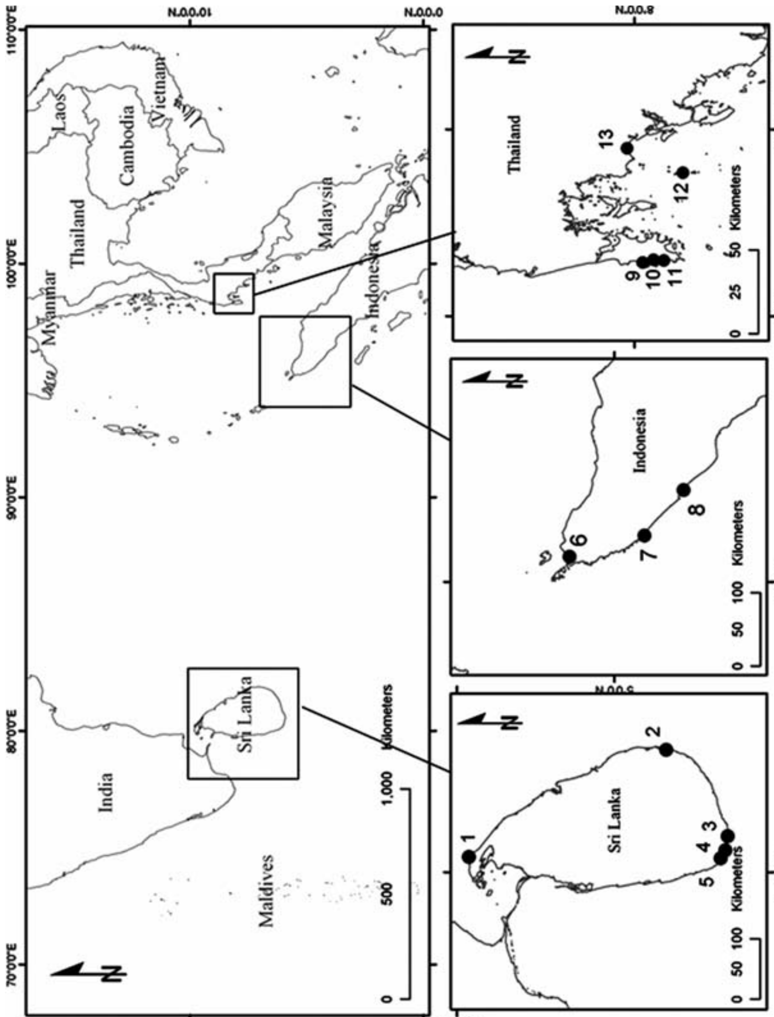

Fig. 1 Locations of eyewitness surveys

The Indonesian research team firstly translated the survey questionnaire into Bahasa Indonesia, the official language of Indonesia. Other earlier surveys had indicated that most interviewees would be illiterate and would most probably converse in Acehanese rather than in Bahasa but that most interviewers would be more confident writers/recorders in Bahasa. Interviewers would approach people staying in the temporary camps in either Banda Aceh or Calang and orally go through the survey questionnaire in Acehanese but record responses in Bahasa. These written transcripts were later translated back into English and scanned to form a digital record. Prior to the survey questionnaire there were plans to use digital voice recorders and record full unstructured accounts that would have been more direct. However, the opportunity to use a more structured approach and to link to research in other countries seemed sensible. Interviewers were postgraduate law students from Syiah Kuala University in Banda Aceh organised by a member of staff who was personally affected by the tsunami. She standardised the interviewing objectives and process of the Indonesian interviewer group with the final translation back into English being completed by Indonesian students in New Zealand.

Figure 1 shows the areas where eyewitnesses observed the tsunami, and Table 2 summarises the number of interviews in each country.

A comprehensive database framework was established using FileMaker, structured around the interview questions. The database is fully searchable by region, location, effect on building, and various aspects of the interviewees' experience. It may also be sorted to reflect particular items of interest, such as building type, building material, or health impacts of the tsunami. The database records detailed descriptions of how the tsunami appeared in the area where interviewees were staying (wave height, speed, any sequence of waves, how the wave broke). It further records qualitative descriptions of how buildings were affected by the tsunami, whether and how people were injured, and the effects on local infrastructure (electricity supply, communications, transport, etc.).

In addition, the project has built a video and photo archive of images taken by our interviewees, and those available on websites, containing several hundred photos and about 30 video clips; and has assembled and reviewed damage descriptions and reports written by other research groups worldwide. There are some obvious limitations to this dataset as a representative sample of those affected. First, it is of course an extremely small sample, and also covers a small

Table 2 Number of eyewitnesses interviewed in each country

Country	Number of eyewitnesses
Indonesia	398
Thailand	35
Sri Lanka	24
Malaysia	3
Maldives	6
India	2
Australia	1

number of the locations affected by the tsumani. Also, as we could only interview survivors, we tend to have more reports from the better building types, and from those further from the shore.

The interviews assembled two different kinds of information, descriptive and quantitative. In many ways the descriptive accounts provide the best evidence of what occurred and what was observed by the eyewitnesses, particularly at the onset of the tsunami; but this information is difficult to summarise or analyse. Accordingly, in the following sections, where possible, a record of what was experienced is provided first through the use of passages from the verbatim accounts; and this is followed by a summary of the more quantitative data in the form of statistics or tables.

3 Eyewitness Accounts from Sri Lanka and Thailand

Most of the eyewitnesses from Sri Lanka and Thailand were tourists staying in coastal resorts. In these areas the earthquake was hardly felt if at all, and the first knowledge of the tsunami was when it arrived at the beach at around 9 to 9.30 on 26th December. There are concentrations of reports from two main areas – Southern and Eastern Sri Lanka, and Southern Thailand (Phuket and neighbouring islands, and Khao Lak).

In Sri Lanka the eyewitnesses were staying either on the South Coast between Galle and Matera, a distance of 50–75 km, or on the East coast, between Pont Pedro and Pottuvil. The first wave of the tsunami struck this section of coastline at about 9.20 am, local time, i.e. about 100 minutes after the earthquake. At this time, on Boxing Day, many of the visitors were still asleep, and were woken by shouting or when the water entered their rooms. None of them reported feeling the earthquake.

The experience in most of this area was a very rapid rise in the water level, not preceded by sea retreat. A more detailed account from Koggala speaks of three closely spaced waves in the first sequence:

> There were three waves which struck at around 9.20 am. There was not much lapse of time between each wave – about two or three minutes. It seemed to happen very quickly and with the sea rising up the beach at a very fast pace. The first two waves were not high curved waves ... but more swells of water (huge volumes) that moved very quickly. The third was a big curved wave (about 12 feet high), which crashed into the first floor of the hotel. [Koggala]

There is some consistency in the reports of the depth to which the water rose inside buildings; it is clear that it reached head height in many of the ground floor rooms.

In Thailand there were 17 eyewitness accounts from the Phuket area, nearly all of whom were tourists, 11 eyewitness accounts from Phi Phi and neighbouring islands all but one of whom were international visitors, and 7 reports from the Khao Lak area, two of whom were offshore at the time, and all of whom were tourists.

All witnesses reported that the sea receded before the tsunami.

> I saw what looked like a giant wave spanning the horizon, around four miles offshore. I just assumed it must be an optical illusion. [Then] I saw the entire bay suddenly drain of water with a quiet roar. The bay is around two miles wide, and the water rushed out to a distance of around one mile offshore.

At Karon Beach, there was an initial wave, followed by a "swell" of water:

> The initial large wave slap[ped] into the concrete buildings as the wave successively reached different points up the beach. Then I saw a swell of floodwater heading our way--- perhaps just a 3-4 foot deep plateau of water with a roiling mass of water and debris at the leading edge.

At Kamala Beach there was an early 'warning wave', followed by much more water and a much more powerful surge – strong enough to carry people along:

> The seawater was up to our ankles and we realised it was still coming in… then I saw much more water coming in… By now the water was knee deep, …[it] poured down over the steps. By the time I got up the water was at chest height. Then the water surged much stronger.

The highest estimate of wave height around Phuket was at Patong Beach, where one witness said the first wave was "about 3 m (above [their] head)". This person said that a second wave, four or five minutes later, was "a lot higher", and a third wave, five minutes after that, was "between" the heights of the first and second waves. The single eyewitness on the east coast of the island saw three powerful "walls" of water coming towards the beach.

The number of waves reported varied between sites – from a single wave on Ko Phi Phi to as many as 15 in Krabi, where an eyewitness said they were "powerful" if not high, and successive waves carried more water inland. Even on the same island, there were some differences between reports, as on Phi Phi where some observers saw one wave, while others saw three.

> [The sea came in again in about 2 minutes]. The Thai crew took the boat out to sea as the tide rose. But it kept rising - maybe 20-25 ft. We all grabbed a child and ran up the very narrow beach to the trees. Then we climbed up the trees away from the water. [Phi Phi]

The initial reaction was one of curiosity for most observers, but this changed to fear when they realised a large body of water was about to hit the beach. Most people were able to run to the safety of higher land, although one group on Phi Phi decided instead to find refuge in a bungalow, which subsequently collapsed on top of them.

> I saw the water rolling in, carrying everything with it. I was struck by how dirty it was and how quick it came in - faster than you could run. It came above first floor level - from the mark left on buildings afterwards it rose to about 10 ft. [Phi Phi]

Of the five onshore eyewitnesses at Khao Lak, three were close to the beach and all mention the sea retreating as their first indication. All those on land give graphic accounts of the first arrival of the tsunami. Two saw it from the beach, but the others were indoors at the time, and they first encountered the water as it surged towards them across the beach:

> A 'white line' appeared on the horizon around 10:05, there was no sense of danger at this point, the boats (which must have been quite big) were moving towards the beach and then with a roaring noise the waves suddenly appeared. About 3 seconds [later], water came gushing in, and we had to cling on to the pillars of the bar not to be swept away. (Southsea Pakarang Resort)

Information on the sequence of waves or surges is scarce, partly because most of these witnesses were struggling for survival after this point; in one account "there was no sequence of waves evident from where we were, only surges of water building up on the one before" but in one account:

> There were two big waves, the two arrived 20 minutes apart in quick succession. The second one was 20 cm lower than the first one. The second one was more frightening. The last big wave we observed was at around 1:00 pm and was 9–10 m high. In between, there were many smaller waves, and none of these broke. (Southsea pakarang Resort)

The maximum height reached by the water varies somewhat in the different accounts. One account from a single-storey bungalow a little way back from the shoreline speaks of water reaching within 10 cm of the ceiling. In another account, the survivor escapes to the second storey roof and jumps into the water as it comes out of the (presumably first floor) window. In the beach restaurant of a hotel, the water reached the first floor bar. One family had run to their hotel, some way back from the beach, and had clambered on to a 2.4 m wall and remained there until the water receded.

The water seems to have reached its maximum height in a matter of a few minutes from its arrival, and some accounts suggest that it began to recede after another 3–4 minutes. One account speaks of the force of the retreat:

> as the water receded, it started to suck everything out of the room, like the air out of an airplane if a window were to get shot out. (Wanaburee)

In addition to those from the four regions described above, there are accounts from the Maldives, South India, and Malaysia; accounts from these areas are similar in nature, depending on the scale of the tsunami, and the quantitative data from these eyewitnesses has been added to the statistical analysis which follows. The main quantitative findings from these surveys can be summarised as follows.

3.1 The Earthquake

The earthquake was not felt in Sri Lanka. About half of the respondents in Thailand felt it; of the 29 who answered this question, 9 said they felt it weakly, 5 moderately, and 15 did not feel it at all. One respondent at Port Blair on the Andaman Islands said that it was felt violently, and one in Malaysia felt it strongly.

3.2 The Tsunami

The questionnaire asked whether respondents were aware of anything strange (such as receding water, unusual animal behaviour) before the tsunami. A majority in both Sri Lanka and Thailand said they were aware of nothing. Of the 44% in Sri Lanka who did notice something, most, in describing it, had been warned of the approaching major surge either by the response of people or animals around them, or by a first smaller surge that did not reach them. Most of the 42% in Thailand who noticed something had observed the sea receding beforehand, and there are several good descriptions of this.

Several eyewitnesses gave approximate arrival times in their descriptions. For South West Sri Lanka these varied from 9.00 am to 9.40 am but some gave (presumably incorrect) times up to 1 hour different. Descriptive accounts of the surge often estimate either the wave height or the inundation depth in the building. Naturally there is a wide variation: for SW Sri Lanka they vary from 1.5 m to 5 m, while reports from Phuket estimate the wave height at about 5 m.

In response to a question about the sequence of waves or surges, the responses are very variable and are summarised in Table 3. Most observers in Sri Lanka were aware of 2 or more and one in five observed 4 or more waves or surges. In Thailand half observed just one surge, and only one in fifteen 4 or more. In both places relatively few (10% in Sri Lanka and 27% in Thailand) said they saw the wave break.

Table 3 Sri Lanka and Thailand: was there a sequence of waves; if so, how many?

Number of waves	1 (%)	2 (%)	3 (%)	4 or more (%)
Sri Lanka	28.0	40.0	12.0	20.0
Thailand	50.0	18.8	25.0	6.3

3.3 The Buildings and Building Damage

Sixty-four of the eyewitnesses were in buildings. About three-quarters of them were in hotels, cabins or restaurants, unsurprisingly, as most were tourists, but a significant number, mostly from southern Sri Lanka, were in family houses. Concrete frame was the material of construction for the vast majority of cases where this was reported, the few brick and stone buildings being in Sri Lanka. Most were in single or two storey buildings: only 11 out of the 37 who responded to this question were in buildings of 3 or more floors. This was in many of those cases a significant factor in their survival.

All but a fifth of those in buildings reported some damage to their building. More than a third of the buildings collapsed or were heavily damaged. Nearly half of respondents were in reinforced concrete buildings. However, such

Table 4 Sri Lanka and Thailand: correlation of damage level to distance from shore

Distance (metres)	Damage level					
	destroyed/ collapsed	heavily damaged	lightly damaged	moderately damaged	Undamaged	Total
15-30		2		3		5
30-60		2	3	2	1	8
60-150	2	4	2	2		10
150-500	2	2	1	3	4	12
>500			1		1	2
Total	4	10	7	10	6	37

engineered buildings were still vulnerable to the effects of the tsunami and about 10% collapsed, while about 20% were heavily damaged. Timber buildings were most vulnerable to damage – none of the four included in reports escaped unscathed.

Only 37 of the eyewitnesses said how far they were from normal high tide and reported building damage, and Table 4 shows a correlation of reported damage rating against distance from the normal high tide. The relationship between distance from the shore and damage is not simple, but there appears to be some correlation. Most of the buildings that were destroyed or heavily damaged were located no more than 100 m from the shore. Relatively more of the buildings located 200 m or more away from the shore escaped with no damage. None of the buildings located 500 m or more from the shore sustained heavy damage or were destroyed.

3.4 Means of Escape

Respondents were asked what factors contributed to their escape and survival. Of 45 responses, 10 said they were at a safe distance or elevation above the water. Of those who were directly threatened, 11 said they (or their party) were able to run or move to higher ground inland; another 5 said they held on to a solid object, in most cases a tree. A further 10 attributed their survival to being in a solid (or reinforced concrete) building; while 11 attributed their escape to the fact they were on, or moved to, a higher floor in their building. Several accounts described watching neighbouring weaker buildings collapsing. One respondent attributed his escape to pure chance. Table 5 summarises these responses.

Table 5 Sri Lanka and Thailand: summary of reasons given for escape or survival

Reasons for escape/survival	Number of responses	Proportion of all replies (%)
Was at a safe distance from the water	10	22
Ran or moved to higher ground	11	24
Held on to a solid object	5	11
Was in a strong building	10	22
Was on, or moved to, upper floor	11	22
Total responses	45	

3.5 Injuries

The questionnaire included questions about injuries both to the interviewees themselves and other members of their party. Among the 37 eyewitnesses who reported injuries in their group, there were 16 fatalities in total. Twenty-one of the witnesses reported cuts and 14 reported bruises (often affecting more than one person). Two people suffered from respiratory disorders (bronchitis or pneumonia), and there were three reports of loss of consciousness.

Not all of those reporting fatalities also stated how far they were from the shore. Sometimes they gave approximations of distance from shore (like "less than half a mile"), or ranges (like 200–300 m). All of the fatalities for which we know distance from shore occurred at less than 500 m from the normal high tide mark. Only one of these incidents (with 5 fatalities) happened at a location more than 200 m from the shore, and most of them occurred at less than 200 m. Table 6 summarises the information on deaths and injuries and their location.

These correlations are undoubtedly affected by our sample of eyewitness survivors. Their very survival indicates that they were not located where there were most fatalities. We suspect they were mainly located in relatively safe areas, and it is unclear how this may affect the distance from shore correlations.

Table 6 Sri Lanka and Thailand: correlation of numbers of deaths and injuries with distance from shore

Distance from shore (m)	Number of fatalities	Number of injuries	Total number of reports	Proportion reporting either fatality of injury (%)
Less than 15	1	1	1	100
15 to 30		2	6	33
30 to 60		4	8	50
60 to 150	3	6	13	46
150 to 500	7	6	13	54
More than 500		1	3	33

4 Eyewitness Accounts from Indonesia

By contrast with the reports from Sri Lanka and Thailand discussed in the previous section, those from Indonesia were concentrated in two locations, the city of Banda Aceh and its surroundings, and the town of Calang about 120 km southwards along the west coast of Sumatra. This west coast area between the Aceh State capital of Banda Aceh and the provincial capital of Meluaboh bore the brunt of the tsunami. It suffered an estimated 235,800 of a total number of 283,100 people (or 82%) of all deaths attributed to the tsunami. Moreover, 80% of 267 kilometres of highway and 147 highway bridges were completely washed away (UNOSAT, 2006). Calang is half way between these two cities and because of its harbour and being the former administrative centre for the area it became a staging point for assistance. And while all 3 of these places were very

Fig. 2 Banda Aceh: 1.5 km from the sea

severely impacted by the tsunami, with inundation heights exceeding 13 m at locations close to the shore (Jaffe et al., 2006) dropping down to 5 m in Meulaboh (Wilkinson, 2005) and quickly abating further south, it was this southern area where many survivors headed in the early months after the disaster. To gain a sense of the impact of the tsunami wave within this Banda Aceh-Meulaboh coast line such a tsunami would have been sufficient to submerge a 3 storey building. This explains why the eyewitness survivors were in most cases at some distance from the shore at the time of the tsunami impact.

The photographs show Banda Aceh approximately 1.5 kilometres from the shore (Fig. 2) and Calang (Fig. 3) and they give a sense of the differences of scale of the impact of the tsunami. The two people in the Banda Aceh photograph are looking towards the sea from where the tsunami would have emerged. In the Calang photograph, the white patches are reinforced concrete ground floor slabs and are all that remain of typically two storey reinforced concrete buildings. The arrows indicate the direction the tsunami: the geography of Calang meant that it was hit from 3 different directions.

From the survey responses, it appears that most people were outside rather than in buildings when the tsunami struck. This may relate to the fact that the earthquake was strongly felt in these three locations, encouraging people to

Fig. 3 Calang (the *arrows* indicate the direction of the tsunami's arrival)

leave their homes for safety from the expected aftershocks. Reports estimate that the tsunami arrived 25, 35 and 45 minutes after the earthquake shaking for Calang, Banda Aceh and Meulaboh respectively (Wilkinson, 2005). It struck Calang at 8.24 am on a Saturday.

Another difference between the Indonesian responses and those from the other countries is that, although the number of separate responses is greater, they are briefer, and contain less by way of a personal narrative. A number of questions were only answered by a few respondents. This has been attributed in part to a sense of frustration evident to interviewers presumably towards the slow rate of aid assistance but also to the immediate history of Aceh. In the 30 years preceding the tsunami (and in particular the last 10 years) fighting between the Indonesian Government and the Free Aceh Movement (GAM) had conditioned people not to be forthcoming with their views (Niksch, 2002). Nevertheless, a number of important overall observations emerge from these reports, summarised below.

4.1 Respondents and Their Location

Among the 388 respondents, 95% identified themselves as residents, a few as local or international visitors; 55% were male, 45% female; most (80%) were between 20 and 50 in age. Many reported the size of the party or group they were in at the time: 17% were alone, 13% in a group of two, and the majority in a group of 3 or more. All reported their location at the time of the tsunami: 72 (18%) were located in Banda Aceh, 119 (31%) were in Calang, 18 (5%) were in the two intervening West Coast districts of Aceh Besar and Aceh Jaya, and the remainder in a variety of other locations. Only a relatively small number (33) indicated how far they were from the shore, mostly those who were inside buildings at the time. Of these only one was less than 50 m from the shore; 14 (42%) were between 800 and 1500 m (mostly in Calang), while 16 (48%) were between 3.5 and 5 km from the shore. These latter were mostly those interviewed in Banda Aceh. Thus almost no-one was very close to the shore at the time of the impact of the tsunami, reflecting the fact that the chance of surviving the tsunami at the shoreline was small.

4.2 Experience of the Earthquake

Almost everyone reported having felt the earthquake, and most reported that it had been either strongly felt (81) or violent (297). A very small number said it was not felt (6) (several were fishermen in their boats at sea), weakly felt (4) or moderately felt (1). The vast majority also said it was felt by everyone. Most (265) reported an estimate of the duration of the earthquake ground shaking which varied very widely, 81 (30%) reporting a duration of shaking less than or equal to 5 minutes, 87 (33%) between 5 and 10 minutes, and 96 (36%) greater than 10 minutes. The action taken by the vast majority was to run outside (225 or 69%), or to move towards others (54 or 17%). 18 (6%) moved towards a doorway, while only 3 (1%) dropped and covered.

4.3 Experience of the Tsunami

Almost nobody had any prior knowledge of tsunamis; 97% said they knew nothing about them.

Almost everyone answered the question "when did you first become aware of the tsunami approaching" and answers were mixed: for 46% it was "when someone warned me", for a further 20%" it was when I saw the trough", for 16% "when I saw the water", for a further 12% "when I heard the water", and for 5% "when I felt the water".

Asked whether they observed a sequence of waves or surges, a majority (70%) answered simply "yes", but some quantified their answer,19% saying that they observed only one, 9% three, and just a few observed 2, 5 or 7 surges. The answers are to some extent place dependent. Respondents were also asked whether they observed variations in the tsunami's impact along the coast and what they thought accounted for that. Most (78%) said they observed no differences, but 21% did observe differences; of these most (65%) attributed the differences to mangroves or other vegetation, while some attributed the differences to the shape of the coast, land slopes, sea cliffs or rock outcrops.

4.4 Injuries and Means of Escape

Asked about injuries to themselves and their group, most reported no injuries but 114 reported either injuries or a fatality. Of these there were only 4 fatalities, most reporting bruises (36%), cuts or scratches (36%) and a few broken bones (8%), animal bites, burns, internal injuries or subsequent infection. Asked about the causes of their injuries, the largest group gave water submersion (40%) as the cause, but significant numbers attributed their injuries to vegetation debris (30%) or building debris (21%), and a number to multiple causes.

In answer to the question "to what do you attribute your survival", only about 50% gave an answer, and there were range of answers were obtained, summarised in Table 7. The largest group (33%) said they took evasive action, and 15% said that they moved to a place of safety, essentially the same thing. A significant number of the respondents had been away from home at the time. Other responses included waiting for help (15%), helped by others (7%), having food to eat (12%), or being protected by vegetation, or good fortune.

Table 7 Indonesia: reported means of escape from the tsunami

Means of escape	Number	%
Took evasive action	50	33
Waiting for help (out of danger)	25	15
Moved to a place of safety	24	15
Away from home	26	15
Had food to eat	20	12
Helped by others	12	7
Good fortune	2	1
Protected by vegetation	1	0.5
Don't know	3	2
Total responses	163	

4.5 The Buildings and Building Damage

A rather small number (25) reported damage to their buildings. Of these 16 were in Banda Aceh (at distances ranging from 3.5 to 5 km from the shoreline) and 9 were in Calang, at distances from the shore of between 1 and 1.5 km. These buildings were a mixture of masonry (brick and adobe), timber and concrete frame. Of those reporting the level of damage, most (75%) said their building was heavily damaged, while 12% reported it destroyed. The small number of cases where building damage was reported makes a comprehensive analysis of the building damage caused impossible, and it is important to note that the buildings described were all some distance from the shoreline. Nevertheless an estimate of the tsunami intensity at these locations can be made and this is reported in Section 5.

4.6 General Comments

Respondents were also asked to give their most important observation on the disaster and any further thoughts, and this provided some insight into people's perception of the disaster, at a period of some 6 months after the disaster occurred. The most common single comment was that the disaster was "punishment from the Muslim's God to Aceh's people". And many spoke of the "fear of the sea" that they had, and a wish to remain far from it. But this did not appear to imply a generally fatalistic attitude to such calamity. There was much in the comments about the need to create a society with a better resistance to disaster, to "guard against disaster", to "keep the memory", and more specific suggestions such as a developing a warning system, replanting protective trees, and providing all houses with life-jackets. Many spoke of the hope and need for outside assistance in rebuilding, to help build houses, schools, health centres, mosques; and also a protective sea wall.

4.7 Discussion

The length of the shaking reported is remarkable, as many as 36% of the respondents reporting that it lasted more than 10 minutes. The rupture duration reported by the US Geological Survey was between 3–4 minutes with a suggestion that in northern Sumatra shaking may have been experienced for up to several minutes (USGS, 2006). As there was no strong motion instrument in Northern Sumatra, shaking duration can only be judged by human observations, so these records may be important.

The classic earthquake response (drop, cover, hold) taught in Japan, New Zealand and California (CDEM, 2006) was adopted by few people. However, such an approach may not be wise given the informal style of construction typical of single storey residential buildings in Aceh. Such buildings are specifically excluded from the requirements of the Indonesian Building code and have been replaced with a "model" design approach using standard sizes. Unfortunately such an approach does not adequately address detailing issues and ignores the structural impact of non-structural aspects such as unreinforced infill brick walls commonly used in Aceh. The assumption made in the richer countries that the building is inherently sound may not hold in developing countries.

Lack of prior knowledge of tsunamis is surprising given the area's history of such events with the last as recent as 1941 (though much smaller, at approximately 3 m). The people on the Indonesian Island of Simeulue which is 300 kilometres south of Banda Aceh have on the other hand a strong oral history of tsunamis and headed away from the coast immediately after the earthquake (McAdoo et al., 2006). Has the institutional memory of tsunamis for Acehanese people been lost? Though subsequent to the tsunami, many Acehanese had a different understanding of a traditional lament about the big wave and some remembered as young children working in rice padis in the hills rather then on the flood plains below. This raises the question as to how it would be best for the community to publicly commemorate the tsunami tragedy.

The survey showed that 54% of people in Aceh had no warning at all of the tsunami. The need for a primary early warning system (given the inability of authorities on 17 July 2006 to effectively warn people in West Java of an impending tsunami) must remain as one of the critical exercises of the Indonesian Government. For Indonesia, this may need to be decentralised to a community level warning system because of the proximity of the tsunami-generating earthquake faults. But there is also a need to provide an early warning for countries more distant from the faults.

A disparity between reports of the number of waves is to be expected. For example, the tsunami in Calang would have been a combination of the 3 different directional wave fronts, as indicated in Fig. 2. What was experienced in Calang would have depended on where you were. Moreover, one significant difference was that the tsunami in Indonesia was one event (consisting of a train of waves of 2, 5 or 7 surges) happening within seconds whereas witnesses in Thailand for example suggested that there were minutes between each successive wave.

The apparent lack of injuries caused by the earthquake - despite it being a magnitude 9.0 strength, close proximity and of shallow depth – is remarkable. Also the generation of debris (particular timber as it floats in water) has been picked out as significantly contributing to injuries. Debris also increases the hydrostatic pressures (and hence loads) on buildings by significantly increasing the density of the tsunami flow. However this finding is without doubt affected by a selection bias in the sample that favoured survivors.

Interestingly, the accounts given of the means of escape are nearly all horizontal evacuations and relatively few elected for a vertical evacuation such as climbing trees or climbing on to roofs. This perception that one can outrun a tsunami in Indonesia is important given that 54% of people had no warning (while the warning by others gave the remaining 46% only a slightly better chance). Given the maximum evacuation times of 20 minutes with an early warning and 5 minutes without (Potangaroa, 2006) in a flat low lying terrain it is probable that in some areas better survival rates might have been be achieved by using vertical evacuation options.

5 Comparison with Other Reports

This section sets out to compare the information obtained from the eyewitness reports with that provided by post-event reconnaissance missions and other post-event studies. Two particular points of possible cross comparison of interest are the local magnitude or intensity of the tsunami, and the damage to buildings.

5.1 The Local Intensity of the Tsunami

Clearly there is a relationship between the height of the incoming wave and the level of the tsunami action on buildings and structures, and information on the wave height and its run-up is found in many of the reports. A distinction needs to be made between the incoming wave height offshore the inundation depth at particular locations and the run-up height (the height above mean sea level which the water reaches inland), which can be considerably greater. The wave height was measured by established tide gauges, while the run-up height was, in many cases, derived from measurement of the water-marks left on the buildings. The tsunami impact on a building or other structure is of course principally determined by the water inundation depth at the position of the building.

For Thailand, DPRI (2005) conducted a survey around Phuket and Khao Lak. On the west coast of Phuket run-up heights of 4–5 m were measured, whereas at Khao Lak the run-up height reached 10 m. For Sri Lanka, summaries of measured wave heights and measured tsunami run-up heights were provided in the EEFIT (2006) and EERI reports (EERI, 2005c) and DPRI (2005). These figures are consistent with the estimates of tsunami height provided in the eyewitness reports. The GSMB (2005) report for Sri Lanka provides both estimated wave heights at shore and the number of waves observed at more than 200 locations in Sri Lanka; in 43% of these locations, just 2 waves were observed, but responses varied from none to 6; this is also quite consistent with the observations of the much smaller eyewitness sample.

The run-up and inundation depth at many points on the Indonesian coast were investigated by Jaffe et al. (2006). Their summary states:

> The maximum tsunami elevations were greater than 16 m and the maximum flow depths were greater than 13 m at all sites studied between Breuh Island and Kuala Meurisi, a distance of 135 km. The inland tsunami flow depths were large along this section of the coast. Tsunami flow depths of 15 m and 10 m were observed at 500 m and 1500 m inland, respectively. In flat-lying coastal areas of northwest Sumatra, the tsunami destroyed all buildings in a zone inland of the shoreline that was at least 500 m wide, and, in valleys and broad coastal plains, more than 1500 m wide.

Our eyewitnesses from Indonesia were nearly all at some distance inland. At Banda Aceh they were concentrated at the city centre, 3–5 km inland. In this area according to Borrero et al. (2006), the flow depth varied from 1.5 to 5.5 m.

In Banda Aceh itself, the study by Borrero et al. (2006) states:

> The area towards the sea was wiped clean of every structure while, closer to the river, dense construction in a commercial district showed the effect of severe flooding. The flow depth was just at the level of the second floor and there were large amounts of debris piled along the streets and in the ground-floor storefronts

No equivalent information is available for Calang.

5.2 The Level of Damage

The best way to correlate eyewitness reports with damage data from the post-event reconnaissance reports is through a measure of intensity. The EEFIT study has proposed an intensity scale for tsunami damage (EEFIT, 2006). The scale (defining intensity levels from I, weak to VI, devastating), is based on the levels of damage (from D1, light damage to D4, collapse) caused to buildings of different typologies (Timber, Masonry and Reinforced Concrete). Intensity levels have been assigned on the basis of the EEFIT damage surveys to 22 locations in Southern Sri Lanka and 6 locations in Thailand. The eyewitnesses interviewed in the present study were all asked to identify both building

Table 8 Tsunami intensity levels derived from eyewitness accounts in eight locations, and comparison with EEFIT intensity assignments

	Number of buildings	This study, intensity range	This study, mean intensity	Intensity description	EEFIT intensity assignment
Unawatuna	3	III to IV	IV	Destructive	IV
Hikkaduwa	4	III to IV	III	Strong	III
Khao Lak	4	VI	VI	Devastating	VI
Phuket, Kamala	3	III to V	IV	Destructive	IV
Phuket, Patong	4	II to V	III	Strong	III
Phi Phi	3	IV to VI	V	Very destructive	VI
Banda Aceh (town)	13	IV to V	IV	Destructive	
Calang (town)	3	IV	IV	Destructive	

typology and damage level, and, where there are clusters of eyewitness reports, it is therefore possible to allocate an intensity level using the EEFIT intensity scale. In Sri Lanka and Thailand, there were six locations at which there were more than 3 reports, Hikkaduwa and Unawatuna in Sri Lanka, and Khao Lak, Kamala Beach, Patong Beach and Phi Phi Island in Thailand, providing some basis for such a cross-comparison. Table 8 shows the range and mean resulting intensity level derived from the eyewitness reports in those locations, and the intensity level assigned by EEFIT.

Surprisingly, there is an almost exact correspondence between the mean intensities from these surveys and EEFIT assignments, the only difference being for Phi Phi, where our eyewitness accounts averaged one level lower than the EEFIT assignment. The close correspondence based on very few accounts is partly fortuitous, but does indicate that eyewitness accounts, if of sufficient number, are potentially a good way to supplement post-event damage investigations. They also provide indicative data when reconnaissance work is not possible.

For Banda Aceh and Calang there were also sufficient reports of building damage for an assessment of the tsunami intensity to be possible using the EEFIT scale. In both locations the intensity turned out to be level IV "*destructive*". Bearing in mind that the eyewitnesses were all at some distance from the shoreline, and given damage descriptions given by others, these intensities are plausible. At higher intensities there would be few survivors. An approximate correlation of intensity level and flow depth is given by EEFIT, which suggests that intensity IV is consistent with inundation depths of 3–5 m, comparable, in the case of Banda Aceh, with the flow depth observed in the city centre.

6 What are the Lessons for Coastal Disaster Management?

From the point of view of occupant safety, it is clear that different buildings performed very differently, depending both on siting and type of construction.

- There is clear evidence that buildings sited more than 100 m from the shore were both less likely to be severely damaged, and also gave their occupants a better chance of survival and escape even when they were flooded.
- Reinforced concrete buildings of two or more floors survived better than masonry buildings; failure of wall panels under flood impact was totally destructive to masonry, but mostly was not destructive to RC frame buildings.
- Timber frame buildings performed very poorly, partly through buoyancy and lack of strong foundations, and often through poor connections.
- Eyewitness reports strongly suggest that survival rates among tourists were better in hotels which had open ground floors. Where ground floors were enclosed, occupants became trapped and were also victims of collapsing masonry infills.
- There was evidence given by numerous eyewitnesses of different levels of impact at different points along the same stretch of coastline attributed to topography, coral reefs, and vegetation.

More general lessons are:

- The eyewitness accounts suggest that there are two situations to be considered when considering tsunami: namely the near case (Indonesia) where there is not sufficient time to make a horizontal evacuation and the far case (Thailand, Sri Lanka and India) where there is sufficient time. In the near case, buildings will need to with stand tsunami loads for life protection, while in the far case this may not be required. The role of an early warning system cannot be overstated for those areas distant from where the tsunami is generated.
- In future all buildings built or rebuilt on the Indian Ocean (and indeed other tsunami-prone) coasts will have to give careful consideration to means of escape in the event of tsunamis. Earthquake building codes now require ordinary buildings to be designed so that they preserve life-safety in the event of ground shaking with a 10% chance of occurrence in 50 years (the 475-year event); this means that they should not collapse at this level of ground shaking. In a tsunami, a non-collapse requirement is necessary but not sufficient to provide for occupant safety. Warning systems and means of escape should, in future, all be considered in relation to providing protection against an event of this magnitude (which may be even larger than the event of 26.12.04).
- In Aceh several mosques survived the tsunami and in particular the mosque at Loknga is a good example of a tsunami proof design that should be

investigated further. The Mosque has large diameter columns, a high level of structural redundancy but perhaps most importantly the roof beams are cantilevered out from these internal columns thus making the exterior concrete wall non-loadbearing. Thus, when subjected to massive horizontal hydrostatic and hydrodynamic loads the local collapse of the exterior walls did not initiate a global collapse of the building.
- There survey results point to a clear need for better education about the risks of tsunamis in earthquake-prone coastal areas. It is a matter of serious concern that just 3% of survivors interviewed in Indonesia had any prior knowledge of tsunamis. The clues that now seem such obvious indications of imminent surges – earthquakes, followed by significant falls in sea level – should have prompted more people to move to safety. The international community and local governments in tsunami-prone areas should act to ensure people understand the risks, and know how to act to reduce the risk to life and health.

Finally what can be concluded about the usefulness of eyewitness reports of this kind in supplementing the more conventional post-event reconnaissance reports and instrumental observations? In several respects the eyewitness reports provide data not available elsewhere. These include:

- Evidence of the duration of ground shaking
- Evidence of the sequence and spacing of surges
- Information on the types and causes of injuries
- Evidence about prior knowledge of the phenomenon of tsunamis
- Evidence of the degree of warning available
- Information about the means of escape of the survivors
- Evidence of variations in the tsunami impact at different points along the coast

In the surveys described here, the number of eyewitnesses surveyed was in most cases insufficient for statistically definitive data to be obtained, but the results indicate nevertheless the potential value of this kind of survey following future events. To be able to conduct it speedily enough and on a sufficient scale, plans need to be in place before the event.

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Towards a Global Response to Large Disasters

C. Scawthorn

1 Introduction

The 1755 Lisbon earthquake was a major event in European and world history. It was felt across broad parts of Europe, and occurred at the height of the Enlightenment and on the eve of the Industrial Revolution. Its massive death toll, and destruction of one of the largest and most beautiful cities in Europe, shook thinkers such as Voltaire whose inherent optimism was deeply shaken by the event, resulting in a fundamental dialogue with Rousseau, who observed "*. . .it was not Nature that collected twenty thousand houses on the site . . . if the inhabitants of this big city had been more equally dispersed and more lightly housed, the damage would have been much less*". (Quoted in Goldberg 1989). From a scientific viewpoint, changes were made in building construction in Lisbon following the earthquake, such as the gaiola (an internal wooden cage for masonry buildings), as well as in the planning of reconstructed Lisbon; however, while the gaiola survived to the 1920s in Portugal, it was little publicized and not utilized elsewhere (Tobriner 1984). Together with the 1783 Calabrian earthquakes, the Lisbon earthquake strengthened nascent European efforts at construction of seismological instruments (Dewey and Byerly 1969), and was a major factor leading to the birth of modern seismology.

Today, on the 250th anniversary of the Lisbon earthquake, it is appropriate that we take a step back from our detailed in-depth studies of seismic effects on the natural and built environments, to consider at a global scale the progress we are making in mitigating the human and social impacts of earthquakes, and what might be done to improve that progress. Towards that goal, this paper first reviews some data on deaths and economic impacts of recent large earthquakes, with the goal of understanding the root cause of large earthquake catastrophes. While the analysis is summary and preliminary, our conclusion is that the increasing concentrations of people and wealth due to increasing

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urbanization is the primary factor in a global trend of increasing earthquake catastrophes. This leads to some broad proposals for international cooperation on preparedness and recovery, as well as scientific collaboration.

2 Global Trends in Earthquake Disasters

Figure 1 is a graph by Munich Re, frequently cited to indicate the increasing trend of natural disasters. It shows insured losses (lower portion of each column, and lower increasing trend) and total economic losses (total column height, and upper increasing trend) due to earthquakes, floods, wind and volcanic natural disasters, for the period 1950–2000. Both economic and insured losses are normalized to current (as of 2000) US dollars. The trend for both economic and insured losses is a dramatic increase in recent years.

In order to examine this trend in some more detail, for earthquakes, Munich Re data for the fifteen most deadly earthquakes for the period 1900–2004 have been combined with several other either very costly or deadly, events (1906 San Francisco, 1994 Northridge, 1995 Kobe, 2004 Niigata, Japan earthquakes, and 2004 Indian Ocean Tsunami), as shown in Table 1.

Actual deaths and economic losses (dollars at time of vent) arising from these events are shown in Figs. 2 and 3. The trend over the last one hundred years is that of decreasing deaths but increasing economic losses, even including the recent Indian Ocean Tsunami mega-catastrophe.

Of course, it might be argued that these trends are due to 'constant lives' but 'appreciated' economic values. To examine these potential biases, Fig. 4 'normalizes' the data to current (2000) population densities, while Fig. 5 updates to

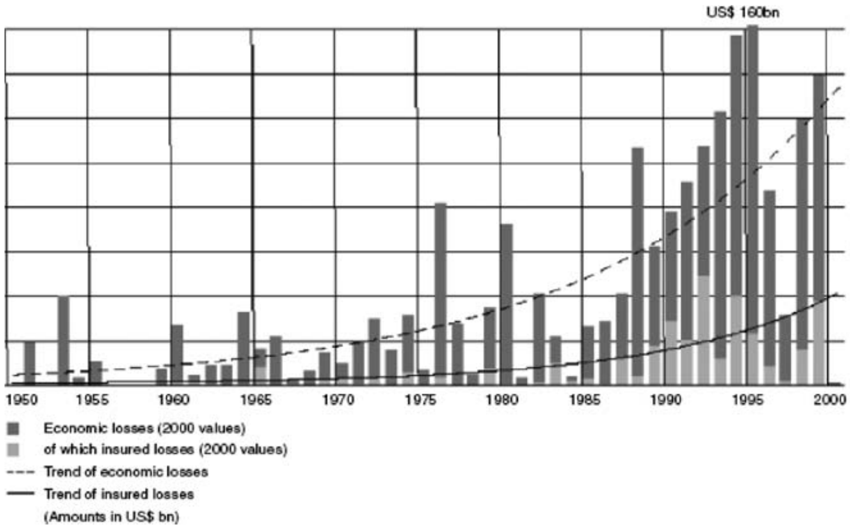

Fig. 1 Trend of worldwide economic and insured losses
(Source: Munich Reinsurance.)

Table 1 Selected large earthquake catastrophes

Event	Year	Deaths	Econ. Loss ($ mns)
San Francisco	1906	2,000	$ 524
Italy, Messina	1908	85,900	$ 116
Italy, Avezzano	1915	32,600	$ 25
China, Gansu	1920	235,000	$ 25
Japan, Tokyo	1923	142,800	$ 2,800
China, Gansu	1927	40,000	$ 25
China, Kansu	1932	77,000	
Pakistan, Quetta	1935	50,000	$ 25
Turkey, Erzincan	1939	32,900	$ 20
Chile, Concepción	1939	28,000	$ 100
Peru, Chimbote	1970	67,000	$ 550
China, Tangshan	1976	242,800	$ 5,600
Guatemala	1976	23,000	$ 1,100
Armenia, Spitak	1988	25,000	$ 14,000
Iran, Gilan	1990	40,000	$ 7,100
Northridge	1994	65	$ 24,000
Kobe	1995	6,600	$ 100,000
Iran, Bam	2003	26,200	$ 500
Japan, Niigata	2004	60	$ 30,000
Indian Ocean Tsunami	2004	243,000	$ 6,000

Source: Munich Re, 2004; Scawthorn 2005a,b

economic losses to current (2000) dollars, and Fig. 6 updates to economic losses to current (2000) dollars *and* normalizes' the data to current (2000) population densities.

The 'normalization' to current population densities is to account for population growth – a comparison of say actual 1906 earthquake fatalities versus a

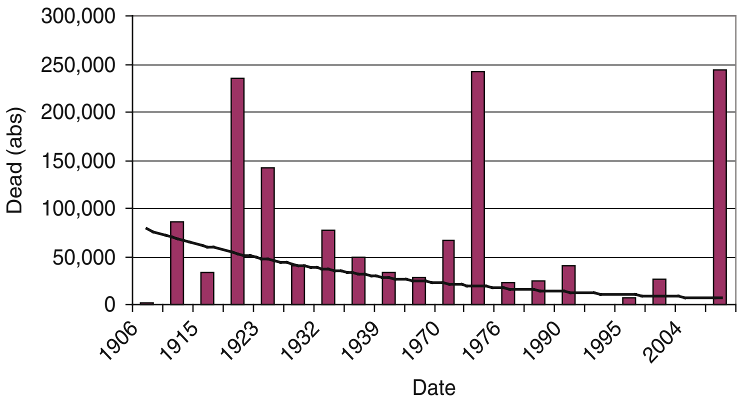

Fig. 2 Deaths for Events shown in Table 1

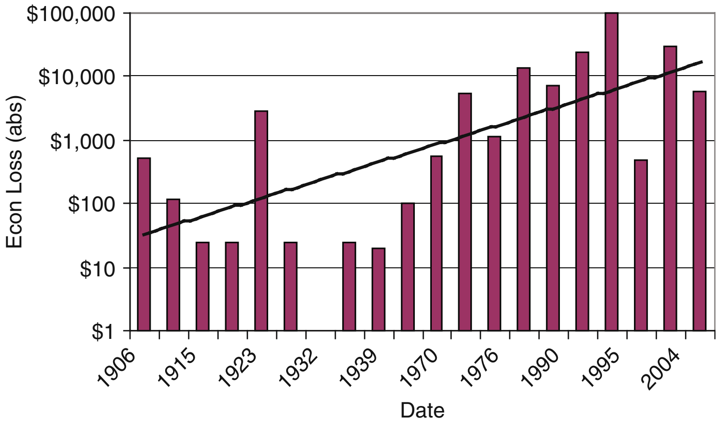

Fig. 3 Economic Losses for Events shown in Table 1

current event fails to account for today's much greater populations at risk. Treating the data in this way shows that deaths still maintain a decreasing trend, of similar order of magnitude. This shows that improvements in construction, emergency response and medical treatment have truly saved lives.

Economic losses still have an increasing trend, but the trend is significantly reduced – from a factor of 100 over about the 100-year period, to a factor of about ten over the period, when only monetary appreciation is accounted for. If that and population growth is accounted for, however, the trend is seen to have a factor of about two over the period. That is, increasing population growth is a major factor in increasing earthquake catastrophes.

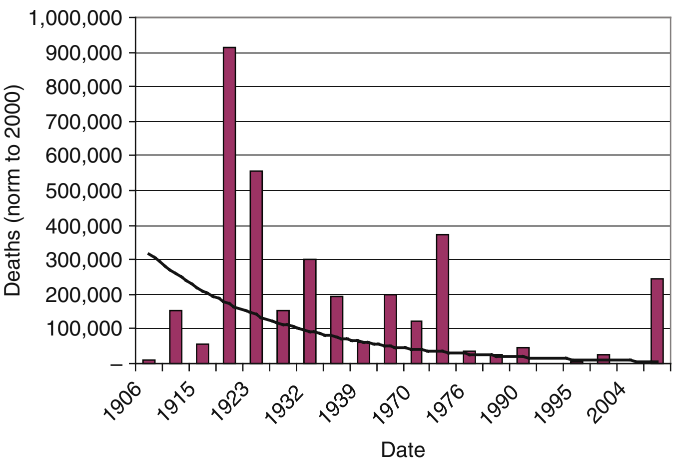

Fig. 4 Deaths for Events shown in Table 1, 'normalized to 2000 population densities'

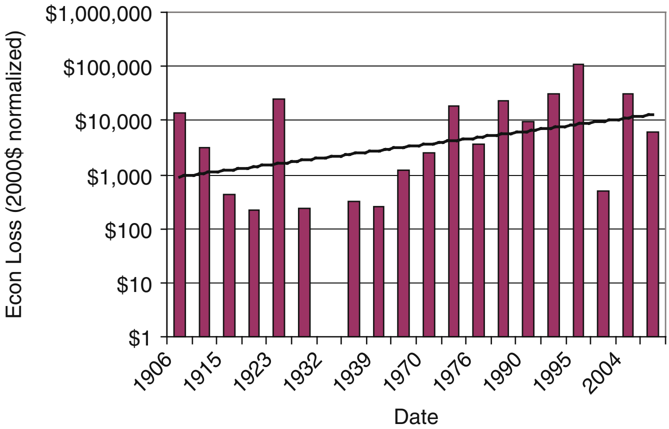

Fig. 5 Economic Losses for Table 1 events, in 2000$

However, in recent decades population growth has been accompanied with another trend – that of *urbanization*. Urbanization – the concentration of people and economic value in large cities – tends to increase the volatility of natural hazards losses. By concentrating assets in cities, everything else being equal, more natural hazards such as earthquakes will occur in sparsely populated areas, with less loss. However, when an earthquake does occur in or near a heavily urbanized area, the 'direct hit' will be a much larger loss, compared with the pre-urbanization situation of a more distributed population. The effect is,

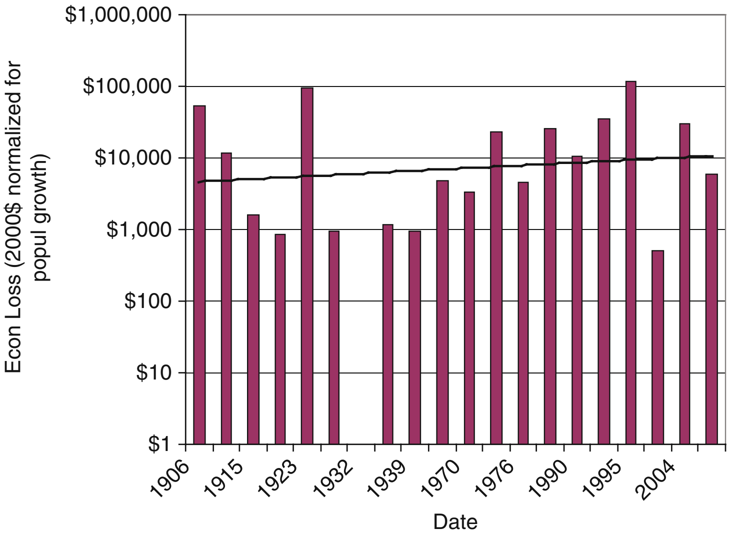

Fig. 6 Economic Losses for events shown in Table 1, in 2000$ and 'normalized to 2000 population densities'

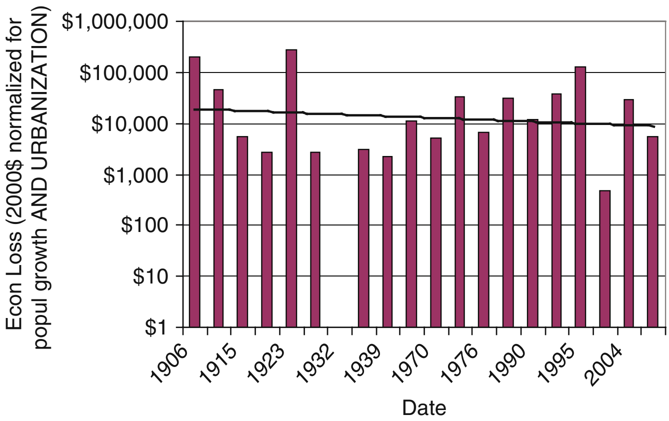

Fig. 7 Economic Losses for events shown in Table 1, in 2000$ and 'normalized to 2000 population densities' and urbanization

fewer but larger catastrophes. To examine the effect of urbanization on economic losses, Fig. 7 increases losses in earlier years by the ratio of average urbanization in 2000, to the average urbanization at the event time. That is, if 27% of the population were urban in 1950, and 57% in 2000, losses in 1950 are increased by .57/.27 = 2.11 to account for the increases in losses that *would have occurred* had the same degree of urbanization prevailed in 1950 as prevailed in 2000.[1] Figure 7 shows a *decreasing* trend in losses with time, by a factor of about 2 over the period.

3 Economic Adjusted Life Years (EALY)

In the discussion so far, deaths and economic impacts have been considered separately. Separating these two measures is typical, since equating or converting human lives to a monetary amount is very problematic, as well as entailing issues of morality and equality. In the health policy field however, an approach termed "disability adjusted life years" (DALY) has been introduced (World Bank 1993), which combine "time lived with a disability and the time lost due to premature mortality" (Homedes 2000).

Calculation of DALYs are based on five factors:

1. *Duration of time lost due to a death at each age* – based on the potential limit for life set at 82.5 years for women and 80 years for men.
2. *Disability weights* – the degree of incapacity associated with various health conditions. Values range from 0 (perfect health) to 1 (death).

[1] Global averages of urbanization are used here, an obvious area for improvement in more refined analysis, where country or region-specific data should be employed.

3. *Age-weighting function* to consider relative importance of healthy life at different ages.
4. *Discounting function* which considers the value of health gains today compared to the value of health gains in the future.
5. *Health is additive across individuals* – two people each losing 10 DALYs are treated as the same loss as one person losing 20 years.

In effect, "Years lost from premature mortality are estimated with respect to a standard expectation of life at each age. Years lived with disability are translated into an equivalent time loss by using a set of weights which reflect reduction in functional capacity, with higher weights corresponding to a greater reduction" (Anand and Kara 1995). While not without controversy in the health policy field (Anand and Kara 1995, argue that the "concept is flawed and its assumptions and value judgements open to serious question"), DALYs have proven useful as a way to more accurately value the overall impacts of various health policy alternatives.

Herein, we define an analogous concept, which we term Economic Adjusted Life Years (EALY), in an effort to better value the overall impacts of a natural disaster. We define EALYs as

$$\mathrm{EALY} = \mathrm{DALY} + \mathrm{EL}/\mathrm{W} \tag{1}$$

Where

$EALY$ = Economic Adjusted Life Years
EL = non-recoverable economic loss
W = average annual wage per capita
and DALY is as defined in the medical field.

EALYs in effect extend the concept of DALYs (which measure the effective loss of total human temporal duration) to include the loss of human time input to capital creation. It does not equate human life to economic goods, but attempts to measure the amount of peoples' lives spent in economic activity, which has subsequently destroyed by a disaster (the workers' lives were not lost, but what they spent their working lives doing, was destroyed).

In the results presented here, we approximated W by two times gross domestic product per capita,[2] and for DALYs assume average duration of time lost due to a death is 40 years (and ignore time lost due to injuries). For example, if a disaster results in a 1,000 lives lost, and a $1 billion in economic loss, and the per capita gdp is $5,000, then EALY = 40,000 + 100,000 = 140,000. If the affected population is 100,000, the EALY is equivalent to each person having been set back 1.4 years of economic production.

[2] PPP GDP, data taken from the CIA Factbook.

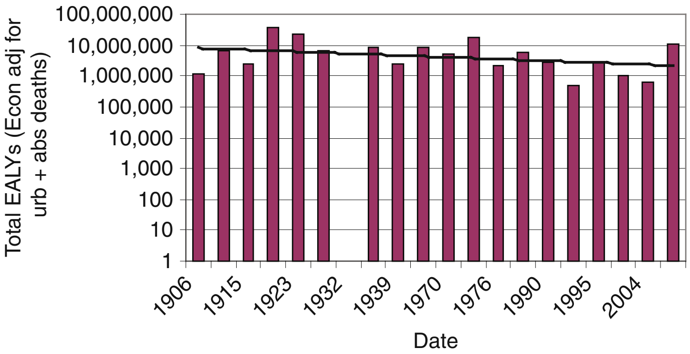

Fig. 8 EALYs for events shown in Table 1

Using this methodology, Fig. 8 shows EALYs for the events in Table 1, as adjusted for urbanization. The trend is decreasing somewhat over the period, although the economic component (i.e., EL/W, Fig. 9) is increasing over the period.

It is interesting to examine the EL/W component, adjusted for 2000$ but otherwise not adjusted (e.g., for urbanization). Figure 10 and Table 2 show such this information, in which it can be seen that the heaviest toll was the 1988 Spitak event, due to the large economic loss factored with the low per capita gdp.

There are a number of caveats to this work, primary of which is probably that the data on economic losses is extremely sparse, and of variable quality, as noted in NRC (1999). However, the overall trend at this time is decreasing life loss, and mildly increasing economic impacts on a per capital basis. The impacts differ widely depending on the economic development.

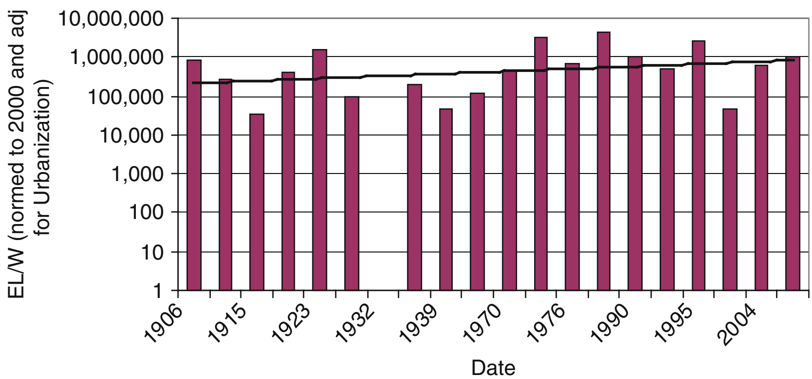

Fig. 9 Economic component, EL/W, for events shown in Table 1

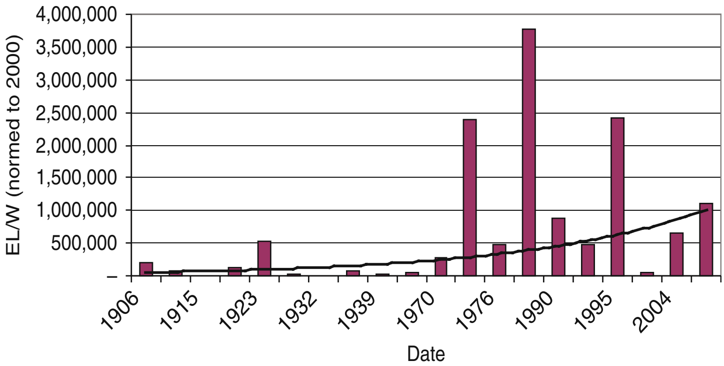

Fig. 10 EL/W for events in Table 1

Table 2 EALYs for selected large earthquakes

Event	Year	Econ Loss ($mns) at time of event	EALY normed to 2000$
San Francisco	1906	$ 524	207,015
Italy, Messina	1908	$ 116	73,192
Italy, Avezzano	1915	$ 25	9,934
China, Gansu	1920	$ 25	133,821
Japan, Tokyo	1923	$ 2,800	525,271
China, Gansu	1927	$ 25	32,062
China, Kansu	1932		
Pakistan, Quetta	1935	$ 25	77,878
Turkey, Erzincan	1939	$ 20	19,908
Chile, Concepción	1939	$ 100	50,806
Peru, Chimbote	1970	$ 550	285,974
China, Tangshan	1976	$ 5,600	2,384,807
Guatemala	1976	$ 1,100	470,226
Armenia, Spitak	1988	$ 14,000	3,783,491
Iran, Gilan province	1990	$ 7,100	887,944
Northridge	1994	$ 24,000	473,400
Kobe	1995	$ 100,000	2,408,926
Iran, Bam	2003	$ 500	47,195
Japan, Niigata	2004	$ 30,000	643,777
Indian Ocean Tsunami	2004	$ 6,000	1,111,111

4 International Response

Currently, the global response to natural disasters such as earthquakes is almost non-existent. Individual nations are typically left to their own resources, with some international aid provided to particularly overwhelmed smaller economies. Perhaps the only coordinated international response is that of international search and rescue (SAR) teams. Of these, the dog teams seem to be particularly well organized, responding very rapidly and in a relatively coordinated manner across nationalities, based on the author's limited observations in a number of disasters. Aside from the dog teams, however, even SAR teams each arrive and do their own thing, using different technologies and methods often independent of local authorities and any central operations center.

The one exception to this pattern was the 2004 Indian Ocean tsunami, which elicited a wide and somewhat coordinated international response. This exception was probably due in large part to the transboundary nature of the event, a relatively rare occurrence in even large natural disasters, as well as the almost unprecedented (in modern times, for a natural disaster) magnitude of the life loss.

A global approach for natural disasters should fundamentally be based on standardized pre-determined protocols for mitigation, preparedness, response and recovery. In addition to these four 'seasons' of the disaster cycle should be added the theatre of learning from disasters. The remainder of this paper discusses each of these aspects, suggesting ways in which a global approach to natural disasters, now totalling lacking, might be developed.

5 Construction

It is a truism that *'earthquakes don't kill people, buildings do'*. Seismic provisions of current building codes in most nations are generally satisfactory, but their enforcement may be quite another story. This is particularly true in rapidly urbanizing regions of developing economies, where the social and administrative infrastructure cannot keep up with the demands for housing and other construction. As a result, substantial portions of cities are today being built in a seismically deficient manner, due to lack of proper training of builders, and lack of enforcement of the building code.

A mechanism for improving the integrity of the building process might exist if the responsibility for construction quality can be made to adhere to someone, indeed anyone. The author recently suggested that the mutual insurance mechanism might be useful in this regard (Scawthorn 2005). The essence of the approach built on the recognition that, compared with Ex-Post financing, a much more efficient approach to disaster reduction might be Ex-Ante risk financing combined with physical risk reduction measures. Currently, for most developing economies, the bulk of donor and development aid is devoted to improving physical and social infrastructure. This matches well with the fact that the bulk of current seismic risk is the large inventory of existing buildings, built prior to modern seismic building practices. In designing a funding vehicle, if the natural hazard risk charge (i.e., actuarial cost of the earthquake, wind or flood damage) were to be included in the overall financing, with associated insurance or contingent credit, then a new vehicle would be created in which the natural hazard risk was first transferred, and then "built down" to an acceptable level.

In the early part of the term of the financing, a significant part of the finance cost would be allocated to risk transfer but, as the risk is mitigated by improved physical and social infrastructure, then the risk is reduced with time, and less of the financing cost is used for risk transfer. The result would be a single combined or *Hybrid* financial vehicle that has two tranches, financial protection and physical protection, which vary during the life of the instrument, as one form of protection replaces the other.

Consider a simple case study of natural hazards risk, and mitigation. Assume a collection of properties, such as a factory or housing, with a replacement value of \$100 (or, \$100 million – this entire discussion is in relative terms). Assume further

- that the property currently has a natural hazards risk that can be expressed in terms of an Expected Annualized Loss (EAL), as 1%. EAL is the annual probability weighted expected value of all future losses.
- that of this EAL of 1%, that 0.75% represents repairable losses, and that 0.25% represents losses such that the property is ruined (i.e., collapses or needs to be demolished).
- That the property can be retrofitted for a cost of \$20, such that the residual EAL is 0.10% (i.e., 10% of the current EAL), and that retrofitting reduces the probability of ruin to zero (i.e., all potential losses are repairable).
- that the property generates profits of \$19 per annum (equivalent to an 8 year payback with 10% return on investment).
- Lastly, assume a real discount factor of 3% per annum.

All of these values are typical of actual projects the author has worked on. These values are summarized in Table 3.

Given this data, the annual damage to the property for example can be computed as the value times the current EAL (i.e., \$100 $\times$ 1% = \$1), and the PV of all future property loss may be computed as this divided by the discount rate, or \$33. Table 4 summarizes this and other sources of loss. In this, "n" refers to "no mitigation" and "m" refers to "mitigation". Further, BI (business

Table 3 Base case assumptions

interest pa	**r_**	3%
VALUE (direct property)	**V**	\$ 100
min ROI pa	**ROI**	10%
payback (no. yrs to recoup Value)	**PB**	8
min Profit pa (assume 1 yr outage)	**P**	\$ 19
EAL (no mitigation) pa	**EALn**	1%
EAL(with mitigation) pa	**EALm**	0.10%
cost of mitigation (one time, now)	**Cm**	\$20
probability pa of damage suff for ru	**Pruin**	0.25%

Table 4 Expected damage, without mitigation

PV (all future property loss, no mitig)	**PLn**
PV (loss of future 1-yr profits, no mitig)	**Bln**
PV (loss of all future profits | ruin, no mitig)	**Cruin**
PV (all future property loss, with mitig)	**PLm**
PV (loss of future 1-yr profits, with mitig)	**Blm**
PV (all losses, no Mitigation)	**PVLn**
PV (all losses, with Mitigation)	**PVLm**

interruption) refers to loss of profits for one year (the assumed period for repairs), and Cruin is the Cost of Ruin (i.e., the loss of the facility, with the attendant loss of all future profits. Note that the cost of ruin is greater than the property loss.

Mitigation: Now, consider the benefits of strengthening or retrofitting the property (referred to as 'mitigating', or 'm'). The cost of mitigation is assumed to be $20, whereas the PV of avoided damage (i.e., the benefits of mitigation) are $90, so that Table 5 indicates that the BCR for mitigation is 4.5. Similarly, the LCC for no mitigation is $94, versus a LCC for mitigation of $24. Both appear very favorable. However, the IRR for mitigation is only 3%, which is equivalent to the discount rate – that is, putting the money in the bank. Note also that most of the benefits of mitigation derive from preserving future profits. In determining the IRR, the costs of mitigation are known and predictable, but the costs of damage occur at uncertain times, so that the IRR in this case was determined using Monte Carlo simulation (30,000 trials of 80 year facility life were employed in the Monte Carlo simulation, in this and subsequent results).

Insurance: Consider next insuring the property, rather than retrofitting it, Table 6. Insuring the property does nothing for saving lives, but protects capital, and also protects the income stream of future profits, since if a ruinous event occurs and the facility collapses, the capital is available to rebuild. In this analysis, the cost of insurance is taken as 1.5 times the pure premium. Insuring eliminates almost all financial risk, so that the benefit of insurance is almost the same as all potential losses. However, while the cost of insuring in any one year is only 1.5% of value, the cost of insuring in perpetuity is equivalent to 50% of property value, which results in a BCR of 1.79, not near as favorable as for mitigation. LCC for insurance is greater than for mitigation, but still favors

Table 5 Benefits of mitigation

Benefit = PV (of mitigation)	**Bm**	$90
BCR	**BCR**	4.5
LCC no mitigation	**LCn**	$94
LCC mitigation	**LCm**	$24
effective reduction in losses pa given mitigation	**Rlpa,m**	$2.71
IRR	**IRRm**	3.0%

Table 6 Benefits of insurance

cost of insurance / EALn =	**Finsur**	1.5
Cost of Insurance pa =	**Cinsur**	\$1.50
PV (all future insurance costs)		\$50.00
Benefit = PV (property losses, now insured)	**Binsur**	\$90
BCR	**BCR**	1.79
LCC no mitigation	**LCn**	\$94
LCC insur	**LCinsur**	\$50
effective reduction in losses pa given mitigation	**Rlpa,m**	\$2.69
IRR	**IRRm**	12.0%

insurance over doing nothing. However, the IRR is now 12%, due to the spread out relatively low annual cost of insurance, versus the one-time upfront cost of mitigation.

Hybrid: Lastly, consider a combined program of insurance and mitigation, insuring the property rather than retrofitting it, such as shown in Fig. 11. In this, a bond or other funding vehicle is envisioned, for a period of T years. The funding from the bond is used to purchase insurance, while the property is retrofitted. The time required for retrofitting may take several years, so it is envisioned there is a one-year planning period during which the risk is 100% of the prior risk. As retrofits proceed, the risk is reduced to a residual risk, and the need for insurance is reduced. Following retrofit, the residual risk is quite small, and may require little or no insurance. The term of the bond is several times the period required for retrofit, which spreads out the cost of retrofitting. From the moment the Hybrid scheme begins, the owners are protected financially via a combination of insurance and mitigation, which transitions from mostly insurance to mostly real physical mitigation. As the real physical mitigation takes over, lives as well as finances are.

Table 7 shows the results for the Hybrid program. As might be expected, the BCR and IRR are intermediate between the Insurance and Mitigation

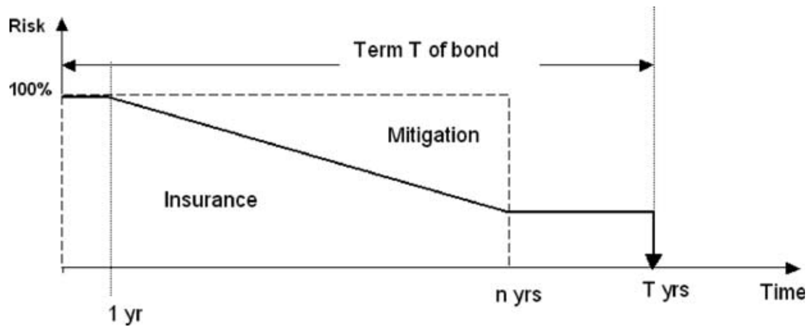

Fig. 11 Hybrid mitigation-insurance program

Table 7 Hybrid program

Bond has term T yrs	**T**	20
Mitigation is accomplished from yr 1 to yr n	**n**	5
Duration of Mitigation = n-1 yrs	**Tm**	4
Cost of mitigation pa	**Cmpa**	$ 5
PV(Cost of Mitigation)	**PVCm**	$ 18
PV Property Damage over T yrs	**PVPDT**	$5.33
PV Cost Insur = Finsur * PVPDT	**PVCinsur**	$8.00
PV Cost of Program = PVCm + PVCinsur	**PVCP**	$26.30
Benefit to Property Owner = Binsur		$90
BCR	**BCR**	3.40
LCC no Hybrid program	**LCn**	$94
LCC Hybrid Program	**LChybrid**	$26
IRR of hybrid program	**IRRh**	9%

approaches. However, the IRR for the Hybrid is 9%, which is quite attractive as compared with the Mitigation only scheme. More importantly, the LCC for the Hybrid is significantly less than for Insurance. In effect, the Hybrid scheme offers full financial protection from day one, at a much reduced cost versus insurance, and an attractive IRR. The implication is that, if a commitment by the owners can be obtained, an insurer or other party can undertake a Hybrid program for much lower cost than an Insurance program. The Hybrid scheme can be priced the same as insurance, with greater profits for the insurer and resulting physical protection for the owners and occupants, or the Hybrid scheme can be priced lower than insurance, providing the owners with lower costs of protection and the insurers with profits similar to or greater than for insurance.

The foregoing is a simple example, but shows that a Hybrid program of mixed physical protection and insurance can be a win-win for all parties. Ironically, this concept is not new – the Factory Mutual system was founded 150 years ago on a program of a long-term commitment by owners to partner with insurers for fire protection. The insurers then engineered sprinkler systems and instituted other protective features, such as employee training, that reduced costs, which the owners (owners not only of the factories, but also of the 'mutual' insurance company) shared in. Unfortunately, this concept has not been applied for natural hazards risks.

6 Mitigation

As noted above, the bulk of current seismic risk is due to the large stock of existing buildings built prior to the advent of modern seismic building practices. Not all of this inventory is seismically deficient, but a substantial portion is, and the identification of which buildings are collapse hazards is a difficult task.

Efficient allocation of resources for potential natural catastrophes requires a risk-based approach. A number of regional loss estimation and risk assessment methodologies (LERA-methodology) exist, which are beginning to evolve into standardized methodologies (HAZUS and CATS in the USA, EXTREMUM' in Russia, LessLoss in the EU, RADIUS developed by UNEP, etc.). Development of differing and competing LERA methods will inhibit international cooperation, which is vitally needed in the face of potential catastrophes.

Loss or risk analysis consists of several basis blocks: a hazard module, to define what nature can do, an inventory module, to define what humankind has put in nature's way, a vulnerability or fragility model, to define the susceptibility of the inventory to the hazard, and an analysis engine, to convolve hazard, inventory and vulnerability so as to estimate potential loss.Other elements include a GIS framework, and a GUI to facilitate I/O. Regarding earthquakes, the USGS and SCEC have begun efforts towards an OpenSHA, or seismic hazard module. The Earthquake Engineering Research Institute (EERI) has begun compilation of a World Housing Encyclopedia, which at least initiates a taxonomy as a first step towards inventory compilations. Professor Gregory Fenves and co-workers at UC Berkeley and PEER have made great strides towards an open vulnerability modeling environment (OpenSEES, Open System for Earthquake Engineering Simulation). These efforts are the budding shoots of an earthquake Open Risk Model. This writer is not aware of similar progress in the wind and flood areas.

A common scientific and methodological basis is needed for risk analysis and for development of a standardized comprehensive methodology for loss estimation and risk assessment that will permit consideration of the interaction of natural, technological and human-induced factors and terrorist mechanisms of triggering catastrophes. The methodology is also needed to provide an opportunity for predictive estimates of losses resulting from future scenario catastrophes in view of system dynamics (industrial facility, municipality, region) as well as the spectrum and intensity of threats. A common methodology will permit:

- Creation of a scientific basis for the transition of safety management to a unified system of risk indices;
- Development of recommendations for: (a) measures to reduce system vulnerabilities (facility, municipality, region etc.); (b) rational allocation of resources; (c) planning of activities for response and reconstruction;
- Identification and prioritization of prevention and mitigation measures;
- Reduction of uncertainty of estimates through reducing model uncertainty;
- Performing evaluation of large-scale engineering projects;
- Assessing the expediency of applying new materials and technologies;

Modifying plans for development of industrial facilities, municipalities, regions in order to ensure their safe and sustainable development.

More work needs to be done in all these areas – development of exposure inventories, of hazards data and methods, of vulnerability data and methods, etc. Hazards are more advanced than vulnerability, due to the institutional structure of government agencies. In the US for example, there are national agencies charged with monitoring and studying the natural environment (USGS, NOAA), but no comparable agency focuses on the built environment. As a result, we have excellent databases on seismicity, meteorology, hydrology, geology, etc, but sparse information on inventory, and vulnerability. Every earthquake today records the strong ground motion, but no useful data is collated on the damage. As this writer has emphasized previously, what is needed is NEED (a Natural Extreme Event Database), discussed further below.

Recent developments in performance-based earthquake engineering have begun to apply mainstream structural engineering's scientific rigour and analytical flexibility to estimate system-level earthquake repair costs, fatalities, and repair durations ("dollars, deaths, and downtime") that loss modelers have been concerned with for decades, but this work is still preliminary and has not yet produced loss models for broad categories of facilities. Some good steps have been taken, perhaps most recently by FEMA in supporting the CUREE-Caltech Woodframe project, where analyses and experiments have led to much more advanced vulnerability modeling of woodframe buildings. The SAC Steel frame project also contributed in that arena. However, much vulnerability information, in earthquake as well as wind and flood, still is judgement-based, or at best based on limited empirical data.

While more work needs to be done in all areas, perhaps the key next step is development of an open source risk analysis engine. Development of an open source risk analysis engine, appropriately crafted for flexibility of I/O, will encourage hazard and vulnerability module development. It will also complete the risk model, permitting open application of risk modeling and multiplying the user base. The open source risk analysis engine should be written generically, not specific to earthquake or another hazard's needs. That is, whether the hazard is earthquake, flood, wind or many other hazards, the analysis of their risk follows a similar structure of the causative phenomena being a point, line or area source and the damaging agents generally decaying with distance from the source. The damaging agent operates on the inventory and, at a first order, the losses are the summation of the effects on the inventory.

In the immediate future therefore, the task is the development of an open source risk architecture, and the development of a generic open source risk analysis engine. Laying out the architecture, and providing the risk analysis engine, will encourage development of both needed databases for input, and a multiplicity of synergistic applications. The vital requirement is open-ness.

7 Preparedness and Emergency Response

Another point noted above is that international response, even of SAR teams, is very uncoordinated. Until recently, this situation existed within the United States between states, and even to within cities in the same state. In the 1980s, a program called FIRESCOPE was developed to enhance response of many different fire departments responding to large wildland fires. FIRESCOPE developed in the 1990s into first the Integrated Command System (ICS), which grew further into what is now termed the Standard Emergency Management System (SEMS). ICS and SEMS are used relatively consistently now within California and, increasingly, SEMS is being implemented at the national level. Currently, the US Federal Emergency Management Agency (FEMA) has supported development of about 75 USAR (Urban Search and Rescue) teams across the US, which are all organized along SEMS lines, are quite similar, and which can be taken anywhere in the country and 'plug and play' interchangeably.

On the international arena, an equivalent to ICS and SEMS is needed, in which an international body develops standardized protocols for disaster emergency response. These protocols should employ a standardized international terminology, with similar equipment, uniforms, methods, training and command structure. When a large disaster strikes, by pre-arranged rapid entry agreements etc, these *'international disaster brigades'* can converge on a city with its dozens to thousands of collapsed structures, and accomplish real life-saving.

8 Development of a Disaster Knowledge Base

Current disaster mitigation is enormously hampered by lack of good quality data, in virtually all aspects of the disaster problem. Nowhere is this more apparent than in the current status of scientific data collection, and the archiving and dissemination of this data. We discuss three aspects of the problem here: international protocols for the standardized collection of data; international collaboration in data collection; and archiving of the collected data. We begin with a discussion of international protocols for the standardized collection of data.

International Data Protocols and Investigation Coordination: Data on disaster impacts, whether in the natural, built or social environments, is of great value in understanding what happens and how to prevent it from happening again. Unfortunately, such data is rarely collected. Even when collected, it is of highly variable quality, with ill or non-existent definitions of what it meant. All too often, many investigators focus on the same dramatic phenomenon, with no attention paid to other opportunities for learning.

An international cooperative effort is needed to develop a standardized ontology of data effects, which is consistent with building codes and other mechanisms for implementing enhanced knowledge. The existence of the ontology will identify knowledge gaps, which will lead to a research agenda which can be used to guide consistent, comprehensive data collection, by a coordinated effort of investigators drawn from any number of nations.

A good step in this direction has been made in the US, where the USGS recently led an effort to develop a plan for the coordination of post-earthquake investigations (USGS 2003). The purpose of *The Plan to Coordinate NEHRP Post-Earthquake Investigations* is to provide for the coordination of domestic and foreign post-earthquake investigations supported by the NEHRP (National Earthquake Hazards Reduction Program) agencies and their partners. Most of the emphasis of the Plan is on domestic US earthquakes, which either (1) result in a Presidential disaster declaration, or (2) are considered by NEHRP agencies to provide an opportunity to learn how to reduce future earthquake losses in the United States. The plan is a framework for both coordinating what is going to be done and identifying responsibilities for post-earthquake investigations. Coordination is addressed in various time frames ranging from hours to years after an earthquake Fig. 12. The plan includes measures for (1) gaining rapid and general agreement on high priority research opportunities, and (2) conducting the data gathering and field studies in a coordinated manner. It deals with identification, collection, processing, documentation, archiving, and dissemination of the results of post-earthquake work in a timely manner and easily accessible format. The plan organizes

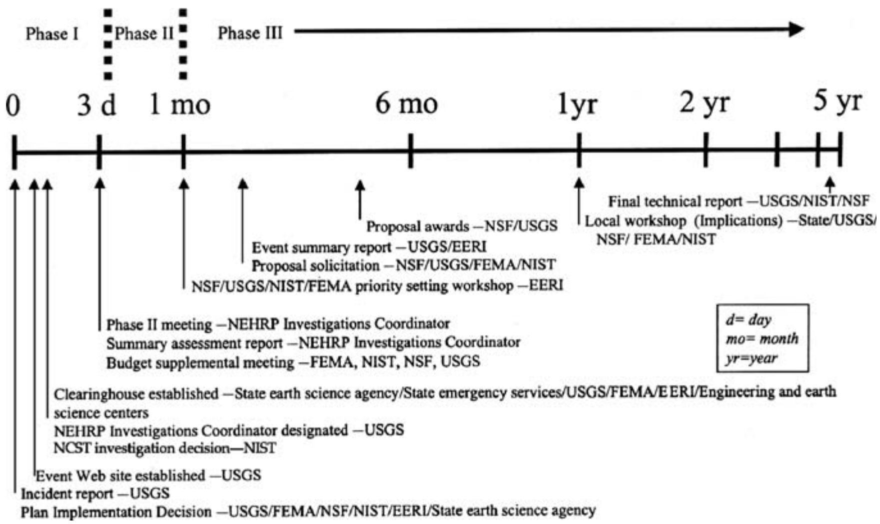

Fig. 12 Activities timeline for NEHRP Post-Earthquake Coordination Plan – Domestic earthquakes. (USGS, 2003)

domestic post-earthquake investigation and information dissemination activities into three phases and a number of steps, each of which is a decision point within the plan – that is, the NEHRP agencies at each step should decide whether the situation warrants further action, or terminate the coordination activities at that point. In summary, the three phases and steps are (see USGS 2003 for details):

- Phase I (immediate to several days): Incident Report and Plan Implementation; Web Site Management; Technical Clearinghouse; appointment of a temporary NEHRP Investigations Coordinator with broad oversight responsibilies; and NCST Investigation;
- Phase II (several days to 1 month): Meeting to identify and report opportunities and needs for data gathering and investigation, and call for proposals for rapid investigations; Budget Supplemental Decision;
- Phase III (1 month to 5 years): Workshop on Investigation Priorities, within 1–2 months; Investigations Solicitation; Information Dissemination; Event Summary Report published within 3 months of the event; Public Conference on the first anniversary of the earthquake; within 5 years, a comprehensive synthesis of research and professional reports.

For foreign earthquakes, which typically are less intensively investigated than domestic earthquakes, the plan recommends that all U.S. post-earthquake investigators inform EERI of plans and schedules of investigations before departure, as well as their ongoing status once in the field. EERI shall regularly report these planned activities and their status to the NEHRP agencies as well as on its Web site. NEHRP agencies shall monitor these plans to avoid interference by visiting U.S. investigators with local experts.

USGS Circular 1242 concluded with recommendations that addressed several deficiencies in current domestic post-earthquake investigations. The deficiencies were identified at an invitational workshop of experienced post-earthquake investigators held in March 2001 as part of the process to prepare the Plan, and were in the areas of coverage and comprehensiveness of investigations of earthquake impacts, including performance of the built and socioeconomic environments; application of new information technology to data collection; and data management and archiving.

International Investigation Clearinghouse: As part of the data collection effort, real-time coordination in the field is essential. In the past, investigators meeting each evening, in a hotel, university or other setting, to exchange observations, ideas and data, have accomplished this, Fig. 13.

However, in larger natural catastrophes, the scale of the disaster as well as differences in language etc, hamper a truly international effort. To address this need, the author and his colleagues at Kyoto University are currently developing a prototype *Virtual Clearing House* (Fig. 14) – that is a, web-based facility which bridges distance, time, language and other barriers, to facilitate international collaboration on data collection in an efficient manner.

Fig. 13 Investigators meet at technical clearinghouse to discuss findings from field investigation of 2001 Nisqually Earthquake (Photo by author)

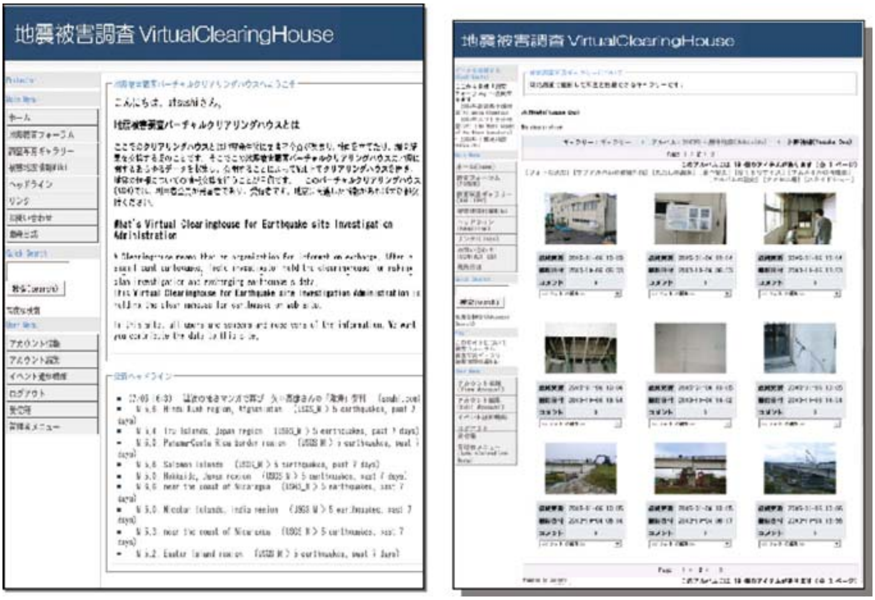

Fig. 14 Screen shot of prototype Virtual Clearinghouse

NEED: An additional problem for development of a knowledge database is the loss of valuable data on natural and technological disasters, collected by disparate investigators and organizations. Experience shows these data are typically lost after a very short period, because of the lack of a central, standardized archive. The author and colleagues are working on development of a *Natural Emergency Experience Database (NEED)* to address this need.

The overall objective is the design, development, prototyping, and pilot testing of a mechanism to preserve and disseminate emergency experience data, that is, data about natural and man-made disasters. There is a wide variety of data related to disasters, ranging across the natural, technological, built and social environments. Perhaps the most important class of these data – and the focus of this proposal – is the detailed information from data-gathering in the field: the raw data that are typically used as a basis for drawing general conclusions, but that are too voluminous to publish conventionally. In the example of earthquakes, these perishable raw data have historically included earth-science information; observations of the structural and nonstructural performance of building, bridges, and other facilities; and near- and long-term data on the economic and human consequences and response. Analogs exist for all other natural and man-made disasters.

Experience has shown that raw and first-reduction data usually become lost or otherwise unavailable in the years following the emergency. The conclusions derived from the data are often published in durable form: journal articles, reports, etc. However, the conclusions are often presented in a form that precludes analysis by others, or comparison with other studies. There are two principal reasons for this: researchers develop new, nonstandard terms to explain the data; and the published summary information does not address aspects of the underlying data that are only later recognized as important. In either case, because the underlying data are lost or unavailable, lessons learned by different researchers cannot be compared, and studies that ask new questions must always gather new data or make questionable assumptions about the old, now missing data.

Historically, the loss of the raw data has been due to its volume and, typically, hardcopy format. With the digital revolution, however, the opportunity exists to archive inexpensively much of the raw data, which is often field-recorded digitally, via PDAs, digital voice-recorders, digital video-taping, GPS, GIS, etc. The loss of these valuable, and now readily storable and retrievable, data from recent and future emergencies can be prevented by creating a national, public, durable, electronic archive for permanent storage and dissemination. More than an online database of past emergency information, the archive (i.e., the *Natural Emergency Experience Database, NEED*) can be the locus for dissemination of data-collection protocols and software, as well as the IT backbone of a system for propagating common definitions and goals for future data gathering, both in the field and in the laboratory.

NEED is *not* a site to store journal articles online; such resources already exist. Rather, it is a place to store the raw data that underlie emergency research, for example databases containing every response to every question on a survey of human responses to a disaster, or the characteristics and digital images of every building examined in a safety survey. These are the data that are analyzed to produce publishable findings, but that are not themselves typically published. The concept for such an archive emerged in the earthquake community, and has been called for by NEHRP in its *Plan to Coordinate NEHRP Post-Earthquake Investigations* (USGS 2003) and by participants at the Earthquake Engineering Research Institute's (EERI) Invitational Workshop: An Action Plan to Develop Earthquake Damage and Loss Data Protocols (2002). We propose to research and develop the prototype NEED using the earthquake community as the testbed and then, based on this experience, extend NEED to all natural and technological disasters.

The problem is fundamentally that of the inefficient utilization of information in the mitigation of disasters. The current process of information flow is shown in Fig. 15. Basically, prior to, during, and after disasters, individual researchers and organizations gather data on the disaster. The data may be depth of flooding, number of earthquake-collapsed buildings, hurricane-caused sheltered persons, or anthrax-related hospital admissions. In some cases, the actual data may have constraints on its use, such as personal privacy issues. When the researcher has collected the data, it is used in some manner: in response organizations, to improve their response; for academics, to create new knowledge, etc. In some manner, the data are usually reduced, and a summary of the data and the researcher's findings are published in a report, refereed journal etc. (the article in Fig. 15. Other researchers, or users themselves, synthesize more application-oriented findings from one or more articles, which are then translated into information for users. The process is similar, whether it is social scientists researching crowd behavior and ultimately

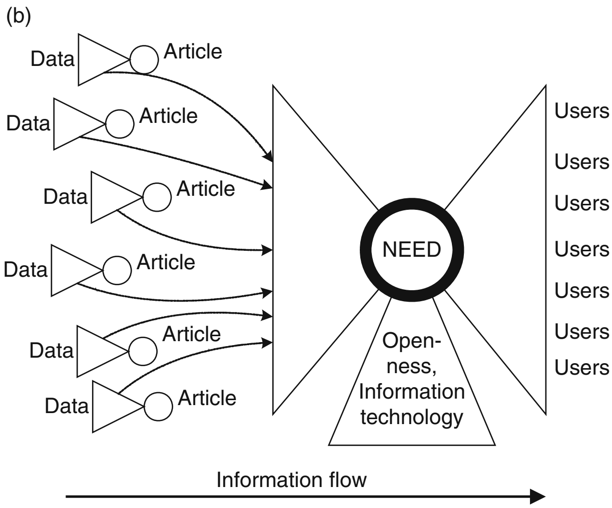

Fig. 15 Information flow without NEED

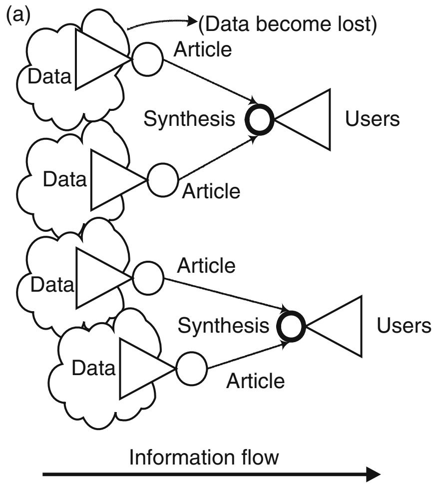

Fig. 16 Information flow with NEED

improving emergency messages, or structural engineers researching beam-column connections and ultimately improving building codes. The inefficiency arises in the lessons being founded on only one or a few datasets. Scatter in the data, lack of broad sampling, and assumptions about the (inaccessible) underlying data, result in very fuzzy knowledge and large inefficiencies.

The solution we propose is shown in Fig. 16. It consists of a voluntary pooling of data in NEED. Data contributions are open to all disciplines and all natural and technological disasters, so that cross-cutting behavior and performance, whether of crowds or of steel beams under inelastic deformations, can be extracted, compared, synthesized, and more efficiently used. Larger datasets enhance the signal-to-noise ratio and reduce bias, thus enhancing efficiency. Data gaps are more easily identified: if the data do not reside in NEED, they need to be gathered. Errors are more easily identified: if a particular dataset has a significantly different trend than others, the question arises as to *why*. In some cases, the difference in trends may be spurious because of differing terminology or initial conditions, but in others, important errors may be found.

9 Concluding Remarks

While some progress is being made in the reduction of losses due to natural hazards, such as earthquakes, substantial progress is needed. Our analysis of selected large earthquake disasters supports others' findings that economic losses are increasing. This increase is due to a combination of population increase combined with increasing urbanization. The strains of rapid urbanization in the developing world overwhelms social and administrative infrastructure, so that even current construction is seismically deficient. On top of this, the

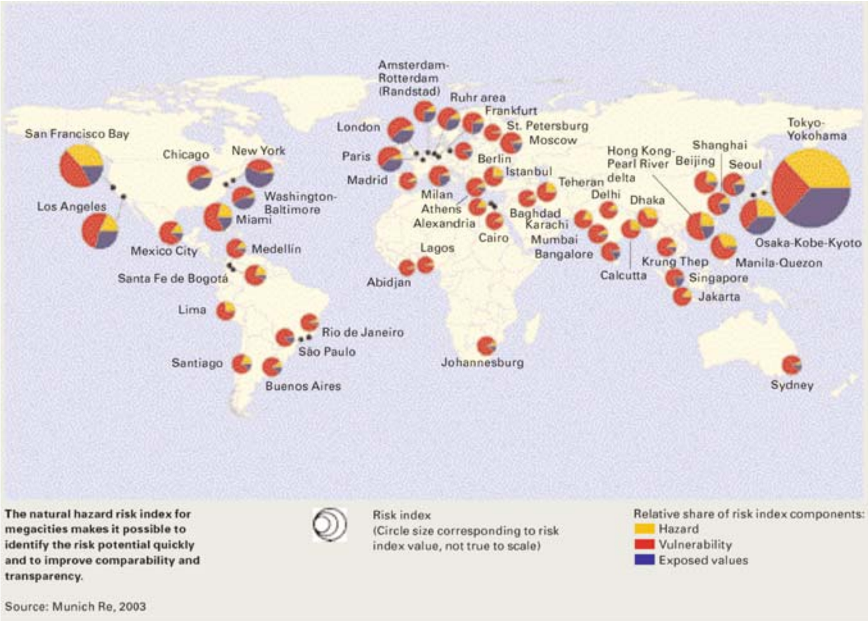

Fig. 17 Munich Re natural hazard risk index for megacities

large inventory of current building stock, generally built before the advent of modern seismic standards, is an enormous problem, with the potential for huge potential losses in megacities such as Istanbul, Tehran, Tokyo, Manila and elsewhere, Fig. 17.

Towards the goal of reducing losses from earthquakes and other natural hazards, this paper has laid out a framework for global collaboration. A new *Hybrid vehicle* for improving construction integrity by adhering responsibility for construction quality via financial mechanisms is proposed.

In order to more efficiently allocate scare mitigation resources, we propose the development of an Open Source Risk software, which can be developed and adapted locally, based on international collaboration.

Current emergency response to large disasters is very limited, due to administrative, technical and language barriers. We propose the development of international disaster brigades, developed, equipped and trained on the basis of international protocols, who can converge rapidly on a disaster anywhere in the world, and cooperate efficiently.

Perhaps most importantly, our knowledge base for disasters is extremely limited, and current data collection efforts are piecemeal and virtually totally uncoordinated. We provide a number of ideas, which we are currently working on, regarding enhanced data collection, archiving and dissemination.

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Post-Tsunami Urban Damage Assessment in Thailand, Using Optical Satellite Imagery and the VIEWS™ Field Reconnaissance System

B. Adams, S. Ghosh, C. Wabnitz and J. Alder

1 Introduction

In the aftermath of extreme events such as earthquake, tsunami and hurricane, rapidly quantifying the extent and severity of urban damage is a high priority. It provides critical information for emergency management personnel, helping national and international organizations gauge the magnitude of response activities, offering insight into the damage sustained in hard-to-reach locations, and directing field teams to the hardest-hit areas. Urban damage data is also employed within the risk management arena to generate loss estimates and to infer the number of casualties.

Over the past decade, the value of optical remote sensing technology for post-disaster damage assessment has been increasingly recognized (see Matsuoka and Yamazaki 2000, Estrada et al. 2001, Yamazaki 2001, Eguchi et al. 2003, Adams 2004, Huyck et al. 2004a, Ozisik and Kerle 2004, Rathje et al. 2005, Woo et al. 2005, Adams and Huyck 2006). Its practical implementation during response activities may be conceptualized as a three-stage "tiered reconnaissance system". From Fig. 1, at stage 1 moderate resolution sensors such as Landsat TM offer a region-wide perspective for establishing the broadscale distribution of damage. Next, high-resolution commercial imagery from sensors including Quickbird and IKONOS enables damage to be identified at the neighbourhood scale (Adams et al. 2003, 2004a,b, Huyck et al. 2003, 2004b, 2005, Saito et al. 2004, Woo et al. 2005, Chiroiu et al. 2006, Shirzaei et al. 2006). At stage 3 this information is integrated into field assessment tools such as VIEWS™, providing decision support, route planning and navigation guidance for the of detailed and efficient survey of damage to individual structures (Adams et al. 2005a).

This paper documents the use of moderate- and high-resolution remote sensing imagery within the tiered reconnaissance framework, to furnish a

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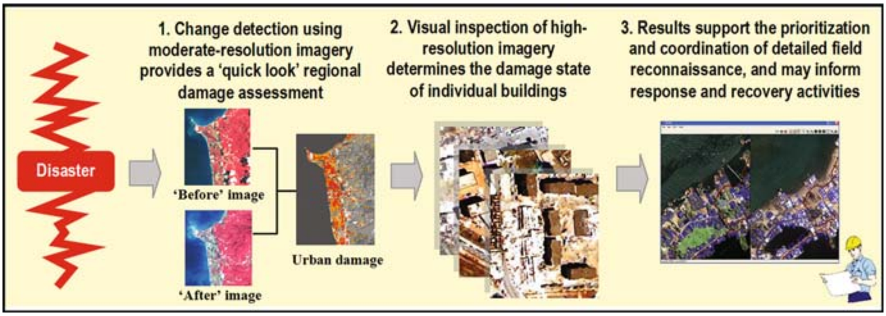

Fig. 1 Tiered reconnaissance system, which utilizes moderate- and high-resolution satellite imagery to rapidly identify urban damage from at regional, neighbourhood and ultimately per buildings scales. This information provides decision support for response and recovery activities

multi-disciplinary international project team with post-tsunami urban damage data. This data is currently being used as part of an exploratory initiative to scientifically investigate reports that mangrove ecosystems significantly buffered the impact of the tsunami, thereby protecting coastal communities (Adams et al. 2005b, Chang et al. 2006).

Focusing on the northern Andaman coast of Thailand (Fig. 2), this paper introduces a region-wide Normalized Difference Vegetation Index

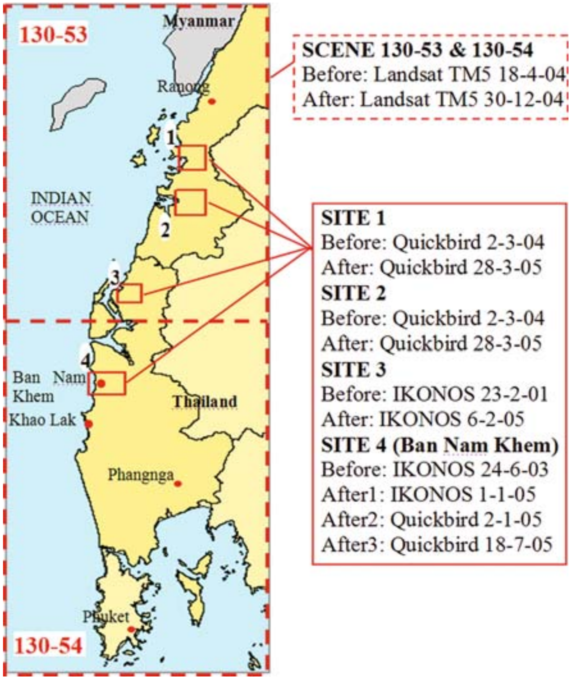

Fig. 2 Thailand study site spanning Ranong and Phangnga provinces, and optical imagery acquired for the Andaman coast study. Debris line mapping was conducted using the coast-wide Landsat TM imagery. High-resolution Quickbird and IKONOS imagery for four sites was used for damage mapping, as base data for the VIEWSTM field deployment, and for accuracy assessment and validation

(NDVI)-based methodology to demarcate the coastal zone within which extreme urban damage was sustained. A temporal sequence of moderate resolution Landsat TM5 imagery is used to identify inundated areas and delineate the position of the tsunami debris line. At the neighbourhood scale, high resolution 'before' and 'after' imagery is used to develop a local building damage map, showing the percentage of collapsed structures for the area of Ban Nam Khem. This is an example of an urban settlement that received no direct protection from fronting mangrove. The paper goes on to describe the acquisition of detailed urban damage survey data during an August 2005 deployment of the VIEWS^{TM} field data collection and visualization system The purpose of this deployment was to verify the damage state of individual structures and to collect ground truth data for validating the remote sensing analysis. Results for the debris line are verified through comparison with high-resolution imagery, and accuracy of the damage map is assessed based on the inspection of VIEWS field observations. The paper concludes with summary of key findings and directions for future work.

2 Debris Line Delineation

Following the tiered conceptual approach described above, demarcation of the tsunami inundated area and the landward position of the debris line provides a regional indication of coastal areas that sustained extreme urban damage. From a disaster response perspective, this could provide a useful quick-look for rapidly identifying hard-hit areas across a wide geographic extent, potentially spanning multiple nations.

Figure 3 summarizes the NDVI-based methodology employed for mapping regional tsunami inundation. The input data comprised 7-band TM5 imagery captured 'before' and 'after' the tsunami. The imagery was provided courtesy of

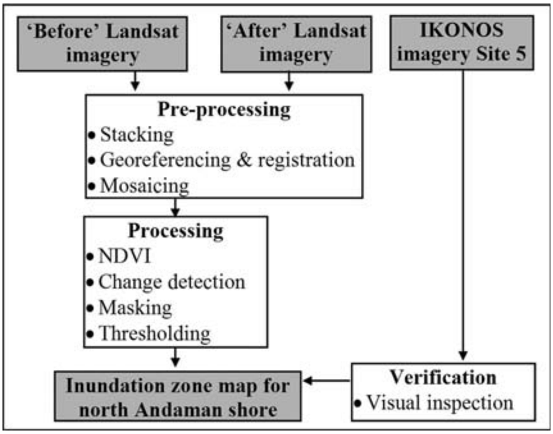

Fig. 3 Methodology employed for demarcating the regional tsunami inundated zone and delineating the debris line

the US Geological Survey and Thailand's Geo-informatics and Space Technology Development Agency (GISTDA). The pre-coverage dates from April 2004, with the post- dataset collected just four days after the tsunami (Fig. 1).

Initial pre-processing for the 'before' and 'after' image sets involved stacking the bands into single images, registering and georeferencing to a pre-georeferenced Landsat TM baselayer image for the Andaman coast, provided courtesy of NASA (WGS84 UTM zone 47 N), and finally mosaicing the 130-53 and 130-54 images into a single coverage (for details of the processing algorithms see RSI 2003).

The extent of inundation was next determined as a function of changes in NDVI readings between the 'before' and 'after' mosaics. This vegetation index was selected as useful indicator for differentiating between land-surface types; preliminary visual assessment of the images suggested that the incoming tsunami wave caused a fundamental change in the land-surface cover by scouring away vegetation and urban surfaces, and replacing them with bare earth, sand and debris.

NDVI was calculated according to Equation (1), utilizing Landsat TM5 band 4 (near-infrared - NIR) and band 3 (red).

$$\mathrm{NDVI} = (\mathrm{NIR} - \mathrm{red})/(\mathrm{NIR} + \mathrm{red}) \tag{1}$$

A change map was then computed as the difference between the 'after' and 'before' scenes for a coastal tract extending 15 km inland from the shore. Inspection of the results suggested that a number of non-tsunami-related changes were present within the imagery that could potentially be confused with surface inundation. As such, cloud and water masks were developed for the 'before' and 'after' cases by applying a threshold of NDVI $<= 0$. The debris line was manually delineated based on the limit of inundation.

Performance of the NDVI inundation methodology was verified for stretches of the Andaman coast near Ban Nam Khem (site 4 in Fig. 2) and Khao Lak (site 5). Inundated areas seaward of the debris line were visually compared with high-resolution satellite imagery, which depicts in detail the nature of post-tsunami land surface cover. Details of the findings are discussed in the subsequent results section.

3 Quantifying Building Collapse

A temporal sequence comprising high-resolution 'before' and 'after' satellite imagery was used to detect building damage within study sites 1 through 4 (Fig. 2). In accordance with the overarching project aim, these study sites were selected based on their proximity to mangrove and mangrove reclamation, together with the availability of high-resolution remote sensing coverage (for details, see Chang et al. 2006).

The imagery comprised Quickbird and IKONOS datasets (Fig. 2), either purchased directly from the vendor, or obtained by the National Science Foundation courtesy of the Clearview license. In terms of timeliness, the 'after' data for site 4 was acquired just 1 week following the tsunami, and as such provides an extremely accurate record of damage prior to clean-up operations. The first available imagery for sites 1 through 3 is less timely, being collected 1–2 months afterwards.

Figure 4 summarizes the interpretation-based methodology employed for mapping urban damage at the neighbourhood scale. Pre-processing was conducted to prepare the imagery for damage assessment; and its subsequent field deployment within the VIEWS field data collection and visualization system. Imagery for site 1 through site 3 was provided in georeferenced true colour pan-sharpened format. Visual inspection indicated acceptable spatial correspondence, and as such, pre-processing was limited to 2% colour and/or Gaussian enhancements to maximize contrast within the scenes. For site 4, the 'before' and 'after' scenes were provided as separate high-resolution greyscale and lower-resolution colour bands. A high-resolution colorized product was produced by combining these bands, using an HSV pan-sharpening algorithm. Due to significant spatial offsets, the pansharpened 'after' scenes were re-registered to the 'before' scenes, using a series of tie points (RMS ~3.5pi). Finally, the pre-processed high-resolution images were output in GIS- and VIEWS compatible formats.

In order to identify focus areas within the four study regions warranting in-depth analysis of building damage, a rapid visual inspection was conducted using the 'before' and 'after' imagery. This exploratory processing step suggested that significant areas of urban damage are present within site 4, while

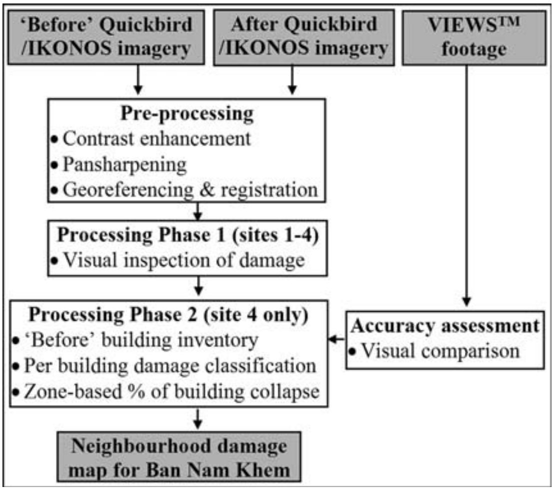

Fig. 4 Methodology employed for mapping neighbourhood-scale urban damage and quantifying the percentage of collapsed structures at Ban Nam Khem (site 4)

damage could not be distinguished at sites 1-3. As such, site 4 was selected as a focus area for subsequent analysis.

For processing step 2 (Fig. 4), a comprehensive pre-tsunami inventory of building stock was performed for the settlement of Ban Nam Khem using the before imagery. The damage state of each structure was then determined by expert interpretation of the 'before' and 'after' coverages (Fig. 5a). A simple 2-level classification scheme was employed comprising "collapsed" versus "non-collapsed". Previous studies suggest that remote sensing imagery is extremely effective at distinguishing extreme damage states, but given nadir viewing limitations is less reliable for intermediate damage levels (see, for example, Yamazaki et al. 2003a,b, 2004a,b, Saito et al. 2004). Finally, the percentage of collapsed structures was computed within a series of zones defined at 100 m intervals from the pre-tsunami shoreline(s) experiencing greatest tsunami exposure (Fig. 5b).

The accuracy of the building damage assessment for Ban Nam Khem was determined through comparison between damage states interpreted from the December 2004 and January 2005 coverage and VIEWS footage collected during an August 2005 field deployment (see the following section) furnished with July 2005 base imagery. Despite the time lapse between these datasets, the ground truth data provides an accurate record of buildings that remain standing 6 months after the event, and by inference did not collapse. Details of the results obtained are discussed in the subsequent results section.

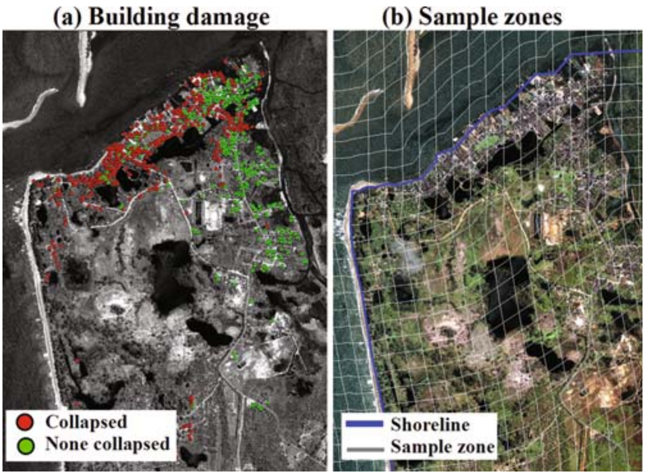

Fig. 5 Building damage assessment phase 2 processing for the Ban Nam Khem region of site 4. (**a**) Damage state determined by expert interpretation for 761 structures; and (**b**) Sample zones for computing % collapsed structures, positioned at 100 m intervals from the open coast and inlet shores that are subject to extreme tsunami exposure

4 VIEWS^{TM} Deployment

The VIEWS field data collection and visualization system (Adams et al. 2004a,b, Ghosh et al. 2005) was deployed to study sites 1 through site 4 from August 16–25th 2005, in order to verify the damage state of individual structures and 'ground truth' the preliminary remote sensing results.

VIEWS is a portable in-field reconnaissance system that can be deployed from a moving vehicle, aircraft, boat or on foot. It was developed by ImageCat, Inc. through funding from MCEER. VIEWS streamlines and accelerates multi-hazard damage survey activities, serves as an in-field decision support and prioritization tool, and has a visualization mode that can be used afterwards to analyze the data collected. VIEWS integrates base layers of satellite or airborne imagery collected 'before' and 'after' the event with: real-time GPS (Global Positioning System) readings; georeferenced digital photographs and/or video; and other geospatial datasets including damage maps and street networks. Post-disaster deployments to date include: the Bam and Niigata earthquakes, and hurricanes Charley, Katrina and Rita (Adams et al. 2004b, Ghosh et al. 2005, Womble et al. 2006).

Fig. 6 August 2005 VIEWSTM deployment at study sites 1–4

For the August 2005 Thailand deployment, VIEWS was equipped with satellite base layers including the high-resolution 'before' and 'after' coverage and derived datasets such as the debris line. Figure 6 shows field deployment by the 4-person team and guides, which was undertaken from a moving vehicle, on foot, and by boat. Georeferenced video and photographs were collected, documenting ground characteristics of urban damage. Additional information concerning urban structural characteristics and other landuses classes was collected, but falls beyond the scope of this paper.

5 Results and Validation

Figure 7 shows the tsunami inundation and debris line map for the Andaman coast study site. According to the Landsat TM5 NDVI-based methodology, severe coastal inundation was experienced at open-coast, inlet-bordering and near-inlet sites to the north of Phuket Island. Inundation has not been identified at back-of-mangrove locations, although some front of mangrove scouring is present within the more southerly study areas such as site 4. These results suggest that the mangrove has performed a dissipative role, protecting areas to the rear by absorbing the force of the incoming tsunami wave.

Inundated areas have a distinctive signature within the NDVI difference map, characterized by extremely negative values ranging from $-0.15 < \text{NDVI Dif} < -0.85$. The graphical representation of NDVI in Fig. 8 summarizes the trend exhibited by a sample of 30 pixels within the inundated zone and 30 pixels in surrounding non-inundated areas of the 15 km wide coastal swath. Within the pre-tsunami imagery, the inundated and non-inundated samples on average exhibit similar NDVI readings, with respective sample means of NDVI = 0.52 and NDVI = 0.55. However, there is a pronounced difference between the post-tsunami responses for inundated and non-inundated regions. Whereas non-inundated areas on average continue to exhibit a similar response to the 'before' case with a mean NDVI = 0.61, there is a substantial fall in NDVI readings within the inundated area to NDVI = 0.20. This decrease in NDVI is manifested as an extreme negative difference within the NDVI change map (depicted by the red areas in Fig. 7). Since the NDVI index is sensitive to surface cover and in particular vegetation, this reduction suggests a decrease in vegetation within the 'after' scene. This is in turn consistent with pronounced surface scouring by the incoming and outgoing the tsunami wave, together with the deposition of waterborne sediment.

In order to verify that these NDVI-defined zones indeed correspond with inundated areas, the Landsat TM5 results were compared with Quickbird/IKONOS footage acquired for site 4. Comparison of Fig. 9a,d indicates that the reduction in NDVI between Fig. 9c,e corresponds with scouring of the land surface. There is a marked loss in the loss of vegetative cover, together with features such as urban structures. The prevalence of bare ground within the

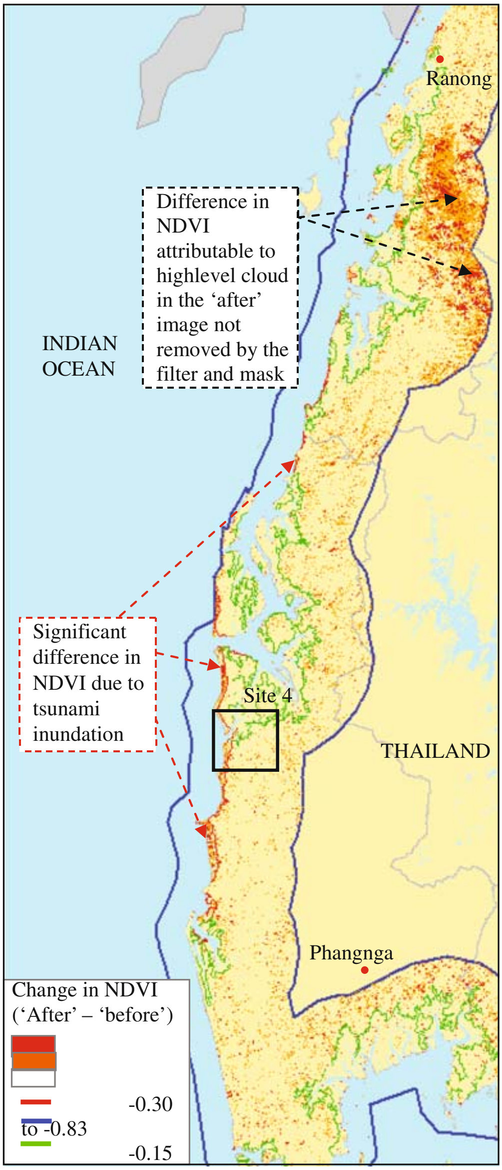

Fig. 7 Inundation area and debris line, identified as function of change in NDVI between Landsat TM5 'before' and 'after' images. Anomalous changes within the northern reaches of the 15 km wide sample zone are attributable to high-level cloud that was not removed by the filter. Site 4 is denoted as the area for which results were verified against high-resolution imagery and VIEWS footage

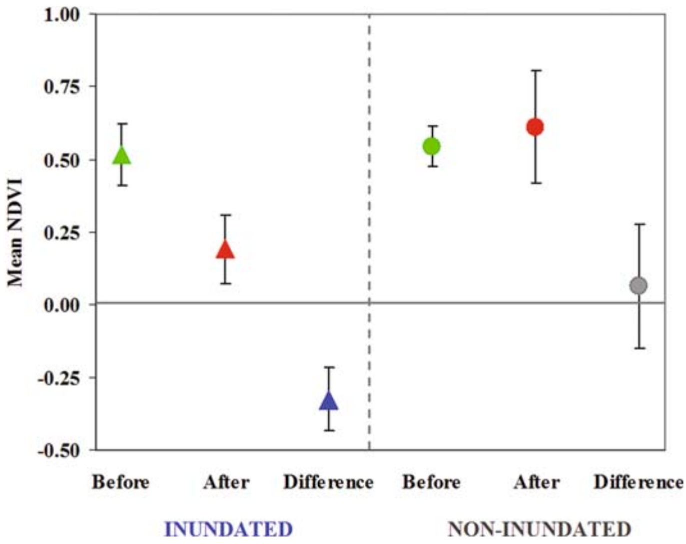

Fig. 8 Mean NDVI readings for samples of 30 pixels within the inundated coastal zone and surrounding non-inundated areas. NDVI values relate to the 'before' image (*grey* symbol), the 'after' image (*black* symbol) and the difference 'after'–'before' (*blue*/*grey* symbol). The error bars represent 1 standard deviation about the sample mean

inundated area is effectively revealed by the increase in return between the pre- and post-tsunami Landsat TM5 NIR-red-green false colour composite images in Fig. 9c,f. Figure 10 provides a photographic record of this scouring effect and deposition of sediments, captured during the August 2005 deployment. Pioneer re-vegetation of the scoured surface has occurred in some locations due to the temporal interval between the tsunami and the field deployment.

Returning to Fig. 7, a negative change in NDVI is also evident within the reaches of the 15 km coastal strip to the south of Ranong. Inspection of the Landsat TM5 images indicates that this is an anomalous result, attributable to the presence of high-level cloud within the 'after' scene that was not removed by the cloud filter.

Figure 11 shows the damage map developed for Ban Nam Khem through expert interpretation of high-resolution pre- and post-tsunami imagery. Of the 761 structures sampled, 449 (59%) were classified as collapsed, with 312 sustaining a lesser damage state. The degree of damage is most extreme bordering the open coast and inlet, with between 50–100% of the houses destroyed. The degree of damage captured by the remote sensing coverage rapidly diminishes moving inland, reaching 0–30% at a distance of approximately 500 m from the shore.

In order to determine the efficacy of interpretation-based damage categorization, an accuracy assessment was performed on the building damage data for

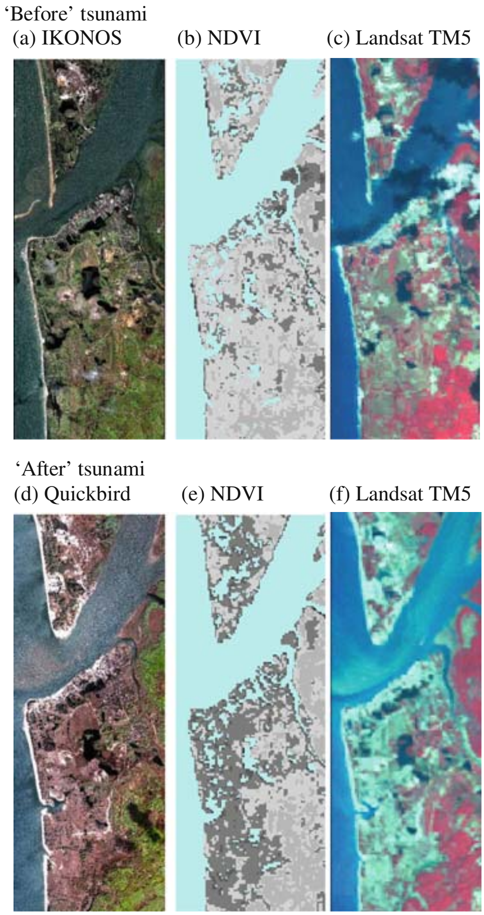

Fig. 9 Verification of the NDVI-defined inundation zone at site 4, using high-resolution imagery. (**a**) Pre-tsunami high-resolution IKONOS imagery showing a vegetated coastal strip with urban development; (**b**) pre-tsunami NDVI image recording relatively high/bright values within the coastal strip; (**c**) pre-tsunami Landsat TM5 false colour composite (bands NIR, *black*, *grey*); (**d**) post-tsunami high-resolution Quickbird imagery showing surface souring and loss of vegetation and urban development; (**e**) a pronounced reduction in NDVI within the post-tsunami image where vegetation and urban development is removed; (**f**) brightening of the post-tsunami false colour composite is indicative of scoured bare earth

Ban Nan Khem. Employing the approach documented by Maingi et al. (2002), a sample of 369 buildings was randomly selected from the inventory of 761. This sample is sufficient to result in an allowable error within 5% of the estimated accuracy at the 95% confidence level, assuming an overall classification accuracy of at least 60%.

The damage state of each point was then verified using a combination of VIEWS georeferenced video footage and a high-resolution Quickbird base layer captured on July 18th 2005. The VIEWS interface an area of Ban Nan Khem is depicted in Fig. 12. Owing to the use of "ground truth" data dating from 7 months after the event, several different scenarios were observed during the accuracy assessment, as shown in Table 1. Collapse was defined as scenario 1 and scenario 2.

Results for the damage classification accuracy assessment are show by the confusion matrix in Table 2. Of the 369 points sampled, a total of 363 were

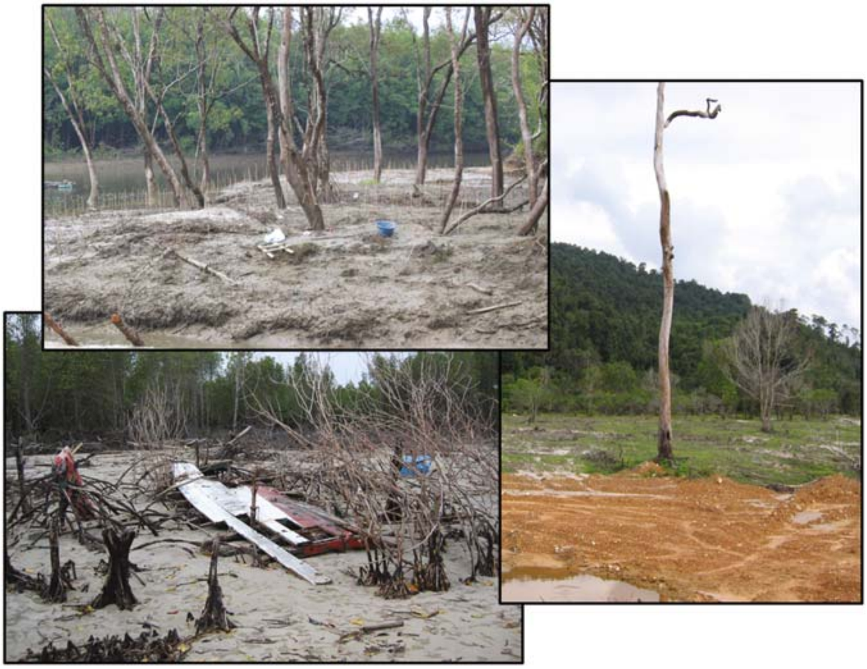

Fig. 10 Photographic record of the inundation zone and debris line along the Andaman coast, captured during the August 2005 deployment

carried forward to the analysis, the others being obscured in the July 2005 satellite base data by could cover. Three different measures of accuracy are employed here: (1) User's accuracy; (2) Producer's accuracy; and (3) overall accuracy (Stehman and Czaplewski 1998). Overall the classification accuracy of 89.5% is high, suggesting that the remote sensing-derived damage map in Fig. 11 provides an effective characterisation of tsunami damage at the neighbourhood scale. For the collapse class, both User's and Producer's accuracies are >85%, indicating that the interpretation of building collapse from the comparison of 'before' and 'after' satellite imagery effectively represents the true collapse status, and that for this study site a high proportion of the collapsed buildings were correctly classified. In the case of not-collapse, the Producer' accuracy is high, indicating that a large proportion of buildings that did not collapse were correctly classified as such. However, the lower statistic for User's accuracy arises because 7 months after the tsunami, a number of buildings that were classified as not-collapsed from the remote sensing analysis, were actually levelled or replaced.

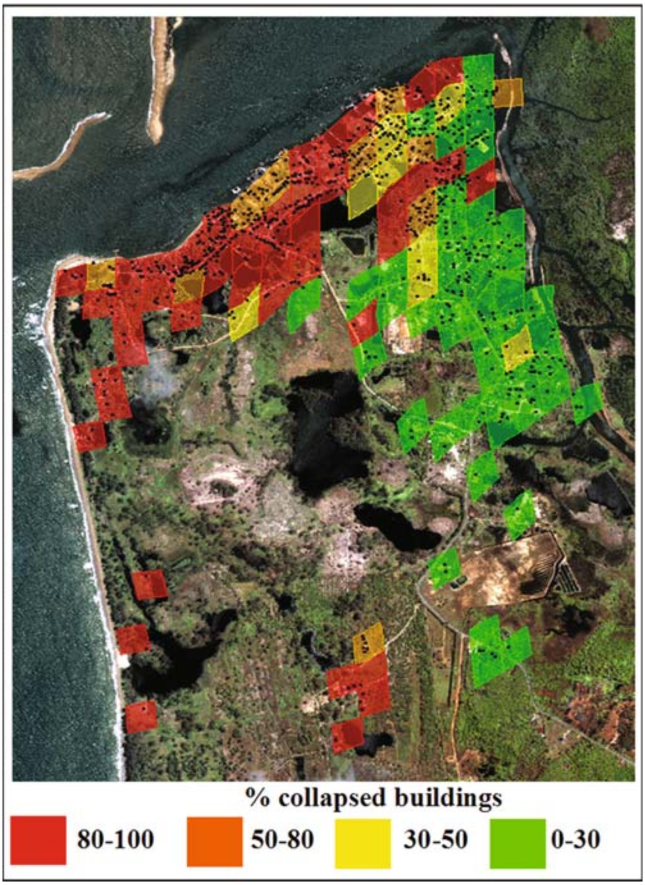

Fig. 11 Damage map for Ban Nam Khem, developed using high-resolution Quickbird and IKONOS imagery. The black symbols represent the 761 buildings that were sampled

6 Summary of Key Findings and Future Work

Key findings from this study include:

- The tiered reconnaissance approach, which employs a combination of moderate- and high-resolution pre- and post-disaster remote sensing imagery, is an effective tool for identifying urban tsunami damage at regional and neighbourhood scales.
- The multi-temporal analysis of Landsat TM5 imagery using an NDVI-based methodology effectively demarcates inundated coastal areas, and enables the delineation of a debris line. Inundation is detected as a function of the change in land surface cover, where the tsunami wave scoured away vegetation and urban development, replacing it with bare earth and/or debris.

- The comparative analysis of pre- and post-tsunami high-resolution satellite imagery supports urban damage mapping at a neighbourhood scale. For the settlement of Ban Nam Khem, 59% of the building stock was classified as collapsed and 41% not collapsed, with an overall classification accuracy of 89.5%. User's accuracy for the non-collapsed class was reduced by a number of instances where buildings identified as visually intact within the December 2004 and January 2005 remote sensing coverage were cleared or replaced during the 7 month period until "ground truthing" was conducted.
- VIEWS data collection and visualization system proved to be an effective tool for recording georeferenced video footage throughout the Andaman coast study sites. This dataset serves as a valuable resource for ground truthing, accuracy assessment and future research.

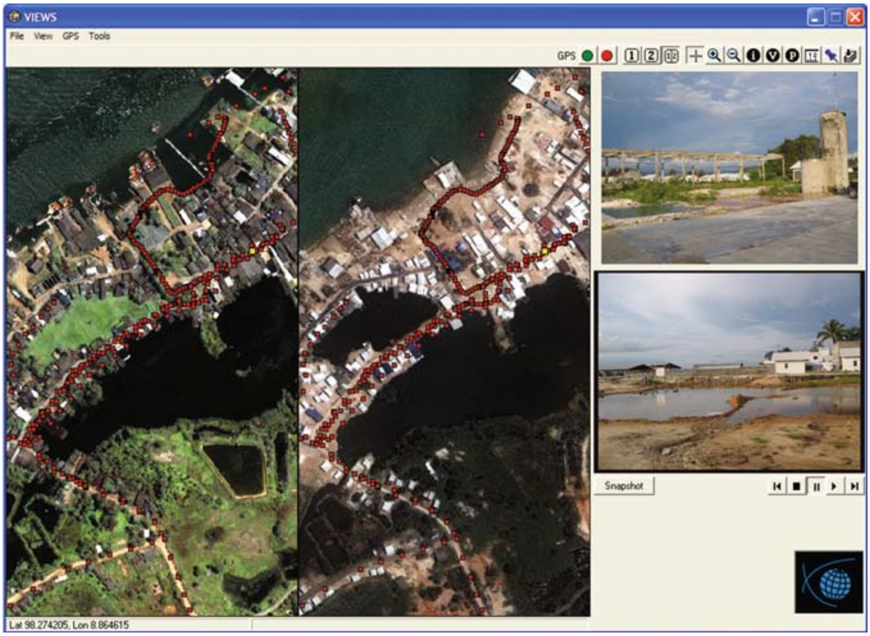

Fig. 12 VIEWSTM interface for Ban Nam Khem. A combination of georeferenced video and satellite imagery from July 2005 was used to verify the damage classification (see Fig. 5)

Table 1 Damage state interpretation scenarios observed in Ban Nam Khem

	'Before' status	VIEWS^{TM} status	Damage state
1	Intact building	Bare earth or foundations	Collapsed
2		New building of different shape, size or colour	Collapsed
3		Building same shape and size. New roof colour	Not collapsed
4		Building same shape, size, colour	Not collapsed

Table 2 Confusion matrix recording accuracy assessment results for the damage assessment conducted for Ban Nam Khem

	Accuracy assessment				
	Collapsed	Not Collapsed	Total	User's Accuracy (%)	Producer's Accuracy (%)
Collapsed	212	7	219	96.8	87.2
Not collapsed	31	113	144	78.5	94.2
Total	243	120	363	**Overall Accuracy 89.5%**	

Future work will focus on extending the damage assessment methodology to consider semi-automated methods of damage assessment that are currently being employed in earthquake- and hurricane-affected regions (see, for example, Adams et al. 2004a, Gusella et al. 2005, Huyck et al. 2004a, Woo et al. 2005). The accuracy of the region-wide inundation mapping and debris line analysis will also be assessed once an independent dataset becomes available depicting the location and extent of inundated areas. Results from the research presented here will be integrated into the overarching project to investigate the relationship between coastal ecosystem degradation/health and tsunami loss. Preliminary findings from this study (Chang et al. 2006) suggest that the presence of healthy mangroves afforded significant tsunami protection to urban areas.

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Lesson Learnt and Implemented Actions After 2002 Molise-Puglia Earthquake

A. Goretti and G. Di Pasquale

1 Introduction

Action plans for seismic risk reduction are formulated within the open ended process of the Emergency Management Cycle. The four phases of the cycle include preparedness, response, recovery and mitigation. Preparedness is planning how to respond in case of emergency and working to increase the resources available to wage an effective response effort. Response is performed during and immediately following a disaster and is it aimed to save lives, minimise property damage and facilitate the beginning of recovery from the incident. Recovery follows the response and continues until all systems return to normal, or near normal, conditions. Mitigation refers to activities which actually eliminate or reduce the chance of occurrence or the effects of a disaster (Galanti et al. 2005).

The occurrence of an earthquake is so sudden and the impact on the victims so devastating that seismic risk can not be reduced relying only on preparedness, response and recovery. Public authorities in seismic-prone countries are typically responsible for protecting public safety and coordinating emergency response and reconstruction. Due to massive financial investments, to long term benefits and to the region or nation-wide social protection, public authorities are also responsible for assessing and mitigating public earthquake risk.

In spite of this, resources are seldom allocated to implement policies and strategies for seismic risk mitigation, while, according the do-nothing-until-it-happens approach, resources are used for peace-time necessities.

This is the main reason why significant innovations and nation-wide action plans for seismic risk mitigation have only been implemented after destructive earthquakes, when society is devastated and the risk highly perceived.

In the past Italy acted predominantly in this way, although significant experience on earthquake recovery dated back several centuries (Goretti and Di Pasquale 2002). In order to understand the innovations introduced after

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2002 Molise-Puglia earthquake, an overview of last century developments will be briefly reported. Further details are reported in Decanini et al. (2004) and Di Pasquale et al. (1999).

Before Italy unification several codes for post-earthquake reconstruction were issued. In 1783 a code was issued by the King of two Sicilies after a destructive sequence in Calabria, while in 1859 another one was issued by the Pope after the Norcia earthquake.

After the 1908 Messina earthquake (83,000 victims, $M_s = 7.3$), the seismic zonation was introduced for the first time in the unified Italy. Just one seismic zone, including Calabria and part of Sicily, was considered. Shear coefficient was assumed about 0.1. At the same time, the building seismic code was also issued addressing both the repair of the damaged buildings and the construction of the new ones.

After the 1915 Avezzano earthquake (30,000 victims, $M_s = 7.0$) Central Italy was included in the seismic zonation. An additional seismic zone was established. South Italy was then classified as zone I, while Central Italy as zone II. Shear coefficients were respectively assumed equal to 0.1 and 0.07.

In 1975 the seismic force profile was considered linear along the height of the building. Up to that time it was considered almost constant. At the same time, the design response spectrum was introduced.

After 1976 Friuli earthquake (989 victims, $M_s = 6.5$) the retrofitting of masonry building and the restoration of the cultural heritage, mainly churches, were addressed. The area stricken by the earthquake, in northern Italy, was introduced in the seismic zonation. In the subsequent years a wide research project, named Progetto Finalizzato Geodinamica (PFG), was launched in order to provide a new seismic zonation based on the analysis of all the historical data, and not on the only earthquakes occurred since 1908.

After 1980 Irpinia earthquake (3000 victims, $M_s = 6.9$) the new zonation based on the PFG hazard results was put in force, with several decrees issued from 1980 to 1984. A moderate seismicity zone III was also introduced with shear coefficient equal to 0.04. However this zone was limited to part of the territories stricken by the 1980 earthquake (99 municipalities).

In 1996 the displacement check was introduced in the building seismic code in order to limit the economic and/or functional damage associated to earthquakes. This was one of the few significant modification implemented in absence of any destructive earthquake. However the Italian seismic code remained unnecessary misleading and unclear. In particular the shear coefficient assumed in the code was based on an implicit behaviour factor of about 4.5 for most of the structural types, without enforcing adequate detailing to guarantee this value. Similarly, poor information were given on the assessment, upgrade or retrofit of existing buildings, and provisions issued for the reconstruction after the 1976 and 1980 earthquakes were used also for the peace-time intervention design.

On the other hand, several revisions of the seismic code and seismic zonation were proposed in Italy in that years. However, they were not enforced, mainly due to the absence of destructive earthquakes. Among them the seismic code proposal in the 80's and the seismic zonation proposal in 1998. The latter one included all the advances in the field of engineering seismology of the previous almost 20 years (the ongoing zonation dated back to 1980 Irpinia earthquake). Most of the Italian territory not yet included in seismic areas was proposed to be inserted in seismic zone II or III, while the most hazardous seismic zone I was proposed to remain almost unchanged. As an example, according to the 1980 zonation, S. Giuliano di Puglia was not included in seismic area, thus allowing to design buildings against only gravity loads, while according to the 1998 proposal it was included in zone II, where shear coefficient was equal to 0.07 (corresponding to design PGA equal to 0.25 g). Surprisingly, the proposed zonation remained unattended and without any valid alternative.

In this framework of advances in research of seismic code and seismic zonation, but stagnation of innovation into practice, Molise-Puglia 2002 earthquake occurred. Despite the moderate magnitude, $M_W = 5.7$, the impact was severe with 30 victims, including 27 children and 1 teacher in the primary school collapse, and 173 serious injuries. Suddenly the public opinion and the government realised that an immediate change of policies was needed to guarantee the building seismic safety. This was particularly important in case of strategic buildings (buildings expected to be used in case of emergency) or buildings with relevant consequences in case of collapse, such as schools. A process of revision of both the seismic building code and the seismic zonation started, together with a national plan for the assessment and retrofit of strategic and relevant buildings, infrastructures and cultural heritage.

In the paper the impact and the emergency management of the 2002 Molise-Puglia earthquake are at the first summarised. Lesson learnt are then highlighted and discussed, to end with the action plan implemented to mitigate seismic risk in Italy.

2 Impact and Emergency Management of Molise-Puglia 2002 Earthquake

The 2002 Molise-Puglia earthquake received widespread attention in Italy (Galanti et al. 2004) and abroad (Foster and Kodama 2004) because of its devastating impact on the residents of the sparsely populated regions of Molise and Puglia. The Molise earthquake sequence began at 11:32 on October 31, 2002, with an Mw 5.7 shock. The earthquake affected an area of about 1700 Km^2 with a population of 370,000. Overall, 91 municipalities were stricken by the earthquake, including 65 in the province of Campobasso and 26 in the province of Foggia. Most of the municipalities were agrarian-based villages and towns: 30% of them have less than 1,000 inhabitants and 60% of them have less than 2,000 inhabitants (Galanti et al. 2004).

The extent of damage varied among the impacted municipalities, with most structural damages, all 30 fatalities, and 61 of the total 173 serious injuries sustained immediately following the initial shock in the small village of San Giuliano di Puglia (population 1,163) (Fig. 1). In addition to the 27 children and one teacher died in the collapsed school, two elderly women were killed in building collapses a few blocks away from the school.

At 16:08 on November 1, an Mw 5.7 aftershock significantly worsened the existing damage, causing additional homeless and anxiety in the population and in the SAR teams. In San Giuliano, the aftershock resulted in the mayor ordering the evacuation and closure of the entire town. In addition to the main shocks, around 200 smaller shocks (M 2.5–3.5) occurred over the first 60 hours following the initial event (DPC 2002).

S. Giuliano suffered damages corresponding to macroseismic intensity I = VIII-IX MCS. The towns surrounding San Giuliano also suffered varying degrees of damage, but with lower felt intensities (I = V-VI to VII MCS), no fatalities, and relatively few injuries.

The damage pattern in the area depended mainly on the distance from the epicentre and on the specific geological, geotechnical and morphological conditions. It was also influenced by the fact that most of the affected villages were not inserted in seismic areas, allowing, thus, the construction of economical, home-made, buildings and the use of materials and detailing inadequate for seismic areas.

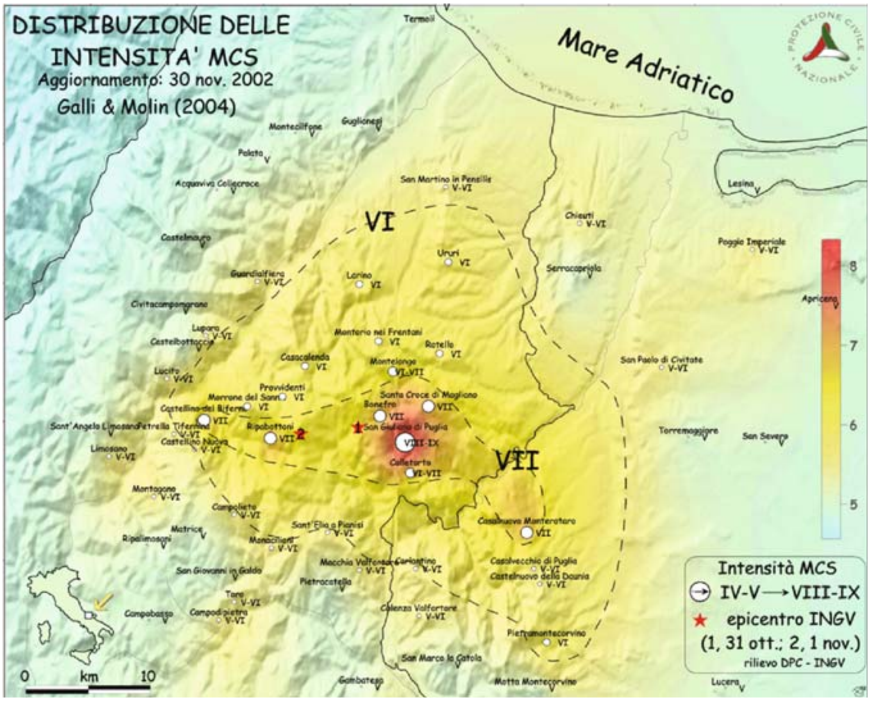

Fig. 1 2002 Molise-Puglia earthquake felt intensities (MCS scale, modified from Galli and Molin, 2004)

Villages dated back several centuries and were located for defensive purposes on hilltops. Many buildings were constructed on steep slopes, with the down-slope side often more than one story greater than the up-slope side. The historical cores of most of the affected villages were located on the firm soils characteristic of such promontories, as in S. Giuliano and Bonefro. More recent buildings were located on degraded shale or soft clay deposits that strongly amplified the seismic waves.

About 80% of the damaged buildings were masonry buildings (Goretti and Di Pasquale 2004). Main vulnerabilities were found in the poor quality of the stone masonry walls, built in an irregular stonework with little engagement between elements, in the vertical bearing walls made of two unconnected portions, in the degraded mortar, in the timber floors and roofs, often thrusting and not anchored to the masonry walls, and in the lacking of iron ties. The layout of the filed stones varies from regular to irregular. Dimensions of the stones varied up to 50–60 cm. In cases of larger stones, small stones were placed in between. In large portion of the walls, mortar was often lacking.

In the 20th century expansion area, buildings were originally one story high, and constructed in field or hewn stone masonry. One or two stories have been added in the '60–'70. Quite all available materials have been used: hollow bricks, perforated bricks with circular hollows called "occhialoni", full bricks. New floors and roofs were usually made of RC and hollow bricks. These buildings had suffered major damages for the presence of rigid and heavy floors placed on vertical, brittle, non resistant, bearing walls. Peculiar of masonry buildings was also the pre-existent damage of the oldest buildings, due to scarce maintenance and abandon.

In the most vulnerable buildings, failure modes due to the break up of the masonry walls prevailed (Fig. 2a). The failure mode originated in the outer portion of the facade, with expulsion of stones, and it was emphasised when the walls were constructed with two adjacent leaves of stones.

When orthogonal walls were scarcely and ineffectively linked together, the out-of-plane component of the seismic action caused vertical or subvertical cracks in the corner regions and, sometimes, masonry displacement, evidencing separation of perpendicular walls and onset of overturning mechanisms (Fig. 2b).

In buildings where the out-of-plane mechanisms were not prevailing, that is in case of effective connection among walls and floors, the observed damage was often related to inclined or X-shaped cracks, primarily due to in-plane actions (Fig. 2c).

The in plane failure mode has been observed in stone masonry, hollow bricks masonry, concrete blocks masonry and full bricks masonry. It has been facilitate by some incorrect details as chimney flue or wide opening, especially at 1st story. In the most damaged buildings, diagonal cracks were spread over all the masonry walls with large sliding.

Not so frequent, but anyway observed, the sliding failure mode characterised by a rigid sliding of the upper portion of the building with respect to the lower

Fig. 2 Observed masonry buildings failure modes: (**a**) wall separation, (**b**) out of plane wall overturning, (**c**) in plane wall shear failure, (**d**) roof collapse

part. The sliding usually localised where discontinuity of stiffness or strength was found.

The emergency response phase began immediately following the initial Mw 5.7 shock on October 31 and continued for nearly three months.

Due to the agrarian-based development of the affected area and to the social impact of the earthquake, it was impossible to address the event with the only resources of the stricken Regions. Consequently, a tiered response was implemented and all of the country's resources were made available. National Civil Protection Department (DPC) promoted an integration and coordination between all response groups and the Italian standardized emergency response structure was employed. Automatic response protocols were implemented by DPC and essential service personnel were assigned at every level of the emergency organisation. Ambulances were dispatched to the affected area and local hospitals prepared to receive the injured. Fire Brigades initiated search and rescue and emergency shoring operations, State Police secured access into the impact area for emergency vehicles, and municipal technicians began damage inspections.

An estimated 3,100 emergency responders and volunteers were available in the affected areas within 24 hours of the event. The total number of response personnel peaked at almost 5,000 between the first and second week, and gradually declined thereafter (DPC 2002).

Response agencies included military personnel (Army, Air Force and Naval Force) who distributed emergency supply containers and pharmaceuticals, provided generators, lighting towers, and heavy machinery, and assisted the Italian Red Cross in setting up outdoor emergency shelters. Police agencies (Carabinieri, Police and Customer Services) provided traffic and access control and escorted heavy equipment into impacted towns. Rangers from the Corpo Forestale Italiano assisted police agencies, while road maintenance technicians (Ente Nazionale Strade – ANAS), inspected roads, and bridges. The Italian Red Cross established and managed emergency shelters and a field hospital. Voluntaries Misericordie provided advanced mobile medical units, and amateur radio operators provided supplemental communications. Within the first two months, about 140,000 man-days were employed, with an average of 1,270 and peaks that occurred in the week from November 2 to November 8 (DPC 2002).

Immediately following the main shock, local technicians conducted initial safety inspections to determine building usability and to assess damage. Requests for assessment of damaged structures were made by residents to the local building department. It quickly became apparent that responding to individual requests in the epicentral area was not efficient and an exhaustive survey was performed in this area.

An estimated 23,000 requests for structural damage inspections were received. Fourteen percent of the total number of public buildings and 23% of private buildings were evaluated unsafe and then unusable (Fig. 3). Another 5% required re-inspection before accessibility could be determined. Inaccessibility was due either to direct damage or to risk of collapse from an adjacent structure (Goretti and Di Pasquale 2004). Inspection requests and building inspections started to slow down after 3–4 weeks. The average number of daily inspections per team turned out to be 8.6 inspections/day at the beginning

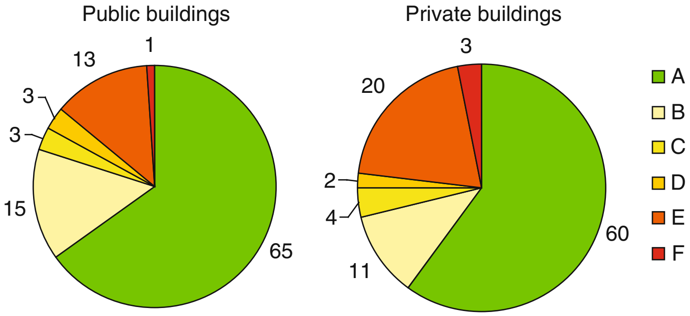

Fig. 3 Percent of unusable public and private buildings. A = Safe and habitable, B = Partially or totally uninhabitable, but habitable after short term counter measures, C = Partially habitable, D = To be re-inspected, E = Unsafe, uninhabitable, F = Unsafe, uninhabitable due to risk of collapse of adjacent structure(s)

of the survey and 6.5 at the end of the survey. This reflects the great difficulty in performing the very last inspections.

Similar activities were carried out in several villages in Puglia, where about 4,000 buildings were inspected.

Due to the consequences of the collapse of the primary school in San Giuliano di Puglia, all the schools in the Molise Region received widespread attention. All schools were closed following the earthquake for damage inspections and seismic evaluations, and remained closed for more than two weeks. Of the schools evaluated, 80% were found to have little or no damage (Augenti et al. 2004).

People that lived in buildings classified unsafe were not able to return in their home and were accommodated in emergency shelters. The number of shelter seekers increased from an estimated 1,100 on October 31, to more than 3,000 on November 1 and rose to almost 12,000 by the end of the first week. Emergency shelter operations continued for more than two months, with the last shelter closing on January 15, 2003 (Galanti et al. 2004). A total of 535 prefabricated homes were provided, 268 of which were constructed in the temporary village in San Giuliano, and the remainder constructed in numerous villages throughout the damaged region. The temporary village, comprised of a primary school, town hall, market, pharmacy, and social centre, was located approximately one kilometre from the damaged town in a less vulnerable site.

Local authorities in the stricken area were not used and prepared for emergency management. Twinning among municipalities stricken by the earthquake and outside regions were then established (Fig. 4). Twinning proved to be very efficient, as trained personnel from outside regions helped local authorities to manage all the emergency operations: from sheltering and assistance to the population to building safety inspections.

Larino primary COM was deactivated on January 24, 2003, but Regional personnel continued to operate there for several months.

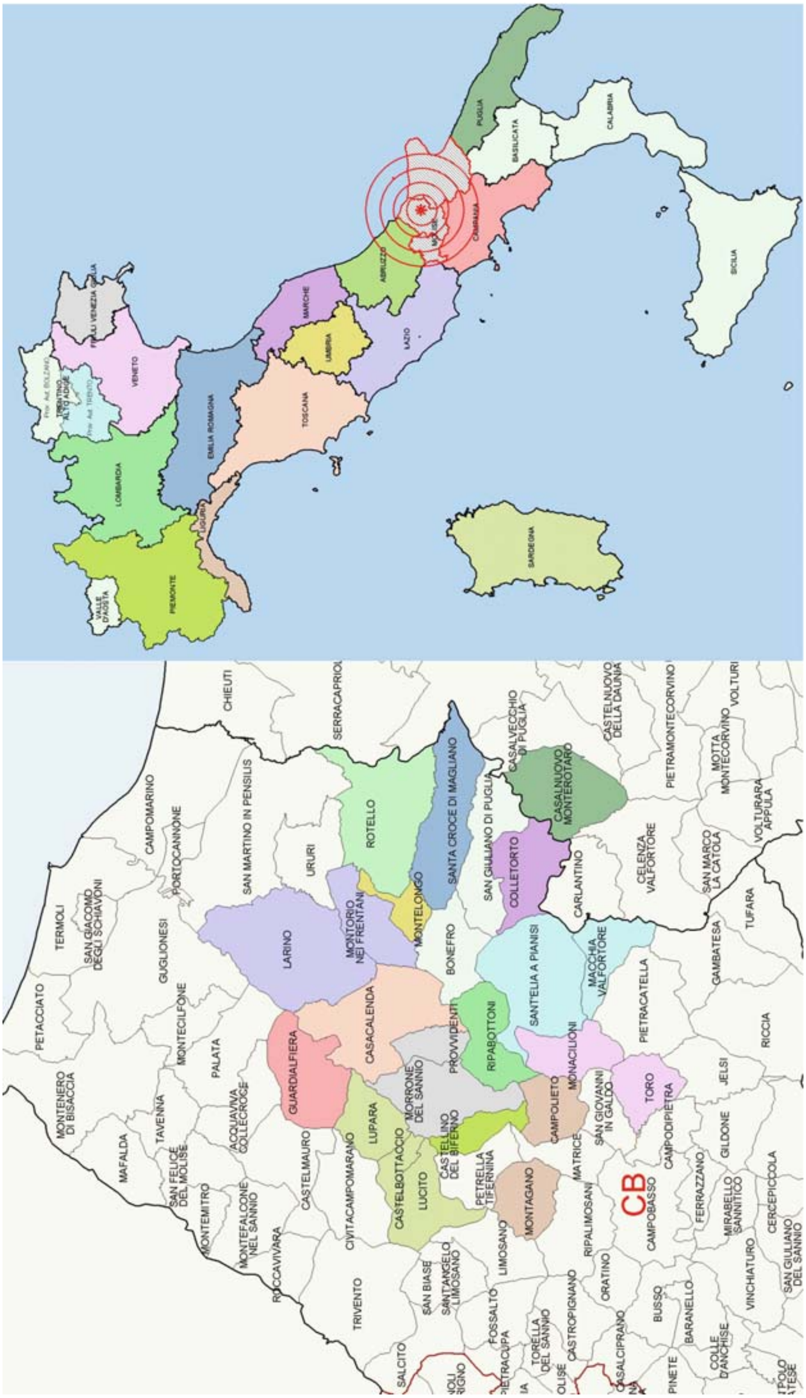

Fig. 4 Regions and municipalities with the same colour established twinning on all the operations of the emergency management

2.1 Lesson Learnt

Due to its devastating impact, the Molise-Puglia earthquake represented a moment of thought that made easier a positive criticism of all the, at that time, ongoing activities on seismic risk reduction: mitigation, preparedness, response and recovery. Several lesson were learnt. They are briefly reported in the following.

In the area earthquakes occurred several centuries ago. After the 1456 earthquake, damages were reported in many villages, but most severe effects were reported after the 1627 earthquake. In 1805 earthquake seismic felt intensity was at the damage onset. No destructive earthquake occurred in the area.

Lesson learnt is to beware of areas with rare events. People forget easily the consequences of seismic events, making difficult developing any local seismic culture in the area.

The historical core of S. Giuliano was built on rock, while the expansion area, where most of the damages are concentrated, together with the collapsed school, were built on soft clay soil. If the expansion area would have been on stiffer soil, the disaster would have not been like that. Lesson learnt is that the disaster has been increased by incorrect land use. The fact that in the past no destructive earthquake was reported in the area is also due to the fact that villages were, once, built on rock and, hence, shaking was less severe.

Since 1998 it was known that the affected area should have been classified as a moderate seismicity area. However involved national institutions did not revise the seismic classification. Lesson learnt is that institutions have to act according to the latest available and consolidated information. If they do not act at national level, it is impossible to ask local authorities for acting at local level. There was an enormous gap between scientific knowledge and transfer of this knowledge into practice.

The Italian building code was obsolete and unclear. The value of the behaviour factor was implicitly assumed in the code, the necessary seismic detailing was only partially specified, and not in the code itself but in a not enforced addendum. The capacity design was not considered. It was probably the most obsolete code among developed, seismic prone, countries. Lesson learnt is the same as before, that is institutions have to act according to latest available and consolidated information. In addition any attempt to keep the code unchanged, to facilitate design and construction, should only be permitted to a limited extent.

Due to the obsolescence of both seismic code and seismic zonation, buildings constructed in the past are not safe enough. Lesson learnt is that there is an urgent need of seismic assessment of buildings and infrastructures with a vital role in emergency, such as strategic buildings, or with relevant consequences in case of collapse, such as schools, churches, etc.

Due to the absence of any seismic awareness, local authorities were not ready to manage seismic emergency. The relief model used by National Civil Protection had to be paternalistic, telling local authorities what to do and how to do. Lesson learnt is that local authorities should not be left alone in emergency preparedness, but central authorities should support and periodically control local authorities preparedness.

Beside the above list of far-reaching lessons, other lesson were learnt on several aspect of seismic risk reduction. As an example, lesson learnt on the technical emergency management, that is building post-earthquake safety inspection, are reported in the following.

During the safety assessment, sometimes occurred that forms different from the official one were used by local authorities. Although it sped up the survey, it also posed difficulties in the harmonization and computerisation of results. Lesson learnt is that the process of homogenisation of procedures and forms, initiated in 1997, should continue all over Italy.

In epicentral areas an exhaustive survey turned out to be very effective. Although it required a slightly greater effort, it considerably reduced the time needed to identify buildings and to access buildings in case the owners evacuated their houses. Lesson learnt is to explore the feasibility of an exhaustive survey also in case of destructive earthquake or in densely populated areas.

During the inspection, it turned out that the management of the safety inspections should be better performed at local level then at central level. It certainly requires more personnel, but, at the same time, it may allow an immediate data validation and the establishment of maps or GIS systems. Lesson learnt is to explore new IT solutions for inspection management, such as palm computers directly connected to GIS.

To increase post-earthquake safety assessment efficiency, twinning with other Italian Regions proved to be very effective, as trained personnel from outside regions helped local authorities to manage all the emergency operations. Lesson learnt is to establish a national protocol for Regional intervention in case of future events.

Inspectors training on post-earthquake safety assessment is essential for a common basis language and hence for a reliable judgment on the building safety. Lesson learnt is that, during the emergency, a permanent staff should be devoted to training. Moreover, the existing training program for public servants should be improved and extended to professionals.

The resources needed, during the emergency, to inspect building, manage inspections, computerise forms, train inspectors were approximately in the range of 1 working day for every 2.3 inspected buildings. Lesson learnt is that procedures have to be revised in order to increase this performance.

2.2 Implemented Actions

Several actions, based on the lessons learnt after the 2002 Molise earthquake, have been implemented in recent years to reduce the impact of future seismic events. They will be summarised in the following. As already discussed, in the area stricken by the 2002 Molise earthquake buildings were not designed according to the seismic code, because the area was not inserted in the national seismic zonation. However hazard studies, performed several years before, showed that accelerations of the order of 0.2 g were expected with a return period of 475 years. This unfavourable situation was mainly originated by the fact that scientific knowledge was not put into practice. The Italian Government established then a Working Group aimed to the revision of both the seismic zonation and the seismic code. The proposals were prepared in a very short time and on March 20, 2003, the Prime Minister issued the by-law n. 3274 concerning “Initial items on the general criteria for seismic zonation of the national territory and on seismic code”. This was a quite unusual procedure, since in the previous 30 years the seismic zonation and the code were updated with a decree of the Minister of Public Works. In recent years, as results of the decentralisation process, the State had to provide the general criteria for the zonation, while Regions had to apply these criteria to establish the seismicity of each municipality. Since this procedure takes normally a long time, the Government decided to put into actions the new code and the new seismic zonation with the by-law procedure. This is a very quick procedure usually adopted for emergency measures, typical of civil protection actions after disasters. This unusual procedure caused several institutional difficulties, although the by-low title clearly stressed the temporary character of the act and the intended progressive return to the standard procedure.

2.2.1 Seismic Zonation

For what concerns the seismic zonation, the Prime Minister by-law n. 3274:

- adopts a new, temporary, seismic zonation of the national territory, using the results of a Working Group established in 1998;
- establishes the general criteria for seismic zonation and requires that Regions provide the final zonation on the basis of these general criteria;
- provides a new seismic hazard map that may be used for future zonations. The new map is based on well established methodologies and up to date data.

After the promulgation of the by-law, all the Italian municipalities were inserted in the seismic zonation, with different seismicity grades: from Zone 1 (highest hazard, PGA = 0.35 g) to Zone 4 (very low hazard, PGA = 0.05 g).

In Fig. 5, the evolution of the Italian classification is shown. It can be seen that significant revisions of the seismic zonation occurred only after great disasters, when the society is devastated and the risk highly perceived.

In Fig. 6, the cross classification before and after the by-law 3274 is presented. The number of municipalities inserted in the most risky Zone 1 almost doubled, while the number of municipalities in Zone 2 remained almost unchanged, although the municipalities were not the same. More than 20% of the Italian municipalities were inserted in Zone 3. Eighty five municipalities that were not previously inserted in seismic areas, were inserted in Zone 2. Few

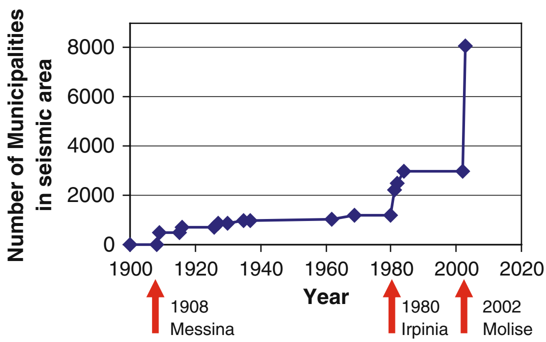

Fig. 5 Number of municipalities where buildings have to be designed against seismic loads versus time. Arrows indicate seismic disasters that facilitated significant revisions of the seismic zonation. Total number of Italian municipalities is 8100

1984\2003	Zone 1	Zone 2	Zone 3	Zone 4	
I cat	368	-	-	-	368
II cat	348	2151	-	-	2499
III cat	-	88	11	-	99
NC	-	85	1623	3427	5135
	716	2324	1634	3427	

Fig. 6 Number of municipalities in seismic area before and after the by-low 3274/2003

months later the promulgation, almost all the Regions adopted, with slight modifications, this new seismic zonation.

2.2.2 Seismic Code

After 2002 Molise earthquake, the Government decided to also revise the seismic code. The new code was included in the same by-law 3274. In more detail, in the new code:

- the seismic action is described in terms of elastic response spectra and in terms of peak ground acceleration corresponding to selected probability of exceedance;
- the effects of the soil amplifications are considered in the spectrum shape and intensity;
- the expected performances of the structure are clearly stated;
- the influence of structure characteristics, such as geometry, regularity and detailing, on the overall ductility are clarified;
- the influence of the ductility on the design spectrum is specified;
- rules to consider brittle and ductile failures of components are given;
- the capacity design is introduced;
- innovative intervention strategies, such as seismic isolation and energy absorption, are considered in the code;
- linear and non linear analysis may be used;
- proper attention is devoted to existing buildings, for which the overall ductility and the performance of brittle elements is much more uncertain.

A relevant part of the new code is devoted to existing structures: it includes the knowledge of geometry, material and detailing, the assessment methodology and the design of upgrade or retrofitting intervention. Attention is also paid to the assessment of historical buildings, which are numerous all over Italy.

Such important changes could not have been enforced abruptly. Hence the law allowed the new seismic code to be used on a voluntary basis for a 18 months period. During this period, engineers, state and local officers and software developers had time to apply it and familiarize with it. Furthermore in the same period, a special Committee was instituted to monitor the new code and to suggest any possible revision. As a matter of fact the Committee received contributions from various representatives of research centres, professionals, associations of material producers, associations of constructors, Regions, offices deputed to check the design, etc. After an intensive work, a revised version of the code was issued several months later (Decree of the Prime Minister n. 3431, 2005).

The new code was disseminated in many congresses and workshops, and a specific training program was launched in cooperation with the professional associations of engineers and architects.

More recently (September 2005), a unified version of all the technical codes was issued by the Ministry of Infrastructure, under the agreement of the National Civil Protection. In this code, which dictates the general principles of the structural design, the new seismic code contained in the by-law 3274 is one of the codes that may be used to design new buildings or to assess existing buildings.

2.2.3 Existing Structures

The benefits of a new seismic zonation and seismic code develop slowly along time because only new constructions, or retrofitted ones, will be safer. In Italy many buildings date back to several centuries ago, most of them with an important cultural heritage. Moreover, for long time municipalities were inserted in seismic areas only after an earthquake occurred there. Only in the 80s the zonation was revised on the basis of an hazard study. For these reasons most of the constructions have been designed for gravity loads only. Today only 1 building out of 4 is seismically designed. Among the unsafe buildings, there are strategic buildings (hospitals, municipalities, fire brigade stations, army headquarters, etc.) and buildings relevant for the possible consequences of their collapse (schools, theatres, stations, airports…). According to the new seismic code, new strategic buildings should be operational after an earthquake and new relevant buildings are supposed not to represent a risk for life. However, due to the lack of aseismic provisions, most of them probably do not guarantee the aforesaid performances. So, in the same by-law 3274, the Government has promoted the seismic assessment of strategic and relevant constructions, with priority for those in seismic zones 1 and 2. It is not possible to estimate the total number of construction to be assessed, because private

Table 1 Number of structures and cost of the assessment and retrofitting plan

	Total	Assessment (%)	Retrofit (%)
Number	6,718	97.0	3.0
Costs	380 M€	38.1	61.9

buildings having mixed occupancy are also included. For buildings, a lower bound estimate is about 70,000. The assessment has to be performed with an engineering evaluation, so it is a very important and demanding task to be completed in five years. Many Regions, State Organizations and also private owners have started to assess their estate, but activities are slowed down by the large amount of funds needed. Other two by-laws have been then promulgated, to provide financial contributions for the seismic evaluation and the retrofitting of strategic or important public constructions. Two hundred millions of Euros have been funded in the years 2004–2005, about two third devoted to Regions, one third to State Organization. The Government contribution ranged between 30% and 50% of the conventional cost of the assessment, and up to 60% of the conventional cost of intervention, depending on the seismic zonation of the municipality where the structure to be assessed or retrofitted was located. The owners had to provide the remaining costs.

Table 1 shows that 97% of the structures included in the plan are to be assessed and only a small percentage are going to be retrofitted. On the contrary most of financial resources are devoted to the structure retrofit (61.9%). This is due to the fact that, on average, the cost of one retrofit corresponds to the cost of about 50 seismic assessments.

In Table 2 the structures to be assessed are grouped according both to the Region or State use and to type of use. State structures are mainly bridges and army headquarters and barracks, while Region structures are mainly schools and secondly Town Halls and Hospitals.

At the same time, another national plan aimed to the seismic safety of the Italian Schools has been established. About 40,000 public schools exist in Italy

Table 2 Percentage of structures included in the assessment plan, grouped by the following categories: A = Schools, B = Town Halls, C = Health, D = Headquarters, E = Infrastructures, F = Churches and Cultural Heritage, G = Other and differentiated between State and Regions

State	A	B	C	D	E	F	G	Total
Number	0.0	0.0	6.6	53.5	33.9	0.0	6.1	1,603
Volume	0.0	0.0	16.9	68.8	1.7	0.0	12.7	28 Mmc
Cost	0.0	0.0	8.4	56.8	29.3	0.0	5.6	69 M€
Regions	A	B	C	D	E	F	G	Total
Number	50.8	23.2	6.1	0.9	11.0	5.8	2.1	4,913
Volume	50.4	16.5	25.7	0.6	0.1	3.7	3.0	33 Mmc
Cost	53.5	18.9	16.2	0.7	3.8	4.3	2.7	75 M€

and the cost of retrofitting is expected to be extremely high, in the order of several billions of Euros. An initial plan for the upgrading or retrofitting of the most risky schools has been funded with about 200 millions of euros. Other 300 millions have been allocated very recently.

2.2.4 Scientific Research

After the Molise earthquake, the attention to seismic risk has raised considerably, and the research activities in the field have experienced a new vigour. The by-law 3274 has also assigned funds to create a network of laboratories of seismic engineering (Reluis, Rete dei Laboratory Universitari di Ingegneria Sismica, www.reluis.it) and a research centre (Eucentre: European Centre for training and Research in Earthquake Engineering, http://www.eucentre.it) aimed to support the progress in knowledge and the transfer of scientific results in best practices, guidelines, pre-normative documents and other tools for the reduction of seismic risk.

2.2.5 Emergency Training

The Molise earthquake has confirmed that it is essential, in order to effectively manage the emergency, to have a robust understanding on the activities to carry out and on the responsibilities to take immediately after the event. To have an efficient response, an accurate planning and frequent practices are both necessary. Several training courses for technical personnel involved in post-earthquake inspections have been performed and will be performed in the next future. An important international exercise was hold in Sicily, simulating the response to a magnitude 6.8 earthquake off the coast of Catania. All the organizations contributing to the National Civil Protection System (Department of Civil Protection, Fire Brigades, Army, Navy, Air force, Red Cross, Regions, Provinces, Municipalities, etc.), international teams and international observers participated to the exercise to test all the response activities: search and rescue, triage, transport of injured, damage assessment, communications. In order to support the exercise, an upgrade of the model for loss simulation in Italy (Di Pasquale et al. 2004) was tested.

3 Conclusions

Lessons learnt and implemented actions after the 2002 Molise earthquake have been summarized. The earthquake by itself was not so strong: in the last 30 years several destructive earthquakes in Italy caused much more damages and victims. Nevertheless the death of 27 classmates and of their teacher, in a zone not considered before as seismic, made clear, also to the wide public opinion, that the safety conditions of many buildings and schools were inadequate and that a wide gap existed between scientific knowledge and implemented actions for seismic mitigation. The earthquake was also a further test of the Italian emergency management system, characterized by a flexible system composed by many bodies (army, regions, municipalities, volunteers, red cross, etc.) under the central coordination of the National Civil Protection. The dynamically allocated functions in the emergency coordination centres and the coordinated cooperation of all the bodies involved proved to be efficient. The need of further training actions at local level on emergency management was highlighted, mainly in the zones where seismic awareness and seismic culture was lacking.

The twinning among municipalities stricken by the earthquake and outside regions proved to be efficient, as trained personnel from outside regions helped local authorities to manage all the emergency operations.

It was evident, as many times in the past, that, after a destructive earthquake, attention to seismic risk raises sharply. The inadequacy of prevention actions was then perceived by the public opinion, and made possible important changes in seismic codes and seismic zonation. This produced initially difficulties in the application of the new rules, but also an extraordinary effort to disseminate the new design concepts that involved professionals, scientific and administrative structures. Risk knowledge has to be accompanied by tangible actions for its reduction: national plans to assess the actual safety of important structures and to retrofit them have been started. All these efforts will certainly raise seismic awareness, and will bring to a safer building stock.

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The Next 1755 – Myth and Reality; Priorities and Actions to Develop in Case of an Earthquake in the Lisbon Metropolitan Area

João Azevedo, Sandra Serrano and Carlos S. Oliveira

1 Introduction

The November 1st, 1755 earthquake, shook violently not only the city of Lisbon, great part of the Portuguese territory and a vast neighbouring geographic zone (Oliveira 2008), but also shook deeply the philosophical thoughts of that time, released fears and apprehension and, at least for some time, the seismic phenomena and their causes and consequences were part of the worries and concerns of the European society. A collapsed Lisbon emerged from the earthquake induced shaking, was shortly after hit by a tsunami with the water invading the Tagus valley and, finally, was further destroyed by the fire (Fig. 1). All powers of nature seemed to reunite to vanish one of the largest and most majestic European cities. At that time, not only in Portugal but also in whole Europe, explanations were advanced for the unfortunate event. Several texts were published referring to the earthquake, most of them being of religious or poetic nature. But, for the first time, there were some methodical reflections about the possible causes of earthquakes and some actions were taken to analyze the effects of earthquakes and implement measures that could diminish their impact. The 1755 earthquake is thus associated, at least in what regards the so called occidental world, to the rising of a new era of thinking and also to a new way to approach the issues related with earthquakes, in what can be viewed as the launching of organized research efforts in the domains of seismology and earthquake engineering.

The rebuilding of Lisbon, following the complete destruction of large areas, constituted a huge intervention that still imprints its characteristics in today's city. It was, historically, a great achievement in terms of urban planning, architectural development and engineering design and construction.

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Fig. 1 November 1st, 1755, Lisbon earthquake, tsunami and fire. Detail of a German engraving – Museu da Cidade, Lisbon

The destruction caused by the earthquake and the fact that for a large period people were frightened by many aftershocks developed the grounds for good reconstruction procedures.

In the past, some other strong earthquakes with probable source similar to the 1755 earthquake struck the Portuguese continental territory, such as in 1356. Others that affected the Lisbon region occurred in 1344, 1531 and 1909. The historical data shows that the zone, although not of very high seismic risk as compared to other zones in the world, is prone to large earthquakes that also have large return periods. These very large return periods make that some generations, as is the case of the present one, do not pass by any significant seismic occurrence and that the collective perception of the seismic risk is reduced. The 1755 earthquake becomes, thus, the most significant and almost the sole important historical reference, justifying why it is, in the contemporary Portugal, a ghost scenario that at the same time disturbs consciences and encourages motivations and actions (Pereira de Sousa 1919).

2 The Memory of 1755

Time dilutes man's consciousness of risk and that is the case regarding the evolution of the Portuguese perception of the seismic risk following the 1755 earthquake. For a large period the quality of construction improved, but, unfortunately, with the passing time the concerns with structural safety and construction quality decreased.

Following the 1909 Benavente earthquake, with the development of seismology and structural engineering and with an international awareness of the devastating effects of earthquakes, some relevant Portuguese seismologists and engineers conducted several scientific studies about the 1755 earthquake. In 1955, in commemoration of the 200 years of the earthquake and organized by the "Commission for Bridges and Structures" of the Portuguese Engineers Association (Ordem dos Engenheiros) a "Symposium on Seismic Actions" was held to debate measures and actions that should be adopted to reduce the effects of earthquakes. The final conclusions of this symposium (Ordem dos Engenheiros 1955) were an alert to the need of more and better attention to the seismic phenomena and included objectives and ideas that, albeit 50 years have passed, are still sensible and sound:

– Portuguese regions present high seismicity. The work undertaken by the seismological branch of the Portuguese Meteorological Service clearly asserts the seismicity of the different regions, but further geophysical studies are needed combining geological and seismological information;
– instruments for seismic data acquisition are necessary, mainly accelerometers to register strong motion, allowing a definition of the seismic intensity but also the assessment of the seismic input to the structures;
– a seismic code relative to seismic resistant structures needs to be urgently adopted, based on already existing knowledge;
– collaboration among the designers and constructors is needed, especially between engineers and architects, to design and build safe structures;
– it is necessary to guarantee not only the resistance of the main structural systems, but also that of accessory elements;
– to diminish the effects of earthquakes it is necessary to take care of civil defense issues: first aid actions, fire defense and safety of transportation and supply systems;
– *in-situ* and laboratory experimental studies are needed to clarify the behaviour of structures and contribute to code improvement and updating;

- design methodologies for seismic resistant structures do not create new problems of structural analysis procedures. The existing procedures are well adapted to the design, needing only development and dissemination;
- some construction processes facilitate the attainment of seismic resistant constructions, in particular the reinforced concrete solutions that allow an adequate resistance at reduced costs;
- it is necessary to introduce and maintain the study of seismology and seismic resistant construction techniques in the schools of engineering and architecture;
- there is a need for a straight collaboration between seismologists, geologists, historians, engineers, architects, other technicians and building contractors, to promote the knowledge about the defense against earthquakes.

Following this symposium some actions were taken, namely, three years later, the introduction of code provisions for seismic design. These were later substituted and complemented by other codes (RSA 1983), both in terms of the seismic action characterization and design codes for the different construction materials (Azevedo 1995). The Eurocodes will soon be enforced, gradually replacing the current codes. One can say that, globally, in terms of non-attained objectives, the need for a good coverage of the country in terms of strong motion instrumentation is still, probably, the most significant.

3 Risk Perception (Sense of Insecurity)

Different studies, assuming different seismic scenarios, have shown that there are some zones of the continental Portuguese territory that are at significant risk. Among those are, for obvious reasons, the great Lisbon area and the populated zones of the south of Portugal. If this sense of insecurity is greatly shared by scientists, researchers and technicians, the same can not be said in terms of the population, at least in general terms. The awareness of risk and the adoption of measures to reduce it is not part of the habits of the Portuguese population, especially if the risk is not correctly known and understood. Even with the knowledge of the “historical” 1755 and with the evidence of dramas in other parts of the world, the “average” continental Portuguese citizen often cogitates that this is a reality of the past, that it happens to the others and not to him and that it is not a risk he should care about. Several causes contribute to this reasoning, the main being: - the fact that in his lifetime he has only felt harmless minor earthquakes; – the sense that, if the risk exists, there is not too much he can do to reduce it; – the believing (hope) that someone has taken care of the problem, because he heard about seismic resistant codes and “anti-seismic” constructions as well as emergency planning. Curiously, this is not the case with other Portuguese citizens from the Azores islands, that, such as people from other countries, like the Japanese or the Californians, have felt strong earthquakes in their lifetime (every 25–30 years there is a significant earthquake that affects one or more islands), know by self experience that earthquakes cause extensive damage at least in non-engineered structures; know that they also have to make emergency planning and that there is much that can be done to reduce the impact of an earthquake.

If a risk is not very well known to the population in general and, consequently, is not assumed as a problem to be solved, one can expect that decision makers and politicians will postpone it in their agenda. In democratic societies, all forms of government are supported by votes and votes are given to those that give more guarantees (or more promises) of solving people's problems. And there are easier and more visible and rewarding problems to be solved. It is thus a mistake to think that steps to diminish seismic risk can be made exclusively or even mainly by lobbying politicians. The "average" politician is also (still) not "aware" of the seismic risk. The conclusion is that if there is seismic risk and if it is to be reduced, one has first to take the following actions towards the common citizen: – make him aware of the risk; – inform him that there are ways to minimize the risk; – convincingly, inform him about the costs/benefits of risk reduction; – give him reliable (and not costly) solutions for seismic risk reduction. All these actions need to be implemented in a comprehensive way, such as with respect to other risks of our contemporary society.

In what concerns seismic risk, one can say that, although some knowledge exists about the possible occurrence of earthquakes, its likelihood is still considered as reduced by the public in general. Consequently, seismic risk reduction is given second priority by public authorities that need to respond to other risks that are considered to be higher, such as, in Portugal, car accidents, human health risks (i.e. AIDS), forest fires, drought, or even, more recently, terrorist attacks. Political will to tackle the situation regarding seismic risk is thus, regardless of the economical situation, conditioned by the overall relative assessment of all risks. Only an unbiased assessment of all risks can adequately define priorities. Ethical principles urge "earthquake people" to make sure that the assessment of seismic risk is not biased in either direction.

4 What is at Risk?

Do the citizens and especially public authorities have a correct perception of the expected consequences of a "new 1755 earthquake" and have they organized adequate planning and identified priorities for action? What is at risk?

One of the undisputed assessments that can be made regarding the seismic risk in Portugal is that, in terms of hazard, 1755 is a repetitive event that sooner or later will take place. This does not mean that other more hazardous events of the same type (distant earthquake scenarios) will not occur and that other unknown or known hazards (as local earthquake scenarios, i.e. 1531) will also not happen. 1755 and past events are thus a minimum boundary in terms of hazard quantification. A quantification that is undoubtedly difficult to make given the limited available information, but surely necessary and deserving a special attention in terms of historical, geological, seismological and engineering studies. In terms of seismic risk it is even more difficult to assess the outcome of a new 1755. But it can definitely be said that it may be devastating. The great Lisbon region has today a population that exceeds three million people, has the larger concentration of industrial facilities in the country and, integrating the Portuguese capital, it also concentrates the largest number of service companies and administrative and governmental bodies. On the other hand, the southern zone of the country, especially the Algarve region, which was also greatly affected in 1755, has a resident population only about one eighth that of the great Lisbon, but has an average population of tourists in high season that almost equals half of the latter. Its importance for the Portuguese economy is very large given that it hosts about half of the tourists that visit Portugal. One can thus say that at near half of the Portuguese population is at high seismic risk and that significantly more than half of the Portuguese economy is at stake. In terms of building stock, Portugal still has many masonry buildings, including the great Lisbon area. Following the 1755 earthquake and for decades, the construction of buildings followed patterns of quality but as already mentioned, with time and without new earthquakes, the construction quality decreased. That is the case of a class of buildings that still exist in a significant number in Lisbon, the "Gaioleiros", so called given that the initial concept of the "Gaiola Pombalina" (a braced timber structure filled up with masonry materials) was theoretically maintained. But as the timber structure was greatly simplified both in terms of quantity and layout of the timber elements, material quality and connections detailing, the system is much less efficient in terms of lateral resistance and ductility. With the advent of the reinforced concrete, buildings started to be constructed with reinforced concrete slabs supported by unreinforced masonry walls and later by a combination of reinforced concrete frames and masonry walls. Also this type of construction has proven to be vulnerable to strong motion.

As previously mentioned, the first code regulations imposing the consideration of seismic loading are dated 1958. Only following the enforcement of the current code, dated 1983, one can say that modern rules for seismic resistant design were introduced. The same cannot be said about the implementation, in practice, of those rules, given that a long existing practice of design verification by public authorities was suspended. Given that more than one third of the great Lisbon area population lives in constructions that had design procedures without any seismic consideration (before 1958) and that a significant part of the other two thirds lives in constructions that either were not designed and/or effectively built according to the current appropriate standards, it is easy to see that a very large number of individuals is at risk. To worsen the situation, it is known that part of the older building stock is degraded, which further weakens its capacity to withstand seismic actions, as is also known that some "rehabilitation" interventions in old buildings ended up removing or damaging part of the existing structures (including some Gaiolas Pombalinas) and introducing new elements that not always are designed to withstand lateral loading. It is also known that in some Lisbon zones (as well as in Algarve), some soils where constructions were built following the 1755 earthquake do not have good soil foundations. What has been said about buildings can not be extrapolated to bridges and viaducts, where a significant part of the existing important constructions have been designed by more experienced designers, already based on the current code provisions and the good practice of design verification was kept at an acceptable level by the public authority for roads. Anyway, there is a series of older bridges and in particular some viaducts in the Lisbon area that are known to not satisfy the current seismic safety standards, albeit having undergone some "lifting" interventions in the recent past. With respect to other infrastructures it is worth mentioning the industrial facilities in general and those that are part of lifeline systems in particular. These are probably the kind of infrastructures whose seismic behaviour is less well known and whose destruction or even malfunction following an earthquake may have a quite large impact in economic and social life. Even because some of these infrastructures were not designed neither by technical staff with knowledge of seismic resistant design, neither following any standards applicable to seismic zones, it is thus urgent to carry out an evaluation of their performance in the presence of earthquake ground motion. Also needing a correct assessment of their seismic behaviour are many structures built with precast elements, both for industrial and commercial facilities. The fact that they are used by a very large number of individuals, that it is known that these structures require special care in terms of connections detailing and finally the fact that in past earthquakes some of these structures have suffered collapses (Oliveira et al. 1995, Burby et al. 1998), makes them especially vulnerable in terms of seismic risk.

5 Priorities for Action – Review of Past Proposals

From time to time, either following the occurrence of an earthquake or motivated by some sort of invigorating meditation, some actions are assumed as a priority by the technical community and to some extent are recognized as needed by the public authorities and governmental bodies. This took place following earthquakes that either occurred nearby Portugal or that had a larger impact due to the media coverage.

Following the 1994 Northridge earthquake, a field trip mission to the Los Angeles area, stated some recommendations (Oliveira et al. 1995) regarding priority actions to be implemented in Portugal, in view of what had occurred in California. These priority actions, that were the result of a careful assessment, can still be considered as actual. They included: - Vulnerability studies for structures of important public buildings – hospitals, schools, emergency services. – Construction quality certification by means of good practice and action of quality assurance bodies - Legislation requiring monitoring of vital structures in the event of a strong motion. – Identification of vital structures of the highway and road network, analysis of its vulnerability and establishment of a retrofitting program - Identification of the seismic vulnerability of lifeline systems as well as infrastructures of collective use and establishment of a retrofitting program - Organization of specialized teams both for immediate and detailed inspection of structures following an earthquake, including the pre-definition of inspection procedures and the creation of legal mechanisms that make possible the inspection and, if needed, evacuation of buildings. - Coordination of human and equipment resources for emergency planning and intervention by the different municipalities of a given region, as is the case of the great Lisbon area.

In particular, the following five points were considered as having emergency character, needing immediate implementation (in parenthesis and in italic are also indicated the significant steps already undertaken): – Assessment of the seismic vulnerability of all important bridges and viaducts in the great Lisbon area, to assure emergency traffic and emergency lines (*A much reduced number of studies was conducted and only minor retrofitting interventions took place in Lisbon. Identification and preliminary vulnerability analysis was carried out in the Lisbon metropolitan area and was initiated for Algarve*). – Establishment of a program for seismic rehabilitation of important, dangerous and special infrastructures, including hospitals, hotels, shopping centers and all types of lifeline systems (*Identification and preliminary vulnerability analysis of lifeline systems was carried out in the great Lisbon area and was initiated and interrupted for Algarve. Recently, studies were conducted for one of Lisbon's main hospitals and some retrofitting criteria were established*) – Instruct personnel and prepare procedures and equipment for building inspection in the event of an earthquake (*intention was announced but never implemented*). – Review the present legislation concerning permission for construction of large or intensive use facilities, subjecting it to a previous assessment of the structural design and the existence of a quality assessment procedure for the design and construction phases. - Commit to the municipal authorities the establishment of disaster scenarios that may serve as a guide for emergency planning (*the Portuguese Authority for Civil Protection – ANPC, developed a study on the seismic vulnerability of the Great Lisbon area. Some municipalities, including the city of Lisbon, have conducted their own studies*).

Ten years after, the outcome is not convincing and what was considered to be at risk is still at risk.

6 Planning Actions to Develop in Preparedness for an Earthquake in the Lisbon Metropolitan Area

Since 1997 ANPC has launched a programme aiming at the development of an earthquake simulator capable of generating earthquake scenarios for different regions of the country. Priority has been given to the regions with higher risk exposition such as the Lisbon Metropolitan Area (AML), around the capital, and the Algarve in the southern part of the country.

The studies carried out by ANPC gathered all the data (having the municipality as the work unit), including for example: estimation of the population mobility for different hours of the day (week, weekends, holidays, etc.); influence of superficial soil layers on the seismic waves amplification as well as the potential for liquefaction and landslide, based on bore-holes available at many locations; and assessment of housing properties based on the building and population Census.

The simulator analyses the different lifelines: roadways; railways, including the subway; energy transportation (electricity, gas, and other combustibles); water; sewage and telecommunications, etc., with the final product carrying out damage assessment to those infrastructures. It also deals with the vital points relevant for emergency management, either due to its operational or political role.

With this knowledge for different scenarios, it is possible to establish guidelines for risk mitigation in general, including emergency preparedness facing a seismic event, in a context of cooperation among different Civil Protection agents and upper-level coordination of elements, as well as definition of policies to retrofit the most vulnerable facilities.

The objectives and the methodology developed for the Emergency Preparedness Planning are further described in the present work.

The AML study is based on a GIS system, containing all the available information on geophysical, geological, housing, important structures, lifelines of all kinds, population, etc. (Coelho et al. 1999) The simulator, for a given event or collection of events (each characterized by a magnitude and an epicentral location and a date, associated to a probability of occurrence) produces information on damage to all different objects under study and on human casualties. Vulnerability assessment was essentially based on the Hazus-99 methodology (NIBS 1977, 1998 and 2002).

6.1 Main Features of the Seismic Simulator

The simulator, which was the result of collaboration between several research institutions (LNEC; IST; FC/UL; CEG/UL; Chiron), (Rocha et al. 2002, Anderson et al. 2004) contains various moduli as referred below.

- The definition of different seismogenic zones affecting the area under study and the characterisation of their most important parameters (maximum expected magnitude and probability of occurrence).
- The propagation of seismic waves from the source to the site including the transference from bedrock to the superficial soils, in order to estimate the seismic action at the elements at risk.
- The local effects, including the potential for liquefaction and landslide.
- The characterization of the building stock having as geographical unit the smallest administrative territorial division and a diversity of parameters for the buildings, such as the age, constructive typology, number of floors, etc. Forty-nine different typologies were identified in the area covered by the project. Their vulnerabilities were evaluated using the Hazus-99 methodology, adapted to the Portuguese constructions technologies.
- The characterization of the population at different periods of the year (seasonal behaviour), of the week (distinguishing between working days and weekends) and of the day (night, rush hours, working periods, in a total of six day periods) was considered in order to determine where people are located at each moment. Estimation of victims (death, serious

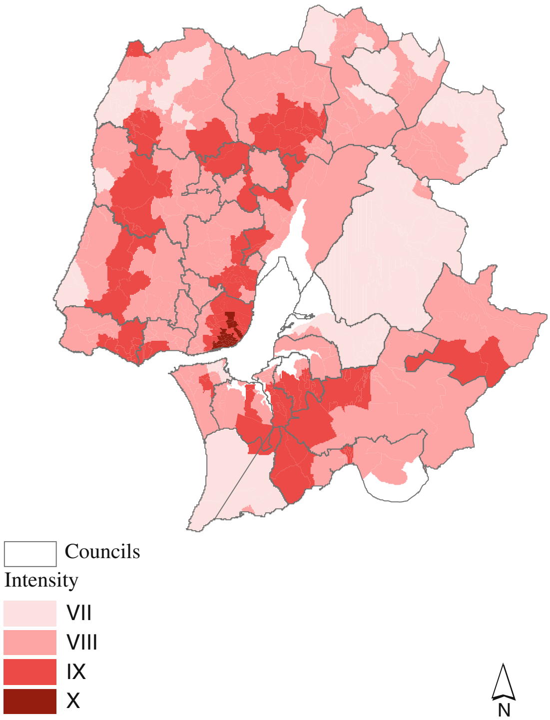


Fig. 2 Generated Intensity (Mercalli Modified) (1755 earthquake scenario)

and light injuries, and homeless) was made according to Tiedmann (1999) and Hazus.

– The characterization of the different lifelines was made with the collaboration of the agencies responsible for each one of them and their vulnerability was evaluated not only by the Hazus-99 methodology, but considering as well their seismic performance in historical events.
– The characterization of about 1000 vital points was made using a direct individual enquiry containing a long list of items, to estimate the vulnerability of their main structures and equipments, and the level of accessibility.

All the above-referred different moduli contain intrinsic uncertainties that are very difficult to control. Thus, the estimates generated by the simulator should be taken with great caution as they represent overall behaviour involving a large scatter.

Figures 2, 3, 4 and 5 respectively show, for a seismic scenario pretending to simulate the 1755 event, the generated intensity and the estimated damage on the building stock, injuries and road network damage.

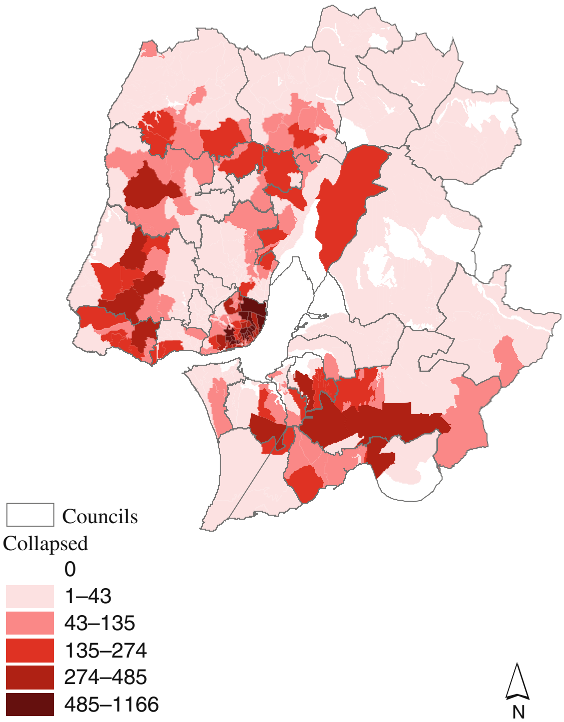


Fig. 3 Estimated damage on the building stock (1755 earthquake scenario)

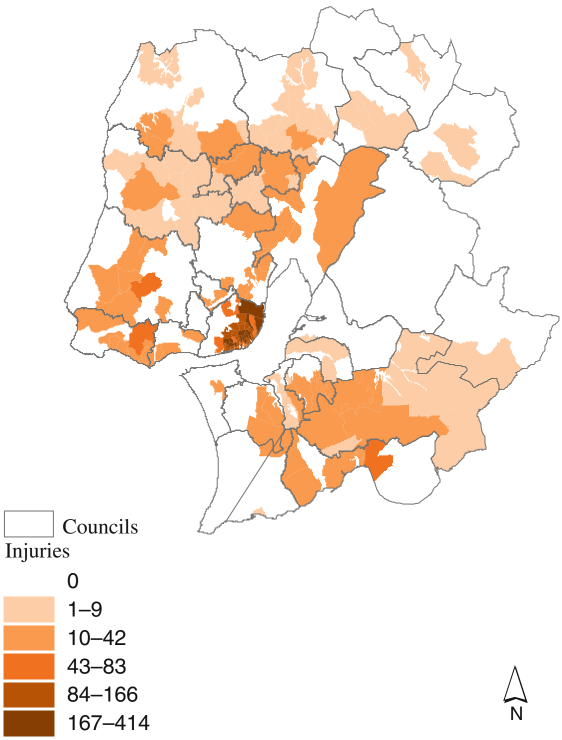


Fig. 4 Estimated injuries (1755 earthquake scenario)

6.2 Emergency Planning

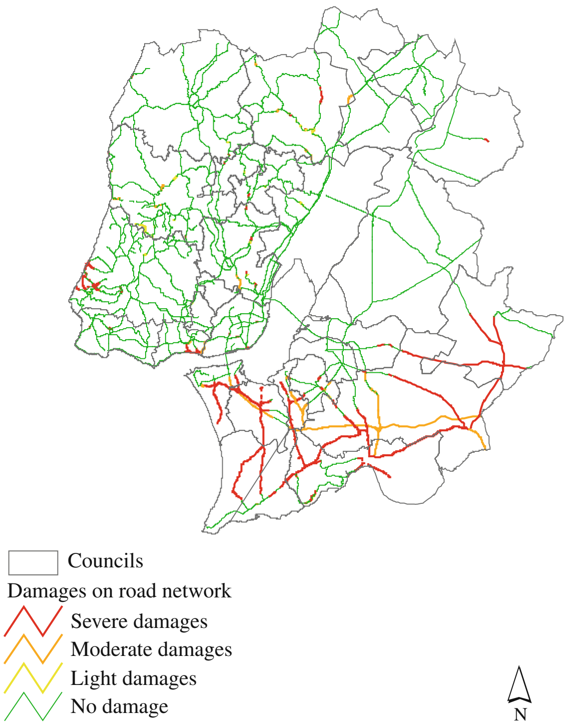
Emergency preparedness and mitigation of earthquake risks in a general sense should include, beyond the operational and response system, all the organizations and entities dealing with the planning process involving various activities from urban planning, code enforcement, prevention and rehabilitation of most vulnerable structures, to education and public awareness.

The above mentioned simulator can be a great help in the definition of priorities to be followed.

6.3 Civil Protection Organization

Even though the Civil Protection organizations differ very much from country to country, there are a few main issues that should always be followed in order to attain a good level of achievement (Sálvano-Briceño 2005), that is to mitigate as much as possible the human, the economic and social losses from disasters:

Fig. 5 Estimated road network damage (1755 earthquake scenario)



– Coordination among the different actors, defining the individual tasks with specified targets and avoiding duplication efforts.
– Open and permanent dialogue with the scientific-technological community and politicians.
– Set up of a credible disaster risk management capable of defining and supporting a strategy for decision-making.

The Civil Protection entities are generally organized in different levels, from the local to regional and national level. Even though in a few countries the local level (administrative unit: council; municipality, regions, etc.) has autonomous management, full coordination among adjacent administrative units has to take place in case of a larger disaster involving several units.

6.4 Preparation and Training

One of the most important requirements of efficient crisis management is that all structures, procedures and installations should be prepared and agreed upon before a crisis happens. No crisis management preparation can be effective without recurrent training exercises. Structures, procedures and technical equipment should be tested at least once a year. Such training exercises do not have to necessarily involve all the technical equipment, they can be confined as a "staff exercise" involving many agencies, such as the Fire Brigade, Police, Health agencies, etc.

The training exercises should be able to include the several decision-makers such as the Secretary-General of the city or the Mayor himself in order to provide the necessary type of inputs from typical decision-markers.

The public awareness and education are essential to protect people and property. To know what to do, when and where, are important keys to save everyone own life.

6.5 International, Regional and National Coordination

Depending on the extension of the disaster, regional and international coordination maybe required to deal with the situation. Establishment of national procedures for international support should comprise the intervening countries and non-governmental organizations, the level and type of assistance (medical, food, clothing, rescue teams, expertise, etc.), and the internal organization for reserving and distributing that international aid. All these topics should be known prior to the disaster and are function of the extension of occurred damage.

6.6 Rapid Damage Evaluation

The simulator can play a very important role in estimating the level and the geographic distribution of damage, right after the disaster. This information includes the more affected areas, damage to the hospitals and vital structures, damage to networks, landslides, etc. To do so, the simulator needs to be updated with information from the real situation that can be provided by witnesses, aerial photographs, satellite images, strong motion data, etc.

This important information will be essential for rapid assessment of the real damage and other consequential effects or impacts, and will constitute the basis for emergency operations (national and international) and for informing the society of a clear picture of what has happened and what are the measures to be taken.

6.7 Alert and Early Warning Systems

Alert and early warning systems are important tools in disaster management only possible due to new technological developments.

Alerts are procedures to be announced prior to the implementation of different levels of emergency planning. They depend on the estimation of damage impact (secondary effects), on the recognition of the earthquake process (possibility of aftershocks or other consequential events), and they may be classified into three alert levels: minor, moderate and major. For an effective alert issue a good link between the emergency authorities and the scientific-technical community (seismology, earthquake engineering, telecommunications, etc.) has to be present all the time.

The importance of creating and improving an effective early detection and warning system is the possibility of developing strategies to deal with the earthquake event prior to the arrival of the most destructive waves, not only of tsunami origin but also the vibratory motion. While for the vibratory motion the early warning is, in general, very short to alert the population, it can be used to close down industrial facilities, reduce velocity of railway traffic, etc. However, this requires a great efficiency in data acquisition, transmission and interpretation, only possible with high-tech resources and expertise.

For the tsunami warning the detection might be difficult for areas located close to the seismic rupture region, creating difficulties on effective alert issues, but for regions away from the rupture, the alert should be very valuable. In the first case, the best solution is educating the citizens for auto-protection measures, which recommend the escape to higher sites when they feel a strong shaking.

6.8 Post-Earthquake Missions

One can divide post-earthquake missions into two main categories, the one dealing with the rescue operations involving specialized teams of medical, pre-hospital immediate care, fire brigades and field engineering, and the other dealing with a detailed survey of damages (Goretti and Di Pascuale 2005).

Only the second type of missions will be dealt with, given that they are of most importance as they inform the population on the safety conditions of their structures and on the need to immediate evacuation, temporary strengthening due to potential aftershock activity and on the total amount of effort to recover the damaged areas. These missions should work with great efficiency and at the shortest time after the event and they require a great deal of prior preparation. Teams of specialized engineers, technicians, building inspectors, etc. should have detailed forms, easy to fill out and assemble the information in a centralized common system. Their decision to close down a building should be well justified and without doubts.

6.9 The Role of Media

The role of media, either in written form or television is of great importance in setting an Emergency Planning due to the strong relation media has with the public and the easy way to divulgate the information. A close link between the

Civil Protection and the media is essential for accuracy in the transmission as well as gathering of information. So, the establishment of preferred channels at different levels (national, local, international) should be viewed as a form to reduce intervention times, accelerate damage assessment, and to help in conveying difficulties or successes in the whole emergency operation.

6.10 Procedures Under Development

To contribute and support the elaboration of the emergency plans for the seismic risk is an ANPC priority. In this direction and taking advantage of the new technologies such as the available seismic simulator, in 2004 the main results obtained with the simulator for a few selected seismic scenarios were distributed to the AML municipalities.

The solution was the elaboration of CDs containing the results of 3 seismic scenarios with estimates of human casualties, damage to buildings and infrastructures for each municipality, in a geographic system (ArcReader program) that allows a simple and efficient visualization. This information was distributed in the Workshop “Planning of Emergency – the seismic risk in the AML”, 2004. This way the municipalities have an opportunity to analyze by themselves the vulnerabilities, to trace the risk zones, to support and develop the special emergency planning. It is intended in the future to obtain and establish more close relations between the involved municipalities.

At this moment ANPC has been elaborating the Special Plan of Emergency for this area, using the results obtained with the simulator. Municipalities can develop their own simulators using data at a much more detailed scale, but boundary conditions throughout adjacent borders should be kept similar in order not to have artificial geographical discontinuities of hazards.

The starting point for the Plan process is the formation of a Coordinating Group and the development of an organized structure. It is important that the Coordinating Group examines the following steps in the planning process:

– Identify agencies responsible for the community’s potential local awareness and for the response preparedness network.
– Identify the hazards that may be produced during an emergency situation.
– Identify the specific community points of contact and their responsibilities in an emergency.
– List the kinds of equipment and materials that are available to respond to emergencies.
– Check the existence of specialized emergency response teams.
– Define the emergency transportation network.
– Establish procedures for protecting citizens during emergencies.
– Set up a mechanism that enables communication exchange with other entities that may have useful information during an emergency.

The above issues cover essentially the major considerations or issues that should be resolved by the Coordinating Group.

7 Priorities for Action – Where to go

It is the point of view of the authors that a great contribution to the reduction of seismic risk would be made if four easier to reach main objectives could be clearly stated and attained:

1. Imposing a quality assurance procedure and inherent safety of the new constructed buildings and other infrastructures, by means of a quality control of both the design and construction. This should be extensive to the design and execution of rehabilitation interventions in existing buildings.
2. Defining the priority interventions to be taken care by the public authorities in terms of seismic retrofitting: This would clearly comprise the most important hospitals and schools, important and dangerous facilities, including those for emergency activities, and vital lifeline infrastructures.
3. Creating a legal and technical framework for rehabilitation actions for existing buildings that includes normative procedures and technical solutions.
4. Defining priorities for interventions in existing buildings and creating partnerships between the building's owners and public authorities to advance with the rehabilitation of the buildings at higher risk.

The order by which these actions should be initiated is not arbitrary. From an ethical point of view, it is not acceptable that an organized effort of rehabilitation of existing buildings takes place while there is not a reasonable guarantee that what we build today follows the standards that have been deemed as appropriate. Also it is not acceptable that an enforcement of new regulations is made while there is not the political will to guarantee that the existing regulations are correctly applied. It would be like promoting health treatments for infected people while not caring to give already existing vaccines to avoid new diseases. It is also not ethical that the state enforces regulations to guarantee the reduction of seismic risk of the existing building stock belonging to the private sector while not actively promoting the reduction of the seismic risk of infrastructures belonging to the public sector, that represent aggravated risk in the event of a major earthquake. Finally, it is not understandable and recommendable that the government intensively allocates funds to retrofit the building stock owned by the private sector, while there is an urgent need to rehabilitate some important, at risk, infrastructures, whose functionality is the responsibility of the state and that can put at major risk a large amount of people. The process of rehabilitation of the existing building stock should be based on rigorous evaluation of the risk associated to each building, carried out by licensed professionals and subject to the same quality control as for the design of new buildings. The owners should be given the responsibility of its conduction, as it would also be the case for the construction of a new building. The subsequent process of rehabilitation should be based on the previous evaluation of the risk and, as much as possible, on the voluntary adherence of the owners and users. In all situations, to avoid the loss of constructions with architectonic value, or the loss of character of urban areas or pure land speculation, a special care must be taken to prevent the destruction and demolition of constructions in name of preventing the seismic risk.

The position of the scientific and technical community regarding these issues also has to contemplate the responsibility of an impartial and ethical positioning (Reitherman 1999). Being an acting part in the process, scientists and technical staff must, first of all recognize that they are an interested part in the process and, consequently, be independent in their assessment of the situation and in the proposal of solutions. Lobbying for seismic risk reduction is not only acceptable but also desirable. Nevertheless, a special care is necessary regarding the way lobbying is carried out and is seen by the other participants in the process, being crucial that it is independent in all reasoning and totally objective and unbiased in all the assessments.

8 Additional Contributions

Having in mind the need of engineering support for these tasks, one could ask if there is, and has been properly used, adequate engineering knowledge to reduce seismic vulnerability. It is clear that in what regards the mitigation of seismic risk for new constructions, there is, in Portugal, seismological and engineering know how and experience that can guarantee levels of risk as low as the ones accepted in other fields of human life and activity. Having in mind that it is impossible to eliminate risk and that one has to think in terms of reducing risk to acceptable levels, one can say that, in our days, it is possible to build earthquake resistant structures. The same conclusions can not be extrapolated when looking at the capacity to guarantee low seismic risk in existing structures, especially when they were built according to non-existing or different standards or non adequate or difficult to model materials and structural solutions. This is why it is difficult to assess the resistance and rehabilitate some constructions like the ones having weak and non-ductile masonry structures and materials. This is also why this is one of the areas where engineering research has to focus its attention. Also necessary is to invest in the study and use of engineering solutions involving new materials and new devices for seismic protection. It is partly non understandable why solutions like base isolation are being used, in building applications, for the first time in Portugal, when they have proven to be highly effective in reducing seismic risk in other countries.

But aspects other than the engineering ones need to be examined if the reduction of seismic risk is to be accomplished. Are there legal, financial and economical instruments that can be used to promote seismic risk reduction? From the point of view of legal aspects, the situation is twofold. On one hand there are currently totally adequate codes and regulations to enable engineers to correctly design and build seismic resistant structures and the criteria for safety assessment are know and clearly defined. On the other hand there are not such detailed legal and technical instruments when engineers need to analyze the safety of already existing structures. It is necessary to establish criteria for the assessment of the seismic safety of existing structures and, in the form of legal or paralegal instruments, make available technical standards applicable to the seismic assessment and rehabilitation of existing constructions. Also, from the legal point of view it is not acceptable that there are not legal rules that enforce the full quality control of the construction product. It is not understandable that the quality assurance of some equipment or simple devices used in daily life can be stricter than the assurance of quality of a product such as the premises where we spend great part of our life. From the point of view of the financial and economical instruments that can be used to promote seismic risk reduction one has to distinguish between new construction and rehabilitation of existing construction. In the first case, it can be said that the cost of constructing with seismic safety in mind is only marginally more expensive than to construct without. In this case, insurance companies can play an important role in guaranteeing quality, if the price of the insurance the entrepreneur has to pay depends on the quality of the final product. In the second case, a combination of long-term loans, tax benefits and cheaper insurance may be used as incentives for rehabilitation.

Have research efforts and education actions been the most adequate? Education towards the mitigation of seismic risk as well as other risks has to be strongly based on educational measures focused on the young generations. Some interesting initiatives were made in the past, namely by the City of Lisbon Civil Protection and the Portuguese National Civil Protection, although their effectiveness is difficult to assess. Drills, preceded by information about earthquakes, should be planned and carried out at schools and other youth organizations. Education at a higher level should also be object of further and new initiatives. In the schools of engineering and namely in the civil engineering curricula, many courses on earthquake engineering and structural dynamics were introduced in the last two decades. It is anyway strongly needed, not only to include topics or courses on the behaviour of masonry structures but also to give more emphasis to the conceptual aspects of seismic resistant design, overcoming an erroneous tendency to believe that sophisticated methods of structural analysis are a panacea for structural design. In the fields of earthquake engineering and seismology, Portugal has long been in the front line of research efforts. The number of researchers working in these fields exponentially increased in the last two decades, demonstrating the great interest these research fields have deserved. The research activities have covered the vast majority of the topics and, globally, research results are of good quality. It is nevertheless necessary that more interdisciplinary studies take place, involving more people and further knowledge areas in the research efforts, with emphasis in the development of new technologies and use of new materials and devices for seismic protection of infrastructures. Another field that deserves continued attention is the applied research related to the improvement of the existing code specifications and the production of standards for the use of seismic protection devices and materials as well as the development of techniques and procedures for the rehabilitation of old constructions including historical and traditional masonry structures.

9 Conclusions

A "new" 1755 is a likely (but not the sole) hazard scenario that needs to be taken into consideration when analyzing seismic risk in continental Portugal and particularly in the great Lisbon area and Algarve, where risk can be considered as high. The citizen's awareness of the risk and the perception of the consequences by the public authorities are quite low. Consequently, insufficient actions have been taken to promote seismic risk reduction, demanding a swift response from all possible interveners. In the past several proposals have been made, consisting on tangible actions to reduce seismic risk to acceptable levels. Some of those proposals still point out in the right direction and deserve to be re-examined. Among the actions that would most contribute to reduce the seismic risk are the reintroduction of quality control procedures in the design and construction activities, selective rehabilitation of important public infrastructures, investments in research and code development for rehabilitation of the existing building stock and establishment of a program that would promote its gradual implementation.

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