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In this issue

2D Finite Element Analysis of the Seismic Response of an Earth Embankment

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Abstract

In this paper the seismic response of a real homogeneous earth dam, located in the south-east of Italy, is studied using a two-dimensional finite element code which implements the fully-coupled effective stress approach. The stress-strain behaviour of the involved soils is simulated through an advanced constitutive model developed within the framework of kinematic hardening and bounding surface plasticity. The dynamic response of the dam is illustrated in terms of signal amplification, permanent excess pore pressures and cumulated displacements during the shaking. The general performance of the numerical tool is tested by changing the size of the time step used in the dynamic simulations.

Introduction

The behaviour of earth dams subjected to seismic actions has traditionally been analysed using semi-empirical approaches or simplified numerical methods. More recently, the fully-coupled effective stress formulation for the solid-fluid interaction [1, 2] has received increasing attention in the field of dynamic analyses of earth embankments. The design of such structures necessarily requires the analysis of the effects of the dynamic loading conditions on their stability and serviceability. In this context, the use of a constitutive model capable of capturing the main features of the cyclic behaviour of soils, such as state dependency, early irreversibility, non-linearity, build-up of excess pore pressures, decrease of nominal stiffness and related hysteretic dissipation, is essential. The implementation of such non-linear constitutive models into numerical codes can significantly improve the predictive capabilities of the stress-strain response of large dams during static service conditions and under seismic loading (e.g. [3-7]).

The paper describes a preliminary numerical study of the dynamic behaviour of a real homogeneous earth dam using the 2D finite element code SWANDYNE II [8] and adopting the Rouainia & Muir Wood model [9] to simulate the mechanical behaviour of the involved clayey soils. The same earthquake record has been scaled to two different peak accelerations and applied at the bedrock level. The seismic response of the embankment is discussed in terms of modification of the input signal along the dam axis as well as earthquake induced permanent displacements and build-up of excess pore water pressures inside the structure. Finally, the performance of the numerical model is tested by changing the size of the time step employed in the dynamic simulations.

Numerical formulation

In the context of finite element analysis, assuming that the relative velocity of the fluid phase is negligible, the system of ordinary equation that results from the *u*-*p* formulation (in which *u* is the displacement of the solid phase and *p* is the pore fluid pressure) can be written as follows [2]:

$$\begin{cases} [M]\,\ddot{\mathbf{u}} + [C]\,\dot{\mathbf{u}} + [K]\,\mathbf{u} - [Q]\,\mathbf{p} = \mathbf{f}^s \\ [Q]^T\,\dot{\mathbf{u}} + [S]\,\dot{\mathbf{p}} + [H]\,\mathbf{p} = \mathbf{f}^p \end{cases} , \qquad (1)$$

where $[M]$ is the mass matrix, $[K]$ is the stiffness matrix, $[C]$ is the viscous damping matrix, $[Q]$ is the coupling matrix between the motion and flow equations, $[H]$ is the permeability matrix, $[S]$ is the compressibility matrix, $\mathbf{f}^p$ is the force vector for the fluid phase and $\mathbf{f}^s$ is the force vector for the solid phase. Frequency dependent viscous damping is usually included via the Rayleigh damping matrix:

$$[C] = \alpha\,[M] + \beta\,[K] \ , \qquad (2)$$

where the factors α and β are related to the modal damping coefficients according to the relationship:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2D}{\omega_m + \omega_n} \begin{Bmatrix} \omega_m \omega_n \\ 1 \end{Bmatrix} . \qquad (3)$$

These coefficients can be calculated by selecting a damping ratio D and two frequencies, ω_m and ω_n, outside which damping is larger than the damping ratio.

The algebraic counterparts of Eqs. (1) can be obtained by applying a time-integration scheme. Assuming that the values of displacements, pore pressures and their time derivatives, $\{\mathbf{u}_n, \dot{\mathbf{u}}_n, \ddot{\mathbf{u}}_n, \mathbf{p}_n, \dot{\mathbf{p}}_n\}$, have been obtained at time t_n, the integration consists of updating $\{\mathbf{u}_{n+1}, \dot{\mathbf{u}}_{n+1}, \ddot{\mathbf{u}}_{n+1}, \mathbf{p}_{n+1}, \dot{\mathbf{p}}_{n+1}\}$ at the next time step t_{n+1} according to the Generalised Newmark scheme. In particular, for the solid phase:

$$\begin{aligned} \ddot{\mathbf{u}}_{n+1} &= \ddot{\mathbf{u}}_n + \Delta\ddot{\mathbf{u}}_n \ , \\ \dot{\mathbf{u}}_{n+1} &= \dot{\mathbf{u}}_n + \left[\ddot{\mathbf{u}}_n + \beta_1 \Delta\ddot{\mathbf{u}}_n\right] \Delta t \ , \\ \mathbf{u}_{n+1} &= \mathbf{u}_n + \dot{\mathbf{u}}_n \Delta t + 0.5\left[\ddot{\mathbf{u}}_n + \beta_2 \Delta\ddot{\mathbf{u}}_n\right] \Delta t^2 \ . \end{aligned} \qquad (4)$$

Similarly for the fluid phase:

$$\begin{aligned} \dot{\mathbf{p}}_{n+1} &= \dot{\mathbf{p}}_n + \Delta\dot{\mathbf{p}}_n \ , \\ \mathbf{p}_{n+1} &= \mathbf{p}_n + \left[\dot{\mathbf{p}}_n + \beta_1^* \Delta\dot{\mathbf{p}}_n\right] \Delta t \ , \end{aligned} \qquad (5)$$

where the coefficients

$$\begin{aligned} \beta_1 &\geq 0.5 \ , \\ \beta_2 &\geq 0.5(0.5 + \beta_1)^2 \quad \text{and} \\ \beta_1^* &\geq 0.5 \end{aligned} \qquad (6)$$

are typically chosen for unconditional stability of the recurrence scheme.

The substitution of the above approximations into Eqs. (1) leads to a system of coupled non-linear equations which can be solved iteratively using the Newton-Raphson procedure.

The constitutive model, used for the first time in the dynamic analysis of a geotechnical boundary value problem, has been formulated for natural clays within the framework of kinematic hardening with some elements of bounding surface plasticity [9]. It is a rate-independent model and it takes into account the effects of damage to structure caused by irrecoverable plastic strains, resulting from sampling or geotechnical loading. The model is an extension of the well known Cam-Clay model. The decrease of stiffness with strain is controlled by an interpolation function which ensures a smooth movement of the elastic domain (which is enclosed in a small bubble) towards the bounding surface during loading. A scalar variable, which is a monotonically decreasing function of both plastic volumetric and shear strain, represents the progressive degradation of the material. The governing constitutive relations of the model are described in detail by [9] and are not reported here.

Case history

The Marana Capacciotti dam is a homogeneous clayey embankment located in Puglia, south-east of Italy. It has a height of 48m and overlays 42m of slightly overconsolidated alluvial silts and stiff overconsolidated silty clays before the bedrock is reached. A detailed description of the geometrical and geotechnical characteristics of the dam and its foundation is presented by [6, 7]. A mesh of 794 isoparametric quadrilateral finite elements with 8 solid nodes and 4 fluid nodes has been used in this study (see Figure 1), assuming plane strain free draining condition for all the analyses.

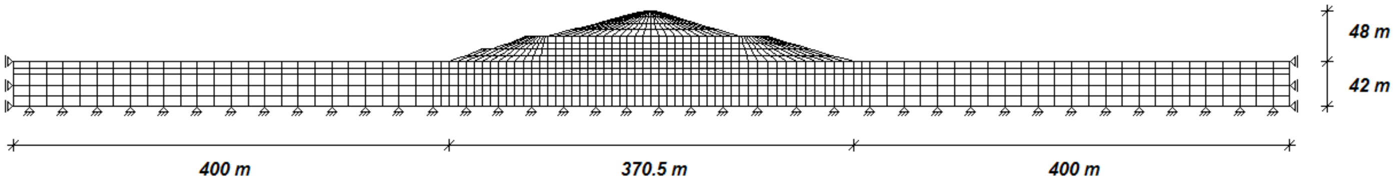

Figure 1. Adopted FE mesh and boundary conditions during static analyses.

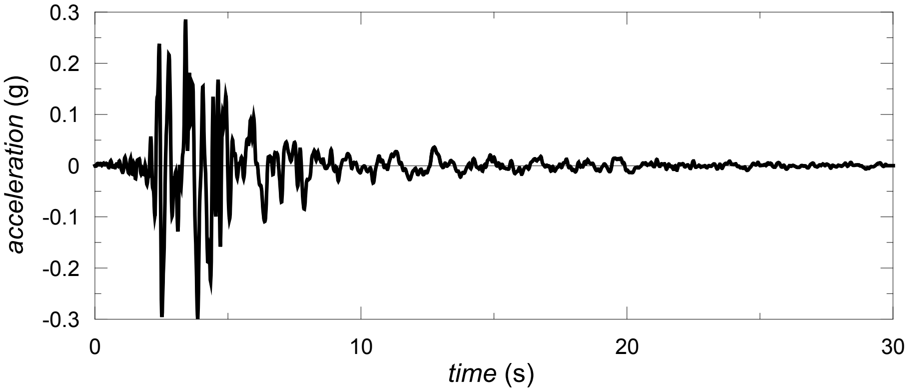

Figure 2. Acceleration time history of the Kalamata earthquake scaled to 0.30g.

The initial static stress state and the corresponding model internal variables have been obtained following the approach described by [6, 7]. The dynamic behaviour of the dam has been simulated by applying the horizontal component of the accelerogram registered at Kalamata (Greece) during the earthquake of September 1986 to the solid nodes at the base of the mesh as prescribed horizontal displacement time history. The input motion, which is characterised by a dominant frequency of 1.63Hz and a length of 29.75s, has been scaled to two different peak accelerations, equal to 0.30g (case 'a') and 0.15g (case 'b'), and filtered to a maximum frequency of 10Hz. The earthquake time history obtained by scaling the signal to a maximum acceleration of 0.30g is shown in Figure 2.

The dynamic analyses have been performed using tied-nodes boundary conditions along the vertical sides of the mesh. No numerical damping has been introduced through the time step integration scheme (i.e. $\beta_1 = \beta_2 = \beta_1^* = 0.5$), while only 2% of Rayleigh damping has been added in the dynamic simulations.

Figures 3a and 3b show the comparison between the input motion applied at the bedrock and the acceleration time histories computed at the base and at the crest of the embankment in terms of Fourier spectra for the two simulations. It can be seen that the amplification of the seismic signal essentially occurred between the base and the crest of the dam due to topography effects: in both cases, the maximum peak of the spectral acceleration is recorded at 0.81Hz with a second peak at 1.64Hz, corresponding to the dominant frequency of the bedrock signal.

The normalised profiles of maximum accelerations inside the dam (see Figure 4) indicate again the significant amplification of the input motion when it reaches the top of the dam. This is consistent with what has been reported in the literature [10].

Figure 5 shows, for both the dynamic analyses, the cumulated vertical displacements of different nodes selected along the dam axis at different depths from its crest (z = 0m). The non-linear and irreversible response of the earth dam subjected to the dynamic action leads to a crest settlement of 0.74m in the case of a peak input accelera-

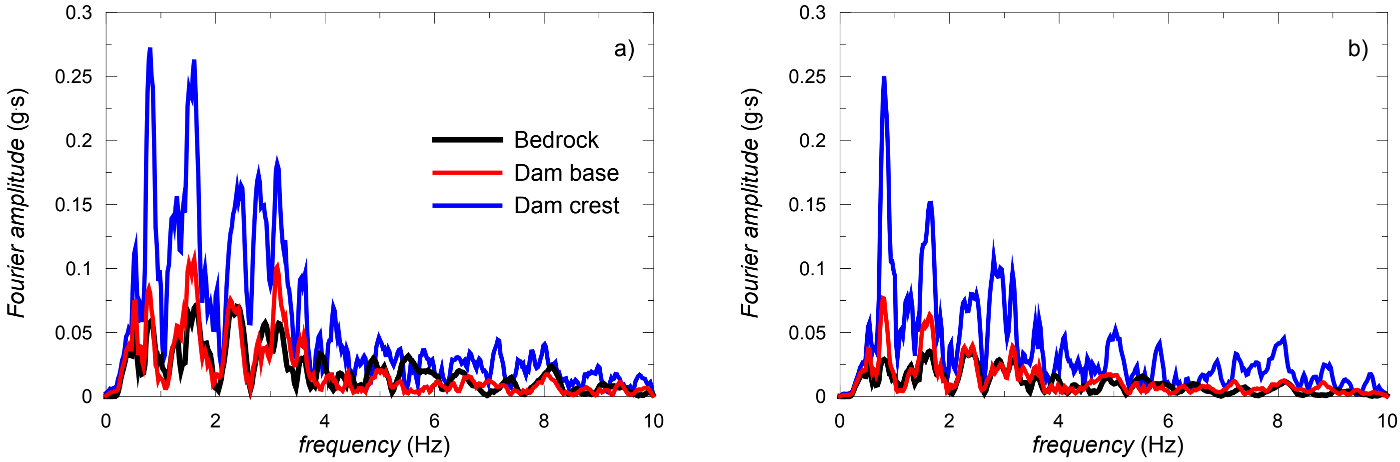

Figure 3. Comparison between Fourier spectra along the dam axis.

tion of 0.30g and to 0.20m in the case of the earthquake scaled to 0.15g, equivalent to the 28% and 8% of the service freeboard of the dam (2.6m), respectively. The overall behaviour of the embankment in terms of displacements indicates a stable response of the structure during and after the seismic action as the recorded time histories become constant immediately after the end of the earthquake in both simulations.

The contour lines of shear strain obtained at the end of the two simulations (reported, respectively, in Figures 6a and 6b in absolute values) show the seismic induced concentration of plastic strains propagating from the toe of the downstream slope into the alluvial silt layer.

This is accompanied by the build-up of excess pore water pressures inside the embankment throughout the shaking during both the dynamic analyses, as shown in Figures 7a and 7b for different depths from the crest along the dam axis. Although it was not directly imposed, the system behaves in undrained conditions during and after the shaking, the materials permeabilities being too low to allow the dissipation of the excess pore water pressures cumulated during the seismic action.

Finally, the numerical performance of the model has been tested by varying the time step sizes adopted in the dynamic simulations, as suggested by [11]. No significant differences were observed in the results of the dynamic analyses when the time step was reduced from 10ms to a minimum value of 1ms.

Conclusion

A preliminary numerical study of the dynamic behaviour of a real homogeneous earth dam using a fully-coupled non-linear approach has been presented in the paper. The adopted constitutive hypothesis, tested for the first time in dynamic conditions, and the employed solid-fluid interaction scheme have allowed to predict realistic permanent displacements of the embankment, due to seismic induced plastic strain accumulation, and the development of excess pore water pressures inside the structure throughout the shaking. No significant differences have been observed in the results of the dynamic simulations by varying the integration time step.

More generally, the paper has presented the use of an advanced numerical tool for the analysis of the seismic behaviour of a specific dam but the same fully-coupled non-linear approach should be employed in the future to investigate the dynamic response of any geotechnical boundary value problem in 2D conditions.

Acknowledgements

The help of prof. Andrew Chan (University of Birmingham, UK) in the implementation of the constitutive model into the finite element code is gratefully acknowledged.

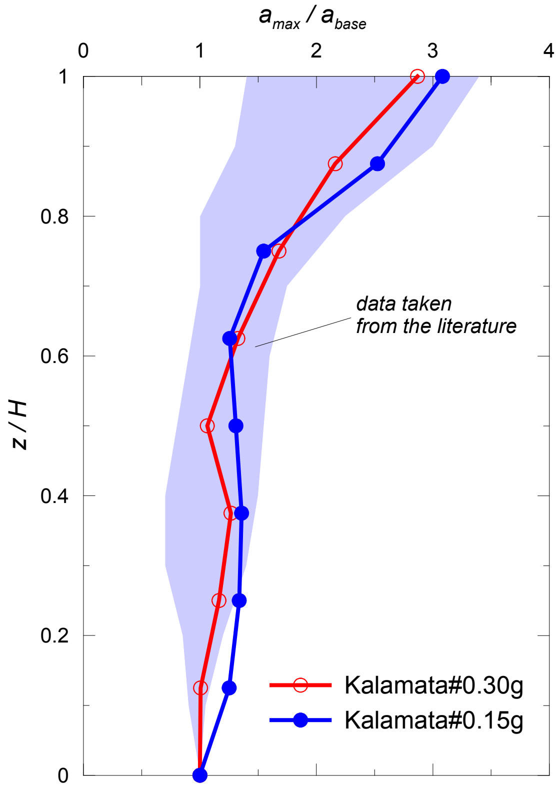

Figure 4. Normalised profiles of maximum accelerations inside the dam.

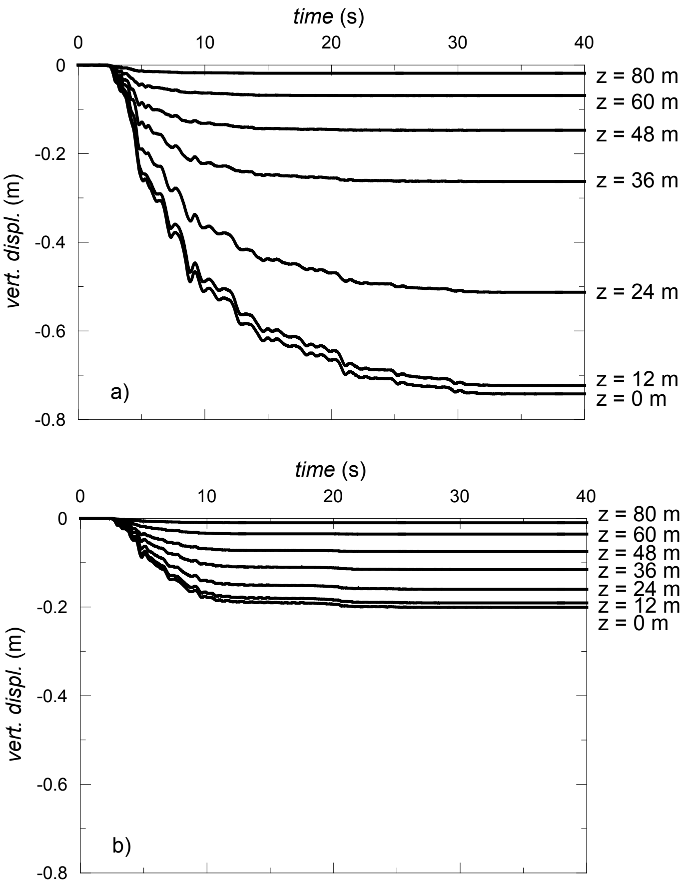

Figure 5. Vertical displacement time histories recorded along the dam axis.

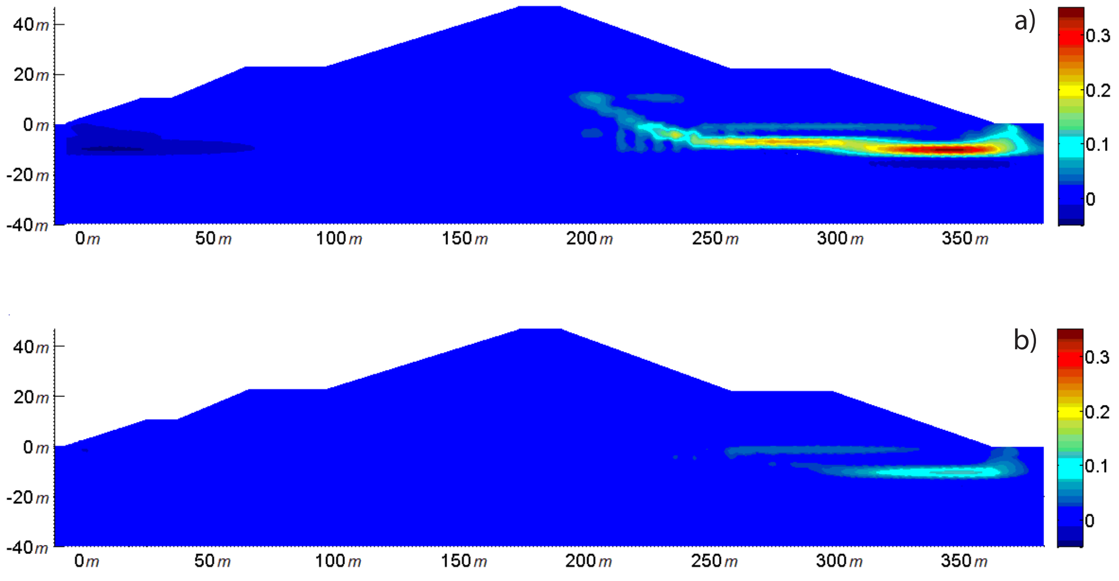

Figure 6. Contour lines of shear strain at the end of the earthquakes.

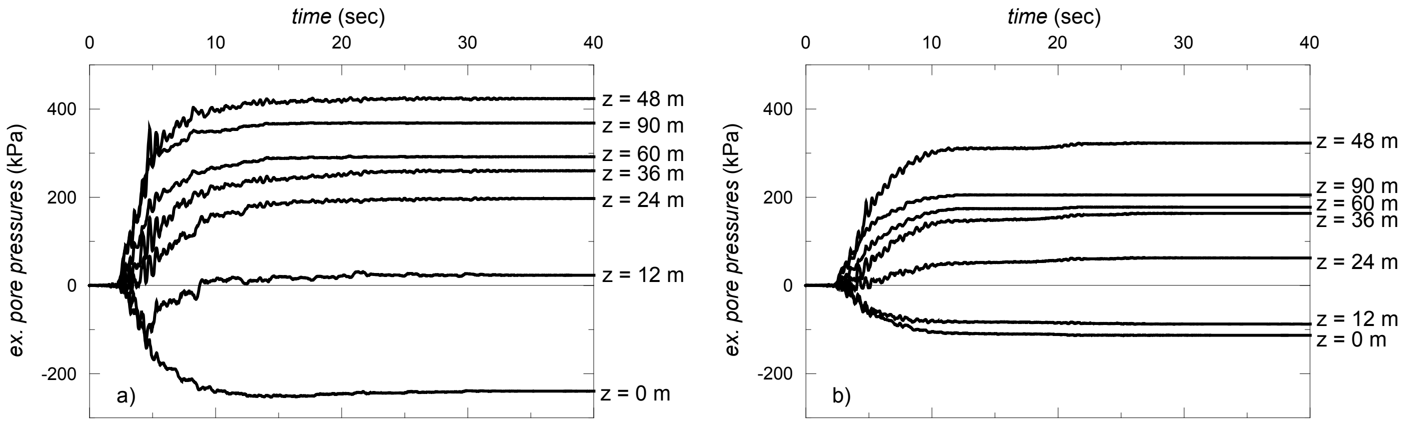

Figure 7. Excess pore water pressure time histories recorded along the dam axis.

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Julian Bommer makes his final contribution as SECED Technical Reporter on Engineering Seismology (a post he has held since 2003 and from which he stepped down at the 2010 SECED AGM) with a report on a major European project to harmonize the assessment of seismic hazard in Europe.

Harmonizing Seismic Hazard Assessment in Europe

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In their letter published in the October 2009 edition of the SECED Newsletter (volume 21, issue 4), Edmund Booth and Bryan Skipp noted that a major European project has been launched that is addressing issues related to the definition of seismic design loads and the harmonization of seismic hazard assessment. The project, known by its acronym SHARE (Seismic Hazard Harmonization in Europe), formally began in June 2009 and is supported by the European Union's Seventh Framework Programme (FP7). The SHARE consortium consists of 18 partners from 13 countries (Algeria, Belgium, France, Germany, Greece, Italy, Montenegro, Norway, Portugal, Romania, Switzerland, Turkey and the UK) led by the Swiss Seismological Service at ETH, Zurich. The UK is represented by the seismology group of the British Geological Survey (BGS) led by Dr Roger Musson.

The primary objective of the project is to produce a framework for harmonized, state-of-the-art seismic hazard assessment throughout Europe and the Mediterranean, including the Maghreb and Turkey. The practice of seismic hazard assessment within individual countries in this region has been very varied, with a number of countries still mapping hazard in terms of macroseismic intensities (e.g., García-Mayordomo et al., 2004; Solomos et al, 2008). As well as representing different ground-motion parameters and sometimes being anchored to different reference site classifications and return periods, these national maps frequently display marked discontinuities across political borders. Pan-European maps, ignoring national boundaries, have been produced as part of the GSHAP (Grünthal et al., 1999) and SESAME (Jiménez et al., 2001) projects. The GSHAP project produced a map of peak ground acceleration (PGA) on stiff soil sites for a return period of 475 years (Figure 1). The SESAME project produced a similar map (Figure 2) as well as comparable maps for spectral accelerations at response periods of 0.3 and 1.0 seconds. Although these maps were useful alternatives to incompatible national hazard maps, the teams that produced them were subsequently disbanded and no framework was established for their maintenance and updating. The SHARE project will produce harmonized state-of-the-art seismic hazard maps for the whole region but it also aims to create a computational framework that will allow the maps to be updated and improved as seismology and ground-motion modelling continue to evolve.

One of the primary targets is to provide a common basis for the seismic zonation maps that will accompany national applications of Eurocode 8 (EC8), which are currently derived by each country independently, such as the UK map produced by BGS (Musson & Sargeant, 2007), which somewhat undermines the objective of the code of harmonizing earthquake protection in Europe. However, the project will not simply produce a map for the Euro-Mediterranean region displaying the 475-year horizontal PGA values on rock that EC8 currently requires to define seismic design actions, but will develop more detailed characterizations of the ground motion that are better aligned with current and emerging trends in earthquake engineering. The current formulation of EC8, in which the Type 1 and Type 2 spectral shapes (for regions of high and low seismicity, respectively) are anchored to PGA to crudely approximate the uniform hazard spectrum (UHS), has been criticized as being significantly behind the current state-of-the-art in code specifications of design loads (e.g., Bommer & Pinho, 2006). There is a specific work package within SHARE on Engineering Requirements and Applications (see Figure 3) intended to ensure that the hazard output enables the current shortcomings in Eurocode 8 to be addressed and corrected in future revisions. The SHARE work package on ground-motion models will develop equations for all of the ground-motion parameters identified for engineering needs, enabling their prediction for rock sites. Additionally, the same work package will provide modified site amplifications factors if those currently embedded in EC8 (Rey et al., 2002) are judged to be in need of revision.

The starting point for the development of ground-motion models will be the current generation of pan-European equations (Ambraseys et al., 2005; Akkar & Bommer, 2010) and those developed for specific countries and regions (see overview by Bommer et al., 2010) . However, the work must also take account of the fact that these models will need to be extended to smaller magnitudes since it has been found that the datasets used to derive empirical models should include events of at least one magnitude unit

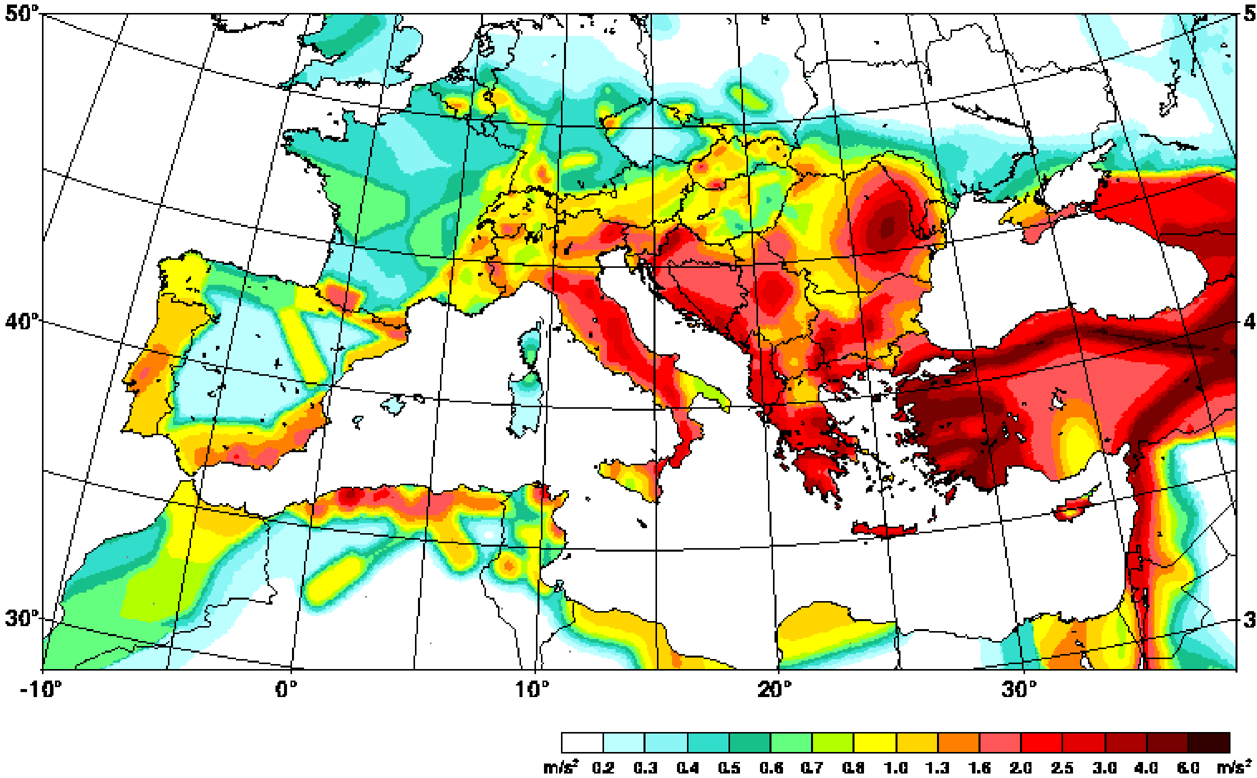

Figure 1. The GSHAP map for Europe, the Mediterranean and the Middle East, showing 475-year PGA values (m/s^2) on stiff soil sites (Grünthal et al., 1999).

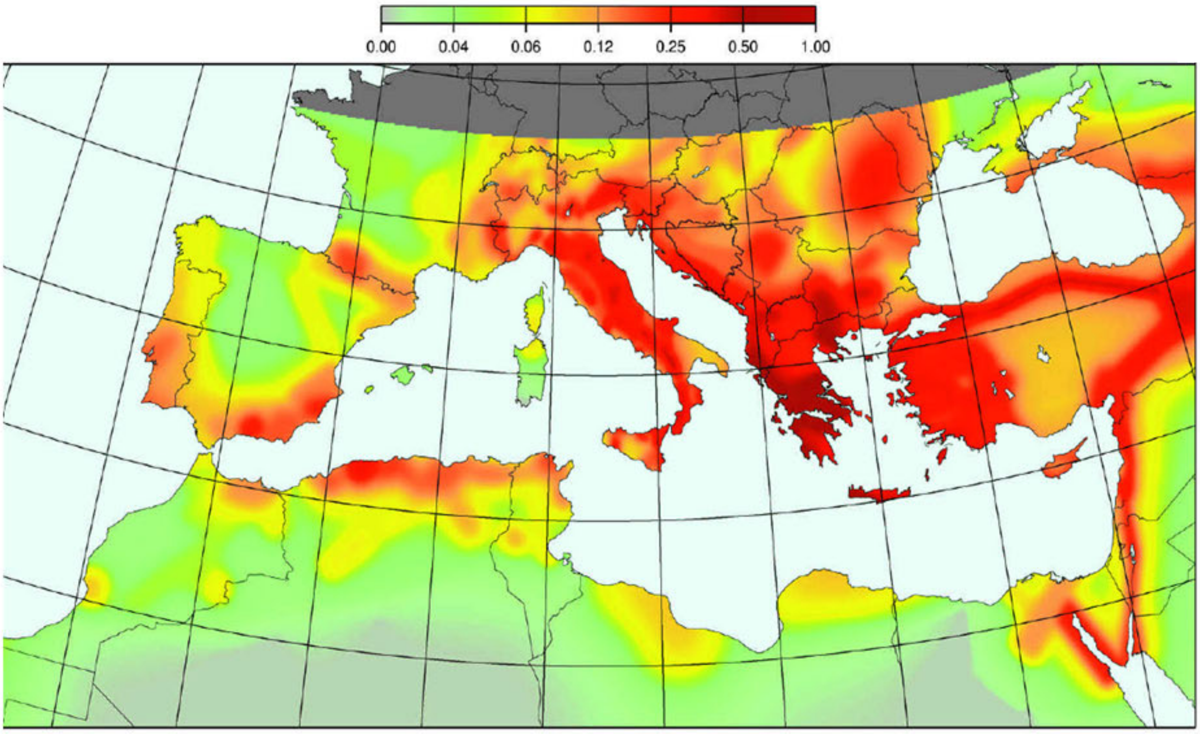

Figure 2. The SESAME map for Europe, the Mediterranean and the Middle East, showing 475-year PGA values (g) on stiff soil sites (Jiménez et al., 2001).

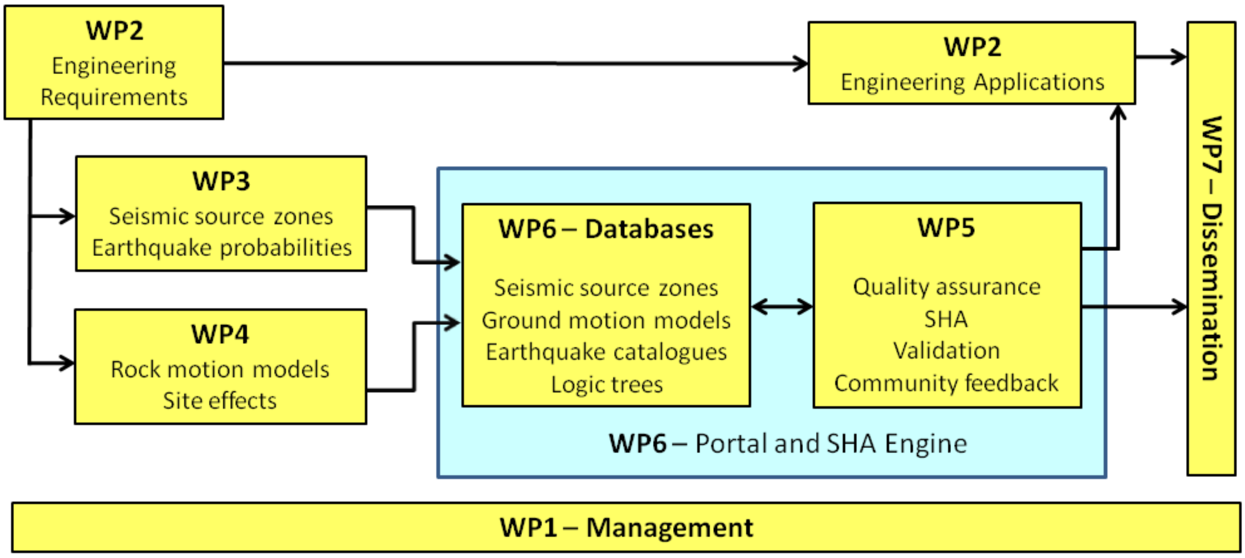

Figure 3. Organization and structure of the SHARE project, in which the targets to be produced by the seismic hazard analyses are clearly defined to meet engineering requirements and adapted to be suitable for engineering applications.

below the minimum magnitude for which they are to be applied (Bommer et al., 2007). Recent analyses of data from California have shown that whilst not apparent at larger magnitude, consistent regional differences can be identified in recordings of small-magnitude earthquakes (e.g., Atkinson & Morrison, 2009).

Another aspect in which the SHARE project may challenge the current status of EC8 is in reconsidering the reference return period of 475 years to define the design motions for the life-safety performance target. This return period, which corresponds to a 10% probability of exceedance in 50 years, was originally adopted for the first probabilistically-derived seismic hazard maps for the United States and their subsequent incorporation to the 1988 edition of the Uniform Building Code. Despite the very arbitrary bases and assumptions on which these reference values were chosen in the United States, reported in detailed by Bommer (2006), they were subsequently adopted by seismic design codes throughout the world. The use of the 475-year return period still persists today in codes such as EC8, despite the fact that US codes have since adopted other reference return periods (e.g., Leyendecker et al., 2000). In SHARE the selection of appropriate return periods will be explored from a risk perspective and also tested through earthquake loss estimations using the resulting hazard maps.

The output from SHARE should provide a sound basis for defining seismic design loads for most buildings and infrastructure, including bridges, in Europe and the Mediterranean. However, whilst the source characterization and ground-motion prediction models may provide a useful starting point for site-specific studies, they will not be sufficient for the assessment of seismic hazard at the site of critical facilities such as nuclear power plants. For such facilities, local seismogenic sources must be thoroughly investigated and site effects must be calculated using in situ measurements and site-response analyses. Additionally, a framework needs to be adopted to ensure that uncertainties are fully captured through structured multiple-expert assessments. A suitable approach is the adoption of the SSHAC framework (Budnitz et al., 1997) and specifically a Level 3 or Level 4 study, which are judged suitable for nuclear facilities (Coppersmith et al., 2010). A proposed outline to apply SSHAC Level 3 procedures for nuclear power plant sites in the UK, and equally applicable to most European countries, has been outlined in Bommer (2010).

In addition to providing an improved and harmonized input to future editions of EC8, another major objective of SHARE is to provide outputs that can be used in seismic risk assessment. There will be very close coordination between SHARE and the current Earthquake Model of the Middle East (EMME) project, and considerable synergy is also expected between both of these projects and GEM (Global Earthquake Model).

More information about all three of the projects mentioned in this article (SHARE, GEM and EMME) can be found on their respective websites:

- www.share-eu.org
- www.globalquakemodel.org
- www.emme-gem.org

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Forthcoming events

Date	Venue	Title	Organiser
16/06/2010 at 09:00	Haydock Park Racecourse Newton-le-Willows Merseyside WA12 0HQ	*Nuclear Design for Extreme Events 2010* For more information, see the back page of this Newsletter	NI & SECED Andrew Campbell (Sellafield)
23/09/2010 at 09:00 (2 days)	Imperial College London South Kensington London	*Seismic Design to Eurocode 8 – A Short Course with Design Workshops*	Imperial College London & SECED Ahmed Elghazouli (Imperial College)

For up-to-date details of SECED events, visit the website: **www.seced.org.uk**

Notable Earthquakes January – March 2010

Reported by British Geological Survey

Issued by: Davie Galloway, British Geological Survey, January 2010.
Non British Earthquake Data supplied by The United States Geological Survey.

			Time			Dep	Magnitude			
Year	**Day**	**Mon**	**UTC**	**Lat**	**Lon**	**km**	**M_L**	**M_b**	**M_W**	**Location**
2010	03	JAN	21:48	8.74S	157.48E	26			6.6	SOLOMON ISLANDS
2010	03	JAN	22:36	8.80S	157.35E	25			7.1	SOLOMON ISLANDS
At least 1,000 people left homeless on Rendova when several homes were destroyed or damaged. Majority of damage was caused by a tsunami.										
2010	05	JAN	04:55	58.17S	14.70W	10			6.8	STH SANDWICH ISLANDS
2010	05	JAN	12:15	9.02S	157.55E	15			6.8	SOLOMON ISLANDS
2010	07	JAN	19:30	55.03N	7.40W	9	1.6			CO. DONEGAL, IRELAND
2010	10	JAN	00:25	7.91S	107.88E	65		5.1		JAVA, INDONESIA
One person killed (heart attack) and two people injured at Kampungbaru.										
2010	10	JAN	00:27	40.65N	124.69W	29			6.5	NORTHERN CALIFORNIA
Around 30 people injured and some damage to hundreds of homes and buildings in the Eureka/Ferndale area.										
2010	12	JAN	21:53	18.44N	72.57W	7			7.0	HAITI
At least 222,570 people killed, over 300,000 injured and around 1.3 million people displaced as over 97,000 houses were destroyed and some 188,000 were damaged in the Port-au-Prince area and in much of southern Haiti.										
2010	15	JAN	18:00	10.45S	63.48W	8			5.5	VENEZUELA
Eleven people injured and some homes damaged at Cariaco.										
2010	17	JAN	09:37	25.56N	105.80E	27		4.4		GUIZHOU, CHINA
Seven people killed, one missing and nine injured after landslides occurred in Guizhou.										
2010	20	JAN	16:18	56.01N	5.84W	6	1.9			JURA, ARGYLL & BUTE.
2010	26	JAN	20:47	55.08N	7.39W	11	1.5			CO. DONEGAL, IRELAND
Felt County Donegal (3 EMS).										
2010	27	JAN	00:42	53.02N	2.18W	2	1.8			STOKE-ON-TRENT, STAFFS
2010	27	JAN	07:51	55.06N	7.39W	9	1.7			CO. DONEGAL, IRELAND
Felt County Donegal (3 EMS).										
2010	28	JAN	06:17	53.02N	2.17W	1	1.8			STOKE-ON-TRENT, STAFFS
2010	30	JAN	21:36	30.27N	105.67E	10		5.1		EASTERN SICHUAN, CHINA
One person killed, fifteen others injured and over 100 homes destroyed and thousands damaged in Moxi.										
2010	02	FEB	08:32	56.24N	3.77W	3	1.8			BLACKFORD,PERTH/KINROSS
Felt Blackford (3 EMS).										
2010	02	FEB	15:22	56.24N	3.73W	2	1.6			BLACKFORD,PERTH/KINROSS
2010	04	FEB	13:32	51.20N	4.58W	19	2.2			BRISTOL CHANNEL
2010	10	FEB	05:23	50.76N	2.92W	9	2.2			LYME REGIS, DORSET
2010	10	FEB	07:09	58.85N	0.86E	16	2.0			NORTHERN NORTH SEA
2010	18	FEB	01:13	42.59N	130.70E	578			6.9	RUSSIA/N. KOREA BORDER
2010	19	FEB	21:09	56.88N	7.42E	27	4.8			EASTERN NORTH SEA
Felt throughout Denmark.										
2010	24	FEB	12:30	51.48N	2.01W	23	2.0			CHIPPENHAM, WILTSHIRE
2010	25	FEB	04:56	25.56N	101.93E	36		5.0		YUNNAN, CHINA
Eleven people injured and several houses damaged in Yunnan.										

			Time			Dep	Magnitude			
Year	Day	Mon	UTC	Lat	Lon	km	M_L	M_b	M_W	Location
2010	26	FEB	20:31	25.93N	128.43E	22			7.0	RYUKYU ISLANDS, JAPAN
2010	27	FEB	06:34	35.97S	72.87W	35			8.8	OFFSHORE CHILE
At least 507 people killed, many more injured and over 200,000 houses damaged by the earthquake and associated tsunami in the Concepcion/Valparaiso area. A pacific-wide tsunami was generated and caused minor damage to a dock and boats in the San Diego area, California.										
2010	27	FEB	08:01	37.75S	75.10W	38			6.9	OFFSHORE CHILE
2010	27	FEB	15:45	24.75S	65.45W	10		6.3		SALTA, ARGENTINA
Two people killed and another two injured.										
2010	04	MAR	03:29	57.27N	3.99W	3	1.6			CARRBRIDGE, HIGHLAND
2010	04	MAR	14:02	13.60S	167.16E	176			6.5	VANUATU
2010	05	MAR	11:47	36.60S	73.23W	18			6.6	OFFSHORE CHILE
2010	05	MAR	16:06	3.77S	100.96E	13			6.8	MENTAWAI, INDONESIA
2010	08	MAR	02:32	38.87N	39.99E	12			6.1	EASTERN TURKEY
At least 51 people killed, 100 injured and over 980 buildings either destroyed or heavily damaged in the Basyurt region.										
2010	11	MAR	14:39	34.28S	71.87W	11			6.9	OFFSHORE CHILE
2010	11	MAR	14:55	34.13S	72.18W	5			6.7	OFFSHORE CHILE
2010	14	MAR	08:08	37.76N	141.58E	32			6.5	HONSHU, JAPAN
2010	16	MAR	02:21	36.22S	73.26W	18			6.7	OFFSHORE CHILE
2010	20	MAR	14:00	3.38S	152.22E	416			6.6	PAPUA NEW GUINEA
2010	21	MAR	01:45	53.86N	3.62W	4	1.7			IRISH SEA
2010	23	MAR	08:38	49.16N	3.25W	8	2.2			ENGLISH CHANNEL
2010	30	MAR	16:54	13.67N	92.83E	34			6.7	ANDAMAN ISLANDS, INDIA

SECED Newsletter

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SECED

SECED, The Society for Earthquake and Civil Engineering Dynamics, is the UK national section of the International and European Associations for Earthquake Engineering and is an affiliated society of the Institution of Civil Engineers. It is also sponsored by the Institution of Mechanical Engineers, the Institution of Structural Engineers, and the Geological Society. The Society is also closely associated with the UK Earthquake Engineering Field Investigation Team. The objective of the Society is to promote co-operation in the advancement of knowledge in the fields of earthquake engineering and civil engineering dynamics including blast, impact and other vibration problems. For further information about SECED contact:

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the Society for Earthquake and Civil
Engineering Dynamics (SECED)

Nuclear Design For Extreme Events 2010 — NDEE2010

One of the key areas that distinguishes the design of nuclear plants from that of conventional industrial facilities is the need to design for more extreme environmental conditions (wind, snow, temperature, flooding and earthquakes), internal hazards (impacts, blast, pipe whip etc) and both accidental and malicious actions. With the increasing activity in the UK in both nuclear decommissioning and potential new nuclear power generation, interest in the techniques, methodologies and criteria developed to deal with these relatively specialist areas of engineering design is growing. This event brings together practitioners, academics and regulators who have been deeply involved in the evolution of the current approaches to these hazards and the development of new techniques and criteria. It should be of benefit to civil, structural and mechanical engineers who wish to increase or refresh their knowledge of developments in these fields and to engineering managers who would like to obtain an overview of the potential effects of these actions, which have historically governed the engineering design of nuclear plant.

A one day conference
Haydock Park – 16th June 2010
To view the full Programme and register for the event please visit the event website
http://ndee2010.nuclearinst.com
alternatively register by completing the form below.

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