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Seismic risk control of nuclear power plants using seismic protection systems in stable continental regions: The UK case

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highlights

- Strategies to reduce seismic risk for nuclear power stations in the UK are analysed.
- Efficiency of devices to reduce risk: viscous-based higher than hysteretic-based.
- Scenario-based incremental dynamic analysis is introduced for use in nuclear stations.
- Surfaces of seismic unacceptable performance for nuclear stations are proposed.

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abstract

This article analyses three different strategies on the use of seismic protection systems (SPS) for nuclear power plants (NPPs) in the UK. Such strategies are based on the experience reported elsewhere of seismically protected nuclear reactor buildings in other stable continental regions. Analyses are conducted using an example of application based on a 1000 MW Pressurised Water Reactor building located in a representative UK nuclear site. The efficiency of the SPS is probabilistically assessed to achieve possible risk reduction for both rock and soil sites in comparison with conventionally constructed NPPs. Further analyses are conducted to study how the reduction of risk changes when all controlling scenarios of the site are included. This is done by introducing a scenario-based incremental dynamic analysis aimed at the generation of surfaces for unacceptable performance of NPPs as a function of earthquake magnitude (M_w) and distance-to-site (R_{epi}). General guidelines are proposed to potentially use SPS in future NPPs in the UK. Such recommendations can be used by the British nuclear industry in the future development of 12 new reactors to be built in the next two decades to generate 16 GWe of new nuclear capacity.

1. Introduction

Nuclear power plays a crucial role in energy supply in the world: around 15% of the electricity generated worldwide is provided by nuclear power stations avoiding around 2.5 billion tonnes of CO_2 emissions (Meiswinkel et al., 2013). The seismic design of new nuclear power plants (NPPs), in order to ensure their safe seismic performance, has received much greater research interest after the Fukushima Dai-ichi accident (Hirano et al., 2012). Currently, it is estimated that around 20% of nuclear reactors worldwide are operating in areas of significant seismic activity (WNA, 2014). In the UK, a tectonically stable continental region that possesses medium-to-low seismic activity (Musson, 1996), strong earthquakes capable of jeopardising the structural integrity of NPPs, although infrequent, can still occur (Musson, 2014). Despite that no NPP has been built in the UK after 1995, a New Build Programme intended to build 16 GW of new nuclear capacity by 2030 is currently under way (NIA, 2012). The necessity of correctly assessing all aspects regarding seismic safety of new generation NPPs in the UK has become a vital issue for the industry (Weightman, 2011). This article is intended to make a contribution towards that aim.

Seismic protection systems (SPS), such as elastomeric-based bearings and energy dissipation devices, have been successfully used in more than 10,000 applications (Martelli et al., 2012). However, only two reactor buildings have been designed with such technology: Koeberg NPP in South Africa and Cruas NPP in France (Forni et al., 2012). Although these two applications were designed more than 40 years ago, extensive research has been conducted since then in order to include SPS in NPPs (Medel-Vera and Ji,

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2014). In the UK, laboratory tests were carried out in early 1990s on small-scale specimens of low-damping rubber bearings and viscous dampers for applications in Liquid-Metal-Cooled Reactors (LMRs) (Austin et al., 1991). Nowadays, several new projects of isolated reactors in medium-to-low seismic areas are currently under way: (i) the Jules Horowitz Reactor (JHR) currently under construction in Cadarache, France (Bignan et al., 2011); (ii) the International Thermonuclear Experimental Reactor (ITER), also under construction in Cadarache, France (Syed et al., 2014); (iii) APR1400, currently under construction in South Korea (Lee et al., 2015). Additionally, some other prominent projects of Generation IV reactors are currently in their early stages: (i) the Advanced Sodium Technological Reactor for Industrial Demonstration (ASTRID) to be built in France (CEA, 2012); and (ii) the Advanced Lead Fast Reactor European Demonstrator (ALFRED) to be built in Romania (Alemberti et al., 2014). All these applications of reactors consider the use of different types of elastomeric-based bearings: JHR and ITER share the same design of low-damping rubber bearings (Sollogoub, 2014), APR1400 uses lead–rubber bearings (Lee et al., 2015), and ASTRID and ALFRED will use lead–rubber bearings and/or high-damping rubber bearings (Forni, 2015; Moretti and Pasquali, 2013). Additionally, other approaches to seismically protect reactor buildings without isolating the entire nuclear island have been investigated: e.g. the Russian VVER-1000 has been the subject of studies that propose the use of high performance viscous dampers to protect its critical components in future applications (Kostarev et al., 2003). This work investigates the suitability of several SPS that can be used in next generation UK reactors subjected to the seismic conditions of the British Isles.

In this work, a sample NPP reactor building based on a 1000 MW Pressurised Water Reactor building equipped with three different types of SPS was analysed: (i) an isolated nuclear island using low-damping rubber bearings plus viscous dampers; (ii) an isolated nuclear island using lead–rubber bearings, and (iii) a non-isolated nuclear island using only viscous dampers located at the critical components of the NPP. The efficiency of these SPS was assessed to achieve possible risk reduction for both rock and soil sites in comparison with a conventional NPP. The risk was calculated following the methodology for seismic probabilistic risk assessment (SPRA) for NPPs in the UK reported by Medel-Vera and Ji (2016a). A representative location of a UK nuclear site was selected and the risk was initially assessed for the single scenario that contributes most strongly to the hazard of such a site. Then, the variation of risk is studied for different controlling scenarios, following a proposed scenario-based incremental dynamic analysis (IDA). Scenario-based IDA, as introduced in this article, aims at the generation of surfaces for the unacceptable performance of NPPs in the UK as a function of earthquake magnitude (M_w) and distance-to-site (R_{epi}). Unacceptable performance surfaces can be a substantial contribution to the UK nuclear industry in order to provide insights as how the seismic risk varies when the NPP is subjected to most (or all) dominant scenarios of the selected nuclear site.

This work is organised as follows: Section 2 describes the two specific objectives of this article and the methodology used. Section 3 provides details about the structural models and their mechanical properties, including the definition of the SPS analysed and the modelling of the soil-structure interaction. Additionally, description of the fragility curves used to characterise the critical components of the sample NPP building is provided. Section 4 presents the seismic input definition for the choice of the nuclear site and summarises the risk assessment calculations performed. Section 5 provides a step-by-step definition of scenario-based IDA and the estimation of unacceptable performance surfaces. Section 6 discusses further aspects regarding the appropriateness of SPS for NPPs in the UK and advantages and limitations of scenario-based IDA and presents the conclusions from this study.

2. Objectives and methodology

This work has two specific objectives: (a) to determine an efficient approach of SPS to reduce the seismic risk of NPPs buildings subjected to the UK seismic conditions including the influence of the foundation soil; and (b) to investigate how the reduction in seismic risk of NPPs buildings changes when considering several or all dominant scenarios for the particular site selected.

All analyses carried out in this work were made considering a simplified structural model based on a 1000 MW Pressurised Water Reactor building. Such a structural model was used to define two types of models: (1) a conventional NPP and (2) a seismically protected NPP. The former models the reference case, i.e. a traditionally built fixed-to-the-ground NPP (Model 1 hereafter), whereas the latter comprises three models that use different types of seismic protection devices suitable for NPPs. The following devices were analysed: (a) low-damping rubber bearings (LDRB) in combination with linear viscous dampers (LVD) aimed at adding a 10% critical (viscous) damping (Model 2a hereafter); (b) lead–rubber bearings (LRB), aimed at adding a 20% critical (hysteretic) damping (Model 2b hereafter); and finally, (c) linear viscous dampers located at the critical components of the NPP, aimed at adding 30% critical (viscous) damping for each local device (Model 2c hereafter). Then, a site was selected that is typical UK NPP site with relatively moderate seismicity, including two types of foundation soil: (i) generic rock site and (ii) generic soil site. The efficiency in reducing the seismic risk of the NPPs was made using the methodology of SPRA reported by Medel-Vera and Ji (2016a). In order to give answer to the first objective of this article, the risk was assessed considering the single scenario (moment magnitude, epicentral distance) that contributes most strongly to the hazard of the site selected. Fig. 1 summarises the tasks performed to comply with Objective (a) of this work.

Regarding Objective (b), the change in seismic risk of the NPP building are assessed considering all dominant scenarios of the particular site selected. For this purposes, it is introduced in this article a scenario-based incremental dynamic analysis (IDA) intended to generate surfaces for the probability of unacceptable performance of NPPs as a function of earthquake magnitude and distance-to-site. Such surfaces were generated for two models, conventional NPP and seismically protected NPP, in generic rock site. Then, the relative performance between those surfaces was studied in order to gain in-depth knowledge about the behaviour for the reduction of risk for all dominant scenarios of the particular site selected. Fig. 2 summarises the tasks performed to comply with Objective (b) of this work.

3. Structural models

3.1. Sample nuclear reactor building

All structural models used in this work were based on a 1000 MW Pressurised Water Reactor (PWR) shown schematically in Fig. 3a. This sample nuclear reactor building is composed of two structural units: (i) the containment structure (CS), composed of a post-tensioned concrete cylindrical wall, and (ii) the internal structure (IS), to which the critical key components of the NPP are attached. These structural units are independent from each other; hence, they are only connected at the foundation level. The height of the CS and IS are 60 m and 39 m, respectively, whereas the total weight of the reactor building is approximately 62,000 ton. Fig. 3b shows the simplified structural model of the sample NPP used in this work. Both the CS and IS are modelled as lumped-mass stick models that are the same in both horizontal directions. Fundamental periods of vibration of the CS and IS are

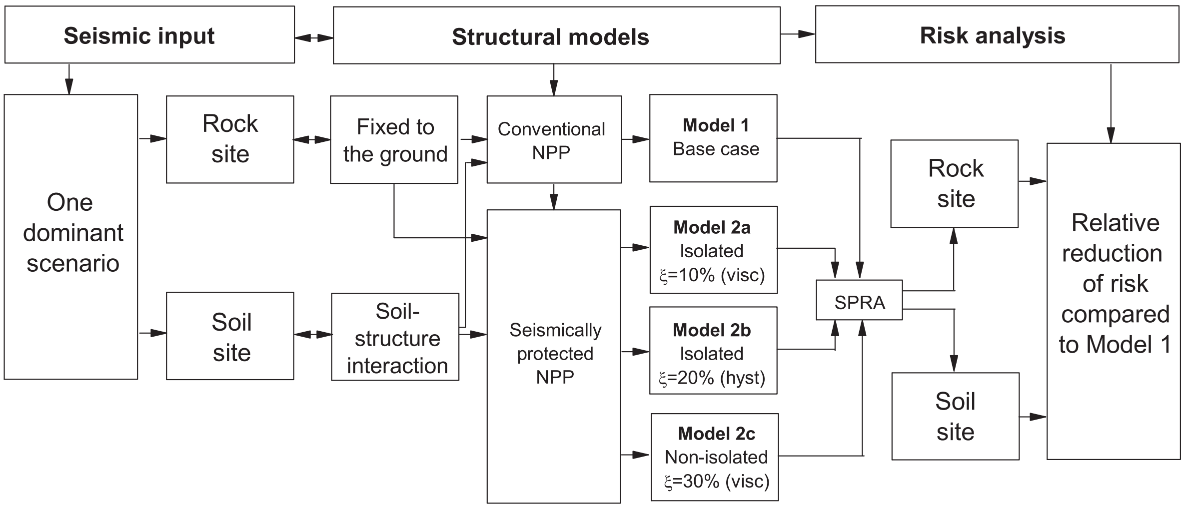

Fig. 1. Relationships among the tasks performed to comply with objective (a).

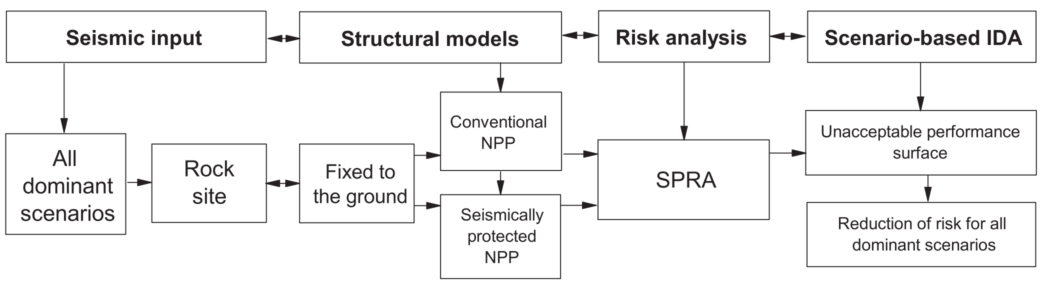

Fig. 2. Relationships among tasks performed to comply with objective (b).

0.23 s and 0.18 s, respectively. This model has been previously used and validated by the authors for SPRA for NPPs in the UK (Medel-Vera and Ji, 2016a).

Following (Huang et al., 2011, 2010), risk assessments of NPPs are focused on their critical components supported by the IS. These components control the cost of a NPP project in terms of design, analysis, construction, testing and regulatory aspects. The critical components of the sample NPP, node assignment in the structural model and their location related to the foundation level are summarised in Table 1.

For simplicity purposes, it was assumed that the plant-system and accident-sequence analyses that involves the definition of event and fault trees for the sample NPP shown in Fig. 3 is the same as used previously by Medel-Vera and Ji (2016a). In short, unacceptable performance of the sample NPP is defined as the failure of any of the critical components indicated in Table 1. As it will be seen in Section 4, nonlinear time-history analyses with the structural model shown in Fig. 3b were performed. In this work, bilinear shear hinges with 3% post-yield stiffness were assigned to all stick elements of the IS for the definition of the structural model. The yield capacity of each hinge was estimated as $0.5 \cdot \sqrt{f'_c} \cdot A_s$, where f'_c is the compression concrete strength (assumed as 35 N/mm^2) and A_s is the shear area of each stick of the IS as indicated in Li et al. (2005).

3.2. Definition and properties of structural models

The properties and particular characteristics of Models 1, 2a, 2b and 2c are defined in detail in this section. It is worth mentioning that Models 2a and 2b are intended to provide seismic isolation to the entire nuclear island, i.e. the CS and IS are supported by a common mat which in turn is separated from the foundation mat by the interface of isolation where the devices are located. Certainly, due to the nature of the simplified structural model of the sample NPP used in this work, the seismic isolators modelled for Models 2a and 2b is an assembly of isolators, i.e. the entire isolation system, rather than all individual isolators. Fig. 4 provides a graphical summary of Models 1, 2a, 2b and 2c studied in this article. The particular characteristics of each structural model used in this work are described as follows:

- Model 1 is intended to represent the base case, i.e. the NPP building that uses conventional construction without any kind of seismic protection device.
- In model 2a, the LDRB system was modelled as a horizontal linear spring as these devices exhibit a visible linear behaviour in shear with little addition of supplemental damping (Naeim and Kelly, 1999). The stiffness of the spring was selected in order to obtain a structural period of $T_{obj} = 2$ s. Additional external damping was included at the foundation level by considering $n = 10\%$ by means of a linear viscous damper whose force–velocity governing equation is linear and the force–deformation hysteresis curve has oval shape.
- In model 2b, the LRB system was modelled using a bilinear constitutive relationship with the properties summarised in Table 2. These properties were calibrated using the direct procedure stated in Naeim and Kelly (1999) considering: design displacement of $u_d = 25$ cm, fundamental period of vibration of $T_{obj} = 2$ s associated to the effective stiffness (peak-to-peak loads), critical damping ratio of $n = 20\%$, and stiffness ratio of $K_u = 10K_d$. As models 2a and 2b are seismically isolated, it is appropriate to consider for the internal damping a lower value than the 5% normally used in seismic analysis of structures. Therefore, the internal damping was modelled considering $n = 2\%$ as recommended in Chopra (1995).

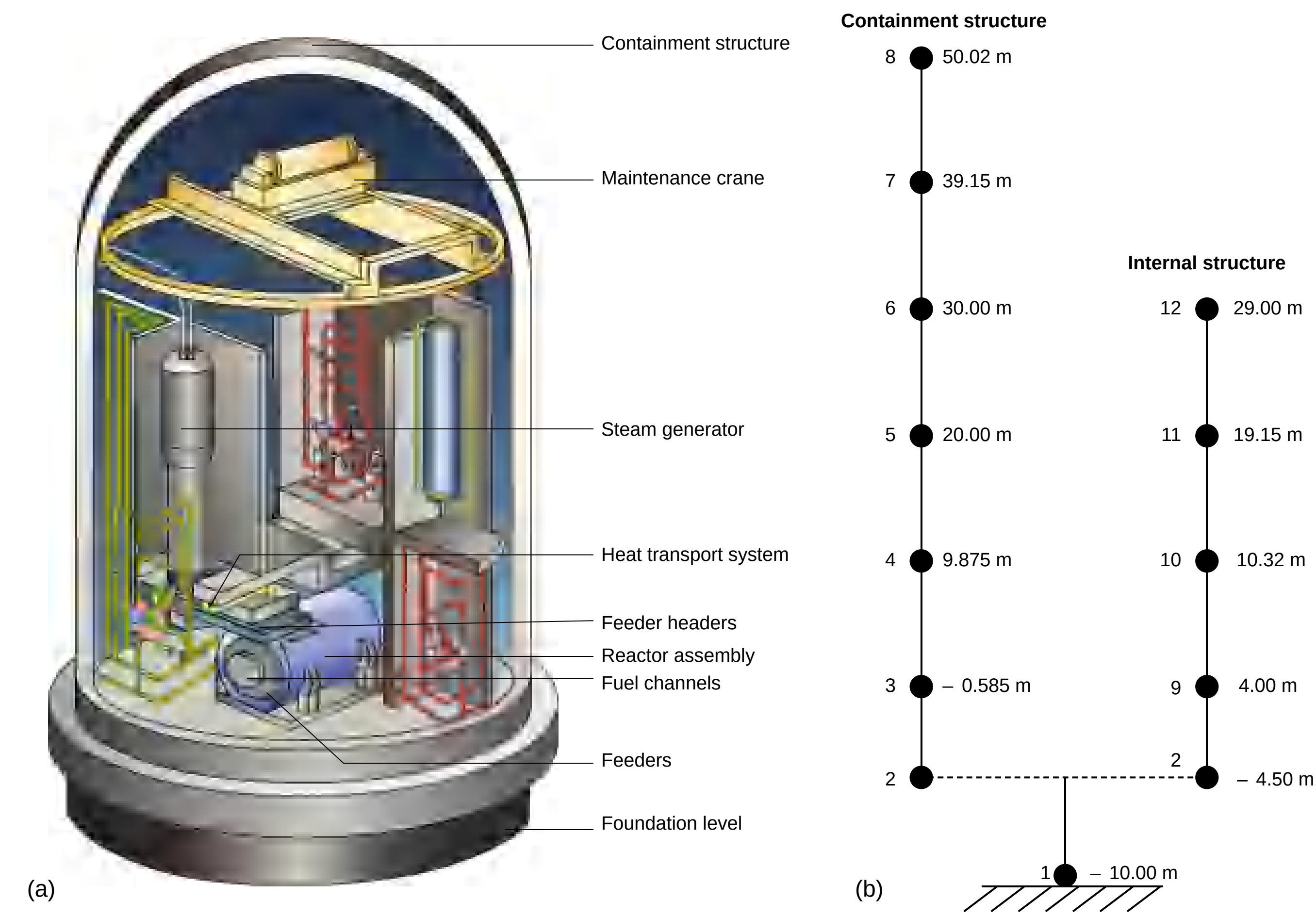

Fig. 3. Sample NPP reactor building. (a) Schematic view; (b) lumped-mass stick model (Li et al., 2005; Medel-Vera and Ji, 2016a).

Table 1
Critical components and their location within the sample NPP.

Critical component	Node number	Elevation (m)
Reactor assembly and fuel channels	2	−4.50
Feeders and feeder headers	9	4.00
Heat transport system	10	10.32
Steam generator	11	19.15
Maintenance crane	12	29.00

- Model 2c is intended to analyse the effect of using LVD to seismically protect each critical component of the sample NPP without the need of isolating the entire nuclear island. Therefore, five viscous dampers were included in the sample NPP. The damping constants for each device were calculated using $C_i = 2 \cdot \xi \cdot m_i \cdot \omega_1$, where m_i $(i = 2; 9; 10; 11; 12)$ is the mass of each node of the IS, ω_1 is the fundamental frequency of vibration of the IS, and ξ is the critical damping ratio of the devices. In this work, a value of $\xi = 30\%$ was used for all devices. This is a simplified procedure to determine the damping constants for connecting adjacent buildings using viscous dampers that has been used elsewhere (Patel and Jangid, 2009). As this model does not consider seismic isolation, the internal damping in the structure was modelled considering the traditional $\xi = 5\%$. It is acknowledged that such a configuration of dampers may not be possible to materialise in an actual application. Nevertheless, it is considered to be a first approximation on the use of dampers to protect critical components.

3.3. Fragility analysis

The estimation of fragility curves for the critical components of the sample NPP used in this work is addressed in detail in Medel-Vera and Ji (2016a). This section only presents a summary that allows reproducibility of the fragility curves used in this work. In this regard, the parameters and results of interest are as follows:

- Demand parameter selected: average floor spectral acceleration (AFSA) over a frequency range from 1 to 33 Hz (i.e. the average of 33 floor spectral ordinates at frequencies from 1 to 33 Hz with increments of 1 Hz).
- Deterministic value $\hat{A}$ for each critical component in terms of AFSA: they were estimated as 2.32, 2.48, 2.85, 3.67 and 5.43 m/s2 for Nodes 2, 9, 10, 11 and 12 respectively.
- Logarithmic standard deviations β_r and β_u: they were estimated as $\beta_r = 0.31$ and $\beta_u = 0.41$.

For illustration purposes, families of 11 fragility curves for Nodes 2 and 12 are shown in Fig. 5a and b (median fragility curves are highlighted) whereas Fig. 5c shows the median fragility curves for the critical components at all nodes of the IS.

3.4. Soil-structure interaction

The influence of the type of foundation soil on the seismic behaviour of models 1, 2a, 2b and 2c was studied by analysing their response on both a generic rock and a generic soil site. These sites

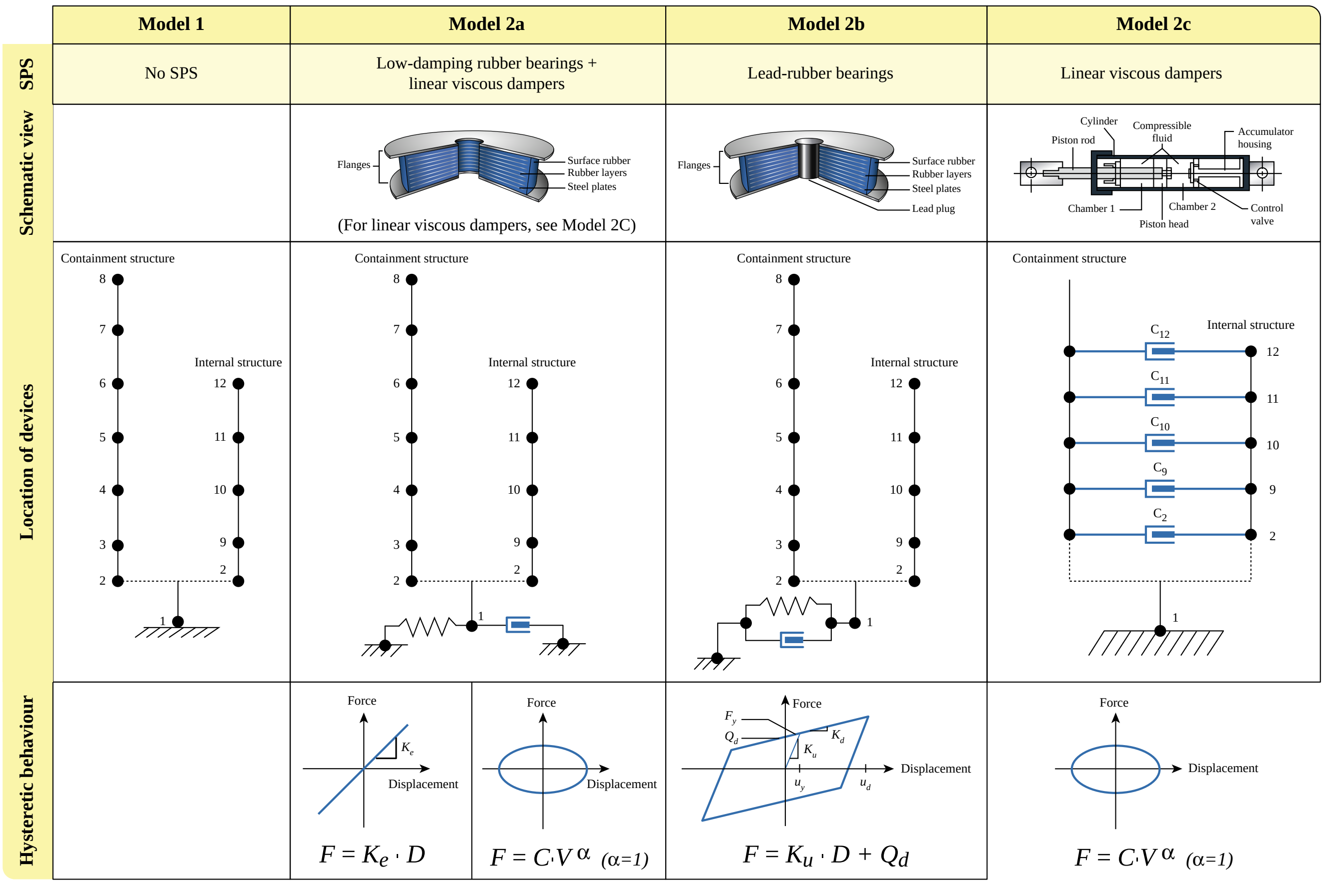

Fig. 4. Graphical summary of the structural models with seismic protection systems studied in this article.

Table 2
Design features of the structural models used in this work.

Model	Seismic protection system	Description
1	None	Conventional NPP
2a	Low-damping rubber bearings (LDRB) + linear viscous dampers (LVD)	T_{obj} = 2 s. n = 10% (viscous)
2b	Lead–rubber bearings (LRB)	T_{obj} = 2 s. n = 20% (hysteretic) u_d = 25 cm; K_u = 10K_d
2c	Linear viscous dampers (LVD)	T_{obj} = 0.18 s. n = 30% (viscous) for each device

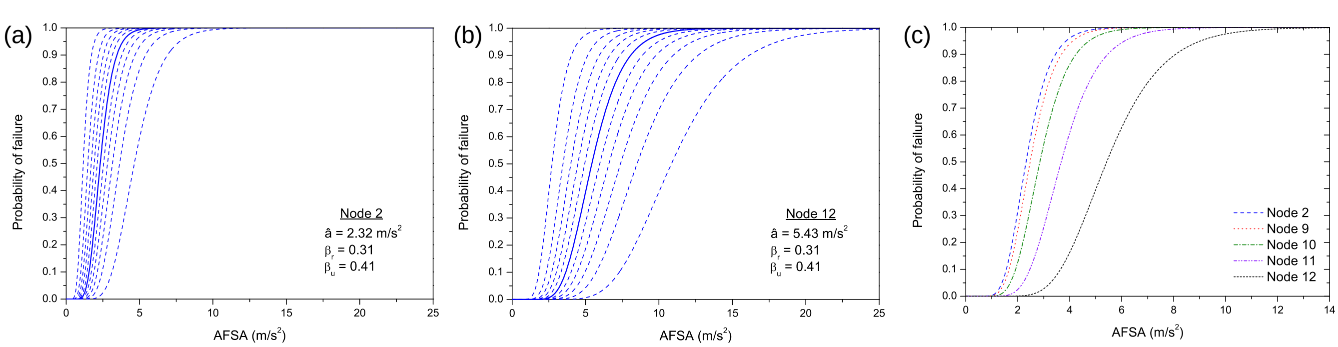

Fig. 5. Family of fragility curves for the critical components at (a) Node 2; (b) Node 12; (c) median fragility curves for critical components at all nodes of the IS.

were characterised by their corresponding average shear-wave velocity of the top 30 m of the subsurface profile ($\overline{V}_{s30}$). In this work, the rock site was considered to possess $\overline{V}_{s30}$ > 760 m/s, whereas for the soil site, it was considered to possess $\overline{V}_{s30}$ in the region of 360 m/s which is associated to a soft-to-medium soil profile. These definitions are in line with those made to analyse the influence of the type of soil on the seismic behaviour of other reactor buildings, e.g. the AP1000 nuclear reactor building (Tuñón-Sanjur et al., 2007).

For the structural models in the rock site, a fixed base connection to the ground was considered. For the soil site, it is acknowledged that there are currently available thorough soil-structure interaction (SSI) analyses for nuclear reactors (Elkhoraibi et al., 2014; Ostadan and Kennedy, 2014; Saxena and Paul, 2012; Saxena et al., 2011). However, such approaches are out of the scope of this work as this article is intended to provide a first approximation on the behaviour of NPPs in soil sites in the UK. Therefore, a simple approach to model SSI in the time domain was followed. Despite several methodologies available to model SSI in the time domain (Wolf, 1989), a straightforward discrete approach based on soil springs was implemented in this work. In such a methodology, the interface soil-structure is modelled by six frequency-independent elastic springs and dampers associated to the six rigid body degrees of freedom. Such springs and dampers model the dynamic stiffness of the soil located both underneath and adjoining the embedded portion of the building. In nuclear engineering research, such an approach has been used previously (e.g. Bhaumik and Raychowdhury (2013) and Wagenknecht (1987)). In this work, the soil springs proposed by Llambias et al. (1993) and validated in Llambias (1993) were directly used as they were proposed for PWR buildings in UK soil sites. Fig. 6 shows the generic case for modelling SSI using soil springs and Table 3 shows the values for such springs proposed by Llambias et al. (1993).

When modelling SSI as indicated in Fig. 6, Table 3, the fundamental periods of vibration of the sample NPP changed to 0.33 s for the CS and 0.27 s for the IS. This increase in the fundamental periods of vibration of the CS and IS of about 30% in comparison with the fixed base case is in line with the increase obtained for other nuclear reactor buildings and nuclear structures when modelling SSI (Bhaumik and Raychowdhury, 2013; Saxena et al., 2011).

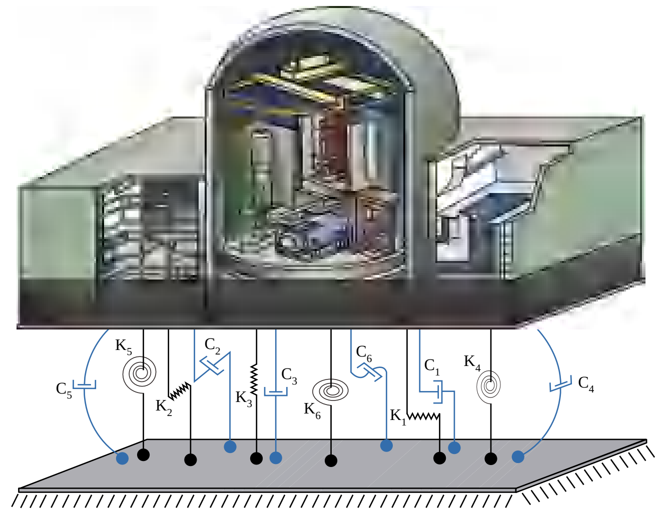

Fig. 6. Generic case for modelling SSI by means of soil springs.

Table 3
Soil springs proposed by Llambias et al. (1993).

Direction	Value
Stiffness terms	
Translational (K_1, K_2) (N/m)	3.87E+10
Vertical (K_3) (N/m)	9.27E+10
Rotational (K_4, K_5) (Nm/rad)	2.64E+13
Torsional (K_6) (Nm/rad)	1.78E+13
Damping terms	
Translational (C_1, C_2) (Ns/m)	1.02E+09
Vertical (C_3) (Ns/m)	3.39E+09
Rotational (C_4, C_5) (Nms/rad)	2.95E+11
Torsional (C_6) (Nms/rad)	1.25E+11

4. Definition of seismic input and risk assessment calculations

The level of seismic hazard of nuclear sites in the UK is defined by a seismic event of an annual frequency of exceedance of 10^{-4}, corresponding to a return period of 10,000 years (HSE, 2011). Therefore, it is of interest to determine the scenario (magnitude, distance) that contributes most strongly to the site's hazard for a 10,000 years return period at the fundamental period of the base case (Model 1). The hypothetical nuclear site selected for risk assessments in this work was the town of Holyhead, Anglesey. This site was considered to be representative of an actual UK nuclear site as the Wylfa Nuclear Power Station was built in a nearby location in Anglesey (Magnox, 2011). The estimation of the dominant seismic scenario was taken from the deaggregation of the hazard curve of Holyhead proposed by Goda et al. (2013) for 10,000 years return period at a structural period of 0.2 s (for simplicity, the hazard defined for a structural period of 0.2 s was assumed to be valid for the fundamental period of the IS equal to 0.18 s). The scenario that contributes most strongly the site's hazard is an earthquake magnitude M_w 5.3 at a hypocentral distance of 15 km (average between 10 and 20 km). Accelerograms compatible with such a scenario both in rock and soil sites will be later required to perform time history analysis of the structural models defined in Section 3. For simplicity, it was assumed a very small focal depth; hence $R_{hyp} \approx R_{epi}$.

Accelerograms compatible with this scenario were simulated using the stochastic ground motion accelerogram model calibrated for NW Europe proposed and validated in Medel-Vera and Ji (2016b). The model is a set of predictive equations of parameters that define a time-modulated filtered white noise process. Such a stochastic process is defined by: (i) a 6-parameter time-modulating function and (ii) a 3-parameter time-varying linear filter. The set of predictive equations for these parameters were calibrated by means of the random-effects regression technique using a small dataset of 220 accelerograms recorded in the stable continental region of NW Europe. The model simulates accelerograms compatible with seismic scenarios defined by earthquake magnitudes $4 < M_w < 6.5$, distance-to-site $10 < R_{epi} < 100$ km and different types of soil (rock, stiff and soft soil). Certainly, the underlying assumption of using this model is that the nature of accelerograms caused by moderate-to-strong earthquakes in Britain would be similar to those caused in the broader region of NW Europe due to geographical proximity.

In Medel-Vera and Ji (2016a), it was reported that a suite of 200 accelerograms was sufficient to estimate the probability of unacceptable performance of NPPs with high statistical confidence. Following this result, two sets of 200 accelerograms each were simulated for both rock and soil sites. For illustration purposes, Fig. 7 shows a graphical summary of the two bins of 200 accelerograms each for the rock and soil sites that are compatible with the seismic scenario that contributes most strongly to the hazard of site selected. Fig. 7a shows a small sample of three simulated accelerograms from each bin. Fig. 7b shows the 5% damped spectral acceleration of 50 simulated accelerograms from each bin. Fig. 7c shows the median, 84th and 16th percentiles of the 200 simulated accelerograms from each bin. These statistics are compared with those estimated using two GMPEs that have been deemed to be suitable for use in the UK: Bommer et al. (2007) and Campbell and Bozorgnia (2008). The former was calibrated for Europe and the Middle East whereas the latter is an NGA model

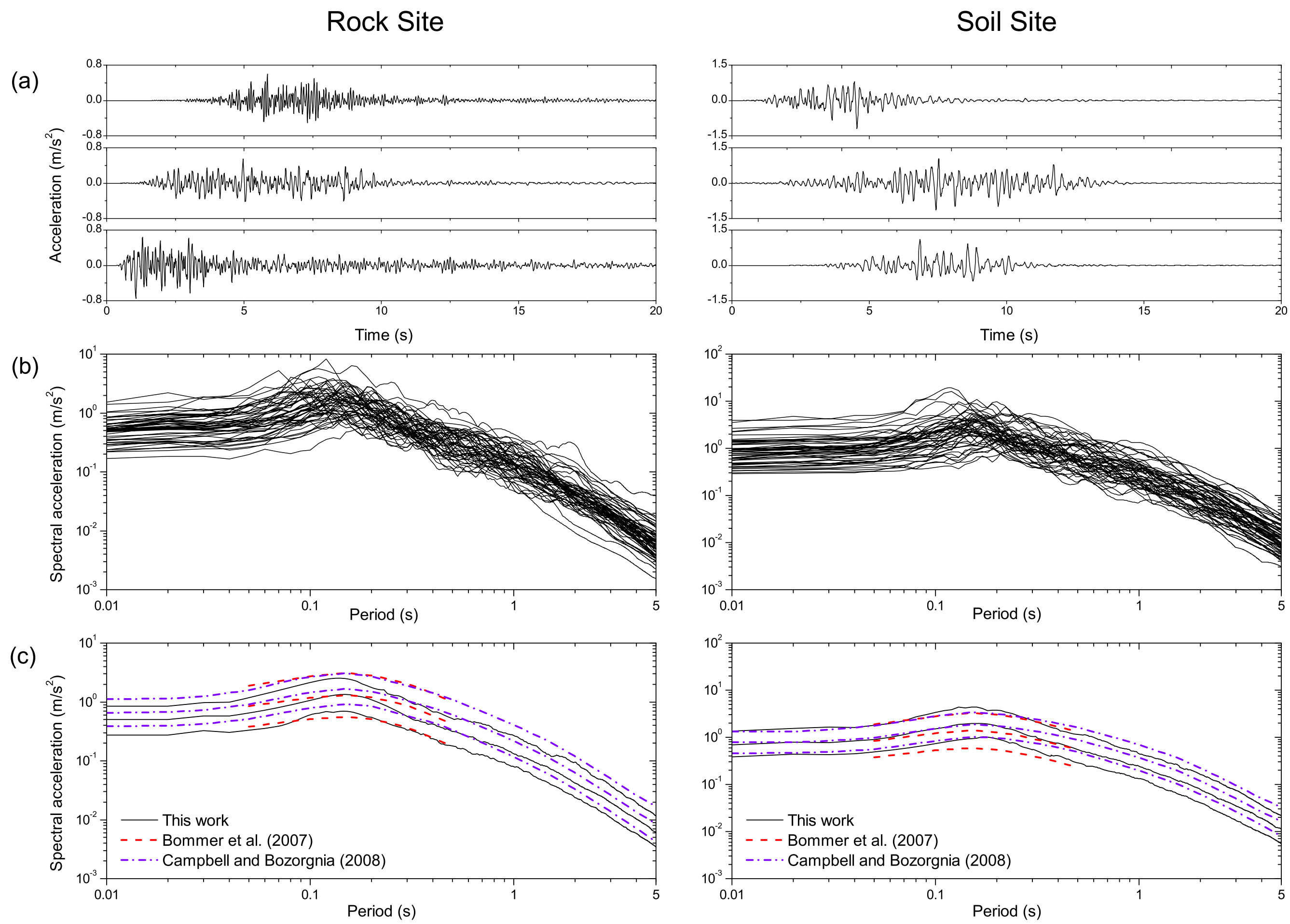

Fig. 7. Summary of the accelerogram definition compatible with earthquake magnitude M_w5.3 at 15 km distance in rock and soil sites. (a) sample of three simulations; (b) 5% damped spectral acceleration of 50 sample simulations; (c) median, 84th and 16th percentiles of the 200 simulations compared with two GMPEs suitable for UK use.

calibrated for active crustal regions. Both GMPEs were used to define the current national seismic hazard map for the UK (Musson and Sargeant, 2007).

Nonlinear time-history analysis were conducted for Models 1, 2a, 2b and 2c using the two bins of accelerograms defined in Section 4 for both the rock and soil sites. For illustration purposes, an example of the damage state for all models as scatter plots of AFSAs in Nodes 2 and 9 is shown in Fig. 8 for the rock site and in Fig. 9 for the soil site. In these figures, the more dots are located in the top-right quadrant, the higher is the seismic demand, and therefore, the higher is the nonlinear behaviour of the structural model. On the other hand, the more dots are located in the bottom-left quadrant, the lower is the seismic demand, and therefore, only linear behaviour can be expected of the structural model. From Figs. 8 and 9, it is possible to visually recognise that in Models 2a and 2b the seismic demand has been significantly reduced whereas in Model 2c the reduction is less effective when compared with Model 1.

Once the damage state for the four models has been calculated, the seismic risk was assessed. Full details about the methodology used in this work to assess the seismic risk can be found in Medel-Vera and Ji (2016a). For the sake of simplicity, this section only presents the results of performing risk assessment calculations. The methodology requires three input variables to calculate the seismic risk of NPPs: (i) the number of fragility curves for each critical component, (ii) the number of observations of the damage state (i.e. the number of accelerograms used to perform nonlinear time-history analysis), and (iii) the number of trials required to estimate the statistical distribution of the probability of unacceptable performance of the NPP. From the results reported by Medel-Vera and Ji (2016a), a sensitivity analysis showed that a robust estimation of the probability of unacceptable performance of the sample NPP is obtained when considering: (i) a family of 21 fragility curves for each critical component, (ii) 200 accelerograms to conduct nonlinear time-history analysis, and (iii) 2000 trials to generate a numerically stable statistical distribution for the probability of unacceptable performance. Therefore, such values were used when computing seismic risk of all structural models presented in this work.

The final cumulative distribution functions for the probability of unacceptable performance for all models are shown in Fig. 10a for the rock site and Fig. 10b for the soil site. The mean value for each distribution was taken as the benchmark value for the seismic risk for each model; these final values are indicated in Table 4. It is worth mentioning that for Model 2a, it was not possible to obtain a robust estimation of the probability of unacceptable performance. This is due to the fact that the seismic demand was considerably reduced: the probability of unacceptable performance obtained was zero when using 200 observations of the damage state. This does not mean that the probability of unacceptable performance is zero as the seismic demand, although notably low, is greater than zero (see Figs. 8 and 9). This means that the probability of unacceptable performance is an extremely small number that needs a much greater number of observations of the damage state

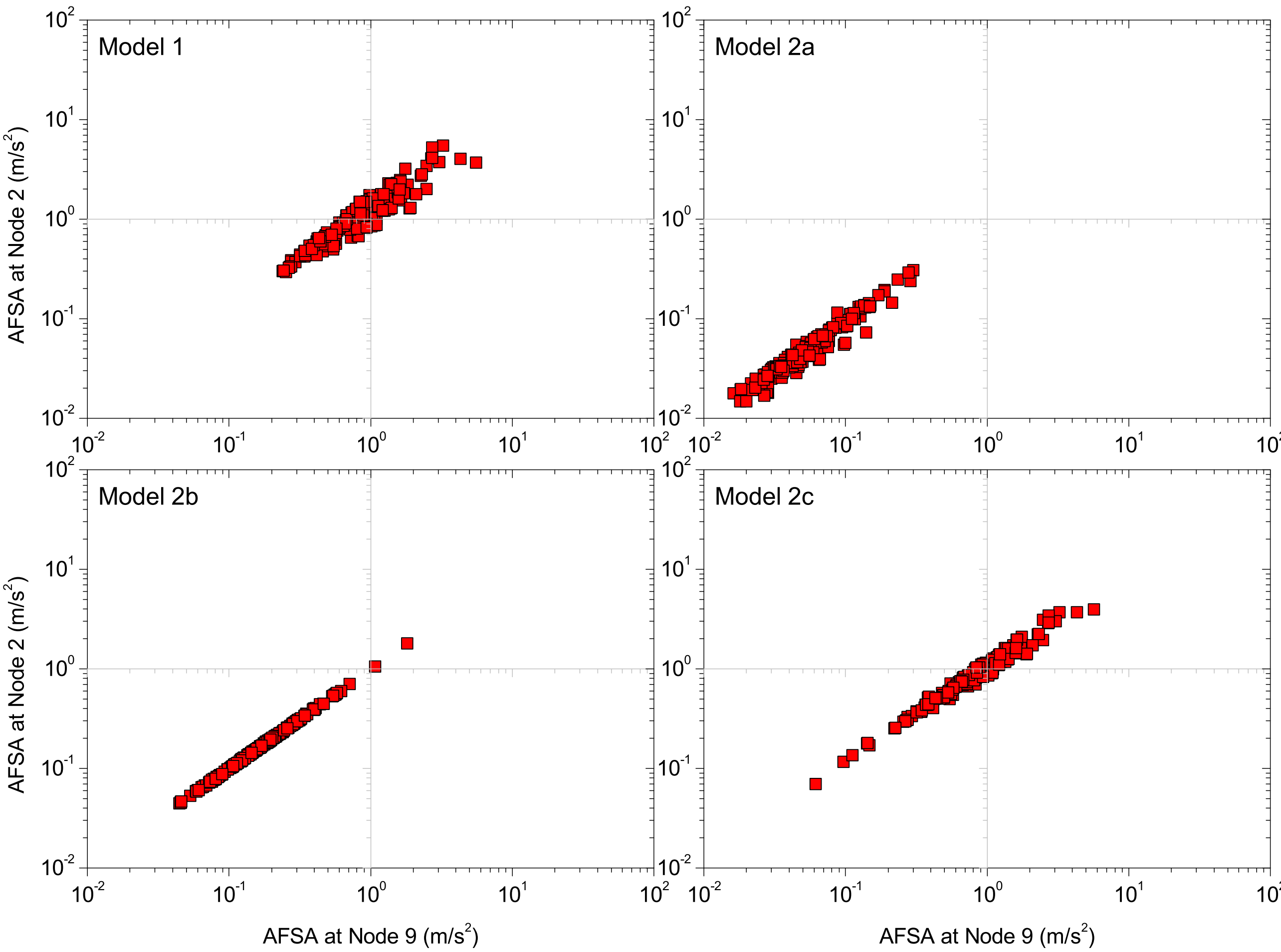

Fig. 8. Scatter plots of AFSAs at Nodes 2 and 9 for all models in the rock site.

and trials to be captured by the methodology used. In order to find a first approximation for the probability of unacceptable performance of Model 2a, the number of accelerograms was enlarged to 400, the number of fragility curves was increased to 201 and the number of trials was augmented to 6000, in order to obtain a value for the probability of unacceptable performance greater than zero. The values shown in Fig. 10 and Table 4 associated to Model 2a are only intended to give a rough approximation for the probability of unacceptable performance as different approaches need to be used to make a statistically robust estimation. However, for structural engineering purposes, the values reported for Model 2a are sufficient to establish a remarkable reduction of risk when compared with the base case.

From Fig. 10 and Table 4, it is possible to see that Model 2a achieves the largest reduction of seismic risk: the use of low-damping rubber bearings plus viscous dampers that introduce a critical damping of η = 10% to the sample NPP can reduce the risk of about 4–5 orders of magnitude depending on the type of soil. Model 2b also achieves an important reduction: the use of lead–rubber bearings that introduce a critical damping of η = 20% can achieve a reduction of about 2 orders of magnitude. In this sense, viscous damping seems to maximise the reduction of risk in comparison with hysteretic damping for UK seismic conditions. Model 2c achieves the lesser effective reduction: the introduction of linear viscous dampers that introduce η = 30% to protect the critical components of the sample NPP without seismically isolating it, reduces the risk ranging from 24.2% to 33.1% for soil and rock sites, respectively. From Table 4, it is also possible to realise that all strategies of seismic protection systems considered in this work were effective in reducing the risk for UK seismic conditions; however, the efficiency is always greater in the rock site than the soil site. These results are stated from a predominantly structural engineering viewpoint and for a stand-alone seismic risk study. Therefore, they cannot be directly used for conditions different to what was assumed in this work.

5. Scenario-based IDA and unacceptable performance surfaces

In previous sections, all seismic risk assessments were made considering the single scenario that contributed most strongly to the hazard of the site selected, namely, an earthquake M_w 5.3 located at R_{hyp} = 15 km. As mentioned in Section 4, this scenario was obtained as a result of the deaggregation of the seismic hazard curve of the site selected. Nevertheless, the determination of the controlling seismic scenario depends on the spatial seismicity modelling and ground motion modelling when the seismic hazard assessment of the site is conducted. Goda et al. (2013) reported that for the UK, "dominant scenario events identified based on different smoothing approaches vary significantly, which may have important implications for advanced earthquake engineering applications (e.g., response spectral shape and record selection for nonlinear dynamic analysis)". For the particular case of the hypothetical nuclear site selected in this work, namely, Holyhead, Anglesey, Goda et al. (2013) showed that controlling scenarios for a structural period of 0.2 s may fall into scenarios that approximately comprises magnitudes $4.5 < M_w < 6.5$ and distances $10 < R_{hyp} < 60$ km depending on the spatial seismicity model and the attenuation

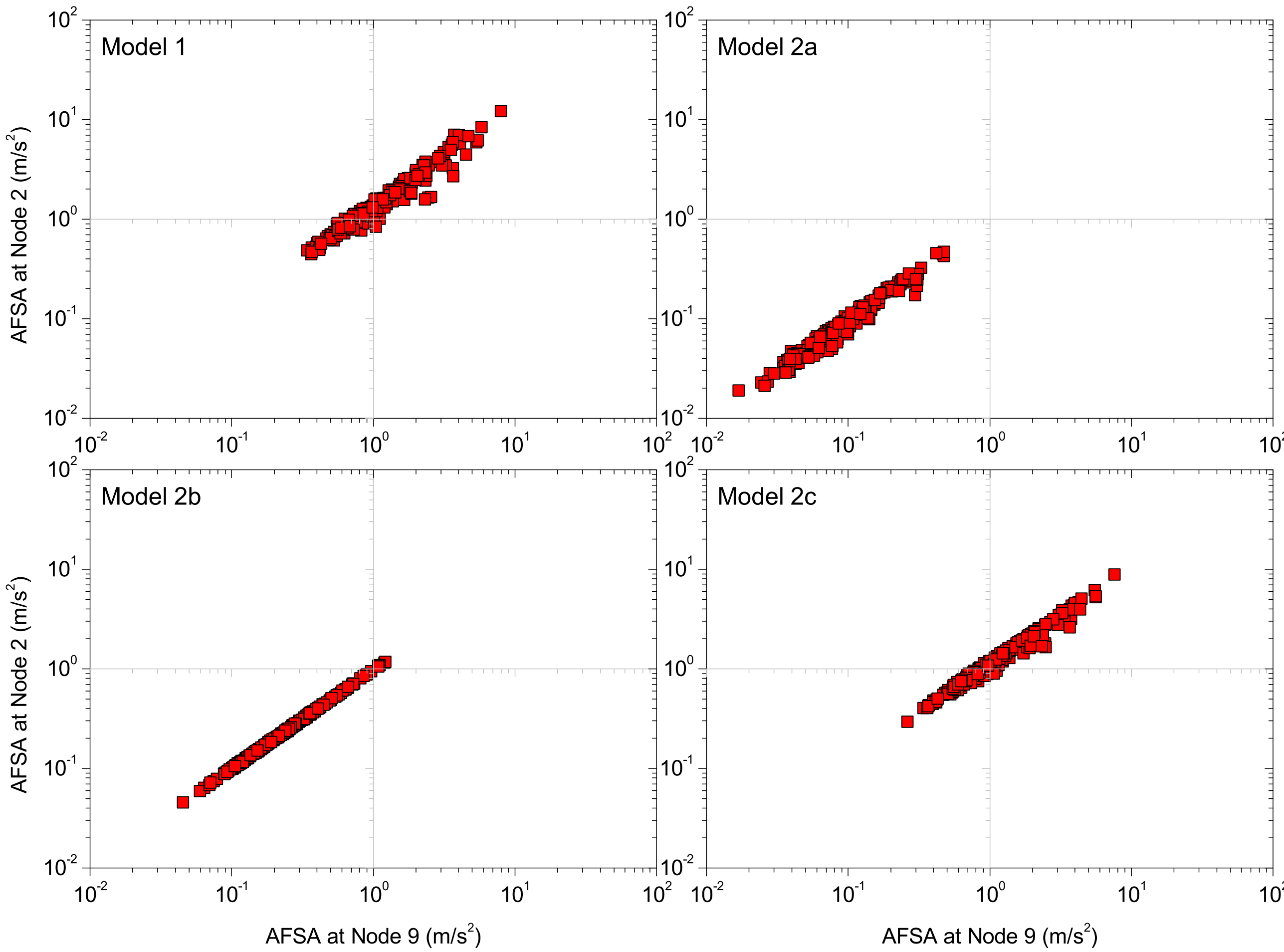

Fig. 9. Scatter plots of AFSAs at Nodes 2 and 9 for all models in the soil site.

models used. The modelling of these two effects plays a role particularly important in low-seismicity regions due to a large uncertainty about seismological and tectonic knowledge present in these regions (Atkinson and Goda, 2011). It is therefore desired to know how the seismic risk of the sample NPP used in this work changes when different controlling scenarios are taken into account. To assess such an effect, it is proposed to use a scenario-based incremental dynamic analysis (IDA) that is a straightforward approach based on the conventional IDA. Certainly, scenario-based IDA, as presented in the following section, is an approach that fits in scenario-based assessments of risk as described in Medel-Vera and Ji (2016a). Potential applications in intensity- and time-based assessments are left as a matter of further research.

Conventional IDA (Vamvatsikos and Cornell, 2002, 2004) is a powerful tool used in structural and earthquake engineering to estimate the dynamic behaviour of a structure that is forced to cover the complete range of structural response, from elastic behaviour to global dynamic instability. IDA establishes a relationship between an intensity measure (IM, e.g. PGA, PGV, 5% damped spectral acceleration at the structure's first-mode period, etc) of a multiply scaled suite of accelerograms and a damage measure (DM) of the structure, i.e. any observable structural output due to a given IM (e.g. peak roof drift, maximum peak inter-storey drift angle, etc.). Although IDA has been widely used in research (e.g. (Liao et al., 2007; Tagawa et al., 2008; Zareian and Krawinkler, 2007)), its main limitation has been regarded as the legitimacy of the rather simple approach to monotonically scale accelerograms to several IMs (Bradley, 2013; Kiani and Khanmohammadi, 2015). Careful selection of the seed accelerograms is needed to appropriately represent the changing characteristics of recordings (intensity distribution, frequency content and time duration) for progressively increasing IMs selected. Extensive literature is currently available aimed at the efficient selection of records in order to reduce the bias in the structural response (Haselton et al., 2009). This has led basis for several variations of IDA, e.g. Progressive IDA (Azarbakht and Dolšek, 2011) that is aimed at the optimal selection of accelerograms reducing the number of recordings needed in comparison with conventional IDA; and Adaptive IDA (Lin and Baker, 2013) that adaptively changes the suites of accelerograms at different IM levels to better reflect the variation of accelerograms' properties as to be in agreement with the site's hazard analysis. In this work, it is proposed to use a Scenario-based IDA for risk assessments purposes of NPPs. In this sense, the main difference between philosophies between traditional IDAs and the proposed methodology is that the latter does not seek to estimate the IM that would produce global dynamic instability of the structure; rather, it intends to estimate the IM that would produce a 100% of probability of unacceptable performance which in turn can be defined according to particular performance requirements of the NPP analysed. Certainly, as the seismic performance required for NPPs is more stringent than for conventional civil structures, it is expected that for the IM that produces 100% probability of unacceptable performance, global dynamic instability is still far from being reached.

The proposed procedure is explained by means of an example of application. This example aims at the estimation of the relative reduction of risk between a conventional NPP and its seismically

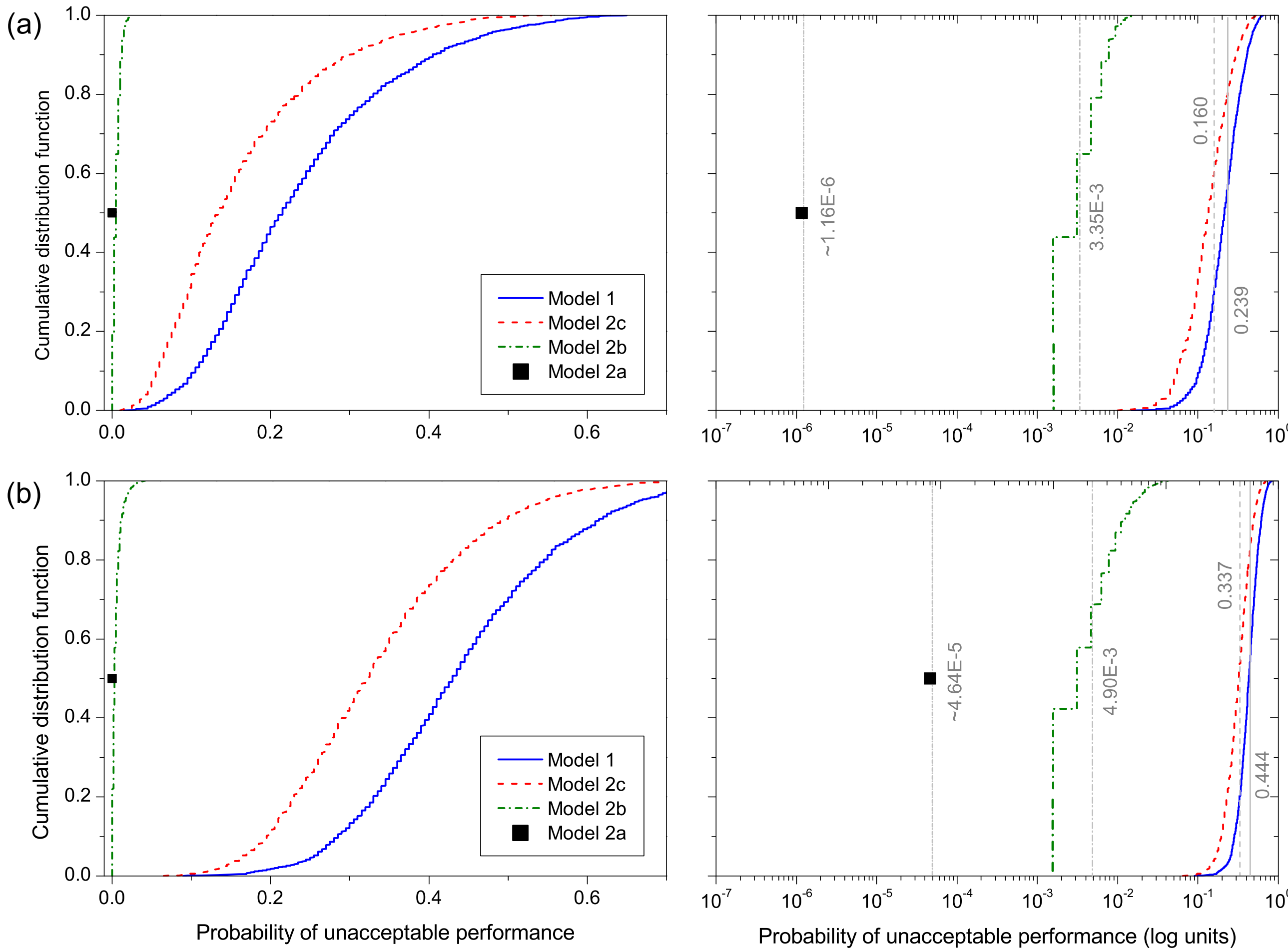

Fig. 10. Final cumulative distribution functions for the probability of unacceptable performance for all models. (a) rock site; (b) soil site.

Table 4
Summary of the seismic risk and reduction achieved for each model.

Model	Rock site		Soil site	
	Risk	Reduction	Risk	Reduction
1	0.239	–	0.444	–
2a	~1.16E−6	~5 orders of magnitude	~4.64E-5	~4 orders of magnitude
2b	3.35E−3	~2 orders of magnitude	4.90E-3	~2 orders of magnitude
2c	0.160	33.1%	0.337	24.1%

isolated version for different seismic scenarios. As shown in Section 4, the use of LDRB in combination with LVD gave the most effective reduction of risk for the particular scenario analysed. Also, Table 4 demonstrated that for UK conditions, seismic protection systems were more effective in rock compared to soil conditions. Therefore, Models 1 and 2a in rock conditions were used in the subsequent example. The following steps were taken to conduct scenario-based IDA:

5.1. Definition of IM

Unlike conventional-type IDAs that normally use only one IM, scenario-based IDA uses two IMs: earthquake magnitude (M_w) and distance-to-site (R_{epi}). In this light, the final product of scenario-based IDA is a surface rather than a curve. For the particular sample NPP used in this work, R_{epi} was increased from 10 km to 60 km in increments of 10 km. Then, for each R_{epi}, the earthquake magnitude M_w was increased from 4.5 to 6.5 in increments of 0.2. Then, for each scenario defined by the pair (M_w, R_{epi}), bins of accelerograms were simulated using the stochastic accelerogram model for NW Europe (Medel-Vera and Ji, 2016b) used also in Section 4, making a total of 66 bins of 400 accelerograms each, i.e. 26,400 simulated accelerograms (it is worth mentioning that 200 accelerograms of each bin were used for Model 1 whereas the 400 recordings of each bin were used for Model 2a, following the results reported in Section 4). In this sense, instead of progressively scale accelerograms as is done in traditional IDA methodologies, in the proposed methodology an epicentral distance is fixed and the earthquake magnitude is progressively increased to then stochastically simulate accelerograms compatible with that given scenario. This definition of seismic inputs explicitly guarantees that accelerograms' properties are preserved when the IMs are increased requiring no scaling, selection and matching techniques. A small sample of 6 bins of the simulated accelerograms is shown in Fig. 11. In this Figure, two accelerograms plus the 5%-damped spectral acceleration of 50 simulations are shown for the bins of

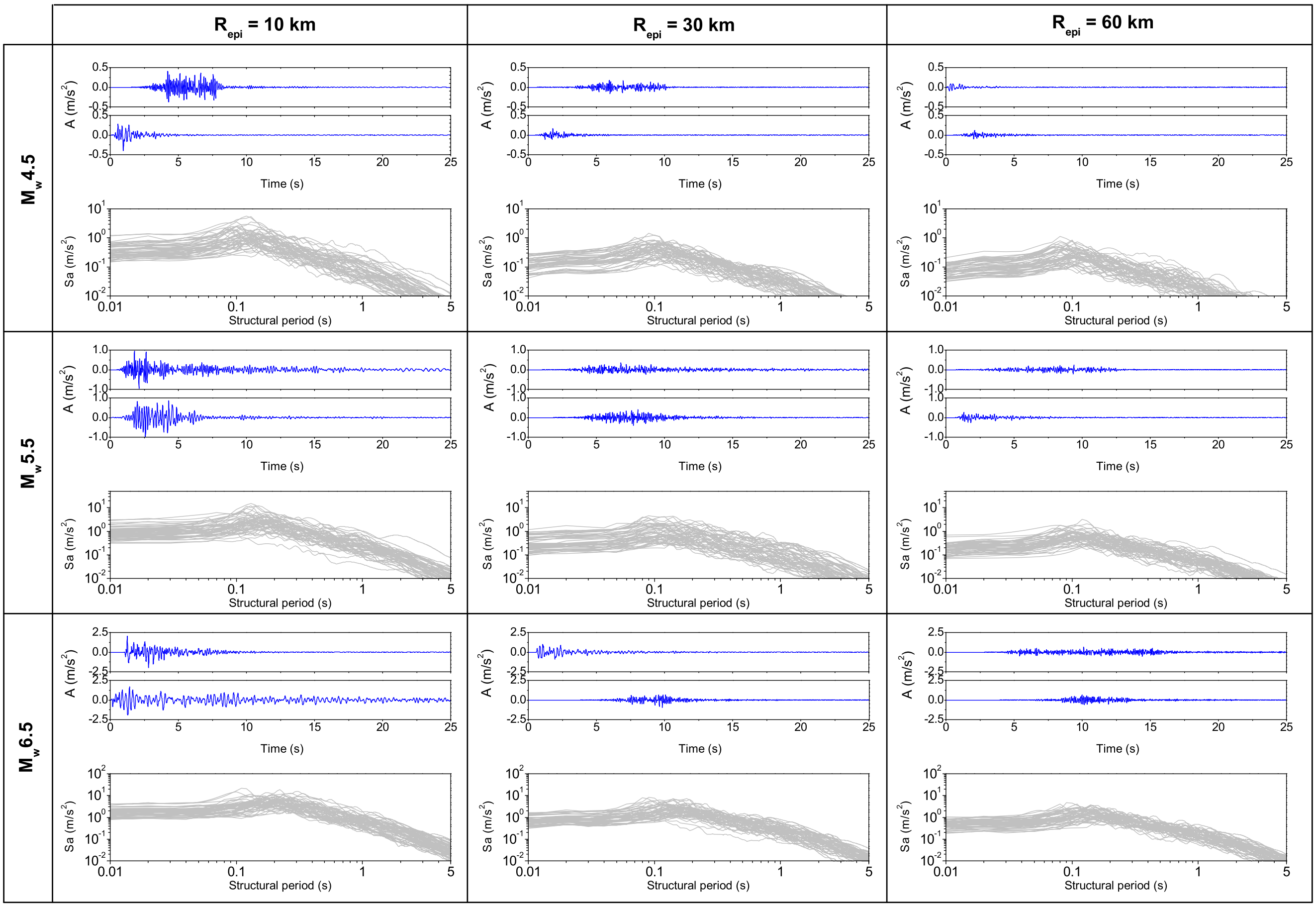

Fig. 11. Sample of bins of simulated accelerograms. two simulations plus 5%-damped response spectra for 50 simulations for the bins of magnitudes M_w 4.5, 5.5 and 6.5 for the epicentral distances R_{epi} 10, 30 and 60 km.

magnitudes M_w 4.5, 5.5 and 6.5 for the epicentral distances R_{epi} 10, 30 and 60 km.

5.2. Definition of DM

As the analysis proposed is intended to use for risk assessments, the DM used in this work was the mean probability of unacceptable performance of the sample NPP as defined in Section 4. Nevertheless, it is worth mentioning that the underlying measure for the seismic demand used to define the probability of unacceptable performance was the AFSA over a frequency range of 1–33 Hz for calculated in all critical components.

5.3. Assessment of risk

Once the IM and DM are defined, nonlinear time-history analyses were performed using Models 1 and 2a and the risk was assessed following the methodology stated in Section 4. For simplicity, the mean value was taken as the benchmark value for the probability of unacceptable performance for both models.

5.4. Generation of unacceptable performance surfaces

After the risk is assessed, the post-processing of the results basically involves the generation of surfaces of the unacceptable performance as a function of M_w and R_{epi}. In this example, interpolation of discrete points was made (i) for the earthquake magnitude in units of 0.1 and (ii) for the epicentral distance in units of 5 km; hence, the matrix of results (magnitude, distance) is of size 21 × 11. For Model 1, the colour map surface for the mean for the probability of unacceptable performance is shown in Fig. 12. From this figure, it is possible to see that the surface has a nonlinear behaviour reaching a maximum of 88.2% for the scenario M_w6.5, R_{epi}10 km.

Similarly, for Model 2a, the colour map surface for the mean probability of unacceptable performance is shown in Fig. 13. All scenarios analysed in this figure were within the linear range of the structural response, reaching a maximum value of 0.13% for the scenario M_w6.5, R_{epi}10 km. For this scenario, the demand of displacement on the isolation system was less than 7 cm for which the devices are expected to fully remain in their elastic range. In this example, both Figs. 12 and 13 must not be taken in absolute terms, but rather, in terms of the relative performance between them. In this sense, it is of interest to study the behaviour of the risk reduction for all the scenarios considered.

As it can be of interest to study the behaviour of risk reduction of the sample NPP for all dominant scenarios, the relative performance shown in Figs. 12 and 13 was also analysed. Fig. 14 shows both surfaces of unacceptable performance of Models 1 and 2a in log units as stacked wire frames. In this figure, it is possible to see that there are several orders of magnitude separating both surfaces suggesting the effectiveness of the isolation system in reducing the risk throughout the set of scenarios considered. In order to gain an in-depth understanding of the reduction of risk, Fig. 15 shows a spline-smoothed contour plot for the reduction of risk defined as the ratio between Models 1 and 2a. In general, this fig-

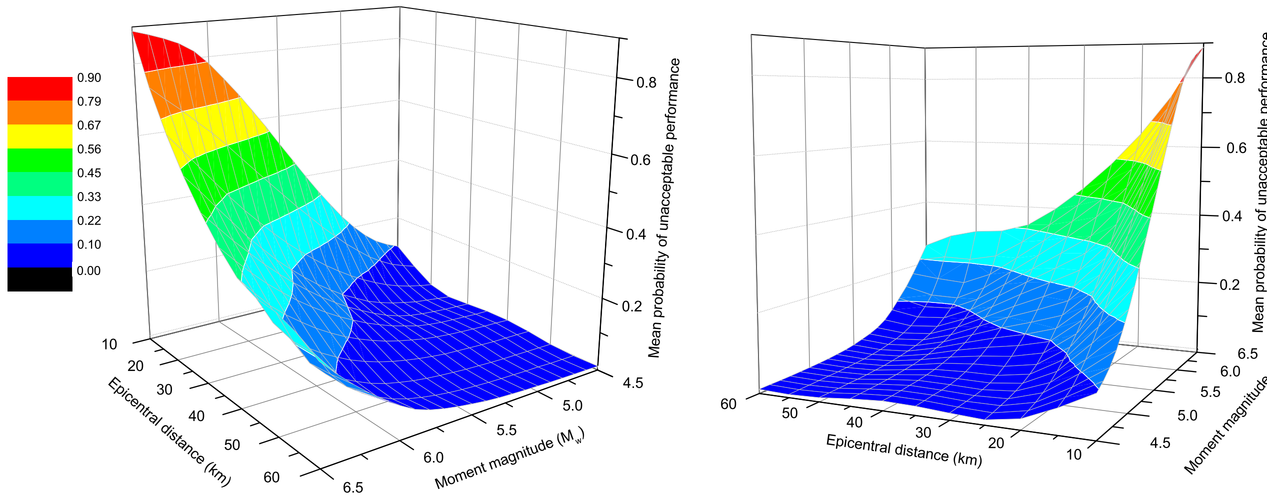

Fig. 12. Colour map surface for the mean unacceptable performance of Model 1. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

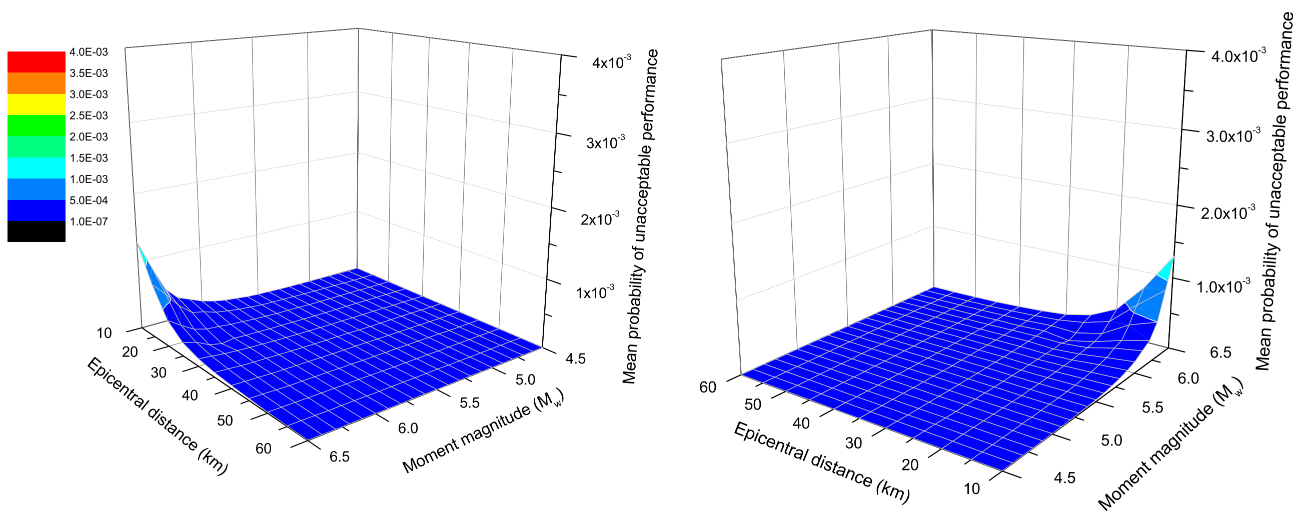

Fig. 13. Colour map surface for the mean unacceptable performance of Model 2a. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

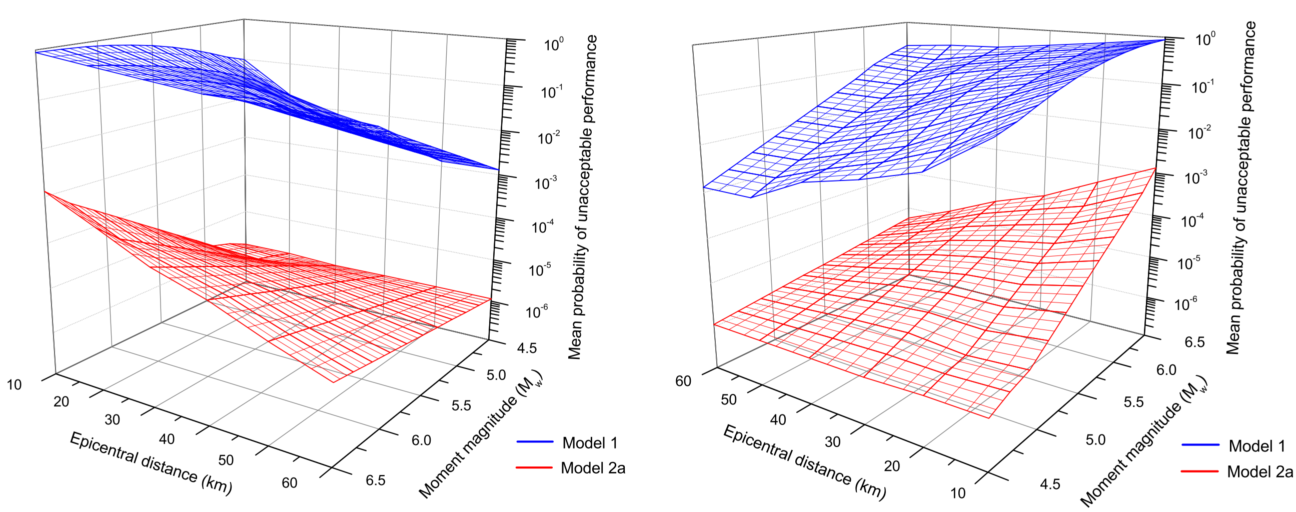

Fig. 14. Mean unacceptable performance surface (wire frame) for Models 1 and 2a (in log scale).

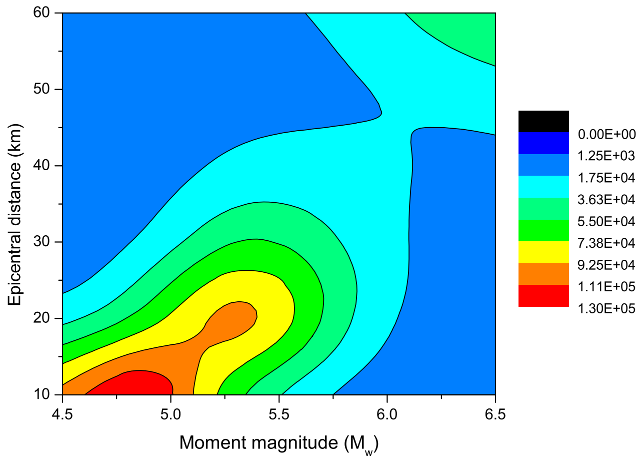

Fig. 15. Spline-smoothed contour plot for the reduction of risk between Model 1 and Model 2a.

ure shows that the reduction of risk ranges from 3 to 5 orders of magnitude while for most scenarios the reduction is in the region of 4 orders of magnitude.

6. Discussion and conclusions

6.1. Discussion

6.1.1. On the determination of the unacceptable performance for seismically isolated models

As mentioned in Section 4, the seismic demand on critical components of seismically isolated nuclear reactor buildings (in particular Model 2a in this work) is significantly reduced by the use of isolation devices. This reduction leads to a probability of unacceptable performance that is an extremely small number which conventional SPRA approaches cannot capture with total accuracy. These events of exceptionally low probability of occurrence fall into the so-called rare events simulation domain for which efficient approaches that can adaptively sample the input are available in the literature. Examples of such procedures are: line sampling (Schuëller et al., 2004), horseracing simulation (Zuev and Katafygiotis, 2011), subset simulation (Au and Beck, 2001), etc. Though the values reported in this work for the probability of unacceptable performance of seismically isolated NPPs are considered acceptable for structural engineering purposes, adaptive sampling procedures could be used to improve the accuracy of the values obtained in this work.

6.1.2. Recommended approach for using SPS in potential NPPs in UK seismic conditions

Results obtained in this work suggest that for UK seismic conditions, the use of low-damping rubber bearings and supplemental viscous dampers would be more effective in reducing the seismic risk than lead–rubber bearings. This result is more in agreement with the approach followed by the JHR and ITER reactor buildings in contrast with the devices selected for the APR1400, ASTRID and ALFRED reactor buildings. Also, this result obtained in terms of probabilistic risk assessment confirms the early attempts (although unfruitful) based on small-scale experimental tests to encourage the use LDRB and VD in nuclear reactor buildings in the UK (Austin et al., 1991). The devices recommended for potential nuclear deployments in the UK have several advantages. Low-damping rubber bearings are: (i) simple to manufacture as the compounding and bonding process to steel is well understood, (ii) easy to model, and (iii) their mechanical response is unaffected by rate, temperature, history, or ageing (Naeim and Kelly, 1999). As these devices are able to dissipate little energy, a supplementary damping system based on viscous dampers seems to be an efficient alternative. Viscous dampers are: (i) activated at low displacements, (ii) require minimal restoring force, (iii) easy to model, and (iv) their mechanical properties are largely frequency- and temperature-independent (Symans et al., 2008). It is worth remembering that this result is stated from a predominately structural engineering viewpoint and for the stand-alone seismic risk study presented in this work. Other important aspects such as through-life operational safety and licensing aspects of the seismic protection systems were not considered in this work.

6.1.3. Use of computer time of scenario-based IDA

It is acknowledged that the proposed methodology of scenario-based IDA to determine unacceptable performance surfaces of NPPs is computationally demanding. The major task contributing to the use of computer resources is the realisation of a large number of nonlinear time-history analyses of a structural model of the NPP. Minor tasks contributing to the use of computer resources are: (i) the simulation of accelerograms compatible with seismic scenarios of interest and (ii) performing the analysis of risk (SPRA) for each scenario of interest. For real NPP projects, structural models could reach several thousands of degrees of freedom possessing a significant amount of systems, equipment and critical components whose seismic response need to be analysed in detail. Nevertheless, the extremely high criticality of NPPs projects are deemed worthy of investing such resources. Nowadays, powerful computational resources are available to the technical community to perform such extremely demanding tasks, e.g. ARCHER, the UK National Supercomputing Service, a system that possesses 118,080 processor cores enabling it to perform 3×10^{14} instructions per second.

6.2. Conclusions

The work conducted for this article has led to the following conclusions:

- The use of low-damping rubber bearings plus dampers that provide a critical damping (viscous) of $\eta = 10\%$ seems to be an effective approach for UK seismic conditions. This approach was more efficient to reduce the risk than the use of lead rubber bearings that provide a critical damping (hysteretic) of $\eta = 20\%$. This is due to the fact that hysteretic damping-based systems need to be subjected to a rather significant demand of displacement in order to develop their full capacity to dissipate energy. A relatively high unused capacity of energy dissipation of these systems is likely to occur if used in the low seismic environment of the British Isles. As viscous damping-based systems are generally activated at low demand of displacements, they are more suitable for the UK seismic conditions. Regarding the foundation soil, the efficiency in reducing such risk was always greater in rock conditions in comparison with soil sites.
- Scenario-based IDA is an intuitive and straightforward methodology to study how the seismic risk of NPPs changes when considering different controlling scenarios. The procedure simplifies the assembly of accelerograms compatible with the seismic scenarios of interest by means of direct stochastic simulation. This methodology avoids the use of procedures normally used in conventional-type IDAs, such as (i) the correct selection of seed accelerograms in order to avoid bias in the structural response, and (ii) the use of scaling procedures in order to match spectral shapes predicted by ad-hoc GMPEs. The final product of scenario-based IDA is a surface rather than a curve as obtained in traditional IDAs as it provides the probability of unacceptable performance of NPPs as a function of

earthquake magnitude (M_w) and distance-to-site (R_{epi}). The necessity of conducting scenario-based IDA for NPPs lies on the high uncertainty in determining controlling seismic scenarios in areas of medium-to-low seismicity. Therefore, it is vital to study how the seismic risk of NPPs changes by covering the entire range of scenarios likely to occur in the site selected using a rational approach to model the seismic input. The main disadvantage of scenario-based IDA is that it is very demanding in terms of computer resources. For real NPP projects in which structural models can reach several thousands of degrees of freedom, the performing of a high number of nonlinear dynamic analyses of a structural model is the main barrier that needs to be overcome. Nevertheless, the necessity of making appropriate risk-informed decisions in mission-critical projects such as nuclear power stations justifies the use of such expensive technical resources. Nowadays, the advent of supercomputing systems can ease the performing of such extremely demanding tasks.

For the example analysed in this work based on a representative UK nuclear site, it was found that: (i) for conventionally constructed NPPs, the seismic risk rapidly increases with a nonlinear behaviour especially for scenarios defined by greater magnitudes and shorter distances; (ii) for seismically isolated NPPs, the seismic risk of the sample NPP is remarkably reduced as the structural response is always in the linear range for all scenarios analysed. In terms of the relative performance of these models, it was found that reduction of risk ranges from 3 to 5 orders of magnitude while for most scenarios the reduction is in the region of 4 orders of magnitude. It can be concluded that the potential use of low-damping rubber bearings and supplemental viscous dampers would be more effective in reducing the seismic risk for the new generation of nuclear reactor buildings in the UK.

References

Alemberti, A., Frogheri, M.L., Hermsmeyer, S., Ammirabile, L., Smirnov, M., Takahashi, M., Smith, C.F., Wu, Y., Hwang, I.S., 2014. Lead-cooled Fast Reactor (LFR) Risk and Safety Assessment. White Paper. Revision 8. Generation IV International Forum.

Atkinson, G.M., Goda, K., 2011. Effects of seismicity models and new ground-motion prediction equations on seismic hazard assessment for four Canadian cities. Bull. Seismol. Soc. Am. 101, 176–189.

Au, S.-K., Beck, J.L., 2001. Estimation of small failure probabilities in high dimensions by subset simulation. Probab. Eng. Mech. 16, 263–277.

Austin, N.M., Hattori, S., Rodwell, E., Womack, G.J., 1991. UK contribution to CEGB EPRI CRIEPI program on seismic isolation. Nucl. Eng. Des. 127, 253–264.

Azarbakht, A., Dolšek, M., 2011. Progressive incremental dynamic analysis for first-mode dominated structures. J. Struct. Eng. 137, 445–455.

Bhaumik, L., Raychowdhury, P., 2013. Seismic response analysis of a nuclear reactor structure considering nonlinear soil-structure interaction. Nucl. Eng. Des. 265, 1078–1090.

Bignan, G., Bravo, X., Lemoine, P.M., Maugard, B., 2011. The Jules Horowitz reactor: a new European MTR (material testing reactor) open to international collaboration: update description and focus on the modern safety approach. In: IAEA International Conference on Research Reactors: Safe Operation and Effective Utilization. International Atomic Energy Agency, Rabat, Morocco.

Bommer, J.J., Stafford, P.J., Alarcón, J.E., Akkar, S., 2007. The influence of magnitude range on empirical ground-motion prediction. Bull. Seismol. Soc. Am. 97, 2152–2170.

Bradley, B.A., 2013. A critical examination of seismic response uncertainty analysis in earthquake engineering. Earthquake Eng. Struct. Dynam. 42, 1717–1729.

Campbell, K.W., Bozorgnia, Y., 2008. NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10. Earthquake Spectra 24, 139–171.

CEA, 2012. 4th Generation Sodium-Cooled Fast Reactors – The ASTRID Technological Demonstrator. CEA (Commissariat à l'énergie atomique et aux énergies alternatives) – Nuclear Energy Division, Saclay, France.

Chopra, A., 1995. Dynamics of Structures. Theory and Applications to Earthquake Engineering. Prentice Hall, New Jersey.

Elkhoraibi, T., Hashemi, A., Ostadan, F., 2014. Probabilistic and deterministic soil structure interaction analysis including ground motion incoherency effects. Nucl. Eng. Des. 269, 250–255.

Forni, M., 2015. SILER Report Summary, Final Report Summary – SILER (Seismic-Initiated Events Risk Mitigation in LEad-Cooled Reactors). European Union, Bologna, Italy.

Forni, M., Poggianti, A., Dusi, A., 2012. Seismic isolation of nuclear power plants. In: Proceedings of the 15th World Conference on Earthquake Engineering.

Goda, K., Aspinall, W., Taylor, C.A., 2013. Seismic hazard analysis for the U.K.: sensitivity to spatial seismicity modelling and ground motion prediction equations. Seismol. Res. Lett. 84, 112–129.

Haselton, C., Baker, J., Bozorgnia, Y., Goulet, C.A., Kalkan, E., Luco, N., Shantz, T., Shome, N., Stewart, J.P., Tothong, P., Watson-Lamprey, J., Zareian, F., 2009. Evaluation of Ground Motion Selection and Modification Methods: Predicting Median Interstory Drift Response of Buildings, PEER Report 2009/01. Pacific Earthquake Engineering Research Center, California, Berkeley.

Hirano, M., Yonomoto, T., Ishigaki, M., Watanabe, N., Maruyama, Y., Sibamoto, Y., Watanabe, T., Moriyama, K., 2012. Insights from review and analysis of the Fukushima Dai-ichi accident. J. Nucl. Sci. Technol. 49, 1–17.

HSE, 2011. Health and Safety Executive – Office for Nuclear Regulation. T/AST/013 – Issue 4: Technical Assessment Guide – External Hazards.

Huang, Y.-N., Whittaker, A., Luco, N., 2011. A probabilistic seismic risk assessment procedure for nuclear power plants: (II) Application. Nucl. Eng. Des. 241, 3985–3995.

Huang, Y.-N., Whittaker, A.S., Luco, N., 2010. Seismic performance assessment of base-isolated safety-related nuclear structures. Earthquake Eng. Struct. Dynam. 39, 1421–1442.

Kiani, J., Khanmohammadi, M., 2015. New approach for selection of real input ground motion records for incremental dynamic analysis (IDA). J. Earthquake Eng. 19, 592–623.

Kostarev, V.V., Petrenko, A.V., Vasilyev, P.S., 2003. A new method for essential reduction of seismic and external loads on NPP's structures, systems and components. In: Transactions of the 17th International Conference on Structural Mechanics in Reactor Technology (SMiRT 17).

Lee, H.-P., Cho, M.-S., Kim, K.-K., Maeng, S.-W., 2015. Development of laminated rubber bearings for the apr1400 nuclear power plant. In: 23rd Conference on Structural Mechanics in Reactor Technology (SMiRT 23), Manchester, UK.

Li, Z.X., Li, Z.C., Shen, W.X., 2005. Sensitivity analysis for floor response spectra of nuclear reactor buildings. Nucl. Power Eng. 26 (in Chinese).

Liao, K.-W., Wen, Y.-K., Foutch, D.A., 2007. Evaluation of 3D steel moment frames under earthquake excitations. I: modeling. J. Struct. Eng. 133, 462–470.

Lin, T., Baker, J., 2013. Introducing adaptive incremental analysis: a new tool for linking ground motion selection and structural response assessment. In: 11th International Conference on Structural Safety & Reliability, New York.

Llambias, J.M., 1993. Validation of seismic soil structure interaction (SSI) methodology for a UK PWR nuclear power station. In: 12th International Conference on Structural Mechanics in Reactor Technology (SMiRT 12), Stuttgart, Germany, p. Paper K07/02.

Llambias, J.M., Shepherd, D.J., Rodwell, M.D., 1993. Sensitivity of seismic structural response to interpretation of soils data. Soil Dyn. Earthquake Eng. 12, 337–342.

Magnox, 2011. Wylfa: Response to EU stress tests following the events at Fukushima, Japan.

Martelli, A., Forni, M., Clemente, P., 2012. Recent worldwide application of seismic isolation and energy dissipation and conditions for their correct use. In: Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon.

Medel-Vera, C., Ji, T., 2014. Seismic protection technology for nuclear power plants: a systematic review. J. Nucl. Sci. Technol. 52, 607–632.

Medel-Vera, C., Ji, T., 2016a. Seismic probabilistic risk analysis based on stochastic simulation of accelerograms for nuclear power plants in the UK. Prog. Nucl. Energy 91, 373–388.

Medel-Vera, C., Ji, T., 2016b. A stochastic ground motion accelerogram model for Northwest Europe. Soil Dyn. Earthquake Eng. 82, 170–195.

Meiswinkel, R., Meyer, J., Schnell, J., 2013. Design and Construction of Nuclear Power Plants. Ernst & Sohn, Berlin, Germany.

Moretti, G., Pasquali, U., 2013. Seismic Isolation of LFR Reactor Buildings. SILER International Educational Workshop, Rome, Italy.

Musson, R.M.W., 1996. The seismicity of the British Isles. Ann. Geophys. 39.

Musson, R.M.W., 2014. UK seismic hazard assessments for strategic facilities: a short history. Bollettino di Geofisica Teorica ed Applicata 55, 165–173.

Musson, R.M.W., Sargeant, S., 2007. Eurocode 8 seismic hazard zoning maps for the UK. British Geological Survey Technical Report, CR/07/125N.

Naeim, F., Kelly, J.M., 1999. Design of Seismic Isolated Structures: From Theory to Practice. Wiley.

NIA, 2012. Capability Report: Capability of the UK Nuclear New Build Supply Chain. Nuclear Industry Association, London.

Ostadan, F., Kennedy, R., 2014. Consistent site-response/soil-structure interaction analysis and evaluation. Nucl. Eng. Des. 269, 72–77.

Patel, C.C., Jangid, R.S., 2009. Seismic response of dynamically similar adjacent structures connected with viscous dampers. IES J. Part A Civil Struct. Eng. 3, 1–13.

Saxena, N., Paul, D.K., 2012. Effects of embedment including slip and separation on seismic SSI response of a nuclear reactor building. Nucl. Eng. Des.

Saxena, N., Paul, D.K., Kumar, R., 2011. Effects of slip and separation on seismic SSI response of nuclear reactor building. Nucl. Eng. Des. 241, 12–17.

Schuëller, G.I., Pradlwarter, H.J., Koutsourelakis, P.S., 2004. A critical appraisal of reliability estimation procedures for high dimensions. Probab. Eng. Mech. 19, 463–474.

Sollogoub, P., 2014. Seismic Isolation of Nuclear Installations in France. SILER International Educational Workshop, Rome, Italy.

Syed, M.B., Patisson, L., Curtido, M., Slee, B., Diaz, S., 2014. The challenging requirements of the ITER anti seismic bearings. Nucl. Eng. Des. 269, 212–216.

Symans, M., Charney, F., Whittaker, A., Constantinou, M., Kircher, C., Johnson, M., McNamara, R., 2008. Energy dissipation systems for seismic applications: current practice and recent developments. J. Struct. Eng. 134, 3–21.

Tagawa, H., MacRae, G., Lowes, L., 2008. Probabilistic evaluation of seismic performance of 3-story 3D one- and two-way steel moment-frame structures. Earthquake Eng. Struct. Dynam. 37, 681–696.

Tuñón-Sanjur, L., Orr, R.S., Tinic, S., Ruiz, D.P., 2007. Finite element modeling of the AP1000 nuclear island for seismic analyses at generic soil and rock sites. Nucl. Eng. Des. 237, 1474–1485.

Vamvatsikos, D., Cornell, C.A., 2002. Incremental dynamic analysis. Earthquake Eng. Struct. Dynam. 31, 491–514.

Vamvatsikos, D., Cornell, C.A., 2004. Applied incremental dynamic analysis. Earthquake Spectra 20, 523–553.

Wagenknecht, E., 1987. Response of a NPP reactor building under seismic action with regard to different soil properties. Nucl. Eng. Des. 104, 187–195.

Weightman, M., 2011. Japanese Earthquake and Tsunami: Implications for the UK Nuclear Industry. Office for Nuclear Regulation – HM Chief Inspector of Nuclear Installations, Bootle, Merseyside.

WNA, 2014. Nuclear Power Plants and Earthquakes, <http://www.world-nuclear.org>. World Nuclear Association, London, UK.

Wolf, J.P., 1989. Soil-structure-interaction analysis in time domain. Nucl. Eng. Des. 111, 381–393.

Zareian, F., Krawinkler, H., 2007. Assessment of probability of collapse and design for collapse safety. Earthquake Eng. Struct. Dynam. 36, 1901–1914.

Zuev, K.M., Katafygiotis, L.S., 2011. The Horseracing Simulation algorithm for evaluation of small failure probabilities. Probab. Eng. Mech. 26, 157–164.