INSTITUTION OF CIVIL ENGINEERS
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ESTABLISHING THE NEED FOR SEISMIC DESIGN IN THE UK

Final Report

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Issue 2

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CONTENTS

SUMMARY

This report presents the finding of a study funded by the Institution of Civil Engineer's Research Enabling Fund. It was carried out by means of a literature search, some desk studies and interviews with engineers from the UK and overseas involved in issues of seismic design for regions of low seismicity. It is a companion study to a review of UK seismic hazard carried out by the British Geological Survey.

The report reviews the vulnerability of construction in the UK to earthquakes, and proposes guidelines for establishing whether or not new construction in the UK warrants an explicit seismic design. It also provides advice for cases where such an explicit design is warranted. The report is mainly concerned with new buildings, although infrastructure for the petrochemical industry, such as LNG tanks and national gas pipelines are also covered. Seismic upgrading of existing buildings and bridges were largely beyond the scope of the study, though it is hoped that bridges may be the subject of similar work in future.

The study was carried in response to the publication by BSI in 2006 of the UK National Forewords to Parts 1, 2, 4, 5 and 6 of Eurocode 8, and the need to prepare the corresponding UK National Annexes. The recommendations, as contained in the drafts issued for public enquiry in October 2007, of both those National Annexes and also the BSI Published Document PD6698 giving background material to the Annexes, draw on this study and the companion study by the British Geological Survey.

1 INTRODUCTION

In December 2006, the Institution of Civil Engineer's Research Enabling Fund awarded a grant of £4,400 for the preparation of a report giving guidance on the need for carrying out a seismic design of UK structures. The grant was prompted by the fact that the year 2006 saw the publication of the UK National Foreword to Part 1 of Eurocode 8 (EC8), the formal title of which is BS EN 1998-1:2004: Design of structures in earthquake regions: general rules, seismic actions and rules for buildings. The Foreword states (*inter alia*):

> There are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN1998 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions. It is the intention in due course to publish separately background information on the circumstances in which this might apply in the UK.

Designers of structures (other than special structures such as nuclear power plant for which regulation, guidance and precedents exist) therefore require some guidance on the circumstances referred to in the Foreword when an explicit seismic design might be warranted. One vital component of this guidance is an authoritative and up-to-date UK seismic zoning map; this has been prepared under a separate report for the Enabling Fund (Musson and Sargeant, 2008). However, the National Foreword states that the need for an explicit seismic design is determined not only by the location of a structure (i.e. seismic hazard at the site), but also by its function (in terms of consequences of seismic failure) and form (in terms of the intrinsic seismic vulnerability of the structure). Moreover, the seismic hazard at a particular site is often significantly modified by very local factors, and in particular by the nature of the soils underlying the site. Allowing for this last aspect was beyond the scope of Musson and Sargeant's work. The scope of the present study was therefore to provide advice on how all of these factors affected the need for seismic design in the UK, and (within the limited resources available) to provide guidance on suitable procedures, if a seismic design were warranted.

The project was conducted by carrying out the following.

1) A review of existing literature from the UK and other low seismicity regions.
2) Discussions with engineers from the UK, France, Germany, Australia and Hong Kong involved in seismic design.
3) A review of the seismic vulnerability of low rise mainly residential buildings in the United Kingdom designed to current building regulations.
4) A review of soil profiles in the UK, with the aim of characterising their properties relevant to site specific seismic hazard.
5) A limited analytical investigation of response spectrum shapes that might be apply to the UK, to investigate the suitability of the standard shapes recommended in EC8 for the UK.

The interim report for this project (Booth, 2007) identifies the literature studied and the personnel consulted.

The work carried out was the main basis for the material contained in the draft Published Document PD6698 to EC8 (BSI, 2007a), and the draft National Annexes to EC8 Parts 1, 4, 5 and 6 (BSI, 2007b, c, d and e). The work was presented at a public meeting organised by SECED (Society for Earthquake and Civil Engineering Dynamics, as detailed on www.seced.org.uk) at the Institution of Civil Engineers on 10th October 2007. There has been a related correspondence in the Verulam column on the Structural Engineer (IStructE 2007).

2 CURRENT REQUIREMENTS FOR SEISMIC DESIGN OF UK FACILITIES

2.1 Introduction

Current requirements to include seismic considerations in the design of structures in the UK apply to structures with high consequences of failure. These consequences may be in three generic categories:

1. Significant detrimental effects to safety or to the environment;
2. Damage to some items of the Critical National Infrastructure;
3. Economic consequences.

Some structures may have failure consequences belonging to more than one of these categories.

A requirement to include seismic considerations in the design may arise from:

- Legislation or regulation;
- Contract;
- Specification.

The three generic categories of failure consequence defined above are now discussed in more detail.

2.2 Significant detrimental effects to safety or to the environment

A structure with safety importance may be either a crowded place or a facility, the failure of which could harm people off-site or affect the environment, eg through the release of toxic substances, or large quantities of water, eg from a dam failure.

Where issues of safety or the environment are involved, there may be an absolute requirement specified by legislation or regulation, but in certain cases not so covered (eg structures with a very high population) the designer should draw the risk to the client's attention, and agree whether seismic effects should be included in the design. The relevant legislation and regulations are as follows.

a) COMAH (Control Of Major Accident Hazards) Regulations, applying to major hazard sites. Facilities are divided by the Regulations into Tier 1 and Tier 2 sites; in only the former is a requirement to consider seismic design specified. The categorisation as Tier 1 or 2 depends on the materials held and on size of inventory. The Regulations are enforced by a Joint Competent Authority, in England and Wales being HSE and the Environment Agency, and in Scotland HSE and the Scottish Environmental Protection Agency. Categorisation as Tier 1 or Tier 2 is not dependant on the demographics of the site.
b) Nuclear safety legislation and regulation (enforced by HSE).
c) Offshore safety legislation and regulation (enforced by HSE's Hazardous Installations Directorate).
d) In the case of certain specialised industries - Explosives (Manufacture, Storage and Transport), Ammonium Nitrate, Hazardous Pipelines, Gas Distribution and Transmission, Biological Agents – the designer should determine if HSE requires seismic design.
e) It is current general practice for panel engineers appointed under the Reservoirs Act (1975) to require a seismic assessment of dams using Charles *et al* (1991). However, there is currently no statutory requirement for a seismic assessment to be performed.

2.3 Damage to the Critical National Infrastructure

A fundamental role for any government is to ensure the continuity of society in times of crisis. This often involves providing extra protection to essential services and systems to make them more resistant to disruption and better able to recover quickly. In the UK, these essential services and systems are known as the Critical National Infrastructure (CNI). The Government views the CNI as those assets, services and systems that support the economic, political and social life of the UK whose importance is such that any entire or partial loss or compromise could:

- cause large scale loss of life
- have a serious impact on the national economy
- have other grave social consequences for the community
- be of immediate concern to the national government

In the UK, the CNI is categorised as ten interdependent sectors:

- Communications
- Emergency Services
- Energy
- Finance
- Food
- Government & Public Service
- Health
- Public Safety
- Transport
- Water

Not every activity within these sectors is critical, but application of the criteria outlined above assists Government and managers within each sector to identify where best to concentrate protective effort. Information as to whether a structure forms part of the CNI is not in the public domain. Senior managers of those organizations having CNI are aware of the level of protection required of a given facility, and may specify appropriate protection, including the requirement for seismic design.

2.4 Economic consequences

In at least one case that the authors are aware of, seismic design has been specified for a UK facility primarily because of the very large economic consequences of the facility ceasing to fulfil its function as a result of a rare seismic event. Where economic consequences may be a factor, the designer should make a client aware of the risk associated with a structure of high economic importance and agree whether seismic design is required or not.

2.5 Current British Standards making reference to seismic actions

The following British Standards make reference to seismic actions. Apart from EC8, it is believed that no other current British Standards do so.

- BS EN 14015:2004 Design and manufacture of site built, vertical, cylindrical, flat bottomed, above ground, welded, steel tanks for the storage of liquids at ambient temperatures and above.
- BS EN 14620-1:2006. Design and manufacture of site built, vertical, cylindrical, flat bottomed, above ground, welded, steel tanks for the storage of refrigerated liquefied gases with operating temperatures between 0 ^{o}C and -165 ^{o}C. General.
- BS EN 1473:2007. Installation and equipment for liquefied natural gas. Design of onshore installations.
- BS EN ISO 19901-2:2004. Petroleum and natural gas industries. Specific requirements for offshore structures. Seismic design procedures and criteria.

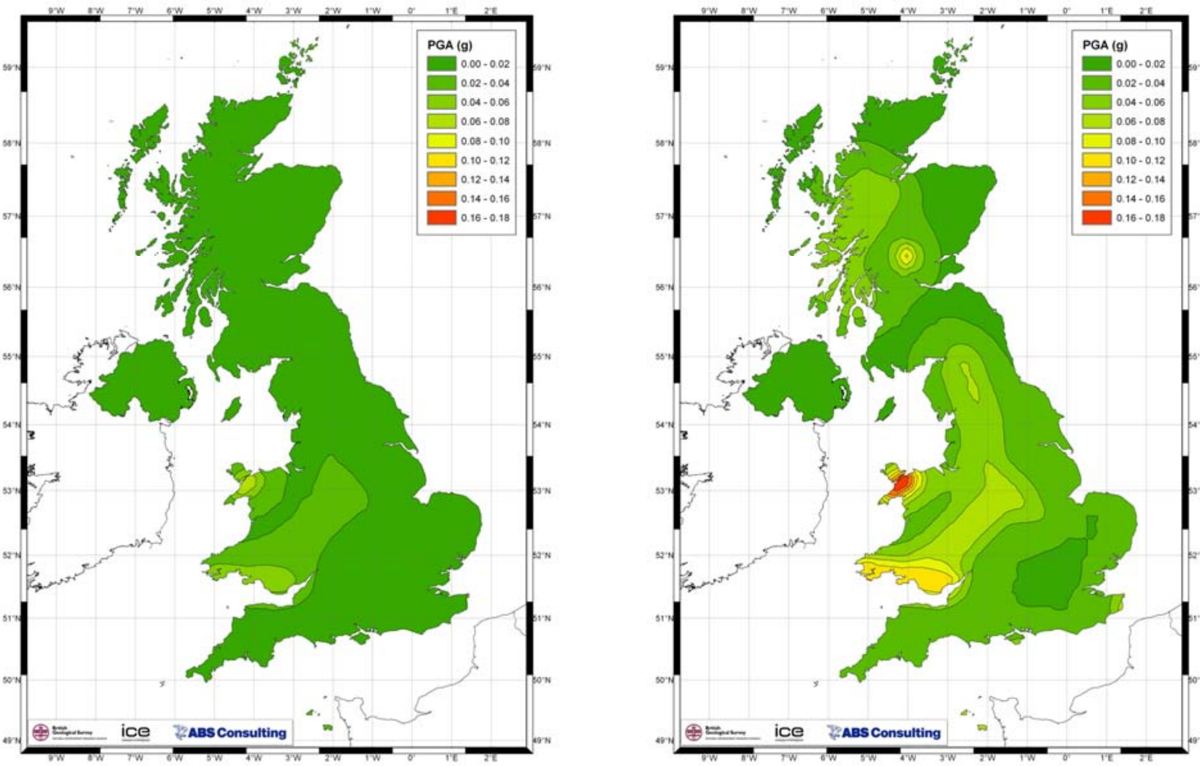
3 DESIGN GROUND MOTIONS FOR THE UK

3.1 UK seismic hazard

Musson and Sargeant (2008) have prepared a review of the regional seismic hazard in the UK, including seismic zoning maps in terms of the peak ground acceleration (PGA) on rock for return periods of 475 years and 2500 years. These maps are shown in Figure 3.1 and are expected to be included in PD 6698 (BSI, 2007a).

However, it is recommended that the values of PGA given by Musson and Sargeant (2008) should not be used directly for the seismic design of structures found to warrant seismic design, except at preliminary scheming stages or for comparison with more detailed hazard estimates or where the consequences of failure are not too great. This is for three reasons. Firstly, the values do not allow for the influence on hazard of such features as local faults, the choice of zone models and the contribution of local soil and topographical features (Musson and Sargeant, 2008). Secondly, the return period appropriate for the design of a particular facility will depend on its function, and indeed there may be regulatory or contractual requirements to consider return periods longer than 2500 years. Thirdly, the values are partly an artefact of the contouring algorithm; this is particularly true of the discontinuous 6% g contour seen in the Midlands in Figure 3.1b, and more generally the hazard values may be unreliable near contour lines. For most structures important enough to warrant an explicit seismic design in the UK, a site specific seismic hazard analysis is generally needed.

For the purposes of design to EC8, it is proposed in section 6.2 that the Musson and Sargeant zoning maps may be used to help screen out buildings in areas where the seismicity is sufficiently low that a seismic design is not warranted.



a) 475 year return period   b) 2500 year return period

Figure 3.1: Seismic hazard map of Peak Ground Accelerations on rock (PGA) for 475 year and 2500 year return periods (from Musson and Sargeant, 2008)

3.2 Return periods for seismic design

EC8 Part 1 defines the 'reference' return period as the return period of the ground motions to be used for the no-collapse (ultimate) limit state design of 'standard' (importance category CC2) structures. This 'reference' return period is then adjusted by applying an importance factor γ_I to the associated ground motions, in order to obtain a longer return period for 'important' structures and a shorter return period for 'less important' structures. The reference return period recommended by EC8 Part 1 is 475 years. However, this return period and the associated importance factors γ_I are subject to national choice.

Design motions for the damage limitation limit state are obtained by multiplying the 'reference' motions by a second factor ν, so that the damage limitation limit state is designed for ($\nu.\gamma_I$) times the reference motions. The factor ν is also subject to national choice, although recommended values are supplied. ν is less than 1, because the return periods for the damage limitation limit state are shorter than those for the no-collapse limit state.

The basis for using 475 years as the reference return period for the no-collapse limit state is discussed by Bommer (2006) and was previously investigated by Booth and Baker (1990). 475 years is currently adopted as the return period for design ground motions by a number of seismic codes, and was originally used in the USA, which however now bases its seismic hazard zoning on much longer return periods (FEMA450, 2003). The following paragraphs discuss the rationale for choosing a particular return period for design.

The annual risk of failure due to earthquakes in a particular structure depends on both the nature of the structure itself (i.e. its seismic vulnerability) and on the ground motions to which it may be subjected during the year (i.e. the local seismic hazard). To estimate the annual risk, the contribution of earthquakes with a range of annual probabilities must be considered and the total risk integrated. Thus, larger earthquakes with a very low probability will pose a greater threat to structures than smaller, more frequent ones, but the overall risk comes from the product of probability of occurrence and structural consequence. As shown by Booth and Baker (1990), the relative contribution to risk of low probability (long return period) earthquakes is much greater in areas of low seismicity than in areas of high seismicity, so design for 475 year motions in the UK would be expected to result in a lower integrated risk of seismic failure than would be the case for a high seismicity area such as Greece. In simple terms, the 'maximum credible' event (or something close to it) is much more likely to occur in any given period at Thessaloniki than it is at Colchester. Conversely, the magnitude of motions appropriate for considering the damage limitation limit state in the UK are relatively much *smaller* in the UK than in highly seismic regions, and in most cases are very unlikely to govern design, even for high consequence of failure structures. Booth and Baker (1990) found that design for ground motions with a return period of around 2000 years would give the same level of reliability at the ultimate limit state as current UK design requirements for wind and gravity loads, although there was a large measure of uncertainty in this figure.

For these reasons, the values recommended by EC8 for design return periods (as well as γ_I and ν) are not suitable for use in the UK. It might be possible to develop values of 'reference' return period and γ_I suitable for use in the UK which would achieve the target reliabilities given in Eurocode: Basis of Structural Design ('EC0'), Annex C. However, this has not been attempted for this study, partly because there is unlikely to be a need for seismic design for standard structures in the UK – i.e. those where there is not an enhanced risk to the environment, economy or population resulting from structural failure. Also, developing appropriate structural vulnerabilities for this calculation would be subject to considerable uncertainty. For these reasons, no recommended alternative values are given for reference return period or γ_I in the UK National Annexes to EC8 Parts 1 and 4. Instead, reference is made to PD 6698:2007 (BSI 2007a), which recommends that an appropriate return period for the design earthquake at the no collapse limit state should be established, on the basis of the appropriate total level of risk appropriate to the project, and accounting for any regulatory requirements. Site specific ground motions should then be derived for this return period. This will usually take the form of site response spectra.

3.3 Choice of γ_I

Table 4.3 of EC8 Part 1 recommends a value of γ_I for Importance Category IV buildings of 1.4, while clause 2.1.4(4) of EC8 Part 6 recommends $\gamma_I = 1.6$ for Importance Category IV silos, tanks and pipelines. Using the relationship between return period and peak ground acceleration (PGA) proposed in the note to EC8 Part 1, clause 2.1(4) implies that the return periods for no-collapse limit state for Category IV buildings is around 1300 years, and around 1950 years for Category IV silos, tanks and pipelines. However, this is based on a return period/PGA relationship appropriate for moderate to high seismicity areas, but inappropriate for very low seismicity areas such as the UK. Using data from Musson and Sargeant (2008) suggests that to obtain equivalent return periods in the UK, γ_I should equal around 1.8 and 2.3 respectively for 1300 and 1950 year return periods, compared with the EC8 recommended values of 1.4 and 1.6. Even this may underestimate the increase in γ_I factor needed for the UK; the values needed to equate total annual risk levels in the UK with those for high seismicity areas would need to be increased still further to reflect the greater relative importance of very long return periods on seismic risk to UK structures referred to in the previous section. Moreover, the EC8 recommended values implicitly assume that the shape of the response spectrum is independent of return period, whereas these shapes are likely to vary markedly with return period, particularly on softer soils.

As explained in the previous section, it is recommended that appropriate design motions for facilities with high consequence of failure should not be on the basis of selecting ground motions for a ‘reference’ return period, and multiplying them by γ_I. Instead, the ground motions should be selected on a direct, site specific basis, using a return period appropriate for the nature of the structure concerned.

3.4 Choice of ν

Just as the EC8 recommended values of γ_I are too low for high consequence category structures in very low seismicity areas, they are too high for ν, the factor to adjust ground motion levels to those appropriate for damage limitation considerations. The value of ν recommended by EC8 is 0.5 for ‘standard’ structures. Using the return period/PGA relationship of EC8 Part 1clause 2.1(4) implies that ν would have to increase to 0.59, in order to represent the return period of 95 years recommended by EC8 Part 1. However, using data from Musson and Sargeant (2008), ν should equal about 0.28 to achieve a 95 year return period in a standard structure in the UK.

No recommended alternative values are given for ν in the UK National Annexes to EC8 Parts 1 and 4. In most cases, the level of design motions appropriate for damage limitation considerations in the UK is sufficiently low that this limit state will not govern, so the parameter is not an important one for the UK. However, checks on inter-storey drift, which for buildings are only specified by EC8 Part 1 for damage limitation checks, may also be important at the no-collapse limit state for a number of reasons, as follow:

- to control P-delta (second order) effects,
- to limit structural damage
- to limit the consequences of non-structural damage for human safety.

It is therefore recommended that building structures in the UK should still be checked for storey drift according to EC8 Part 1section 4.4.3.2, using a value of ν equal to 0.5. This is the value of ν recommended by EC8 Part 1 for ‘standard’ importance class II buildings.

3.5 UK soil profiles

A review of the principal geomorphological settings (mainly in lowland Britain) for a coarse categorisation of soil profiles to a depth of 30m has been prepared. Perhaps the most pertinent general finding, not new to engineering geologists, was the predominance of glacial or related soils throughout the landmass, excepting in the rivers and estuaries.

In EC8 the choice of "soil type" leads, for the chosen design spectrum, to a magnification term S, that could indicate that certain soil profile types could be troublesome. Although there is little empirical evidence to support this in the UK, it is conceivable that certain soil profiles could have been troublesome for example at Wivenhoe in 1884.

Appendix A presents a review of typical soil types in the UK. The appendix suggests that the most typical soil profile in the UK corresponds most closely to Soil Type B in EC8 Part 1 clause 3.1.2(1) Table 3.1. There is however an important rider; in many cases, 'rock' corresponds to material with a shear wave velocity V_s of typically 600m/s rather than 800 m/s, the definition of rock in EC8 Part 1. Here, 'rock' is defined as the level at which there is a sharp increase in stiffness, and hence V_s.

It is therefore recommended that for the purposes of soil classification to the standard Eurocode 8 scheme, soil class in the UK may be selected from EC8 Table 3.1, but with a modified definition of rock level as follows.

The level at which either $V_s \geq 800$m/s

or $V_s \geq 600$m/s below 'rock' level AND

$V_s \leq 300$m/s above 'rock' level.

A significant problem issue identified in this study is that for some UK profiles, it may be hard to decide whether they belong to soil type B or E (see Appendix A section B5.0). As shown in Figure 3.2, for type 2 earthquakes the recommended spectral shapes are the same for these two soil types, but the spectral values are 20% greater for soil type E than type B, for a given ground acceleration on rock.

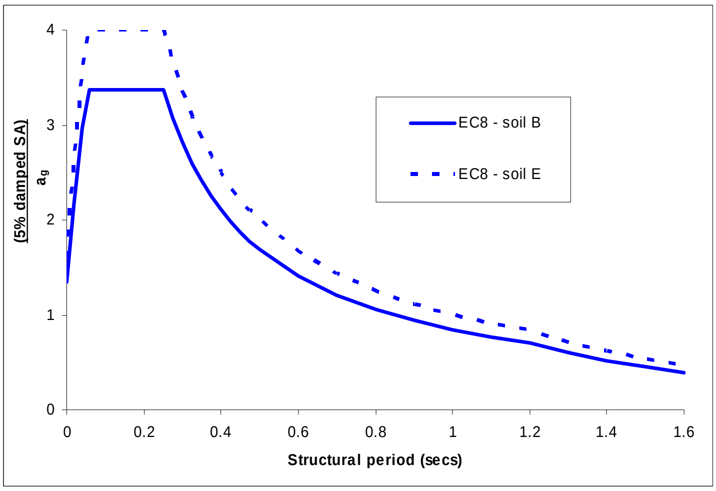


Figure 3.2 Comparison of EC8 recommended spectral shapes for soil types B and E (Type 2 earthquakes)

3.6 Design response spectra

3.6.1 Introduction

EC8 Part 1 recommends standard response spectrum shapes for five standard soil profiles. The shapes depend on the earthquake magnitudes governing seismic hazard at the site, with Type 1 spectra being appropriate for sites where the surface wave magnitude M_s governing hazard exceeds 5.5 and Type 2 for spectra with lower governing magnitudes. The resulting spectral shapes are given for a normalised peak ground acceleration (PGA) on rock of 1; to produce design spectra, therefore, the spectra are multiplied by the design PGA at the site. However, the values of the parameters governing spectral shapes (Tables 3.2 and 3.3 in EC8 Part 1) are subject to national choice.

The study by Musson and Sargeant (2008) worked in terms of moment magnitude M_w, rather than M_s. The dominant contribution to earthquake hazard at 475 and 2500 year return periods was found to come from earthquakes M_w with less than 6. Campbell (1985) indicates that for M_w less than 6, the value of M_s is likely to be less than M_w. From this evidence alone, it appears that the whole of the UK would fit the EC8 definition as an area for which Type 2 spectra was appropriate, and this conclusion appears to be the undisputed consensus of the UK seismological community.

The question then arises as to whether the Type 2 spectral shapes recommended in Table 3.3 of EC8 Part 1 are appropriate for the UK. A number of issues arise. *Firstly*, it might be expected that the shapes might not be suitable for a very low seismicity area such as the UK, since they were derived (presumably) for areas of low or moderate seismicity, where EC8 would routinely apply. *Secondly*, as discussed in section 3.2, the return period appropriate for design in the UK will vary according to the needs of the project, and will certainly tend to be very much longer than for more seismic regions. It would be expected that governing earthquake magnitudes would tend to increase with increasing return period, affecting the appropriate spectral shapes so that the shape would depend on the applicable return period. *Thirdly,* the spectral shapes recommended by EC8 are independent of the PGA. However, there is a wide consensus that soil amplification factors are significantly affected by PGA (IStructE/AFPS 2008). At low levels of PGA, soil strain and hence soil damping is low, leading to higher amplification than for high levels of PGA. This is explicitly accounted for in the US code ASCE 7-05 (ASCE 2005a). EC8 contains no direct allowance for this effect; there is however an indirect allowance because Type 2 areas will usually be of lower seismicity than Type 1 areas, and the soil amplification factors for Type 2 are significantly greater at short period than they are for Type 1.

3.6.2 Comparative study of spectral shapes

In order to investigate these issues further, a study was carried out to compare the spectral shapes produced for low seismicity areas from various sources, as shown in Table 3.1. The results are shown in figure 3.3. The spectral shapes are all normalised with respect to peak ground acceleration (PGA) on rock. Two values of PGA on rock, 7%g and 18%g, were chosen as typical of the likely range of motions that might need to be considered in the UK. The spectra investigated were as follows. For comparison purposes, the EC8 hard site spectrum appears on all the charts.

1) The recommended EC8 shapes, which are shown for soil types A to D and earthquake type 2.

2) The PML (1981) spectral shapes, which are given for three soil conditions defined as hard, medium and soft. Hard soil conditions are taken as equivalent to EC8 soil type A, medium conditions as type B and soft conditions as type C or D. These spectra have been widely used for design in the UK nuclear industry for 25 years.

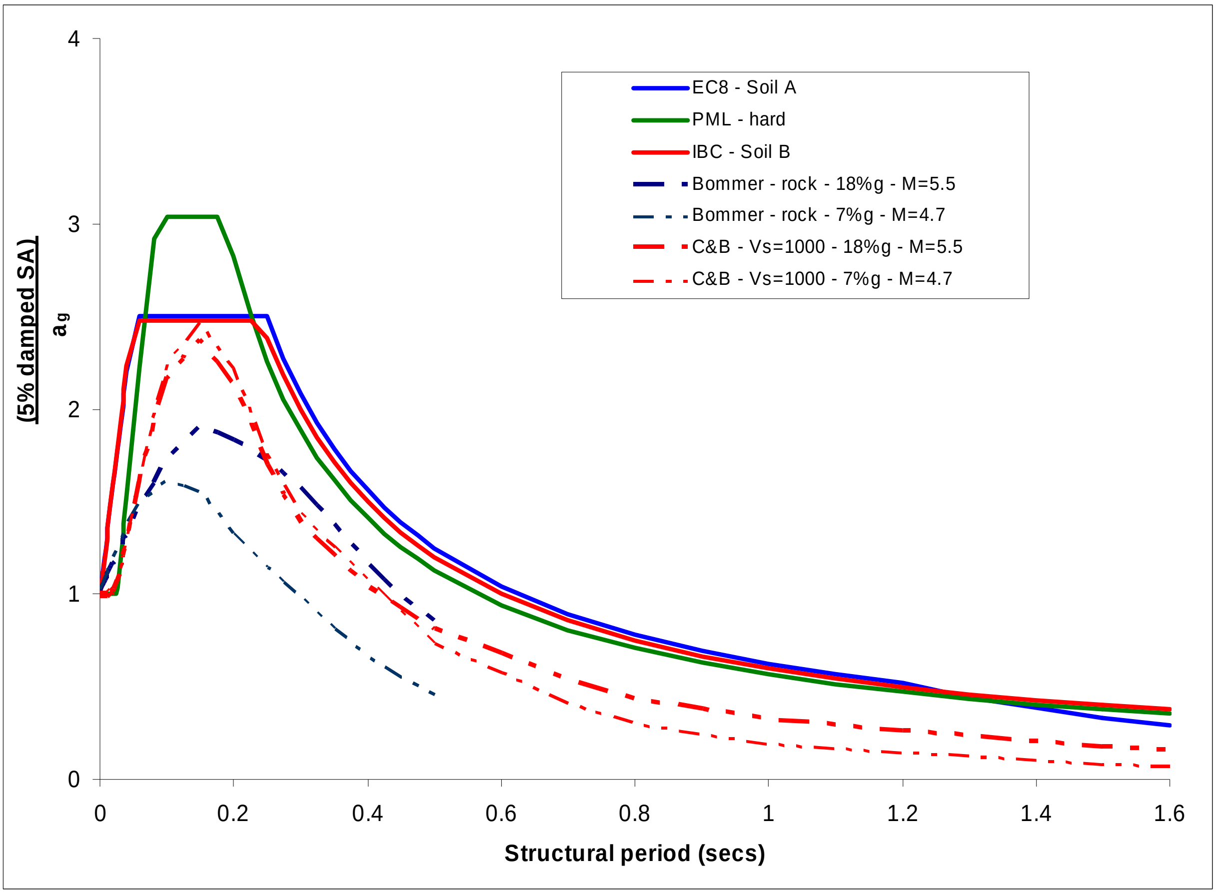
3) The US International Building Code (IBC) spectral shapes, which are defined in ASCE 7-05 (ASCE 2005a). The IBC soil types are defined in a rather similar way to those in EC8, but include an extra soil type, namely hard rock type A which does not appear in EC8. Type B in IBC therefore corresponds to type A in EC8, and so on. The response

spectra are not defined in terms of the peak ground acceleration on rock; instead they are constructed from the spectral ordinates at short period (S_s) and 1 second (S_1). Values of S_s and S_1 were chosen so that the resulting spectra for rock (IBC type B and EC8 type A) matched each other as closely as possible for the two PGAs considered (7% and 18%). Unlike the EC8 and PML spectra, the spectral shapes for other than rock depend on the intensity of ground motion.

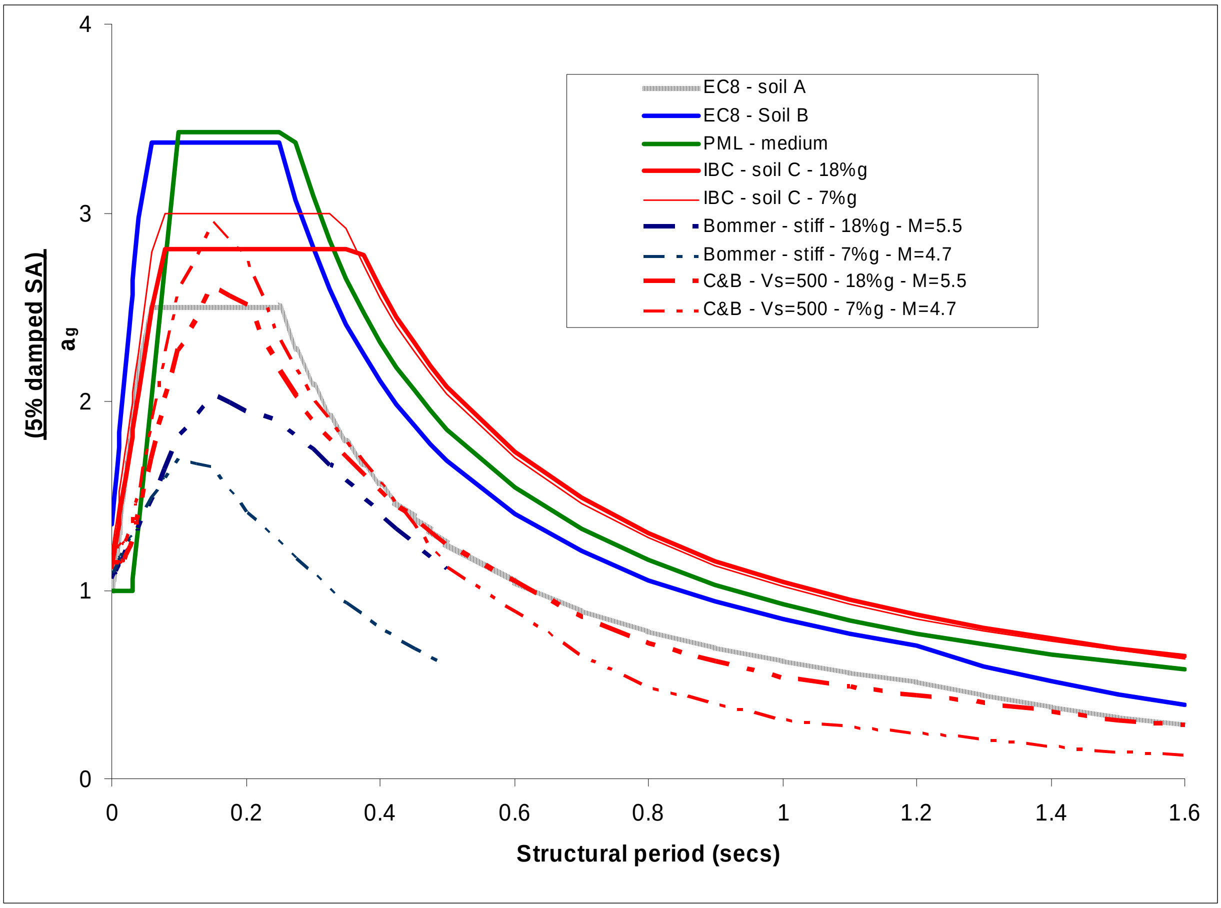
4) The two ground motion equations, according to Bommer *et al* (2007) and Campbell and Borzognia (2006), as used in the UK seismic hazard mapping exercise by Musson and Sargeant (2008). Bommer *et al* specify soil type as rock, stiff or soft, while Campbell Borzognia (C&B) specify the shear wave velocity. In both cases, strike slip faulting was assumed for this study, since this was assumed by Musson & Sargeant (2007). For the lower intensity spectrum (PGA=7%g), a magnitude of earthquake of 4.7 was selected, and the Joyner Boore distance (i.e. distance from the site to the possibly notional surface expression of the fault) adjusted until a PGA of 7%g was achieved for rock or hard conditions. The magnitude and distance were then kept constant for stiff and soft soil calculations for the lower intensity event. A magnitude of 5.5 was chosen for the higher intensity spectrum (PGA=18%g) and a similar procedure was followed.

	Spectrum name	Soil types	Other parameters
1	Eurocode 8	A, B, C, D	Type 2 earthquakes; EC8 recommended values
2	PML (1981)	Hard, medium and soft	
3	IBC: 18%g	B, C, D, E	S_s = 0.69g S_1= 0.165g
	IBC: 7%g		S_s = 0.27g S_1= 0.064g
4	Bommer et al (2007): 18%g	Rock, stiff and soft	‘Other’ faulting M_w= 5.5 R_{JB} = 1km
	Bommer et al (2007): 7%g	Rock, stiff and soft	‘Other’ faulting M_w= 4.7 R_{JB} = 3km
	Campbell and Borzognia (2006): 18%g	Vs = 1000 m/s (hard) = 500 m/s (medium) = 200 m/s (soft)	‘Other’ faulting M_w= 5.5 R_{JB} = 1km R_{RUP} =8km Z_{TOR} = 5, $Z_{2.5}$ = 2 delta = 90
	Campbell and Borzognia (2006): 7%g	Vs = 1000 m/s (hard) = 500 m/s (medium) = 200 m/s (soft)	‘Other’ faulting M_w= 4.7 R_{JB} = 3km R_{RUP} =10km Z_{TOR} = 5, $Z_{2.5}$ = 2 delta = 90

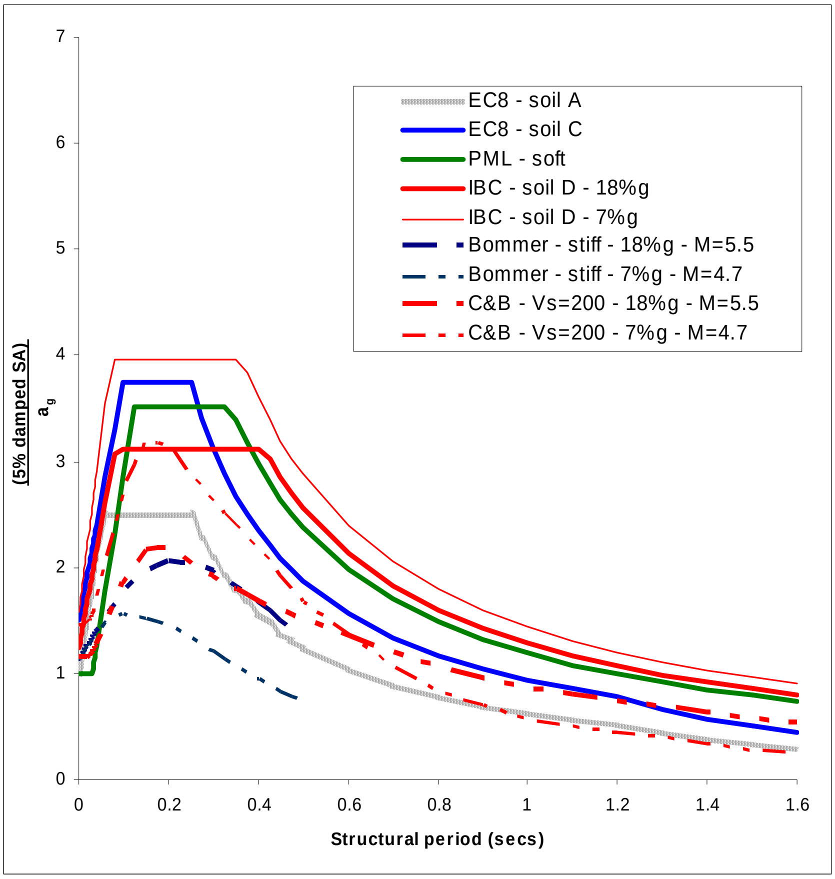
Table 3.1: Parameters specifying response spectra shown in Figure 3.3



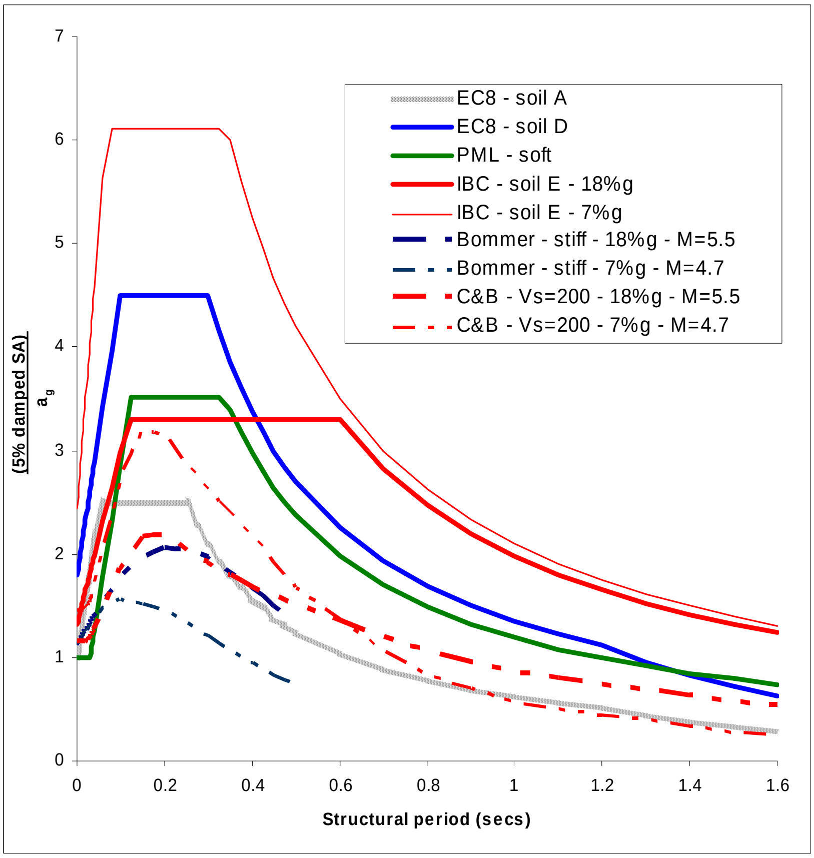
a) Hard sites



b) Stiff sites



c) Soft sites – (i) 'medium soft'



d) Soft sites – (ii) 'very soft'

Figure 3.3 – Comparison of spectral shapes

3.6.3 Discussion of spectral shapes

Figure 3.3(a) represents the basic motions on rock, as interpreted by the various sources, with parameters chosen so that there is an exact agreement between PGA given by all the sources. Figures 3.3(b) to (d) then represent how these basic rock motions are modified by the presence of increasingly soft layers of soil overlying bedrock, according to each source. It can be seen that, while there is a measure of agreement on spectral shapes on rock, at any rate among the codes, the match between sources becomes increasingly fragile with decreasing soil stiffness. For ease of comparison, the EC8 rock spectrum has been added as a thin chain dotted line to Figs 3.3(b) to (d).

A limited study such as this raises perhaps as many issues as it answers. One obvious inference is that robust definition of spectral shapes for a site is likely to be fraught with difficulty, particularly in the presence of soft soils. Some other observations are as follows.

1) The spectra have all been normalised with respect to peak ground acceleration on rock, because this is the governing parameter in EC8, and was used by Musson and Sargeant as the basis of their mapping exercise. Other normalisation parameters would have led to significantly different forms in Figure 3.3. An obvious alternative would have been to normalise with respect to the peak spectral acceleration on rock, more or less equivalent to the parameter S_s in IBC, and this type of normalisation would have led to a different apparent match between the various sources.
2) The code spectra agree quite closely for hard sites. This is hardly surprising for EC8 and IBC, since the parameters in the latter were chosen to match those in the former. What is notable is that the PML hard spectrum agrees quite well with the EC8 rock spectrum, although it overshoots the EC8 spectrum at short period by 22%. This good match also generally holds for the medium and soft spectra, although EC8 soil type D is rather more onerous than PML soft.
3) Amplitude dependent effects on spectral shapes are not significant for stiff sites, according to IBC, but the effect becomes very marked for the softest sites, leading to IBC being much more conservative than EC8 for EC8 soil type D (IBC type E).
4) The two ground motion equations produce results which are generally well below those of the code spectra, for a given peak ground acceleration. Bommer *et al* in particular predicts a much lower amplification at the peak of the spectrum than is the case for either of the codes or for PML. In fact, ground motion equations, from the way they are derived, are not well set up to evaluate spectral peaks due to soil effects, and generally underestimate them (see Gazetas, 2006), but the lower amplification applies to all soil stiffnesses, particularly for Bommer *et al.*
5) The dependence of the spectral shapes on motion intensity shown by the two ground motion equations is not primarily due to the soil effects referred to in section 3.6.1; indeed, Bommer *et al* (2007) specifically exclude these effects from their consideration, due to the difficulty in capturing it. Instead, the marked difference in shape between the higher and lower intensity events is due primarily to the change in magnitude from M_w = 5.5 to M_w= 4.7.
6) As noted by Musson and Sargeant (2008) the two ground motion equations give similar values for peak ground acceleration, but it can be seen that they are widely different at other spectral periods.

This limited study tends to support the recommendation in section 7.2.1 that for design in the UK, the type 2 spectral shapes recommended by EC8 may be suitable for preliminary design, but site specific studies are usually required to establish spectra for final design purposes, at any rate for very high consequence of failure facilities such as LNG tanks.

4 GEOTECHNICAL ISSUES

4.1 Geotechnical hazards

A study by Arup (1993), which investigated seismic risk to the infrastructure (excluding high risk installations) in the UK, concluded that foundation failures caused by earthquake actions in the UK are unlikely to be life-threatening. This included the possibility (considered low) that soil liquefaction might contribute to such failures. The study accepted that landsliding of slopes close to instability under static conditions might be triggered by UK earthquakes, but considered that landsliding of such slopes triggered by other causes – particularly a rise in pore water pressure caused by heavy rain – is much more likely, and seismic causes add little to that risk. The study found that the direct risk from surface fault movement is very low. It was therefore concluded that geotechnical hazards resulting from earthquakes need not be pursued in the context of 'standard' structures.

Work in the 14 years since the Arup study does not point to the need to change these conclusions – i.e. that seismically induced failure of naturally deposited soils is usually a negligible risk. Failure of earth dams or retained soils however may need to be considered. For high consequence of failure structures such as nuclear facilities, geotechnical hazards due to earthquakes will need to be carefully considered, but experience has generally been that seismic design of such structures in the UK is dominated by the vibratory effects of earthquakes, rather than by geotechnical hazards. For the design of high consequence of failure, buried pipelines of substantial length in the UK, landsliding may well be much more important, but for most structures, issues of geotechnical hazard are unlikely to be very significant.

4.2 Requirements for additional ground investigations

Appropriate site investigations are needed to identify ground conditions in accordance with the types given in 3.1.2 and also to fulfil the requirements in EC8 Part 5 where additional information is provided. See also the advice in the EC8 Manual (IStructE/AFPS 2008).

For cases where EC8 is used in the UK for the design of special facilities, a site specific hazard and risk study is generally recommended, at any rate for very consequence of failure structures. This may require more intensive study of geological structure and a specialised evaluation of soil dynamic properties using intrusive and non-intrusive geophysics (see Geological Society, 2002) and laboratory testing.

Extended facilities such as pipelines require a screening procedure to establish such vulnerable aspects as landslides and liquefaction. Non intrusive geophysics has economic attractions for this work.

Where geophysical methods are used they should tied in to carefully logged boreholes.

4.3 Liquefaction assessment in the UK

There has been one generally accepted observation in the UK of what has been taken as a minor "liquefaction" phenomenon and some observations of water level response in boreholes. Studies of liquefaction potential for events of reference return period 475 years are therefore not pursued. But where events of much lower probability need to be considered, as they might well do for strategic or high consequence of failure facilities, the possibility of liquefaction failures cannot be ruled out. Some guidance on liquefaction is given in Annex B of EC8 Part 5; see however the comments on this in the EC8 Manual (IStructE/AFPS 2008). A comprehensive review of current liquefaction matters is given in PIANC (2001). See also USNRC (2003).

5 STRUCTURAL ISSUES

5.1 Seismic vulnerability of structures complying with current UK building regulations

5.1.1 Steel and concrete buildings

It is widely accepted that the robustness and minimum lateral strength requirements of current non-seismic codes of practice provide a significant degree of earthquake resistance, and field experience from earthquakes supports this view. However, it is also evident that some types of structural arrangement which are quite satisfactory for non-seismic loads, and which do not fall foul of building regulations, may become highly dangerous in an earthquake. These issues are discussed by Booth and Skipp (2004), Booth and Pappin (1995), and Arup (1990). Booth and Pappin (1995) made specific proposals for ways to improve seismic resistance which would be much easier and less costly to implement that the full requirements of EC8.

As noted by Booth and Baker (1990), very tall buildings become progressively more susceptible with height to wind loads, as they extend up into levels where the wind speed is less reduced by surface effects. However, their period of vibration becomes longer, and so they detune from the likely predominant earthquake motions and this is particularly true for UK conditions, where the relatively low magnitudes of earthquake involved give rise to mainly short period motions. Therefore, wind effects tend to get progressively more important with height, and earthquake effects progressively less so. The only exception would be on sites with very soft soils, where longer period motions would be present. Overall, however, very tall buildings are unlikely to require explicit seismic design in the UK, despite their economic importance.

5.1.2 Low rise domestic buildings

The seismic vulnerability of UK low rise domestic buildings was assessed by CAR (1988). That report was reviewed by Peter Watt of the Charlwood Partnership Ltd, and his findings are presented as Appendix B.

CAR (1988) found that only a very limited number of features allowed by the then current UK building regulations might prove vulnerable to UK levels of earthquake, and Appendix B presents evidence that overall, subsequent changes to the regulations have produced further significant improvements in seismic resistance. The only aspect which appears to need further study is the vulnerability of timber or steel frame houses with a brick cladding. The concern here is that the frame, although adequate for providing lateral resistance, is much more flexible than the brick cladding, which therefore might be prone to shaking off during an earthquake. This could form a hazard in itself; moreover, if it occurred in only one storey, then that storey could fail as a 'weak storey'. Buildings with nineteenth century brick cladding over mediaeval timber frames proved vulnerable in the 1884 Colchester earthquake, and although of course current construction is vastly different, a concern remains. However, no research into this aspect has been found.

It is understood that a significant general European research programme is proposed into the seismic performance of masonry buildings in low seismicity areas of northern Europe. This might produce further evidence on which augmented advice could be based. The current judgement is that for low rise domestic buildings, the risk caused by other natural hazards such as extreme winds, foundation movements, landslides or flooding is very significantly greater than that caused by earthquakes.

5.1.3 Non building structures

This study has found no record of earthquake damage to engineered non-building structures in the

UK. However, steel and reinforced concrete structures using modern principles of engineering design only date back a century or so, and in the context of requirements for very low levels of risk, that good performance record is not in itself sufficient to dismiss seismic risk as negligible.

Worldwide, bridges built in the last 30 years have only been found to perform badly in earthquakes at durations and amplitudes of shaking which would be extremely rare in the UK. The exception might be the settlement of approach embankments to an extent likely to cause a significant hazard to vehicles or to threaten derailment of railway trains. However, recent experience suggests that poor maintenance and human error are likely to be a much greater threat to railways than low probability extreme hazards such as earthquakes.

As referred to in section 4, pipelines may be prone to large displacements along their length due to seismically induced landsliding and, possible for very long return periods, due to surface faulting.

Liquid storage tanks, even where seismically designed, have failed in large earthquakes, due to buckling of the tank walls, rupture of pipelines connecting to the tanks and overtopping of the tank contents due to sloshing. Tanks on soft soils prone to large amplification of long period motions may be particular susceptible to failure. Silos are also prone to failure in large earthquakes. However, as far as is known, there are no cases of significant failure occurring in tanks and silos at the amplitudes and durations of motion applicable to most UK situations.

Harbour walls and quays have proved particularly vulnerable to large earthquakes, because they are often loaded and supported by loose saturated granular soils which are prone to liquefaction. The failure of these dock structures has rarely proved life threatening, and so the main consideration is the potentially large economic consequences.

5.2 Seismic design for areas of low seismicity

5.2.1 Eurocode 8 requirements

EC8 does not require any seismic design in areas of very low seismicity. In areas of low seismicity, the basis requirement is to perform a seismic analysis, using a q (behaviour modification factor) not exceeding 1.5 (or 2 in the case of some steel structures), and provide sufficient strength according to non-seismic Eurocodes to resist the seismic actions thus calculated. Structures in low seismicity regions are exempt from the requirements of sections 5 to 9 of EC8 Part 1 for capacity design and seismic detailing. The only additional requirement for concrete structures is the use of reinforcement class B or C (as opposed to A) but use of class A reinforcement is not generally anticipated in the UK.

5.2.2 French codes and guidance

In France, two documents (classed as non-conflicting complementary information) will be issued to complement the requirements of EC8 Part 1. The first document applies to one and two storey buildings. Applying to all seismic zones of France, it allows a design based on detailing rules alone without an explicit seismic analysis. This document will be based on the existing French code NF P 06-014 ISSN 0335-3931 which is currently being revised to be compatible with EC8. It is understood that this document will take some time to prepare.

The second document (for which there is no current equivalent in France) will provide detailing rules for Zone 1b (see section 6.1.1), which corresponds broadly to low seismicity as defined in EC8. It applies to steel, concrete, masonry and timber buildings of failure consequence class CC2 (and possibly also to a few of class CC3 – see section 6.2.1.1) without restriction on height. The intention is to provide an enhanced level of seismic protection for construction in the band of weak seismicity extending across the centre of France, without involving the need for seismic analysis or for incurring significant additional cost. It is understood that this document is in its early stages and it is not clear when it will be issued. The earthquake at Annecy of 1996, in the Alpine region of France, produced peak ground acceleration of 30%g pga and damaged many chimneys and weak non-structural elements but produced only minor cracking in engineered structures. Hence the judgement was that detailing rules only were required, particularly for non-structural elements.

AFPS (2005) provides guidance on the seismic design of steel, concrete, timber and masonry buildings and also bridges. This is intended for application to all regions of France, which include areas which are significantly more seismic than any in the UK.

5.2.3 Swiss code

SIA (2002) is the Swiss counterpart to Eurocode 2 and covers concrete structures. It contains a 1½ page section (numbered 4.3.9) entitled 'Earthquake dimensioning situation'. The code allows q (behaviour factors) of 1.5 in non-ductile structures, increasing to 2 if class B or C reinforcement is used. No further requirements are given (except, presumably, providing sufficient strength to resist the calculated seismic actions). Some further rules are given for 'ductile' structures for which q factors up to 4 are allowed.

SIA (2005) is the Swiss counterpart to Eurocode 6 and covers masonry structures. It contains a 1 page section (numbered 4.7) entitled 'Earthquake dimensioning situation'. This provides for a minimum wall thickness of 150mm and a maximum unrestrained wall height of 17 times the thickness. Some very simple rules for 'ductile' reinforced masonry are provided, giving minimum reinforcement percentages, including rules for reinforcing around wall boundaries and openings.

5.2.4 German Code

Currently, the German standard DIN4149 provides rules for seismic protection. It covers only buildings in steel, concrete masonry, and timber (not composite) and at 80 pages is a relatively compact document. Use of DIN 4149 is currently mandatory for zones 1 to 3, although most of Germany is zone 0.

It is understood that updating of this standard, and preparation of German National Annexes to EC8, are currently in preparation.

5.2.5 Australian code

As referred to in section 6.1.4, AS 1170.4—2007: Earthquake actions in Australia provides design advice for buildings in areas of low seismicity.

6 ESTABLISHING THE NEED FOR SEISMIC DESIGN IN THE UK

6.1 Practice in other low seismicity areas

6.1.1 France

The technical basis for the seismic zonation of France is a recently completed and extensive seismic hazard evaluation study lasting around two years. A special spectral shape is proposed for the whole of France, somewhere between the Type 1 and 2 spectral shapes as recommended in EC8, but closer to 2. Soil types adopted follow the standard EC8 definitions, but the spectral shapes (as noted above) are non-standard. In principle, the reference return period is 475 years; in practice some of the zonation is based on a longer return period. Mainland France is divided into a number of zones, as follows.

- 1A for which no seismic consideration is required (PGA less than about 5%g)
- 1B (faible sismicité) where seismic detailing but no seismic analysis is required (PGA roughly between 7% and 10%g)
- 2A & 2B: moderate seismicity, for which the full provisions of EC8 apply, except for low rise buildings
- Higher zones apply to the Antilles and French overseas territories.

6.1.2 Germany

It is understood that there is a current debate on how EC8 should apply in Germany. As discussed in section 5.2.4, most of Germany is currently exempt from seismic design, which however must be applied to all structures in certain specified regions.

6.1.3 Switzerland

As discussed in section 5.2.3, Switzerland has issued counterpart codes for concrete, steel and masonry buildings, which contain brief sections on seismic requirements. An earlier Swiss code contained simple seismic rules for bridges, but it appears that this is no longer current.

6.1.4 Australia

Australia is currently finalising revised seismic design requirements, which will be published as AS 1170.4-2007: Earthquake actions in Australia. It is understood that the standard is in press, and will contain the following provisions.

1) Three seismic design categories are defined, which depend on the importance level of the structure, the regional seismic hazard, the type of foundation soil and the height of the structure.
2) Category I requires a very simple lateral strength check and a few detailing rules.
3) Category II requires a conventional equivalent static analysis and more detailing rules apply.
4) Category III requires a dynamic seismic analysis and still more detailing rules apply.
5) Domestic housing under 8.5m in height is generally exempt from seismic design where the hazard falls below a specified threshold, but adobe, hay bale, random stone and other materials not covered by current standards require checking. One page of simple design rules is provided for the design of housing under 8.5m in areas exceeding a specified threshold level of seismicity.
6) Domestic housing over 8.5m in height is designed to design categories I, II or III depending on the regional seismic hazard, the type of foundation soil and the height of the building.

Wilson & Pham (2007) provide a discussion of the new features of the code.

6.1.5 Singapore

A statutory requirement is being considered for a lateral strength check to be carried out on tall buildings on soft soils, without the need for further seismic design or detailing. It is proposed that tall buildings on firm ground and all low rise buildings should be exempt. The timescale for possible implementation of this statutory requirement is not known.

6.2 Proposed methodology for the UK

6.2.1 Proposed procedure for steel and concrete buildings

6.2.1.1 The proposed screening process

Since the UK National Foreword defines the whole of the UK as an area of very low seismicity, there is no statutory requirement from BS EN 1998-1 (i.e. EC8 as it applies to the UK) to consider seismic loading. However, the UK National Foreword advises that there may be circumstances in which an explicit seismic design is warranted; a screening process is therefore proposed below to identify those circumstances.

Explicit design for seismic actions is unlikely to be required for 'standard' buildings which complied with current UK building regulations. As discussed in section 5.1.1, compliance with building regulations in itself confers a considerable level of seismic resistance, and ensures the absence of some seismically unfavourable structural features, such as low lateral resistance, lack of overall structural continuity and very poor seismic reinforcement and connection detailing. The judgement, supported by work such as Arup (1993), is that this confers a level of seismic resistance which is likely to reduce the annual risk of significant damage to levels well below that from other natural hazards in the UK, and should achieve a level which is acceptable for all 'standard' or 'low risk' buildings, defined as those classified by Eurocode: Basis of Structural Design ('EC0') Table B1 as being in consequence classes CC1 or CC2. These consequence classes correspond to importance classes 1 and 2 in EC8 and other Eurocodes. It is therefore recommended that such buildings do not require explicit seismic design anywhere within the UK. An explicit seismic design would of course be required if there were contractual or statutory requirements for this to be done.

Where consequence class CC3 applies and in the absence of statutory or contractual requirements, it is proposed that a screening process should be carried out, based on three considerations, as follows.

1) Exceedence of regional seismic hazard at the site above a specified threshold
2) The presence of soils overlaying rock at the site which might lead to a high amplification of seismic actions.
3) The presence of unfavourable structural features.

The screening process proposes that seismic actions should be explicitly considered in class CC3 structures where at least two out of the three factors listed above apply. The three factors are quantified and discussed further in the next sub-sections.

The screening procedure is based on a qualitative judgement, namely that where unfavourable structural features are not present, it would take an exceptionally high degree of seismic hazard, relative to the rest of the UK for the risk to be significant – i.e. that both a higher than normal regional hazard and unfavourable soils would both need to be present. It is further judged that the presence of an unfavourable structural feature would only pose a significant risk if the local hazard was higher than average – i.e. *either* high regional hazard *or* poor soils present (but not necessarily both).

A further judgement is that, in the presence of unfavourable structural features and also unfavourable soils, seismic design may be warranted even for sites in those parts of the UK that are shaded light or dark green in Figure 3.1(b). This is because Figure 3.1 is only intended as a broad indication of hazard level. As discussed in section 3.1, local features, such as potentially active faults, are not taken into consideration and the method of drawing the hazard contours affects the outcome, particularly in areas close to a contour line. More generally, any seismic mapping exercise is subject to considerable uncertainty, particularly in areas of low seismicity where data are relatively sparse. It is widely considered that a magnitude 5½ or 6 earthquake could affect any part of the UK, not just the areas coloured yellow or red in Figure 3.1(b) and considerations of civic protection suggest that it is prudent to ensure that important public infrastructure has an above average level of seismic protection.

These are however broad judgements, and it will remain the responsibility of the engineer in consultation with the client and other stakeholders to make the decision in particular cases. Considerations such as the level of hazard indicated on Figure 3.1(b), the severity of the unfavourable feature and the consequences of failure in an earthquake should influence the decision on whether an explicit seismic design is required, where two out of the factors in the screening process are positive. For example, a structure on poor soils with a mass-stiffness eccentricity of 17% (see table 6.2(d)) might be acceptable without carrying out a seismic analysis if the site was in a dark green area of Figure 3.1(b), and/or if the overall lateral strength of the structure were relatively high compared with its weight and/or if the structure was not particularly important for civic protection after a damaging earthquake or if its failure was unlikely to affect a large number of people. Of course, even where an explicit seismic design is carried out, it may prove that other considerations, particularly design for wind and for robustness, still govern design.

A detailed screening process is proposed in the following sections; more refined recommendations may be able to be developed in the light of future work and experience.

6.2.1.2 Threshold level of seismic hazard

EC8 recommends that areas where the 475 year PGA exceeds 4%g on rock should be considered as areas of low seismicity (as opposed to very low seismicity) where a seismic analysis should be carried for all structures, irrespective of importance class. According to the BGS study (Musson et al, 2008) shown in Figure 3a, only a small part of the UK exceeds this threshold, as indicated in figure 6.1.

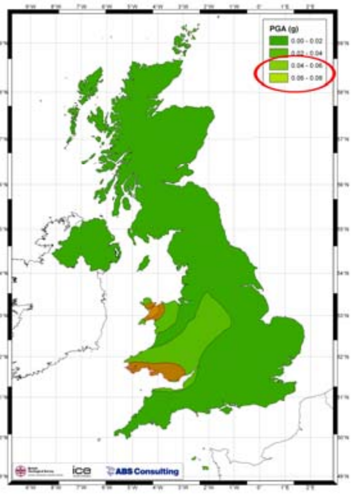


Figure 6.1 Areas in the UK exceeding 4%g PGA on rock for 475 year return period (after Musson and Sargeant, 2008)

An alternative recommendation in EC8 is that the threshold of low seismicity is where the 475 year PGA exceeds 6%g on soil. A slightly larger area of the UK would probably be included in this alternative definition. Note that the EC8 definitions are recommendations only, and can be modified by national choice. Effectively, the statement in the UK National Foreword that the whole of the UK is an area of *very* low seismicity overrides the EC8 recommendation.

For the purposes of the screening process proposed in this report, the threshold level is proposed as a 2500 year return period PGA on rock of at least 6%g, as shown in Figure 3.1b, taken from Musson and Sargeant (2008). 2500 year, rather than 475 year, return period figure was chosen both because the associated map gives a better gradation of hazard, and also because of the importance of longer return periods on seismic risk in the UK, as discussed in section 3.2. 6%g is an arbitrary choice; it applies to about 10% of Great Britain, all of the Channel Islands but excludes Northern Ireland, as shown in the areas shaded pink in Figure 6.2.

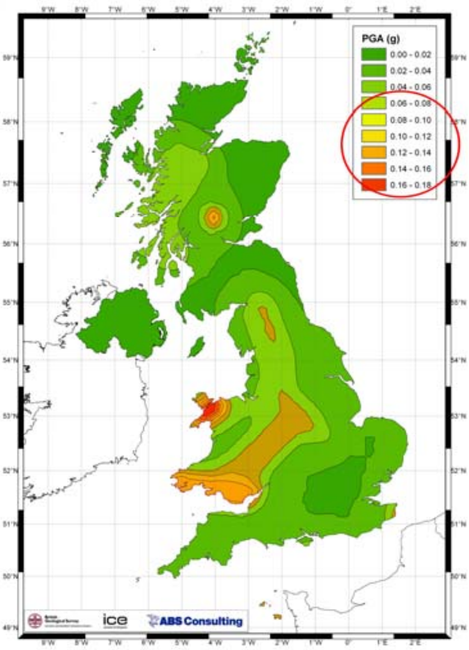


Figure 6.2 Areas in the UK exceeding 6%g PGA on rock for 2500 year return period (after Musson and Sargeant, 2008)

6.2.1.3 Seismically unfavourable soils

Based on the review of UK soil profiles reported in Appendix A, and on work presented by Arup (1993), the soils shown in Table 6.1 are proposed as being particularly prone to amplification of seismic motions in the UK. Table 6.1 is based on the supposition that dangerous amplification of motions for low to midrise buildings may occur where soft or loose soils of moderate depth overlay bedrock and where there is a sharp impedance contrast (i.e. sharp change in shear wave velocity) at the rock to soil interface.

Table 6.1– Seismically unfavourable soil profiles

3m to 10m of sand, silt or clay overlaying rock (note 1)
10m to 30m of sand with SPT<15 overlaying rock (note 1)
10m to 30m of clay with cu < 100kPa overlaying rock (note 1)

Note 1: Rock is defined in section 3.5

Future work and experience may enable additional soil profiles to be added and the existing ones to be modified. 2 and 3-D effects in sedimentary basins and topographical effects may give rise to further local amplification of ground motion (see for example Faccioli, 2002).

6.2.1.4 Unfavourable structural features

The following is a list of features which have proved particularly unfavourable in previous earthquakes. It is not intended to be exhaustive, and some judgement is needed in its application.

Table 6.2– Seismically unfavourable structural features

a)	More than 50% of the mass is concentrated in the upper third of the height of the structure.
b)	Formation of a yielding mechanism under lateral forces by means of a single plastic hinge at the base of not more than one of the building elements.
c)	The design horizontal strength under lateral loading of any storey is less than 65% of that of the storey above. The potential of cladding elements to increase the lateral strength of the storey above that being considered should be allowed for in this calculation.
d)	Structures where the centre of stiffness is displaced from the centre of mass at any level by greater than 15% of the total plan dimension of the building in the direction in which the displacement takes place
e)	Structures where the first predominantly translational mode of vibration in either principal direction has a lower period than that of the first predominantly torsional mode.

These features relate to structures which have 'irregularities' in plan or elevation, which, as shown by field experience of earthquakes, lead to poor seismic performance. The five features above are all directly based on EC8 Part 1, as follows.

a) and b). These structures correspond to 'inverted pendulum' structures, as defined in Table 5.1 of EC8 Part 1, and which attract unfavourable (low) q or behaviour factors.
c) This corresponds to one of the most important conditions used by EC8 to define vertical irregularity, and is associated with formation of a weak storey. The limiting factor of 65% is based on IStructE/AFPS (2007).
d) This is associated with horizontal irregularity. The limiting factor of 15% is based on IStructE/AFPS (2007).
e) This condition is associated by Fardis *et al* (2004) with torsionally flexible systems, as defined in Table 5.1 of EC8 Part 1, and which attract unfavourable (low) q or behaviour factors.

Future work and experience may enable additional structural features to be added and the existing ones to be modified. Recommended design procedures for buildings where a seismic design is found to be warranted are provided in section 7.2.

6.2.2 Low rise domestic timber and masonry housing

Following the discussion of section 5.1.2, it appears unlikely that explicit seismic design would be required in the UK for low rise domestic housing, and it is recommended that none should be considered in the absence of contractual or statutory requirements. Where consequence class CC3 structures are involved, a similar screening process to that recommended in the previous sections for steel and concrete buildings would be suitable. Seismically unfavourable structural features include the following.

- Parapets, chimneys and gable ends with poor lateral restraint.
- Masonry cladding attached to a flexible timber or steel frame.
- Unsymmetrical arrangements in plan of masonry walls, partitions and cladding leading to significant eccentricity between centres of mass and stiffness.
- Arrangements of cladding which lead to a significant reduction in the lateral resistance of one storey compared with the one above.
- Partial height masonry infill in concrete frames, creating ‘short columns’ prone to shear failure.
- Large openings in masonry walls which are not framed by tensile boundary members.
- Poor connections between floors and their supporting structure.

Floors with large openings which inhibit their ability to distribute seismic loads to the elements providing lateral resistance.

6.2.3 Transportation infrastructure

Major bridges in the UK have in the past been subject to seismic checks and it is recommended that this should continue. This study did not review the needs for seismic resistance of UK transportation infrastructure, which it is hoped might be the subject of further studies in future.

6.2.4 Petrochemical industry infrastructure

National distribution gas pipelines and LNG tanks in the UK have in the past been subject to seismic checks, and are subject to statutory requirements (see section 2). Where statutory requirements are not applicable, in principle a vetting procedure such as that proposed in section 6.2.1 for steel and concrete buildings might be possible, though considerable judgement would be required. It may be possible to develop more specific advice in future.

6.2.5 Hazardous chemical and nuclear industry infrastructure

The situation is similar to that discussed for the petrochemical industry above. Seismic design requirements have long been in place for the nuclear industry. For hazardous chemicals for which there are no statutory provisions, in principle a vetting procedure such as that proposed in section 6.2.1 for steel and concrete buildings might be possible, though considerable judgement would be required. Nuclear installations are specifically excluded from the scope of EC8, although many of its provisions may be relevant.

7 RECOMMENDATIONS FOR SEISMIC DESIGN IN THE UK

7.1 General

Where a seismic design is considered necessary for UK structures, and in the absence of project specific design requirements, design to the provisions of EC8 for areas of low seismicity would generally be appropriate for structures falling within the scope of EC8. This involves carrying out a seismic analysis, using a q or behaviour factor not exceeding 1.5 (or 2 for certain steel structures), but essentially no other seismic detailing requirements. The exception is that additional measures may be needed where resistance to very low probability (long return period) events is required; this could take the form of some degree of seismic detailing to provide enhanced ductility.

These procedures are discussed in more detail in the following sections for steel and concrete buildings structures, and also, more briefly, for other types of construction.

7.2 Steel and concrete buildings

7.2.1 Preliminary design

The most effective measure at preliminary design stage is likely to be reduction of the number of unfavourable features affecting the structure. For example, if one of the unfavourable structural features in Table 6.2 applies, then an alternative design eliminating that feature could be developed. If an unfavourable soil type shown in Table 6.1 applies to the site, relocating to a new site without such soils could be considered or a stiff, strong foundation type such as a deep basement founded directly on stiff material might be adopted. Note that merely piling through soft material is unlikely to resolve the seismic issue, because the lateral stiffness of the piles may be relatively low and the piles may themselves be very vulnerable to seismic damage at the interface between soft and stiff soil layers.

If at least two unfavourable features still apply, then preliminary seismic design should follow the procedures given for structures of low dissipative (DCL) behaviour in EC8 Part 1, which is appropriate for areas of low seismicity. This involves a seismic analysis to ensure that there is sufficient lateral strength, using a behaviour factor q not exceeding 1.5 for concrete structures or 2 for steel structures. There are no further specifically seismic requirements for steel structures, except that where q is taken as 2, cross-sectional class 4 members (as defined in EC3) may not be used to resist the seismic actions. However, the provisions of EC3 and other relevant Eurocodes still of course apply. In reinforced concrete structures, the only additional requirement is that Class 2 or 3, rather than Class 1, reinforcement must be used, but in practice Class 1 reinforcement is unlikely to be found in the UK.

Carrying out a seismic analysis to EC8 Part 1 section 4 requires definition of the reference ground acceleration a_{gR} on Type A ground (rock) and the importance factor I. The product $a_{gR}I$ equals a_g, which is used to define the design response spectrum, which is a function of a_g, q and the soil type, as specified in EC8 Part 1 section 3.2.2.5.

For the purposes of a preliminary design, it is suggested that a_{gR} may be taken as the mapped acceleration for a 2500 year return shown in Figure 3.1(b). The importance factor I may be based on the recommended values given in the note to EC8 Part 1 section 4.2.5 clause (5). The response spectrum shape may then be derived from the recommended values given in Table 3.3 of EC8 Part 1, chosen for the most applicable soil type at the site, chosen with regard to the comments in section 3.5.

The reasons why conditions in the UK may lead to different values from this suggested approach are discussed in sections 3.2, 3.3 and 3.6 of this report. The use of a 2,500 year return period for a_{gR} rather than 475 years, as recommended in EC8, recognises the appropriateness of much longer return periods for design in areas of very low seismicity and is likely to result in reasonably conservative results.

7.2.2 Final design

The preliminary design may indicate that the lateral strength provided in a structure for non-seismic reasons (for example wind resistance) substantially exceeds that required for seismic resistance. In this case, more detailed seismic design and analysis may not be needed.

If a more detailed final design is required for very high consequence of failure facilities, it is recommended that the design response spectrum should be established on a site and project specific basis, with the return period for design selected for the level of reliability appropriate to the project, and due allowance made for local features including local soils. This would involve a site specific hazard analysis. For facilities of lower consequence of failure, the preliminary procedures recommended in 7.2.1 may suffice, if conservatively applied. Judgement, probably supplemented by expert advice, will be needed on the level of sophistication required.

Generally, seismic design for low-dissipative (DCL) behaviour will be sufficient. This consists of establishing that the lateral strength provision is adequate, with only the minimal additional requirements noted in the second paragraph of section 7.2.1 above. However, some highly critical facilities in the UK may also require a demonstration that performance is acceptable in events beyond the design basis. One way of achieving this may be to provide some degree of dissipative performance, for example by adopting some of the seismic detailing provisions required by EC8 Part1 for Ductility Class Medium (DCM) or Ductility Class High (DCH) structures.

7.3 Low rise domestic timber and masonry housing

There are a number of sources of advice for the provision of simple measures to improve the resistance of domestic housing in areas of low seismicity, including the Australian seismic standard AS 1170.4-2007: Earthquake actions in Australia referred to in section 6.1.4 and the French standards referred to in section 6.1.1. There are many sources of advice for simple construction rules in areas of high seismicity (e.g. IAEE 1986), though these obviously require adaptation for UK conditions.

EC8 Part 1 itself has some simple rules for the design of low rise masonry buildings(see section 9.7 of the code), which do not involve carrying out a seismic analysis, but instead require minimum cross section areas of wall, depending on the level of seismicity and height of building, as specified in Table 9.3 of the code. In addition, certain other construction details are required. A simple check for minimum wall areas seems a promising approach, but the minimum areas proposed (but not mandated – they are subject to national choice) are substantially greater than would often be used in the UK. The seismic hazard in Table 9.3 of EC8 is modified by a ‘corrective factor’ k which depends on the structural arrangements; this was an attempt at a late stage in the code drafting to make the Table 9.3 requirements less onerous, but the basis for the k factor is open to question. The version of Table 9.3 provided in the French National Annex to EC8 Part 1 dispenses with k and provides substantially lower requirements for wall areas. However, the French Table 9.3 does not provide values for unreinforced masonry, so is of limited value in the UK.

The resistance of domestic buildings to low intensity earthquakes is a subject of on-going research, and it appears that a European research programme may be imminent which will provide further valuable information.

7.4 Transportation infrastructure

This study did not attempt to develop recommendations for the seismic design of UK transportation infrastructure, which it is hoped might be the subject of further studies in future. Seismic guidance for steel and concrete bridges is provided by AFPS (2005).

7.5 Petrochemical industry infrastructure

It is considered that EC8 Part 4 provides appropriate guidance for UK conditions in cases where seismic design is required. Generally, design ground motions should be based on a site and project specific study.

7.6 Hazardous chemicals and nuclear industry infrastructure

Nuclear and other ‘special’ structures are specifically beyond the scope of EC8, although many aspects may be applicable. It is possible (and is considered by the authors of this report to be desirable) that the European nuclear industry should develop seismic nuclear design guidelines based on EC8 and the Eurocode system, although as far as is known there are currently no plans for this. Extensive nuclear design advice exists elsewhere, for example ASCE (2005b).

7.7 Dams

A review of specific seismic design measures for UK dams was not carried out for this study. Charles *et al* (1991) address the issues directly.

8 CONCLUSIONS AND RECOMMENDATIONS

1) Compliance with the requirements of current UK building regulations confers a level of seismic resistance which will be adequate for most UK structures without the need for any explicit seismic design.
2) Seismic considerations are unlikely to prove governing in very tall structures in the UK, because the wind loads in such structures tend to increase with height, whereas the seismic loads, expressed as a fraction of the weight, tend to decrease with height.
3) Unless statutory or contractual requirements state otherwise, it is recommended that seismic design need not be considered for structures in the UK which are in consequence categories CC1 and CC2, as defined in Eurocode: Basis for Design, Table B1.
4) In the absence of statutory or contractual requirements, a screening process is recommended for consequence category CC3 buildings, as follows. It is stressed that judgement, perhaps supported by expert advice, is required in applying the screening process.

   An explicit seismic design is recommended where *at least two* of the following *three* conditions apply.
   1) Exceedence of regional seismic hazard at the site above a value of 6% for a 2500 year return period, as shown in Figure 6.2.
   2) The presence of soils overlaying rock at the site which might lead to a high amplification of seismic actions. A list of such soils, which may need to be augmented and modified in the light of further experience, is provided in Table 6.1.
   3) The presence of unfavourable structural features. A list of such features, which may need to be augmented and modified in the light of further experience, is provided in Table 6.2.

5) A preliminary design may be carried out based on a peak ground acceleration corresponding to the values shown in Figure 3.1b (i.e. for a 2500 year return period), multiplied by the appropriate γ_I value recommended in EC8. Also for preliminary design purposes, a Type 2 spectral shape from the values recommended in EC8 Part 1 Table 3.3 may be used, but the spectral shape is subject to large uncertainty. For final design, a site specific investigation of seismic hazard is generally recommended, particularly for high consequence of failure facilities such as LNG tanks. However, the preliminary procedures recommended above, if conservatively applied, may be judged adequate where for cases of lower consequences of failure.
6) Explicit seismic design is unlikely to be required for domestic low rise housing.
7) This study has not attempted to formulate seismic design recommendations for critical transportation infrastructure (which may be the subject of future work) or for dams (which is the subject of recommendations by Charles *et al* (1991)).
8) A number of sources of advice from overseas are listed in the report, which may be useful in assisting seismic design of structures in the UK.

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Appendix A: Review of UK soil types

Preamble
This report is divided into Part A and Part B. Part A is devoted to the findings from a map study and a limited literature study generally confined to the last two decades of work on the Glacial and Peri-Glacial deposits of the UK. Part B covers a diverse collection of borehole data with an emphasis on the sparse information available on shear wave velocity in superficial deposits especially tills.

Part A: Findings from map study and literature search

A1.0 Background
Following consultations, it was agreed to focus on how well the soil profiles A-E in EN 1998 Part 1 could represent the conditions in the UK and in particular how significant could be those soil profiles that might present the most severe actions. Other factors such as the importance of deep geology and basins for example have not at this stage been pursued.

A2.0 Methodology
The work has followed the following lines;

a) Examination of a sample of the BGS 1:50000 and 1:63360 drift maps to identify the character of the superficial deposits, the extent of superficial cover and some indication of thickness
b) A short study of the literature on Glacial and Periglacial deposits of the UK
c) A short study of the distribution and character of river, estuarine and coastal deposits
d) A review of published data on shear wave velocities in superficial deposits and weathered rocks
e) Study of a selection of borehole data from a variety of sources that contain data on shear wave velocities or sufficient N values to compare with Table 3.1 in EN1998-1:2004.

Maps from the following localities have been selected:
a) Solway Firth
b) West Cumbria
c) West Lancashire
d) Cheshire
e) North Wales
f) Staffordshire
g) Leicestershire
h) Monmouth
i) Warwickshire
j) East Anglia
k) Severn Valley
l) Thames Estuary
m) Kent, Sussex Coast

The list of maps examined is given in Table A1.

The age of the maps extends from the late 1960s to 2006. There has been a notable increase in details of the drift presented. This may truly reflect the relative simplicity of origin as for Anglesey (Maps 92,93 1974) with its Blown Sand, Alluvium, Marine Alluvium, Glacial Gravel and Boulder Clay or the predominant interest in the Pre-Cambrian bedrock. Elsewhere and thirty years later, the impact of Quaternary studies in an area with little “hard rock” interest, the omnibus term “Drift” contains 26 named units.

A3.0 Area coverage of various units

A rough quantitative estimate of the proportion of the map area covered by various units has been made. This could be, in principle be rendered more accurate by planimetry or integration of the digital map data. Three levels have been used, as follows.

1: Predominant	70% of map area with surface expression of a unit
2: Major	20% to 70% of map area with surface expression of a unit
3: Minor	<20% of map area with surface expression of a unit.

This classification is of course constrained by the flowering of detail on the Holocene and Quaternary in the past two decades.

A4.0 Thickness of superficial deposits

Time and cost restraints have limited the search to the maps themselves. In this regard some of the earlier maps give spot data on the superficial cover derived mainly from water wells. Many maps have sections from which a coarse estimate of “drift” can be made and some give more detailed information from motor-way sections. There is other information in a number of published papers, for example in papers on the use of shallow seismic surveying methods and on the search for an EPSRC geotechnical testing site.

A5.0 Summary of findings

Table A1 contains a summary of the findings from study of the 1:50000 series of maps

TABLE A1
EC8 Soil Profile factor

Map No.	Date of map	Location	"Drift" cover (Note 1)	No of drift units	Max thickness of drift (m)	Solid	Remarks
E17	2003,2005	Solway E&W	1		3	Mercia Mudstone Group (MMG),	Special sheets, Buried valley
E37	1999	Gosforth	1	23	80	Ord	
E67	1991	Garstang	1	18	50	Trias Kirkham Mudstone	Motorway M55 sections
E74	1989	Southport	1	9	40	MMG	
E75	1940	Preston					
E81	1991	Patrington	1	16	~80	Flamborough Head/Burnham Chalk	
E83	1974	Formby	1	8	~25	Keuper Marl (MMG)	Borehole data on map
E90	1990	Grimsby	1	13	~50	Welton Chalk, Flamborough, Burham Chalk	
E92, 93	1974	Anglesey	1	5		Pre-Cambrian	See also 105
E98	1962.1993	Stockport				Carboniferous	
E105	1974	Anglesey				Pre Cambrian	See 105
E109	1965	Chester					
E122	1967	Nantwich					
E139	1974	Strafford					
E176	1996	Lowestoft	1 or 2	26	~25	Crag	

Map No.	Date of map	Location	"Drift" cover (Note 1)	No of drift units	Max thickness of drift (m)	Solid	Remarks
E191	1996	Saxmundham	1	18	~30	Red Crag, Corraline Crag, Palaeocene	
E200	1974	Stratford upon Avon	2	11	~10	Inferior Oolite, UL, ML, LL,	
E201	1982	Banbury	2	6		MMG,UL,Cornbrash	4 Avon terraces Landslides, Till on higher ground
E234	1972	Gloucester	3	16		Jurassic,LL, Triassic, MMG	Extensive area (10%) of landslip/foundered strata
E241	1978	Chelmsford	1 or 2	22	~40	Eocene,Bagshot, Claygate, London Clay	
Special	1996	Inner Bristol Channel &Severn Estuary	2	18	~20		
Special	1992	Inner Thames Estuary	2	12	~20	Eocene, Bagshot, Claygate London Clay, Cretaceous, Upper Chalk	5 peat layers at Tilbury. 20m to Upper Chalk
E305,306	1974	Folkstone, Dover A	1 or 2	17	~10	Cretaceous, Middle, Upper Chalk, Wealden, Hastings Beds	
E17	2003,2005	Solway E&W	1		3	MMG, LRPA	Special sheets Buried valley
E37	1999	Gosforth	1	23	80	Ord	
E67	1991	Garstang	1	18	50	Trias Kirkham Mudstone	M55 sections
E74	1989	Southport	1	9	40	MMG	

Map No.	Date of map	Location	"Drift" cover (Note 1)	No of drift units	Max thickness of drift (m)	Solid	Remarks
E75	1940	Preston					
E81	1991	Patrington	1	16	~80	Flamborough Head/Burnham Chalk	
E83	1974	Formby	1	8	~25	Keuper Marl (MMG)	Borehole data on map
E90	1990	Grimsby	1	13	~50	Welton Chalk, Flamborough, Burham Chalk	
E92, 93	1974	Anglesey	1	5		Pre-Cambrian	See also 105
E98	1962.1993	Stockport				Carboniferous	
E105	1974	Anglesey				Pre Cambrian	See 105
E109	1965	Chester					
E122	1967	Nantwich					
E139	1974	Strafford					
E176	1996	Lowestoft	1 or 2	26	~25	Crag	
E191	1996	Saxmundham	1	18	~30	Red Crag, Corraline Crag, Palaeocene	
E200	1974	Stratford upon Avon	2	11	~10	Inferior Oolite, UL, ML, LL,	
E201	1982	Banbury	2	6		MMG,UL,Cornbrash	4 Avon terraces Landslides Till on higher ground
E234	1972	Gloucester	3	16		Jurassic,LL, Triassic, MMG	Extensive area of landslip/foundered strata 10%
E241	1978	Chelmsford	1 or 2	22	~40	Eocene,Bagshot, Claygate, London Clay	
Special	1996	Inner Bristol	2	18	~20		

Map No.	Date of map	Location	"Drift" cover (Note 1)	No of drift units	Max thickness of drift (m)	Solid	Remarks
		Channel &Severn Estuary					
Special	1992	Inner Thames Estuary	2	12	~20	Eocene, Bagshot, Claygate London Clay, Cretaceous, Upper Chalk	5 peat layers at Tilbury 20m to Upper Chalk
E305,306	1974	Folkstone, Dover A	1 or 2	17	~10	Cretaceous, Middle, Upper Chalk, Wealden, Hastings Beds	

Note 1: Drift cover is defined in three classes, as follows (see section A3.0).

1: Predominant 70% of map area with surface expression of a unit
2: Major 20% to 70% of map area with surface expression of a unit
3: Minor <20% of map area with surface expression of a unit.

A6.0 Some generic issues

The maps have shown that the UK is covered by superficial deposits with maximum depths around 40 m with 80m in buried valleys. These deposits are broadly divided into glacial materials and alluvium in river valleys and estuaries. Although the intra-ice fluvial deposits, lake deposits and melt streams have hydrodynamics and sedimentation process common to non-glacial systems, the materials carried and deposited by ice have a random fabric of rock-derived material ranging in size from boulders to “rock flour” (Ehlers *et al* 1991). Successive ice advances and warm periods modify the lithological sequence and the consequences of ice loading including over-consolidation.

The complexities of successive ice advances and retreats have been studied deeply in Great Britain and Ireland and this report draws heavily upon Ehlers *et al* (1991).

An interim conclusion from Part A of this study is that the appropriate S value for use with EN1998:Part 1, under guidance from the recommended soil profiles, should be consistent with the Vs – depth profiles that could obtain over large tracts of the UK covered by glacial deposits.

Part B: Examples of V_S versus depth.

B1.0 Introduction

In Part A, the results of an examination of BGS drift maps that was undertaken to yield a rough estimate how much of the surface was shown as covered by superficial deposits and what is the range of their thickness. It was concluded that most of the surface was so covered and thickness range from a few meters to 40m with buried valleys of 80m deep.

Although in this estimate glacial deposits and river alluvials were lumped together it is clear that the geotechnical character of till and related glacial units was radically different from river and estuary sediments.

B2.0 Shear wave velocity

In EC8 Parts 1 and 5, the profile of shear wave velocity with depth is the preferred way of classifying the ground although it is recognised that SPT N values are more likely to be to hand. SPT tests in glacial tills and even the fluvio-glacials are however difficult to use in a meaningful way.

Both intrusive and non-intrusive seismic methods developed over the last decade and the quality of their product has improved vastly (see CIRIA/GSL 2002). It is conventional standard practice to identify the principal seismic shaking as from horizontally polarised body shear waves propagating vertically upwards (or at some steep angle). There is some evidence that in some circumstances the usual assumption that the velocity of a surface Rayleigh wave is very nearly the same as the shear wave does not hold (see Richart *et al* 1987). Butcher *et al* (1997) show that while the Vs versus depth results derived from surface Rayleigh waves, downhole and crosshole S waves were close in medium dense sand, they were not so in stiff to very stiff over-consolidated clays. This brings in the matter of cross anisotropy and it would seem that in very stratified profiles cross-hole methods are to be preferred.

A selection of examples from relevant settings has been examined, and they are reported as follows.

Site a

This site in Fife has some 17-20m of lithologically undifferentiated glacial deposits overlying Palaeozoic rocks. The values were derived from cross-hole and down-hole seismic procedures (Ricketts et al 1995). Two units within the glacial deposits were modelled on geophysical grounds as having Vs = 300m/s but different densities. The bedrock velocities increased from about 1000m/s at 20m depth to 2000m/s at 90m depth.

Site b

This site is on the Cumbrian coast and has been intensively used and investigated over several decades. Recent studies have, as well as yielding N values, through cross hole, uphole and down hole seismic methods, which provide Vs values for fill, a cohesive glacial/fluvial unit, a granular glacial/fluvial unit, and mudstone.

UNIT	AOD (m)	Mean Vs (m/s)	Range of Vs (m/s)
Hydraulic fill		280	200-400
Cohesive glacial/fluvial deposits Upper	Above –10	300	200-400
Granular glacial/fluvial deposits	-5 to –15	300	200-500
,,	-15 to	400	200-650
,,	-20	600	400-800
,,	-25	650	450-800
Cohesive glacial/fluvial deposits, Lower	-10	500	350-650
,,	-30	750	600-900
Mudstone	-25	850	650-1100
,,	-40	1050	800-1250

Site c

This site was studied for the siting of vibration generating plant. It is situated on Middle Coal Measures with about 15 of superficial deposits. Crosshole seismic measurements were made using three boreholes. The results are set out below

UNIT	Depth below ground level (m)	Mean Vs (m/s)	Range of Vs (m/s)
Made ground and lagoon deposits	5	250	200-350
Upper Glacial Till	5-11	300	200-400
Glacial sands and gravels	11-15	350	250-500
Middle Coal Measures	15-25	500	260-700

Site d

The ground profile at this site near the Severn has Made Ground on Alluvium overlying weathered Mercia Mudstone, leached and unleached Mercia Mudstone resting on Triassic Basal Conglomerate. It has been intensively investigated by rotary coring and crosshole , uphole, down hole seismics and borehole sonic logs. The table below summarises the findings in terms of best estimates

Layer	Depth range (m)	Vp (m/s)	Vs (m/s)	Comments
Made Ground	0-4.2	700	250	
Alluvium	4.2-7.7	600	160	
Weathered Mercia Mudstone (1)	7.7-8.5	1100	250	Depth to ground water 8.5m
Weathered Mercia Mudstone (2)	8.5-9.2	1300	250	
Leached Mercia Mudstone (1) Siltstone	9.2-11.2	1650	600	
Leached Mercia Mudstone (2)	11.2-12.6	1650	750	
Leached Mercia Mudstone (3)	12.6-13.7	1650	350	
Leached Mercia Mudstone (4) *Undifferentiated*	13.7-22.	2500	650	
Unleached Mercia Mudstone (1) *Mainly sandstone*	22.6-31.2	3200	1000	
Unleached Mercia Mudstone (2) *siltstone*	31.2-33.4	2600	800	
Triassic Basal Conglomerate	33.4-35.0	4500	1500	
Devonian	Below 35m depth	2800	950	

Site e

This site on the north shore of the English Channel has been well investigated over several decades. The upper 39m of Holocene deposits above the Lower Cretaceous Hastings Beds are divided into Storm Beach, Upper Shoreface, Lower Shoreface and Offshore Apron and Basal or “Lag” Gravel. The seismic velocities were obtained mainly by crosshole, downhole and uphole methods supplemented by a non intrusive SASW technique. A summary of the findings is given in the table below

Layer	Depth to base (m)	Material	Stratum	Vp (m/s)	Vs (m/s)
1a	3.3	GRAVEL <20% sand	Storm Beach	Linear increase 170-305	Linear increase 100-178
1b	8	GRAVEL <20% sand	Storm Beach	Liner increase 2000-2200	Linear increase 178-290
2	18	SAND& GRAVEL	Upper Shoreface	Constant @2000	Linear increase 290-320
3a	24	SAND<10% gravel	Lower Shoreface “Middle Sands”	Linear increase 1600-1700	Linear increase 200-295
3b	36.5	SAND<10% gravel	,,	Constant @1700	Constant@ 295
3c	39	SAND<10% gravel	,,	Linear increase 1700-1850	Constant @295
4	39.5	GRAVEL with sand	Lag Gravel	Linear increase 1850-1880	Constant @295
5a	44	Very stiff CLAY	Weathered Hastings Beds	Linear increase 1880-2160	Linear increase 295-460
5b		Hard CLAY	Unweathered Hastings Beds	Linear increase 2160 to e.g. 2420 @ 70m	Linear increase 460 to e.g. 600 @ 70m

Site f

The layering of several slopes (1:10 to 1:4) in Wales has been studied using surface seismic refraction methods. The superficial deposits consisted of 5-10m of “head”, typically clayey sands and gravels with Vs ranging from 150-300m/s, resting on weathered and fractured sands and mudstones with Vs ranging from 380- 745m/s.

Site g

This site in Gravesend, Kent was studied with the surface to down hole method

Borehole	Material	Depth Range (m)	Vp (m/s)	Vs (m/s)
1	Chalk	2-4		230
	Chalk	2-13	2230	
	Chalk	5-8		480
	Chalk	14-23	3820	
3	Thanet	0-2		190
	Thanet	3-12	3350	430
	Thanet/Chalk	0-4	790	
	Chalk	5-14	1620	
	Chalk	15-30	3350	

Site h

This site is in the Plymouth area and has been intensively investigated. Down hole, crosshole and seiscone methods have been used. The superficial deposits consist of 25-30m of river and marine alluvials on a slate bedrock. Values from a representative profile are shown below.

Depth (m)	Vs – cross hole (m/s)	Vs - downhole (m/s)
5	240	120
10	158	190
15	175	140
20	181	200
25	203	210
28		+ 800

Site i

This is site in the Wakefield area yielding further Vs values from downhole and crosshole methods. Alluvial and glacial deposits, 10-15m thick rest on Lower Magnesian Limestone.

Depth (m)	Vs – cross hole (m/s)	Vs - downhole (m/s)
0-3		190
3-4		100
4-7	190	100
7-9	190	280
9-11	300	270
11-14	400	180
15-19	300	
19-24	700	

Site j

The seiscone was used on a site in Stockport where Boulder Clay (till) rested on Coal Measures. Six tests were made and 20m was penetrated. The test were all similar in outcome yielding an average value of Vs of 200m/s

Site k

Four CPT tests in 23m of lake and river deposits in Glasgow resting on the Upper Limestone Group are summarised below:

Depth (m)	Vs range (m/s)
5	133-170
10	180-230
15	200 – 250
20	200-230

Site l

A seismic one test in 4-5m made ground over alluvium in Cardiff is summarised below

Depth range (m)	Vs (m/s)
0-4.6	160
4.6-5.7	170
5.7-6.6	155
6.6-7.6	135
7.6-8.6	150
8.6-9.6	190
9.6-10.5	220

Site m

Caerphilly – Sandy Boulder Clay(till) penetrated 9m.

Depth (m)	Vp (m/s)	Vs (m/s)
2	310	470
3	315	810
4	340	380
5	440	510
6	410	340
7	500	400
8	490	440
9	1050	400

Site n

Liverpool. Sand above till resting on pebble beds of the Sherwood Sandstone Group. Results for four boreholes are shown.

Depth (m)	Vs (m/s)			
	Borehole 1	Borehole 2	Borehole 3	Borehole 4
0	125	150	165	175
1	150	255	190	230
2	170	225	175	180
3	180	200	200	180
4	240	180	330	260
5	240	310	280	285
6	270	310	255	280
7	225	275	200	270
8	240	285	270	275
9	275	280	310	300
10		270	310	280
11		360	325	325
12		425	460	

Site o

Coventry – 0 to 3m Made Ground, 3m+ Meriden Formation, Mudstone and Sandstone

Depth (m)	Vs (m/s)			
	Borehole 1	Borehole 2	Borehole 3	Borehole 4
2	160	170	150	190
4	410	450	180	320
6	360	370	300	350
8	370	460	300	350
10	420	500	320	370
12	450	500	434	430
14	410	470	440	460
16	470	570	440	470
18	460	600	500	500
20	450	540	320	460
22	520	540	500	500
24	520	520	470	550

Site p

Margate – Lower Chalk

Depth (m)	Vs (m/s)			
	Borehole 1	Borehole 2	Borehole 3	Borehole 4
2		600	700	
4	500	700	750	
6	650	800	900	
8	800	850	900	700
10	800	1000	1000	800
12	800	800	1000	850
14	1250	1000	1200	850
16	1200	1200	1150	850
18	800	900		800
20	1050	1200		950

Site q

Pembroke downhole seismic testing. Clay 0-10m then limestone, Clay to 27m then mudstone and at 37m, limestone

Depth (m)	Borehole 1		Borehole 2		Borehole 3	
	Material	Vs (m/s)	Material	Vs (m/s)	Material	Vs (m/s)
5	Clay	200	Clay	250	Clay	200
10	Clay	180	Clay	200	Clay	205
15	Clay	300	Limestone	510	Clay+?	270
20	Clay	250	Limestone	610		370
25	Clay	270				400
30	Mudstone	500				370
35	Mudstone/Limestone	1000				320
37	Limestone					

Site r

Grimsby –firm to silty slightly sand clay with occasional gravel fragments, cobbles and thin sand lenses.

Depth (m)	Material	Vs (m/s)	
		Borehole 1	Borehole 2
2	See text	300	360
5		220	220
10		240	240
15		330	330

Site s

Boston Spa.Clayey Made Ground 0-3m, Brotherton Formation (cyclic sequences of dolomite, anhydrite and mudstone)

Depth (m)	Material	Vs (m/s)			
		Borehole 1	Borehole 2	Borehole 3	Borehole 4
5	See text	250	250	300	350
10		490	700	450	300
15		755	755	700	750
20		550	600	750	490
25		750	950	1000	850
30		1200	1400	1250	900

Site t

Lanharan. Eyeball envelope 2 holes

Depth (m)	Material	Vs (m/s)	
		Borehole 1	Borehole 2
5		200	300
10		200	400
15		360	420
20		400	450
25		400	500
30		450	600

Site u

Angelsey CPT1 0-2.75m, 140m/s; 2.75-3.75m, 145m/s; 3.75- 4.75m, 165m/s

B3.0 Comments on the assembled data

The data has been collected from both published papers and through the good offices of individuals, e.g. G.Rickets (Soil Mechanics) D. Mallard (SHWP), J Pappin (Arup). A number of techniques have been used to acquire the data and the procedures are described in geotechnical and geophysical publications. On a number of recent projects several different techniques have been used. The constraints of time and funding have not permitted close comparison to be made between the different methods. In the scoping evaluation a distinction has not been made. Furthermore, because some of the work is reported from on-going projects, the background geological and geotechnical content is sparse.

The examples given are considered to be of high quality in that shear waves have been clearly detected. In most cases P wave data is also available but its use as a useful check on the quality of the work has not been undertaken.

A coarse division of the data into the categories Glacial, Alluvial, Rock has been made. A simple examination of the distribution of values (all techniques lumped) indicates that the mode for the total “samples” of about 70 is in the bin 250-300m/s. The maximum depth of these samples was about 30m, with most from 10-15m below ground level.

It should be noted that the increase in Vs with depth has not been taken into account.

A similar exercise with the data (35 samples) from deposits described as alluvial indicate a mode of 150-200m/s. This finding is not surprising since the Glacial deposits include a variety of compact tills.

A feature of a number of the sites is the Vs assigned to the “rocks” of the “solid geology”. The sedimentary rocks of the Mesozoic and Cainozoic are often weathered and fractured. Unlike the Pre-Cambrian of ice scoured Palaeozoic rocks of northern Britain, they do not present a sharp impedance contrast to superficial and probably over-consolidated material. This prompts the question of whether the value Vs = 800m/s is appropriate in the UK as the criterion for defining bedrock.

B4.0 Discussion of the impact on the National Annex

The hazard map in preparation is based upon a reference return period of 475 years with a supplementary value at 2450 years and an assumption of Ground Type A, as defined in BS EN 1998-1:2004 Table 3.1.

The coverage of glacial deposits and the paucity (certainly in the lowlands) of rock with Vs=800m/s within 5m of the rock ensures that Ground Type A is a rare.

It is therefore necessary to consider the implications of the adoption of Ground Type B as of general application in the UK. The impact of such an approach depends upon the shape of the elastic response spectrum. If Type 1 spectrum (Fig3.2) is used then the S or soil factor multiplier (Table 2.2) is 1.2 whereas if Type 2 (Fig 3.3) is used the value of S is increased to1.35 (Table 3.3).

Examination of Table 3.1 does not throw up a soil profile that matches the conditions in the UK better than Type B although Type E has hints of a catch all formulation. There is a possibility of a soft normally consolidated clay of recent origin resting on a scoured rock platform and of course this is a condition the engineer should be on guard against.

Other exotic soils or settings should be borne in mind; soils having the properties of S1 and S2 in Table 3.1. Perhaps the most questionable material could be a laminated glacial lake deposit. Thames Brick Earth, which is of Aeolian origin, has collapsing properties but no record of troublesome response to shaking.

B5.0 Conclusions and recommendations

The conclusions to the study within the framework of the methodology are embodied in Table B1. This table is based upon the measurements of Vs by a variety of means and on that count alone it is far from homogeneous. Further more it has proved difficult to be sure of the “bedrock” with Vs ≥ 800m/s. Many boreholes did not go deep enough especially where the “seiscone” is used. In the few SSI models analysed for the UK, sensitivity studies have explored the effect of depth to a “bedrock”.

Several profiles have a layer of material Vs 100-300m/s 10-20m thick on soft rocks that seem to be nearer to ground type E although Vs may be less than 800m/s.

The time and cost restraints have limited the study. In only a few cases has a 1D profile response (e.g, SHAKE, WAVE, SIREN) been done. In view of the opinion expressed by Rey et al 2002 that subsoil class B presented problems it is recommended that a limited number of cases assessed as B ground type in the UK be studied a set of time histories consistent with the design spectrum ultimately favoured for the UK.

It has been found difficult to assess with confidence the distinction between types B and E. It has been noted that type E is something of “catch all”. It may be noted that excepting very simple cases, the soil profiles over large areas of superficial deposits can be difficult to assign to the EC8 categories with confidence.

Table B1: Tentative assessment of Ground Type according to EC8

Site	Geomorphology (Note 1)	Nearest equivalent EC8 ground type	Remarks
a	Glac/Per	B	
b	Glac /Per	B	
c	Glac /Per	B	
d	Alluv	B	
e	Alluv	B	
f	Alluv	B	
g	Alluv	B?	Soft rock bedrock
h	Alluv	E?	Bedrock slate
i	Glac	E?	
j	Glac	B/E?	Low Vs to 10m bgl
k	Alluv	?	Low Vs no bedrock
l	Alluv	?	Low Vs no bedrock
m	Glac	B	
n	Glac	B?	No bedrock encountered but probably sandstone at +12m bgl
o	?	?	Soft rock
p	Alluv	?	Soft rock
q	Glac	B	
r	Glac	?	
s	?	B	
t	?	?	
u	Glac?	?	Low Vs

Note 1: The following abbreviations are used to describe the geomorphology.
Glac: Glacial
Glac /Per: Glacial/Periglacial;
Alluv: Alluvial, river, lacustrine, estuarial.

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Appendix B: Seismic vulnerability of low rise residential buildings in the UK

These notes were prepared by Peter Watt of the Charlwood Partnership Ltd. They refer to the report by CAR (1988) listed in the reference section of this report.

General

The original CAR report still has much of relevance and is not too outdated to be of use.

It is noted that the initial sections dealing with risk of seismic damage show very low incidences indeed and then only at lower damage levels with specific items such as cracked walls and unstable chimney stacks to the fore.

Building Regulations have, of course, been updated several times since 1988 and the most recent A.D. "A" has the enhanced guidance for robustness design and for updated disproportionate collapse requirements. The masonry code of practice BS5628 has been updated in 2005 to take account of these changes which includes disproportionate collapse design requirements and enhanced stability/robustness.

BS8103 for low-rise buildings design is actually mentioned in the original CAR report, but only Part 1 was available at that time. All four Parts have been available for over a decade now and are all currently being revised. A.D. "A" and the Scottish Building Standards and Northern Ireland Building Regulations all list BS8103 as complying design documents. BS8103 is compatible with A.D. "A" simple masonry design rules, but goes a bit further than the A.D., particularly with regard to certain aspects of stability tying etc.

Both A.D. "A" and BS8103 only cover up to 3 storey residential and related construction. Therefore the tying and stability rules are geared to that total storey height limitation. BS5628 is not so limited and its stability tying rules are enhanced for 4 storey buildings and a disproportionate collapse design would be needed basically for 5 storeys and above, so the robustness/stability rules become enhanced automatically as the "economic" importance of the structure increases.

A.D."A" contains 3 building classes (basically risk classes) in Section 5. Classes 1 and 2A require only general robustness/stability design (A.D."A" simple sizing rules or BS8103 approach as alternative would do). Class 2b would involve full disproportionate collapse design to a code of practice. Class 3 is the highest (risk) class and a full risk assessment is required. This would include seismic considerations if appropriate.

BS6399 Part 2 has been introduced since the 1988 report was issued. This replaces CP3 and is generally considered by practitioners to be significantly more severe in its determination of wind pressures. Therefore, the wind loads have effectively increased, compared with CP3.

BS5268 Part 6.1 and 6.2 for timber framed design have been around for some time now. Both are currently being revised as BS's and they are expected to increase the storey height scope from 4 to 7 storeys, including any masonry veneer. Timber frame is also subject to disproportionate collapse design basically above 4 storeys.

There is also of course the Eurocode 1 Part that deals with disproportionate collapse actions. A.D. "A" was in reality amended last time around to deflect/encompass most of these Eurocode requirements.

All of these changes mean that certainly masonry construction requirements have moved forward since 1988.

Specific

The original CAR report makes a number of recurring comments on specific issues on masonry construction and some of these are worth looking at more closely in terms of developments over the past 20 years or so.

1. Chimneys and gable spandrels. All of the design documentation now requires specific tying requirements for walls, including gable spandrels, to flooring and roofing elements. A 90mm min. bearing of floors onto loadbearing walling is only allowed up to 2 to 3 storey construction, and for tall buildings horizontal tying is required under all circumstances. Gable spandrels now have specific tying requirements at the verges and in the case of higher rise spandrels at the ceiling level as well. Higher pitch roofs have become more common. Chimney stack projections above roof level and parapet walling are thickness to height limited by regulations, as the original CAR report states.

2. All trussed rafter roofs are required to be diagonally and longitudinally braced and in some cases braced across internal struts/ties too. BS5268 Part 3 has recently been republished and recommendations have been revised. Bracing requirements for all roofs will now be tighter than they were in the 1980's. This is also true for traditional cut framed and pitched roofs where building regulations now refer to the need to consider bracing for this type of roofing construction as well. Previously it was not required at all.

3. Roofing tile and internal shelving movements due to earthquakes are mentioned in the report. These of course are not structural issues in respect of building regulations. Tiles used as a roof cladding are not subjected to building regulations as part of structure. However, since the introduction of BS6399 Part 2 all roof tile manufacturer's have updated their roof tiling design guidance and nailing of tiles is now a more critical requirement, as is anchoring of tiles at verges and along eaves.

4. BS6399 Part 2 has introduced a step change in most structural design as wind loads have generally increased. The report frequently mirrors wind design with seismic effects for key elements of structure. To the extent that improving wind resistance also improves seismic resistance – which generally is the case - robustness will have increased simply by having to use BS6399 Part 2; particularly for those key specific elements mentioned in the report.

5. The report relates to wall ties and other tying metalwork. In 1988 and for a period thereafter galvanised wall ties were still acceptable and widely used. Since the late 1990's stainless components have taken over and building regulations effectively make stainless wall ties mandatory. Most straps are now also stainless, although there is not a history of galvanised strap corrosion as there is for some wall ties in sensitive locations of use.

6. The original CAR report mentions cavity walls as being the only wall form allowed by building regulations, but that was not the case then and is not the case now. Solid walls have always been allowed in the building regulation simple sizing rules, but generally they are not built (some modern variations are, but not many in number).

5. The timber frame design code BS5268 Part 6.1 and 6.2 are in the final stages of amendment. These are expected to increase potential construction heights from the current 4 storey level to 7 storeys in terms of the use of the codes of practice. Masonry veneer is allowed full building height.

Conclusions

The original CAR report provides some comfort on the degree of seismic vulnerability, as it identifies low risks and low damage rates against masonry construction forms that would nowadays be somewhat updated in their basic technical approach. The effect of the current upgraded requirements should be to reduce actual damage in a UK earthquake even further than that identified in the original CAR report.

Basic robustness.stability requirements have tightened, particularly with the latest version of A.D. "A" which again is helpful.

For more "economically" sensitive masonry buildings the disproportionate design requirements are again a significant step and would reduce seismic loading actions consequences greatly.

The use of stainless steels as wall ties and restraint straps has removed the longer term doubt over corrosion and loss of efficiency. This will be particularly beneficial to cavity wall construction and basic building stability interaction.

BS6399 Part 2 has increased wind loadings. This has effectively meant that stability design has been tightened by default for sensitive elements like gable spandrels.

For timber frame, 7 storey construction with full height masonry veneer is now a practical design reality. This probably represents an increase in seismic risk, but BS6399 Part 2 may again offer an offsetting effect against any increased risk.