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SEISMIC HAZARD TO WELDED STEEL PIPELINES IN THE UNITED KINGDOM

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Summary

The United Kingdom Onshore Pipeline Operators' Association (UKOPA) is considering the need for guidance on seismic risk to major accident hazard pipelines and installations. The guidance would seek to offer a consistent and proportionate approach to seismic design requirements for onshore pipelines within the current UK goal setting regulatory regime.

This report considers the history of seismic hazard assessment for engineered structures in the UK and the decisions on appropriate annual probability of exceedance values (return periods) for seismic actions.

Several sources of seismic wave generation are identified and discussed including natural earthquakes and induced earthquakes caused by underground mining, explosive detonations and hydraulic fracturing.

The underlying cause of British earthquakes is discussed based on the extensive work of the British Geological Survey.

The requirements for seismic design to Eurocode 8 and the variations through UK National Annexes and guidance from PD 6698 are presented. This clarifies the need to demonstrate the adequacy of the seismic resilience of major accident hazard pipelines and installations, either by virtue of the low seismic hazard or the low vulnerability achieved by structural design.

The inherent low vulnerability of modern welded steel pipelines to seismic hazard is demonstrated through a review of pipeline performance in past earthquakes.

Seismic hazard in the UK is presented in terms of peak ground motions and seismic intensity (a measure of damage potential) at several return periods. This allows the significance of the return period on the seismic hazard to be illustrated and for the regional variation in natural seismic activity to be presented.

Possible criteria are presented and discussed for the potential screening of seismic hazard to enable an appropriate level of assessment and design to be determined.

Finally, a comprehensive technical study by consultants working on seismic policy development for National Grid is briefly reviewed.

Conclusions

Earthquakes

1. Earthquakes are a release of strain energy due to the fracture of rock and the transmission of some of that energy as seismic waves.

Natural earthquakes in the UK are attributed to the failure and movement of existing geological fault structures primarily due to crustal compression associated with tectonic plate movements.

Induced earthquakes in the UK have been attributed to underground mine collapses, rock blasting and hydraulic fracturing in deep boreholes. Hydraulic fracturing includes oil and gas reservoir stimulation, geothermal reservoir stimulation and shale gas extraction (fracking).

2. Earthquakes can be quantified in terms of a magnitude scale according to the amount of energy released. Several magnitude scales exist according to the determination method. The cumulative number of earthquakes in a region above a particular magnitude is found to decrease with magnitude according to the Gutenberg-Richter log-linear relationship. On average, the UK may expect a local magnitude 3.7 earthquake every year, a magnitude 4.7 earthquake every 10 years and a magnitude 5.6 earthquake every 100 years.

The threshold earthquake magnitude of engineering significance is a moment magnitude of ~4.5. There have been ~27 earthquakes across the UK of this magnitude or greater in a ~300 year period.

3. The anticipated maximum magnitudes for natural and induced earthquakes in the UK are:

- Natural 5.5-6.5
- Mining 3.2-3.5
- Geothermal 1.9
- Fracking ~3.0

4. Earthquake effects at surface can be measured quantitatively in terms of ground motions or assessed qualitatively through the observed damage. The normal ground motion measurement is the variation of horizontal and vertical ground acceleration with time which can be integrated to obtain velocity and displacement ground motions. Damage is assessed and assigned to a seismic intensity scale. Several intensity scales exist although the scales in most common usage are the 12-point Modified Mercalli Scale and the 12-point European Macroseismic Scale.

5. Natural earthquake activity is probably due to the release of localised crustal stress build-up associated with the interaction of the current stress field with major crustal inhomogeneities and pre-existing faults. Earthquakes are positively correlated with geological faults within crystalline basement rocks particularly on strike-slip faults with a favourable orientation in the current stress field.

6. Natural earthquake activity is variable across the UK mainland showing higher levels of activity in the west than in the south and east of the country.

Seismic Hazard

7. Seismic hazards to buried pipelines and above-ground installations include:

- Transient loading due to seismic wave propagation (ground shaking).
- Permanent ground movement due to:
    - Geological fault displacement at surface.
    - Ground movement due to slope instability.
    - Ground movement due to liquefaction.

8. Seismic hazard assessment in the UK has been under development since the early 1970’s with particular motivation from the nuclear industry. Earthquake instrumental monitoring in the UK has expanded since the early 1970’s to reach a current network of over 100 stations operated by the British Geological Survey.

9. A seismic hazard study completed in 1991 for the Department of Environment concluded that there was no justification for the inclusion of earthquake resistance into the design of conventional structures. However, certain structures whose failure could pose a hazard to a significant number of people should incorporate earthquake loading in their design. The study identified an outstanding need for guidance on how to decide if a structure or installation was sufficiently hazardous to justify a seismic design.

10. Eurocode 8 (BS EN 1998 Parts 1-6) for the seismic design of structures in Europe was issued in 2005 and 2006. The national annexes for the UK decisions on nationally determined parameters for the application of Eurocode 8 in the UK were issued in 2008 and 2009. The UK National Annexes are supported by PD 6698 issued in 2009.

Special structures, such as nuclear power plants, offshore structures, large dams and long span suspension bridges, are excluded from the scope of Eurocode 8. This is due to particular aspects of regulatory and detailed design performance for these categories of structure.

The UK decision on seismic design is that there are no requirements for most structures but certain types of structure may warrant an explicit consideration of seismic actions. PD 6698 identifies major hazard sites and major accident hazard pipelines as categories of structure that should be designed to withstand very low probability events, including earthquakes.

PD 6698 recommends that consequence class CC3 (defined in BS EN 1990:2002 as 'High consequence for loss of human life, *or* economic, social or environmental consequences very great') pipelines (and installations) are assessed on a project specific basis to determine if an explicit consideration of seismic actions is required. The structural vulnerability, location and consequence of failure are factors to be considered in deciding on any need for seismic design.

Facilities assessed as posing a large risk to the population or to the environment require a site specific hazard assessment to establish appropriate design ground motions. Other high consequence facilities may use the peak ground acceleration according to a seismic hazard map in PD 6698 for a return period of 2500 years (annual probability of exceedance of $4x10^{-4}$).

11. A particular challenge for major accident hazard pipeline operators is to interpret which pipelines and installations should be considered for seismic design and which seismic hazard assessment approach is appropriate (site specific or hazard map).

12. The selection of a return period for the determination of a seismic action for the design of structures to prevent local or global structural collapse varies as follows:

Facility/Structure	Return Period (years)
Eurocode 8: CC3 buildings e.g. schools, assembly halls, cultural institutions etc. & CC3 silos, tanks & pipelines. [high risk to life]	~800
Eurocode 8: CC3 buildings e.g. hospitals, fire stations, power plants etc.	~1300
Eurocode 8: CC3 silos, tanks & pipelines. [exceptional risk to life]	~2000
PD 6698: CC3 structures.	2500
BS EN 1473: LNG facilities.	5000
Category III dams, Buildings on chemical manufacturing sites, Nuclear power stations.	10,000

Category IV dams[1].	30,000

This summary appears to indicate that a suitable return period for major accident hazard pipelines is ~2500 years and for major hazard sites is 5000 or 10,000 years. The differing return periods for pipelines and installations reflects the difference in the potential failure point and consequence (variable for pipelines but fixed for installations).

Soil Amplification

13. PD 6698 provides seismic hazard maps for peak ground acceleration (PGA) on rock for return periods of 475 years and 2500 years. The PGA values for local site conditions can be determined using soil amplification factors according to Eurocode 8.

Pipeline Vulnerability

14. A review of the performance in past earthquakes of modern welded steel pipelines with full penetration butt welds indicates a low vulnerability to damage. A seismic intensity level of ~IX appears to be the threshold level for damage. This intensity level is associated with the collapse of weak structures, very heavy damage to ordinary buildings and damage to some specially designed structures.

A seismic hazard analysis suggests that the annual probability of seismic intensity level IX on the UK mainland is less than $1x10^{-4}$ per annum (equivalent to a return period of greater than 10,000 years). A maximum credible earthquake for the UK would need to occur within ~40 km of a site for any credible prospect of achieving intensity level IX. The chance of intensity level IX at the epicentre is estimated at 1 in 6.

15. Pipeline stress state calculations for the effect of propagating seismic waves suggest that tensile failure is highly improbable in the UK. Compressive failure in the form of longitudinal buckling is also unlikely unless pipelines are operating at elevated temperatures.

16. Pipelines may be vulnerable to damage due to slope instability or to ground movement associated with liquefaction. However, this would require a pipeline to be routed across unstable or metastable slopes or to traverse very loose or loose susceptible granular soils.

Criteria are available to assess landslip and liquefaction potential in susceptible deposits.

[1] Category IV dams are structures with a downstream population at risk of exceeding 1000. The International Commission for Large Dams recommends a return period of 10,000 years or a safety evaluation based on the maximum credible earthquake for the region.

Above-Ground Installation Vulnerability

17. Above-ground installations are susceptible to dynamic amplification of earthquake forcing vibrations at ground supports or via supporting structures. This depends on the natural period of vibration of the above-ground piping relative to the forcing frequency of the input motions.

Dynamic amplification can be quantified by a response spectrum which varies according to the local site conditions and the frequency content of the input motions. Design spectra are provided in Eurocode 8.

Screening Criteria

18. The ASME B31.8S code 'Managing System Integrity for Gas Pipelines' indicates that pipelines may be susceptible to extreme loading at locations where the pipeline crosses a fault line, the soil is subject to liquefaction or the ground acceleration exceeds 0.20 g.

19. ASCE guidance for the seismic design of natural gas distribution systems identifies a criterion threshold of 0.15 g for the 1 second spectral response acceleration for a return period of 2500 years. Seismic design or mitigation should be considered when the seismic hazard exceeds the threshold level.

20. A seismic intensity threshold of VII for a return period of 2500 years or a threshold of VIII for a return period of 10,000 years would limit seismic design for major accident hazard pipelines and installations to the more seismically active areas of the UK. This includes; north-west Wales, South Wales through Herefordshire to the Midlands and north along the Pennines to Cumbria, the west coast of Scotland and the area around Comrie.

Building Research Establishment Guide

21. An engineering guide to the seismic risk to dams in the UK may provide a suitable model framework for a similar guide for major accident hazard pipelines and installations.

National Grid Policy Direction [January 2014]

22. Consultants working for National Grid have developed a comprehensive strategy for seismic design considerations associated with gas transmission pipelines, above-ground installations, buildings, mechanical plant and electrical equipment.

Pipelines and installations are assigned to an importance class according to safety (potential casualties within the hazard zone) and security of supply (outage time). Safety is considered in terms of the number of casualties within the hazard zone: none, up to 10, up to 100, up to 1000 & over 1000. Security of supply is considered in terms of repair time: ~1 day, ~1 week & more than 2 weeks.

The potential for casualties of over 1000 dictates a site specific seismic assessment.

Each importance class is assigned an importance factor. The importance factor is used to determine the appropriate return period for the seismic action.

The importance factors are as follows:

Importance Class	I	II	III	IV
Importance Factor γ_I	n/a	0.45	1.0	2.0

The approach anticipates the usage of the PD 6698 seismic hazard map for a return period of 2500 years. However, the PGA inferred from the map is increased by a factor of 1.5 to convert from geometric mean value to maximum component value and to introduce an allowance for the possibly that a site specific study might determine a higher hazard level.

The design ground acceleration on soil is determined as,

$$a_g.S = \gamma_f.\gamma_I.a_{gR}.S$$

where

a_g is the design ground acceleration on rock.
γ_f is a load factor (= 1.5).
γ_I is the importance factor.
a_{gR} is the reference ground acceleration on rock for a return period of 2500 years according to PD 6698.
S is the soil amplification factor according to Eurocode 8.

The seismic design requirements are determined according to the importance class and the value $a_g.S$ as follows:

<table>
<tr><td rowspan="2">Importance Class</td><td colspan="2">$a_g.S \leq 0.1\ g$</td><td colspan="2">$a_g.S > 0.1\ g$</td></tr>
<tr><td>Buried Pipelines</td><td>Above-Ground Installations</td><td>Buried Pipelines</td><td>Above-Ground Installations</td></tr>
<tr><td>I</td><td colspan="4">None</td></tr>
<tr><td>II</td><td colspan="3">None</td><td>Simplified</td></tr>
<tr><td>III & IV</td><td>None</td><td>Simplified</td><td colspan="2">Full</td></tr>
</table>

Recommendations

1. UKOPA should confirm the need for an engineering guide to the seismic risk to major accident hazard pipelines and installations in the United Kingdom (UK). The guide should take account of the model framework developed by the Building Research Establishment for the seismic risk assessment of large reservoir dams in the UK.

Key decisions include:

- Any need to differentiate major accident hazard pipelines or installations in terms of population density within the hazard zone.
- Any need to differentiate between major accident hazard pipelines and installations in the selection of return periods for the seismic action.
- The choice of actual return periods.
- The criterion to be applied in the selection of the appropriate level of seismic assessment.

2. As far as possible any guidance should utilise the seismic hazard map for a return period of 2500 years as published in PD 6698. Any new hazard maps should be sourced from the British Geological Survey e.g. seismic intensity if this is deemed to be the basis for regional screening.

3. UKOPA should engage with other pipeline operators in countries of Northern Europe with regions of similar seismicity to the UK e.g. France, Netherlands, Germany & Denmark, to determine their seismic design practices for major accident hazard pipelines and installations. This should assist in developing and benchmarking a proportionate approach to the seismic risk issue in the UK.

4. UKOPA should seek to clarify seismic hazard screening criterion for gas pipelines in the USA according to ASME B31.8S and ASCE TCLEE[2] monograph no. 8. This may assist in the development of seismic screening criteria for use in the UK.

[2] Technical Council on Lifeline Earthquake Engineering.

Glossary of Earthquake Terms

Earthquake

A release of strain energy (generally along a pre-existing fault) due to rock fracture and the conversion of that energy into heat and vibrational energy. The vibrational energy is transmitted through the surrounding rock as seismic body waves.

Epicentre

The point on the earth's surface that lies directly above the focal point of an earthquake.

Focal Depth

The depth below the earth's surface to the point at which the earthquake originates (point of fault rupture).

Magnitude (earthquake)

A quantitative measure of the amount of energy released by an earthquake. Several magnitude scales exist:

- Richter Local Magnitude (M_L) – is based on the maximum seismic wave amplitude A (in microns) recorded on a standard Wood-Anderson seismograph located at a distance of 100 km from the earthquake epicentre. The earthquake magnitude is calculated as,

$$M_L = \log(A) - \log(A_o)$$

  where A_o is a calibration factor

- Moment Magnitude (M_w) – is based on a function of the seismic moment $M_0 = G.A_f.\Delta u$. The earthquake magnitude is calculated as,

$$M_w = 0.67\log(M_0) - 10.7$$

  where
  M_0 is expressed in ergs [1 erg = 1 dyn cm or 10^{-7} J]
  G is the shear modulus of the material involved in the fault rupture.
  A_f is the area of the fault rupture
  Δu is the average relative slip across the fault

- Surface Wave Magnitude (M_s) – is based on the amplitude of surface waves. The earthquake magnitude is calculated as,

$$M_s = log\left(\frac{A}{T}\right) + 1.66\ log(\Delta) + 3.3$$

where A is the wave amplitude in microns
T is the period in seconds
Δ is the distance in degrees

Seismic

Related to an earthquake or vibration of the earth from natural or artificial causes.

Seismic Intensity

A measure of the severity of ground shaking through observations of damage to structures and ground failures (cracking, liquefaction, landslides). Intensity is quantified on a numerical scale. Several intensity scales are in common usage:

- Modified Mercalli (MM) Scale – a 12 level scale used in North America.
- European Macroseismic (EMS-98) Scale – a 12 level scale used in Europe.
- Japanese Meteorological Agency (JMA) Scale – a 7 level scale used in Japan.
- Medvedev-Sponheuer-Karnik (MSK) Scale – a 12 level scale developed in central and eastern Europe.

Contents

1. Introduction

The United Kingdom Onshore Pipeline Operators' Association (UKOPA) is considering the preparation of guidance on seismic assessment for onshore major accident hazard pipelines (MAH) in the United Kingdom (UK). The guidance is needed to ensure consistency in the appraisal of onshore pipelines for seismic design.

The purpose of the guidance would be to:

- Explain the nature of the seismic hazards to welded steel pipelines and installations.
- Provide an overview of UK seismicity and seismic hazard.
- Clarify current regulatory, code & standards requirements in the UK for seismic design of pipelines and installations.
- Suggest criteria for the selection of onshore pipelines for seismic assessment.
- Provide a structured approach to determining the level of seismic assessment required.

This report has been commissioned by UKOPA as an initial step towards the preparation of guidance on the seismic assessment and design requirements for onshore pipelines and installations in the UK. The work was authorised under UKOPA purchase order number UKOPA/PO/14/84 to:

- Review historic and current code requirements in the UK for seismic design of pipelines and installations.
- Identify the background to the need for seismic design in the UK.
- Indicate any policy direction under development by National Grid for high pressure gas pipelines.

The aim of the work is to assist in identifying the need and scope for UKOPA guidance on seismic hazard and to clarify the circumstances that would justify or require a seismic assessment or design of a pipeline or installation.

2. Background

Musson[1] describes the historical development of earthquake hazard assessment for strategic engineering facilities in the UK with particular emphasis on nuclear installations.

Irving[2] suggests that prior to 1970 the only structures in Britain built to accommodate significant earthquake forces were a number of dams in Scotland.

In the early 1970's the Kessock Bridge near Inverness was designed to incorporate seismic resistance in the structure. Construction started in 1976 and was completed in 1982. The bridge includes seismic buffers at the north abutment near the line of the Great Glen Fault. This fault was asserted to be the only tectonic feature in the UK definitely known to be associated with earthquakes[3].

The first seismic hazard map for the UK was produced by Lilwall[4] in 1976 at the Institute of Geological Sciences (IGS). This is presented as contoured areas of seismic intensity for a return period of 200 years using the Modified Mercalli (MM) intensity scale[‡]. The map indicates localised areas of intensity 7, equivalent to the onset of structural damage to some buildings. Lilwall also gives numerical examples of seismic risk in terms of maximum intensity, maximum ground acceleration and maximum ground velocity for differing return periods and differing British seismicity situations. A maximum ground acceleration of 0.20 g or greater was determined for a return period of 10,000 years. Lilwall's results are summarised in Appendix II.

Earthquake hazard assessment work for the UK nuclear industry began in 1973 at the Central Electricity Generating Board[5] (CEGB). The CEGB decided on designing nuclear reactors for safe shutdown following an earthquake event with a probability of exceedance of equal to or less than 10^{-4} per year per site. The initial work at the CEGB used seismicity data from the IGS and ground motion attenuation data from the USA to complete a seismic hazard analysis. At the 10^{-4} annual exceedance level the peak horizontal ground acceleration (PGA) was 0.15-0.25 g and the peak ground velocity (PGV) was 100-260 mm/s depending on alternative interpretations of the IGS data. The subsequent seismic design for the Heysham II nuclear power station adopted a PGA of 0.25 g.

Significant new seismic hazard studies for the nuclear industry were completed by Principia Mechanica and Soil Mechanics Limited in 1982[6,7]. These studies influenced the safe shutdown earthquake (SSE) conditions for a 10^{-4} annual probability of exceedance as follows:

- Average national uniformly distributed seismic activity gave a PGA of 0.20 g.
- The Sizewell site and its surroundings treated as part of an area of uniformly distributed seismicity for the whole of East Anglia gave a PGA of 0.15 g.
- The Sizewell site based on historic epicentres and local major faults as source zones gave a PGA of 0.10-0.15 g.

‡ See Appendix I for scale description.

- A major seismic hazard at 0.25 g SSE design level was found to be dominated by earthquakes of magnitude 4.0-5.0[§] within 30 km of the site.

UK earthquake monitoring using modern seismometers started in 1967 (operational from 1969) with a 7 station network in Scotland of short period vertical component seismometers[8]. The seismic monitoring network was gradually expanded from 1976 initially in part for the North Sea oil and gas industry[9]. The network expanded in response to several factors to reach ~140 stations[**] across the UK by 2005[8].

By the late 1980's it was clear that the forthcoming structural Eurocodes would include a code on the design of structures for earthquake resistance. This influenced the Department of the Environment to commission a study into the seismic risk to the built environment in the UK and to determine whether seismicity should be taken into account in future planning decisions. This study[10] was undertaken by Ove Arup and Partners between 1989 and 1991. The study findings included the following:

- An estimated UK regional seismic intensity of 7-8 (MSK scale[††]) for a 10^{-4} annual probability of exceedance.
- A magnitude 5.5 (M_s[††]) earthquake would be expected to produce a seismic intensity of 7 and lead to in excess of 100 fatalities. Major damage to non-building structures and infrastructure would be unlikely.
- In addition to nuclear facilities, earthquake loading should be incorporated into the design of other structures including some fuel and chemical storage installations, dams and some unusual structures.
- Guidance is required on how to decide when a structure or installation is sufficiently hazardous to require a design to consider earthquake loading and, if so, what level of earthquake ground motion is appropriate.

In 1997 Musson & Winter[11] identified a need for general-purpose maps of seismic hazard in the UK and published sample maps of seismic intensity for a return period of 475 years and PGA for a return period of 10,000 years. The predicted maximum intensity is 6 on the EMS-98 scale[††]. The maximum PGA is above 0.25 g in the area of Inverness, Comrie, The Pennines, North-West Wales, Hereford and Pembroke. The easterly, south

[§] The BGS advise that on average, the UK may expect a local magnitude 3.7 earthquake every year, a magnitude 4.7 earthquake every 10 years and a magnitude 5.6 earthquake every 100 years.
[**] BGS has subsequently continued to upgrade the network with the addition of broadband sensors (~38 by 2012) and strong motion accelerometers (~26 by 2012).
[††] See glossary.

easterly and southern areas of the country generally exhibit the lowest levels of seismic hazard. The hazard maps are presented in Appendix III.

Musson[12] introduced a Monte Carlo simulation approach to probabilistic seismic hazard assessment and this approach was subsequently used to produce seismic hazard maps for the UK[13].

In 2006 the Institution of Civil Engineers commissioned the preparation of a report giving guidance on the need for carrying out seismic design for UK structures. This report[14] contributed to the subsequent content of a BSI published document[15] on the design of UK structures for earthquake resistance and to the UK National Annexes to BS EN 1998, *Design of structures for earthquake resistance* (Eurocode 8).

The ICE study report[14] suggests that '*hazardous pipelines*' and '*gas distribution and transmission*' are categories of specialised industry where a requirement for seismic design would be at the discretion of the Health and Safety Executive.

PD 6698[15] indicates that certain UK engineered structures by virtue of their function, location or structural form may warrant an explicit consideration of seismic actions. In relation to the 3 factors:

- Function: there is specific mention by example of high pressure gas pipelines as a category of structure where failure poses a threat of death or injury to the population. This category would also embrace all major accident hazard pipelines.
- Location: the guidance refers to the significant regional variation in seismic hazard across the UK and to the local influence of superficial deposits in modifying seismic ground motions.
- Structural form: certain structural design elements may possess inherently poor resistance to seismic ground motions.

In 2008 the Institution of Gas Engineers provided guidance (through IGEM/TD/1 edition 5[16]) on the need to consider seismic activity in the design of high pressure gas transmission pipelines. This guidance includes a recommendation for an initial screening appraisal to determine if an explicit seismic hazard assessment and seismic resistance design is warranted.

The progressive emergence over the last 3 decades of seismic hazard as a potential issue for certain engineered structures in the UK has prompted UKOPA to clarify any need or justification for seismic design to major accident hazard pipelines and installations.

3. Seismic Hazard to Pipelines

The seismic activity along a pipeline route may justify an appraisal to determine any requirements for earthquake resistant design. This may involve higher strength pipe and fittings or protection/mitigating measures.

Earthquake activity can be expressed by the magnitude of the event (seismic energy released) and the return period (time interval or frequency). In a specific region the earthquake frequency reduces as the earthquake magnitude increases, figure 1.

Buried pipelines and above-ground piping may be designed for seismic loading according to a specified probability of a selected design seismic event being exceeded in the life of the pipeline. The earthquake history of the region will determine the characteristic seismic event for the chosen probability of exceedance level.

Current practice for the detailed seismic design of engineered structures generally involves two design events corresponding to differing return periods for two levels of structural performance:

- Damage limitation state – structure remains operational after seismic event.
- Ultimate limit state – structure sustains severe damage but without structural failure or collapse.

Earthquakes are also related regionally to intensity levels associated with the damaging influence of seismic events of differing magnitude. The estimated maximum intensity level for the design seismic event may be sufficient to discount seismic loading from the design.

The seismic hazards that affect pipelines are:

- Vibrational motion due to seismic waves (ground shaking).
- Permanent ground movement triggered by the ground shaking.

The most common sources of permanent ground movement are:

- Relative movement at or near surface across a geological fault.
- Lateral spreading due to soil liquefaction.
- Soil settlement due to liquefaction or vibratory compaction.
- Landsliding.

If the selected design seismic events are considered significant then analysis of seismic actions on the pipeline may include:

- Dynamic analysis of above-ground piping due to seismic movement of supports.
- Static analysis of above-ground piping due to permanent differential movement of supports.
- Transient analysis of buried pipe subjected to propagating seismic waves.
- Static analysis of the buried pipeline subjected to permanent ground deformations produced by seismic fault displacement, landsliding, liquefaction or compaction.

Further information on ground strain and curvature due to seismic body and surface waves, fault rupture, liquefaction and landsliding is presented in Appendix IV.

4. Seismic Hazard in the United Kingdom

4.1 Earthquakes

The UK is located near the western margin of the continental crust of the Eurasian tectonic plate. The immediate offshore areas are also continental crust but further west the continental crust terminates and oceanic crust extends westward to the Mid-Atlantic spreading ridge at the boundary with the North American tectonic plate.

The current tectonic stress field and the oceanic and continental topographic relief in the region of the UK are shown in figure 2. The continental shelf around the UK and Ireland land masses is shown in white. The Mid-Atlantic spreading ridge and the southern boundary of the Eurasian tectonic plate are delineated by a continuous black line.

The maximum principal stress is horizontal and compressive with an orientation of northwest to north-northwest in central and southern England and the southern North Sea. The orientation appears to swing to the north in Scotland (although there are limited measurements in this region). The minimum principal stress is also horizontal with the intermediate principal stress in the vertical (gravity) direction. This orientation of principal stresses is conducive to the activation of strike-slip faults, particularly those with a favourable orientation to the stress field.

The current stress field is consistent with the easterly to south easterly ridge push of new oceanic crust from the Mid-Atlantic ridge and the north to north easterly movement and impingement of the African plate onto the Eurasian plate in the Mediterranean. The current stress regime in Great Britain is weak relative to that required to produce orogenic deformation (folding and shearing of intact rock).

Earthquakes in the UK are probably caused by the release of localised crustal stress build-up associated with the interaction of the current stress field with major crustal inhomogeneities and pre-existing faults. This explanation is characteristic for intraplate (within tectonic plates) earthquakes[17]. Shedlock[18] suggests that intraplate earthquakes are often associated with former tectonic plate boundaries within existing plates. These represent weaker structures within the crust where stresses may be concentrated. The larger earthquake events (magnitude 6 or more) are usually associated with extensional 'rift' zones within continents. Other characteristics of intraplate earthquakes include:

- They are less frequent than interplate earthquakes (<10% of earthquakes occur within continental interiors).
- Causative faults are rarely recognised at surface.
- Seismic wave attenuation is slower due to more competent rock in the vicinity of the energy release.
- Crustal flexure due to changes in ice or water load or to upwelling mantle plumes may contribute as a causative effect.

Continental drift and the collision of tectonic plates has contributed significantly to the formation of the geological character of Great Britain (and surrounding islands). Chadwick et. al.[19] describe the geological evolution of the UK crust and present a terrane (crustal blocks) map of the UK landmass and adjacent North Sea. Each terrane has a similar geological character (properties & structure) and is 'sutured' to neighbouring terranes by fault systems and shear zones. Terranes amalgamate by accretion through tectonic plate collisions.

Chadwick et. al.[19] investigated the possibility of a correlation between current UK seismicity and bulk crustal properties, upper crustal structural blocks ('terranes') or structural faults and folds. The study concluded that present-day seismicity is most likely a result of interaction between adjacent crustal blocks on major fault systems (principally Precambrian[‡‡] and Palaeozoic[§§] faults). Other observations from the study for the UK landmass included:

- No correlation could be identified between current seismicity and variations in singular crustal properties of thickness, depth and heat flow.
- Some correlation was identified with deep crustal faults and shear zones but no correlation was found with comparable structural types in the upper mantle.
- Earthquake locations are found to be associated with (positive correlation) Precambrian and Palaeozoic fault systems. They are

‡‡ More than ~545 million years ago.
§§ Between ~248 & ~545 million years ago [Cambrian to Permian inclusive].

negatively correlated (i.e. not associated with) Mesozoic[***] and Cenozoic[†††] fault systems.
- The current weak crustal stress field is not conducive to the reactivation in the dip direction of either deep crustal thrust faults or shallow basin-controlled normal faults. However, thrust faults may exhibit some strike-slip movement and current strike-slip faults may be reactivated.
- Post-glacial isostatic rebound may exert a localised influence on seismicity.

The low seismicity of the UK is therefore attributed to the weak stress field and its orientation relative to the geological fault systems.

4.2 Blasting

Explosive detonations in earth materials for mining, quarrying and civil engineering activities produce seismic waves of the type generated by earthquakes i.e. elastic body and surface waves. However, blasting operations tend to involve much smaller energy releases than earthquakes and produce seismic waves at higher dominant frequency levels.

Experimental tests for the American Gas Association[20] and for British Gas[21] indicate a clear trend between pipe stress and peak particle ground velocity, figure 3.

National Grid adopts a control limit of 75 mm/s on ground vibration in the vicinity of high pressure gas pipelines[22]. This corresponds to an upper bound transient pipe stress of ~70 MN/m^2 based on the sample data in figure 3.

National Grid specification T/SP/GM/4[22] also provides an empirical expression for pipe stress due to blasting vibration as follows:

$$\Delta\sigma = 0.3.\,v$$

where

$\Delta\sigma$ is the transient stress in MN/m^2
v is the peak particle velocity in mm/s

Dowding[23] suggests that longitudinal pipe strain associated with blast induced vibration is primarily due to bending. This may be associated with the high frequency/short wavelength of the dominant seismic waves.

*** Between ~65 & ~248 million years ago [Triassic to Cretaceous inclusive].
††† Between the present and ~65 million years ago [Tertiary & Quaternary].

Pipeline bending stress and strain produced by longitudinal curvature due to seismic waves associated with earthquakes is anticipated to be secondary to uniform axial stress and strain produced by longitudinal direct extension/compressive. A control limit on peak particle ground velocity needs to take account of this observation. A control level may be determined according to a selected permitted transient axial stress limit and a lower bound effective horizontal propagation velocity for the critical seismic wave type.

4.3 Fracking

Shale gas extraction involves the artificial fracturing of suitable sub-surface rocks by fluid injection at high pressure. Two types of induced seismicity are associated with hydraulic fracturing:

- Seismicity due to the creation and propagation of engineered fractures.
- Seismicity due to the presence of a pre-stressed geological fault.

In 2011 a number of seismic events (~50) were detected in the vicinity of a drilling site at Preese Hall in Lancashire. The events occurred during a period of several fluid injections to hydraulically fracture several zones within a shale formation. The seismic events were not at a level that would cause damage to the drilled borehole or to any structures at the surface. However, some of the events were more significant than expected from shale rock fracturing operations. The Department for Energy and Climate Change (DECC) took the decision to immediately suspend all fracking operations for shale gas pending a thorough investigation of the cause of the seismic events and the scope to mitigate the risk of unacceptable induced seismicity from future similar operations for shale gas exploration and extraction.

The Preese Hall drilling site is operated by Cuadrilla Resources Limited (Cuadrilla) who commissioned their own investigations into the causes of the seismic activity and to suitable future mitigation measures. DECC commissioned several reports[24,25] including a review of the specialist studies commissioned by Cuadrilla[26].

Heszeltine[24] noted:

- An important factor in shale gas extraction is the requirement for a high density of drilling sites, typically at 3 km spacings.
- Two principal earthquakes near Blackpool in April and May 2011 were attributable to fluid injection operations at a depth of ~2-3 km at the Preese Hall drilling site. The earthquakes were of relatively

modest magnitude (M_L=2.3 & 1.5) but also at relatively shallow depth (~3.6 km & ~2 km).

- Similar magnitude earthquakes have been induced by coal mining, filling of large water reservoirs, deep geothermal exploitation and oil production offshore.
- There is a high risk of further seismicity associated with shale fracturing but the effects are likely to be small. Better seismic monitoring is needed.

In their review of study outcomes commissioned by Cuadrilla, Green et. al.[26] agreed that the principal earthquakes were attributable to the hydraulic fracture treatments at the Preese Hall drilling site but disagreed with an opinion that no further earthquakes would occur due to similar treatments in a nearby well.

Green et. al.[26] suggest that a realistic upper limit for fracturing induced earthquakes is ~3 M_L, equivalent to a seismic intensity at surface of 4-5 on the EMS-98 scale. This is unlikely to cause structural damage but would be strongly felt near the epicentre. The report recommends several steps to mitigate the risk of future earthquakes including a halting of fracturing operations if a seismic event is detected above a threshold of 0.5 M_L.

A Royal Society/Royal Academy of Engineering (RS/RAE) report[25] for the UK Government's Chief Scientific Advisor confirmed that the major risks of unacceptable seismicity and groundwater contamination due to hydraulic fracturing for shale gas could be effectively managed. Seismic risk could be mitigated by:

- Local stress and geological fault surveys in the target shale formations.
- Seismic monitoring before, during and after hydraulic fracturing.
- A traffic light monitoring approach [Green – injection proceeds as planned, Amber – injection proceeds with caution possibly at reduced rates and monitoring is intensified, Red – injection is suspended]. The traffic light thresholds may be magnitude based or ground motion based.

In December 2013 DECC announced that it was satisfied that appropriate controls were available to mitigate the risk of undesirable seismic activity due to shale gas exploitation. New fracking proposals would be considered subject to strict operational controls including a 'traffic light' seismic monitoring system adopting a cautious Red 'STOP' threshold of magnitude 0.5 M_L.

The studies of induced seismic activity in Lancashire have identified that the probable cause of the two principal earthquakes was fluid migration into a north-south running geological fault which failed due to a reduction in effective stress from the high fluid pressure. North-south trending faults are known to be conducive to strike-slip failure in the current tectonic stress regime[19].

The RS/RAE report[25] notes that in relation to the anticipated maximum potential induced earthquake magnitude of ~3 M_L due to fluid injection, this is similar in terms of surface vibration to the passing of a truck.

Guidelines on best practice for UK onshore shale gas wells have been produced by UKOOG[27] with input from DECC, HSE[‡‡‡] and EA/SEPA[§§§].

It is concluded from the technical opinion on hydraulic fracturing of UK shale formations and on the proposed future controls that any related induced seismicity can be neglected as a source of unacceptable disturbance to major accident hazard pipelines and their installations.

4.4 Other

Styles and Baptie[28] note that induced seismicity has been a common occurrence in UK coalfields, nearly always associated with post-mining hydrogeological recovery and mine flooding. They suggest a maximum reported magnitude of 3.2 from the Midlothian coalfield.

Kusznir et al.[29] present observational evidence of induced seismic activity associated with active longwall coal mining in the North Staffordshire and South Wales coalfields. The maximum magnitude of induced seismicity was 3.5 M_L in North Staffordshire and 1 M_L in South Wales. The difference in maximum magnitude in the two coalfields was attributed to the occurrence of pillar failure mechanisms in previously worked seams in the North Staffordshire coalfield.

Bromley & Mongillo[30] discuss induced seismicity due to hydraulic fracture stimulation of geothermal reservoirs. They report that the maximum magnitude seismic event recorded in the UK was 1.9 M_L at the Rosemanowes site near Penryn, Cornwall.

UKOOG report[31] that hydraulic fracturing has been adopted in the UK to enhance hydrocarbon recovery from conventional onshore wells. It is suggested that ~200 wells have been hydraulically stimulated since the mid to late sixties. These activities do not appear to have produced any concerns over induced seismicity.

‡‡‡ Health and Safety Executive

§§§ Environment Agency/Scottish Environmental Protection Agency

5. UK Seismic Design Requirements

5.1 Structures within the Scope of Eurocode 8

Eurocode 8[32,33,34,35,36,37] provides a unified approach to the seismic structural design of a broad spectrum of structural types involving a wide range of construction materials. The code provides a treatment of seismic loading and resistance, structural modelling, performance criteria and geotechnical aspects. Some aspects of Eurocode 8 are subject to National variation through the mechanism of national annexes. The UK National Annexes[38,39,40,41,42] and BSI published document PD 6698[15] were published in 2008 and 2009 following draft publications in October 2007.

The UK national requirements are based on two principal study reports[13,14].

The National Forewords to BS EN 1998-1[32] and BS EN 1998-5[36] state the following:

> "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 EN 1998 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions."

PD 6698:2009 indicates that high pressure gas pipelines represent a category of structure where failure poses a large threat of death or injury to the population. The function of these pipelines is such that failure due to very low probability events, including earthquakes, might need to be considered.

The UK National Annex to BS EN 1998-4:2006[35] indicates that the UK decision on additional requirements for facilities associated with large risk to the population or the environment is that a site specific hazard analysis should be performed. This may have implications for some sites designated as top-tier under the COMAH[****] regulations. The annex also indicates that the UK decision on a reference return period of the seismic action for the ultimate limit state of a tank, silo or pipeline structure is 2500 years.

Special structures, such as nuclear power plants, offshore structures, large dams and long span suspension bridges, are excluded from the scope of Eurocode 8. This is due to particular aspects of regulatory and detailed design performance for these categories of structure.

**** Control of Major Accident Hazards

5.2 Reservoir Dam Structures

Large raised reservoirs in the UK with a water holding capacity exceeding 25,000 m^3 are subject to the statutory requirements of the Reservoir Act 1975. Reservoir safety is regulated by a requirement for:

- Periodic inspections.
- Appointment of a supervising engineer.
- The keeping of a register.

Enforcement of reservoir safety requirements is assigned to the EA for England, Natural Resources Wales for Wales, local authorities for Scotland (with a phased transition to SEPA from April 2015). Northern Ireland does not have specific legislation covering reservoir safety (a Reservoir Bill is currently at committee stage in the Northern Ireland Assembly).

Guidance on seismic risk to UK reservoir dams is provided by a Building Research Establishment report published in 1991[43]. The guidance uses a qualitative total classification factor based on the summation of weighting points assigned to four individual factors; capacity, height, evacuation requirements & potential downstream damage. The total classification factor is used to assign a dam to one of four possible dam categories. The dam category dictates the return period to be used for peak ground acceleration (PGA) selection for the safety evaluation earthquake. The return periods and PGA levels are:

Dam Category	Return Period (years)	Horizontal PGA (g)[††††] [varies with assigned seismic zone]
IV	30,000	0.25-0.375
III	10,000	0.15-0.25
II	3000	0.10-0.15
I	1000	0.05—0.10

ICOLD recommend[44] the design of large raised reservoirs to be based on a Safety Evaluation Earthquake which can be either the Maximum Credible Earthquake for the region (with ground motions based on mean + one standard deviation) or the Maximum Design Earthquake determined for a return period of 10,000 years.

Guidance is provided on the nature and extent of earthquake safety evaluation according to the dam category. A pseudo-static slope stability analysis using a seismic coefficient of 2/3 PGA may be appropriate for category I-III dams but a full dynamic analysis may be needed for category IV structures.

†††† Based on two papers in Earthquake Engineering in Britain. Thomas Telford, London, 1985: Irving, J. *Earthquake hazard in Britain*. & Long, R.E. *A ground motion probability analysis for Britain based on macroseismic earthquake data.*

It is considered important to identify those dams with features that would contribute to a high susceptibility of damage in an earthquake. This may be due to condition, abnormal behaviour or foundation/embankment material type.

The recommended level of safety evaluation for embankment dams is as follows:

Dam Category	Dam Height (m)	
	< 15	>15
IV	Vulnerability checks & full dynamic analysis	
III	Vulnerability checks	Vulnerability checks & pseudo-static analysis
II	Vulnerability checks	
I	None	Vulnerability checks

The recommended level of safety evaluation for concrete and masonry dams is as follows:

Dam Category	Dam Height (m)	
	< 15	>15
IV	2-D dynamic analysis	
III	Pseudo-static analysis	2-D dynamic analysis
II	Pseudo-static analysis	
I	None	Pseudo-static analysis

Some consideration should be given to adequate visual surveillance and instrumental monitoring proportionate to the dam category and the quality of design/construction knowledge.

Criteria according to earthquake magnitude and epicentral distance are provided to capture dams that require a thorough post-earthquake examination.

5.3 Nuclear Sites

The Office for Nuclear Safety (ONR), an agency of HSE, regulates nuclear safety and security at 37 licensed sites in the UK[45].

There are currently nine operating nuclear power stations: one Magnox station operating since 1971, seven advanced gas-cooled reactor stations variously operating from 1976-1988, and one pressurised water reactor station operated since 1995.

The safety of nuclear facilities to seismic hazard is ensured through a conservative and robust civil and mechanical design with strong independent regulatory oversight by ONR.

ONR requires nuclear site licensees to create and implement adequate arrangements for compliance with conditions[46] attached to each nuclear site license. License conditions 14: *Safety documentation* and 16: *Site plans, designs and specifications*, are likely to embrace seismic design considerations. ONR safety assessment principles[47] for nuclear facilities indicate a need to derive a Design Basis Earthquake (DBE) and an Operating Basis Earthquake (OBE). Buildings, structures and plant must be designed to withstand ground motions due to the DBE event. There should be no impairment of function of any structure, system or component under repeated ground motions due to the OBE event.

Seismic hazard is a factor in regulatory consent for siting of a nuclear facility[47].

Individual and societal off-site risk targets for nuclear facilities are:

Risk Level	Individual	Societal (100 or more fatalities)
Basic Safety Level (BSL)‡‡‡‡	$1x10^{-4}$ pa	$1x10^{-5}$ pa
Basic Safety Objective (BSO)§§§§	$1x10^{-6}$ pa	$1x10^{-7}$ pa

Newell & Roberts[48] indicate that the majority of new nuclear installations are designed to elastic stress limits for a 'Safe Shutdown Earthquake' (SSE), based on a probability of exceedance of 10^{-4} per annum. The term SSE has now been replaced by the equivalent term DBE. Reserve structural strength is required beyond the SSE/DBE and this is typically set at a margin of at least 40% on earthquake ground motion.

5.4 LNG Facilities

Onshore LNG facility safety is regulated and enforced through the COMAH regulations[49] by a competent authority.

LNG storage and import facilities designed to BS EN 1473:2007[50] must be capable of maintaining overall integrity and containment under a 'Safe Shutdown Earthquake' with a mean recurrence interval of 5000 years.

‡‡‡‡ This is the benchmark limit to 'tolerable risk level'. It is the highest to be acceptable but must be ALARP so could be lower.

§§§§ This is the benchmark 'broadly acceptable level'. No further improvement required by ONR but must be ALARP so could be lower.

5.5 Chemical Manufacturing Sites

The Chemical Industries Association (CIA) recommends[51] that occupied buildings on chemical manufacturing sites should be designed so that they will protect their occupants against the hazards which may be expected to occur with a maximum return period of 10,000 years.

The CIA guidance makes no mention of seismic hazard. The emphasis is on the threat to building occupants from on-site fire, explosion or toxic gas.

6. Pipeline Code Requirements (Steel)

PD 8010-1[52] identifies earthquakes as a potential environmental loading hazard and lists several aspects that should be taken into account.

IGEM TD/1 edition 5[16] indicates a requirement for consideration of seismic loading on pipelines and installations in accordance with BS EN 1998-4[35] and the relevant national annex.

IGEM TD/1 edition 5[16] notes that the UK national seismic hazard map for a return period of 2500 years (available within PD 6698) may be used for initial screening to determine if an explicit seismic design is warranted.

An explicit seismic design should include a geological and seismological investigation to identify pre-existing faults and their potential for movement. Static and dynamic effects of ground shaking should be considered for installations. Installations and pipelines should also be assessed for acceptable stress and strain conditions due to potential earthquake induced permanent ground movements.

IGE/TD/12[53] identifies earthquakes as an occasional load which may be combined with normal sustained loading to provide an abnormal load case as part of structural analysis for pipework design. The permitted Von Mises equivalent stress level for an abnormal load case is approximately 12% higher than the permitted stress level for normal sustained loading.

BS EN 1594[54] indicates that a consideration of seismic hazard may be justified depending on the seismic activity in the area of the pipeline route. An informative annex is provided. Ground shaking is indicated to be a major design consideration for above-ground sections of pipeline due to dynamic amplification of ground motions.

BS EN 1993-4-3[55] indicates that earthquake loads should be considered where appropriate and makes reference to Eurocode 8.

BS EN 14161[56] classifies earthquakes as an environmental load and indicates the effects to be considered in the pipeline design.

The UK National Annex[40] to BS EN 1998-4:2006[35] provides (by referral to PD 6698) the UK values for nationally determined parameters as follows:

- The additional requirement for facilities associated with large risk to the population or environment is that a site specific seismic hazard analysis should be performed.
- In the absence of a project-specific assessment a reference return period of 2500 years should be used for the seismic action associated with the ultimate limit state.
- In the absence of a project-specific assessment a return period of 95 years should be used for the seismic action associated with the damage limit state. Alternatively, a reduction factor can be used to obtain the seismic action from the value associated with the ultimate limit state.
- When a reference return period of 2500 years is used for a CC3[*****] (high consequence) structure the importance factor[†††††] should be taken to be unity. For a project-specific assessment the importance factor should be selected on a project-specific basis.
- In the absence of a project-specific assessment the reduction factor[‡‡‡‡‡] relevant to the seismic action for the damage limitation state should follow the recommended code values.

The UK Annex[40] also indicates that the informative Annex B of BS EN 1998-4:2006[35] may be used in the UK.

7. Public Safety

The risk to public safety from onshore steel pipelines is influenced by the pipeline operating stress level and the minimum proximity to normally occupied buildings. These are adjusted according to the area population density and the hazard potential of the conveyed substance. Additional protection measures and surveillance can also contribute to reduced risk.

The standard operating stress restrictions and minimum proximity limits for onshore steel pipelines according to PD 8010-1:2004[52] are summarised in figure 4. These limits may be altered if justified through quantitative risk assessment.

[*****] Consequence class for reliability differentiation according to BS EN 1990:2002+A1:2005 – Eurocode. Basis of structural design. BSI. July 2002.
[†††††] A factor to increase the seismic action to take account of return periods other than the reference return period.
[‡‡‡‡‡] A factor (= 0.4 or 0.5) used to reduce the seismic action to take account of a shorter return period for the damage limitation state.

Guidance appropriate to high pressure gas pipelines is provided by IGEM/TD/1.

Pipelines conveying dangerous fluids are generally excluded from urban (high population density) areas although natural gas pipelines are permitted to operate at up to 7 bar and may be located at a minimum proximity distance of 3 metres from buildings. However, the operating stress level in these gas pipelines is very low.

In suburban and rural areas the operating stress level and proximity is controlled to reflect the differing population densities within the influence of the pipeline.

Pipelines conveying dangerous fluids and within relatively close proximity to normally occupied buildings or population clusters are generally operating at low stress levels. This provides considerable reserve strength to accommodate accidental or occasional external loading on the pipeline structure.

8. UK Seismic Hazard Maps [Earthquakes]

8.1 Introduction

Seismic hazard maps are a convenient way of showing the regional variation of ground motion or seismic intensity levels for a chosen annual probability of exceedance. This form of hazard presentation is appealing to the engineer for its convenience in the potential selection of design ground motion values. Application to pipelines is also appealing given their often significant geographical extent. However, Mallard, D.J. et. al.[57] indicate that site specific studies have suggested that typical hazard maps for regions of modest seismicity (such as the UK) are only indicative at low annual probability of exceedance levels (the order of 10^{-4}). Normally, the results from any site specific study would be expected to be higher than those from a typical hazard map.

Musson & Sargeant[13] indicate that the selection of design coefficients from hazard maps rather than from site specific studies is not generally considered to be best practice.

8.2 Peak Ground Acceleration

PD 6698[15] provides seismic hazard maps for reference return periods of 475 years and 2500 years based on work carried out by Musson & Sargeant[13] at the British Geological Survey. The hazard is expressed as peak horizontal ground acceleration (PGA) on rock. The PGA is defined as the geometric mean of the two horizontal components. These values are expected to be ~13% lower than the largest component values[§§§§§].

The seismic hazard map for a return period of 2500 years is presented from PD 6698 in figure 5.

An interpretation of the probabilistic seismic hazard analysis approach developed by Musson & Sargeant[13] has been implemented[******] to provide indicative UK hazard maps for PGA according to site conditions.

A simplified flow chart is presented in Appendix VI for PGA calculations.

The PGA hazard maps for 3 site conditions are presented in figure 6 for a return period of 2500 years and in figure 7 for a return period of 10,000 years.

The hazard maps indicate that the PGA at 'soft' sites is on average ~15% larger (range ~10%-~20%) than at 'rock' sites. The PGA at 'stiff' sites is on average ~10% larger (range ~5%-~15%) than at 'rock' sites.

A comparison of the seismic hazard produced by the national seismic source zoning used by Musson & Sargeant[13] (23 source zones) and the seismicity assessed at a local level from a site specific study is presented in figure 8. The site specific study adopted 9 area source zones within a 300 km x 250 km area and a single fault zone. The site specific study used the attenuation model of Principia(1982)[6] for area zones and of Principia(1985)[58] for the fault zone. The comparative analysis for the larger regional seismic source zones of Musson & Sargeant[13] uses the attenuation model of Principia(1982)[6] based on the expression given by Douglas[59,60]. The results suggest that in this case the higher level national seismic zoning model has adequately captured the seismic hazard at the site.

Musson & Sargeant[13] elected to calculate an average PGA value from the attenuation models of Campbell & Borzorgnia[61] and Bommer et. al.[62]. These were used in 2007 to produce the UK seismic hazard maps for PD 6698[15]. The site specific seismic hazard analysis cited above was also completed in 2007. This specific study elected to use the attenuation models developed by Principia[6,58] for the nuclear industry.

A comparison of the seismic hazard according to the different attenuation models is provided in figure 9. The PGA values from the Principia attenuation models have been reduced to adjust maximum component values to geometric mean values. The results indicate that given the

§§§§§ $PGA_{largest\ component}$= ~1.15 $PGA_{geometric\ mean}$ so $PGA_{geometric\ mean}$ = ~0.87 $PGA_{largest\ component}$.
****** This is a replication of the BGS Monte Carlo approach to seismic hazard mapping based on an interpretation of the method as described by Musson & Sargeant[13].

same seismicity inputs the choice of attenuation model can have a significant influence on the predicted hazard.

8.3 Peak Ground Velocity

It is shown in Appendix V that longitudinal pipe strain can be conservatively assessed (by assuming no slippage) from the peak ground velocity (PGV) and the effective seismic wave propagation velocity along the pipeline.

American Lifelines Alliance guidance[63] provides $\frac{PGV}{PGA}$ ratio values according to ground type, earthquake moment magnitude and source to site distance. However, the minimum earthquake moment magnitude is 6.5 compared to an historic maximum for UK onshore earthquakes of ~5.0[††††††]. Also, the dependency on magnitude and source to site distance is inconvenient for application to PGA values calculated by probabilistic seismic hazard analysis.

The results of a direct assessment of PGV (geometric mean) using the probabilistic method and seismicity inputs of Musson & Sargeant[13] and incorporating the attenuation models of Campbell & Bozorgnia[61] and Akkar & Bommer[64] are presented in figures 10 and 11 for return periods of 2500 years and 10,000 years, respectively.

The hazard maps indicate that the PGV at 'soft' sites is on average approximately double (average ratio ~1.9, range ~1.8-~2.1) the level at 'rock' sites. The PGV at 'stiff' sites is on average ~50% larger (average ratio ~1.5, range ~1.3-~1.5) than at 'rock' sites.

The PGV for an annual probability of exceedance of 10^{-4} is a factor of ~1.9 times larger (range ~1.6-~2.7) than the PGV for an annual probability of exceedance of 4×10^{-4}.

PGV hazard maps may be convenient for screening pipeline routes using a PGV limit criterion such as that used by National Grid for ground vibration control near high pressure gas pipelines. A criterion limit value would depend on selecting an acceptance transient stress increment and a lower bound effective propagation velocity for seismic waves.

The hazard maps indicate that at an acceptance limit of 75mm/s (as used by National Grid for other sources of ground vibration) most of the UK is below this hazard level at the 4×10^{-4} annual probability of exceedance level (return period of 2500 years).

[††††††] Moment magnitude.

8.4 Seismic Intensity

Seismic intensity is an indication of the damage level produced by seismic activity.

The results of a direct assessment of seismic intensity using the probabilistic method and seismicity inputs of Musson & Sargeant[13] and incorporating the attenuation model of Musson[65] are presented in figure 12.

An updated intensity attenuation model is available[66] to take account of more recent earthquakes and to express the attenuation in terms of moment magnitude.

The maps suggest the following:

- At an annual probability of exceedance of $2x10^{-3}$ (a 1 in 10 chance in 50 years) seismic disturbance is limited to strong shaking (but negligible damage) at intensity level V over the north, east and south of the UK mainland and to widespread slight non-structural damage to some buildings at intensity level VI in Wales and in an area extending up the west side of England to Cumbria. A small area in the west of Scotland is also at risk of non-structural damage at this probability level.
- At an annual probability of exceedance of $4x10^{-4}$ (a 1 in 50 chance in 50 years) seismic disturbance is limited to slight non-structural damage to many buildings at intensity level VI over the east and south of England and most of Scotland. Two principal zones in England and Wales (North-West Wales and a continuous zone from South Wales through the West Midlands and up into Cumbria) and two zones in Scotland (west coast & around Comrie) at intensity level VII are at risk of moderate damage to well-built ordinary buildings and considerable damage to some older or poorly constructed buildings. Negligible damage is expected to buildings of good design and construction.
- At an annual probability of exceedance of $1x10^{-4}$ (a 1 in 200 chance in 50 years) seismic disturbance is associated with extended areas of Scotland, England and Wales at intensity level VII subject to moderate damage to ordinary buildings and considerable damage to less robust buildings. Areas of North-West Wales, South Wales, the Pennines, and Scotland at intensity level VIII may experience significant damage to some ordinary buildings and some weak older structures may collapse. Damage to engineered structures in these areas should be slight.

At seismic intensity level VIII there may be fatalities. In 1989 a magnitude 5.6 M_L earthquake at a focal depth of ~11.5 km occurred at

Newcastle in Australia producing a maximum MM intensity of VIII. There were 13 deaths, at least 150 injuries and tens of thousands of buildings were damaged[67].

9. Pipeline Performance in Past Earthquakes

An indicative potential damage assessment according to isoseismal intensity levels is presented in Table 1. This is based on an evaluation of steel pipeline performance in historic earthquakes, see Appendix VII. The results indicate that the threshold for damage to welded steel pipelines is a seismic intensity of VI on the EMS-98 or MM scales, equivalent to a ground acceleration exceeding ~0.09 g. At this level of disturbance the evidence suggests that poor performance is associated with fillet welded socket and spigot connections, welded double bell and internal chill ring joints or butt welded joints with poor root penetration or lack of good fusion.

The damage threshold for modern pipelines with full penetration butt welds is a seismic intensity level of IX on the EMS-98 or MM scales, equivalent to a ground acceleration exceeding ~0.65 g.

The seismic intensity attenuation model of Musson[66] suggests that the probability of a maximum credible earthquake producing intensity level IX[‡‡‡‡‡‡] is ~15% (1 in 6 chance) at the epicentre, ~5% (1 in 20 chance) at 10 km from the epicentre, ~1% (1 in 100 chance) at 20 km from the epicentre and negligible beyond 40 km from the epicentre.

10. Above-Ground Installations

Above-ground pipeline installations may be subject to dynamic amplification of the seismic ground motions due to the natural frequency of vibration of the piping system relative to the frequency of the ground motions. The seismic ground motions are applied at the pipe supports or base of supporting structures. Actual motions of the above-ground piping may be larger depending on the natural vibration frequency of the system.

The amplification of acceleration for a single degree of freedom (SDF) oscillator (mass, spring & dashpot) subject to a harmonic (sinusoidal) input motion is presented in figure 13.

When the natural frequency of the oscillator is very small relative to the forcing frequency, the amplification factor approaches zero. The system mass has insufficient time to respond to the high frequency excitation.

‡‡‡‡‡‡ This is based on the earthquake depth and maximum magnitude values and weightings used by BGS in the preparation of the seismic hazard maps for PD 6698.

This is known as the flexible range. In this range the system forces due to mass acceleration approach zero. The system mass movement approaches zero and the relative movement of the system approaches that of the input movement.

As the oscillator natural frequency approaches the forcing frequency, the amplification factor becomes large (and is infinite for an undamped system). This is known as the resonant range.

When the oscillator natural frequency is large relative to the forcing frequency, the amplification factor approaches unity and the acceleration of the mass becomes the same as the input motion. This is known as the rigid range. In this range the loadings on the piping system are due only to the acceleration of the masses and there is no relative movement between the piping and the supports.

The dynamic amplification factor (for horizontal acceleration) as implied by BS EN 1998-1:2004[32] for Type 2 earthquakes (as recommended by the UK National Annex[38]) is presented in figure 14 according to system natural vibration frequency and ground type to BS EN 1998-1:2004[32]. PD 6698[15] indicates that structural response is particularly uncertain for sites underlain by soft soils and that more reliable site specific dynamic responses should be established for certain structures and facilities.

The structural performance of above-ground piping under seismic loading can be analysed using one of three methods. In order of improving quality of simulation but increasing complexity:

- Lateral force method or sometimes referred to as quasi-static analysis. This involves a static elastic analysis using applied forces calculated according to the mass of the piping and an assigned level of acceleration. The acceleration should include dynamic amplification of the local peak ground acceleration. The static analysis may involve both vertical and horizontal seismic loads.
- Modal analysis. The natural frequencies and vibrational mode shapes of the piping system are determined using a lumped mass model. An equivalent static force vector for the piping system is determined for each mode of vibration using an acceleration value from the elastic response spectrum for the appropriate natural frequency of vibration. The results (structural response) of the individual modal loads are calculated and summed to provide the total system response.
- Time history analysis. This is a time-stepping technique involving the application of a series of individual time dependent force pulses to the piping structure to represent the earthquake motion. The equations of motion are solved in each time step to obtain incremental displacements, velocities and accelerations. The stress

state is obtained from the cumulative structural displacements at the end of each time step. This method of analysis uses a step-by-step direct integration approach to obtain the response-history of the structure to the earthquake input motions.

It is important for the integrity of above-ground piping that ground failure at supports is avoided. This requires attention to foundation design to accommodate both static and dynamic bearing loads and to foundation level and type on potentially unstable soils (under seismic loading).

11. National Grid Policy Development for Gas Pipelines

National Grid has commissioned Jacobs to assist in the development of a seismic policy and associated specifications and procedures for the UK high pressure gas transmission network.

The assets within the scope of the seismic policy are:

- Above-ground installations and buried steel pipelines and piping.
- Building structures.
- Mechanical plant.
- Electrical equipment.

Some of the key points, mainly in relation to buried pipelines and above-ground installations, from a report[68] issued in January 2014 are noted in this section.

In relation to the UK seismic hazard as expressed in PD 6698 it is recommended that the PGA values for a return period of 2500 years are increased by a factor of 1.5. This converts the mapped geometric mean values to maximum component values (~ 1.15) and additionally takes account of a tendency for site specific studies to determine higher hazard levels than generic studies (~ 1.3). The PGA values for a return period of 10,000 years should be obtained by multiplying the 2500 year values by a factor of 2.0 (referred to an importance factor).

Jacobs determine threshold criteria for pipelines to identify a need for seismic design. At the appropriate return period the seismic design requirement threshold is a PGV of 150 mm/s. In lieu of a PGV estimate a conservative threshold for PGA of 0.10 g can be adopted.

An importance class is assigned to systems according to safety consequence and potential network outage:

Importance Class	Area Type [Safety]			
Outage	*Remote*#	Rural [≤10 potential casualties]	Suburban [≤100 potential casualties]	Town [≤1000 potential casualties]
Short [~1 day]	I or II*	II	III	IV
Medium [~1 wk]	II or III*	II or III*	III	IV
Long [>2 wks]	III or IV*	III or IV*	III or IV*	IV

no risk of casualties, * applies to Critical National Infrastructure

An importance factor is associated with the importance class as follows:

Importance Class	I	II	III	IV
Importance Factor γ_I	n/a	0.45	1.0	2.0

A site specific seismic risk assessment should be carried out where the number of potential casualties is greater than 1000.

An initial seismic screening procedure is presented based on the importance class and design ground acceleration.

The design ground acceleration on rock, a_g, is obtained according to the following expression,

$$a_g = \gamma_f . \gamma_I . a_{gR}$$

where

γ_f is a load factor (= 1.5)
γ_I is the importance factor
a_{gR} is the reference ground acceleration on rock for a return period of 2500 years according to PD 6698.

The ground acceleration on soil is $a_g.S$ where S is the soil amplification factor according to BS EN 1998-1:2004.

The seismic design requirements are determined according to the importance class and the value $a_g.S$ as follows:

<table>
<tr><td rowspan="2">Importance Class</td><td colspan="2">$a_g.S \leq 0.1\ g$</td><td colspan="2">$a_g.S > 0.1\ g$</td></tr>
<tr><td>Buried Pipelines</td><td>Above-Ground Installations</td><td>Buried Pipelines</td><td>Above-Ground Installations</td></tr>
<tr><td>I</td><td colspan="4">None</td></tr>
<tr><td>II</td><td colspan="3">None</td><td>Simplified</td></tr>
<tr><td>III & IV</td><td>None</td><td>Simplified</td><td colspan="2">Full</td></tr>
</table>

The following strain acceptance limits are recommended for the structural performance of buried pipelines:

Strain	Condition	Damage Limit State	Ultimate Limit State
Compressive	Load controlled	Min [1%, 0.4t/D]	Min [1%, 0.4t/D]
	Displacement controlled	Min [1%, 0.4t/D]	Min [4%, 1.76t/D]
Tensile	Load controlled	0.5%	0.5%
	Displacement controlled	1.5%	3.0%

The recommended acceptance limit for the structural integrity of above-ground piping is a Von Mises equivalent stress equal to the specified minimum yield strength of the pipe material.

12. Discussion

12.1 PD 6698 hazard maps

Two studies formed the basis of the technical content of PD 6698. The work of Musson & Sargeant[13] provided the hazard maps and the work of Booth et. al.[14] explained the requirements for seismic design in the UK.

The key decisions influencing the construction of the model for the hazard maps were based on the consensus view of leading experts established at a meeting held at the Institution of Civil Engineers on 26 April 2007.

The threshold earthquake magnitude of engineering significance was chosen to be a moment magnitude of 4.5. There have been ~27 earthquakes of this magnitude or larger across the UK over a ~300 year period.

The weaknesses in the hazard maps of PD 6698 to seismic design of specific facilities are known to include:

- The potential influence on hazard of local faults which could be assumed to localise seismic activity closer to the site.[§§§§§§]
- The potential limitations of a Gutenberg-Richter magnitude-frequency relationship.[*******]
- The use of the conventional geometric mean of the horizontal ground motion rather than the maximum component.[†††††††]
- The standardisation of focal depth and maximum magnitude weightings over all seismic source zones.
- The influence of assuming only strike-slip fault focal mechanisms.
- The potential influence of local soil conditions on ground motion amplification.[‡‡‡‡‡‡‡]
- The potential influence of ground motion amplification due to significant topographic features.[§§§§§§§]
- The potential need for a return period of in excess of 2500 years for some facilities.

The potential limitations of the PD 6698 hazard maps has led experts in seismic hazard analysis to prefer site specific studies for structures that require an explicit seismic design.

The use of a soil factor according to Eurocode 8 Part 1 for local site amplification of the reference seismic action on rock is anticipated to provide some conservatism relative to direct assessment using the method underlying the hazard maps of PD 6698. Indicative calculations suggest the amplification factor S compares as follows:

Ground Type	S [Eurocode 8 Type 2]	S [Generic Mapping]
B $360 < V_{s30} < 800$	1.35	~1.10 [max 1.16]
C $180 < V_{s30} < 360$	1.50	~1.15 [max 1.23]

Eurocode 8 offers the benefit of providing soil amplification factors for weaker soils that are outside the range of application of the attenuation models used for the generic hazard maps.

[§§§§§§] There may be considerable difficulty in identifying geological faults and their potential for activity at any specific site in the UK.

[*******] This is associated with the earthquake catalogue showing more or less earthquakes of a particular magnitude than suggested by the G-R relationship. However, this anomaly could be attributable to the short length of the catalogue.

[†††††††] Using the maximum component may overstate the damage potential associated with multiple cycles. This is particularly true of slope instability and liquefaction.

[‡‡‡‡‡‡‡] This can be handled with code amplification factors.

[§§§§§§§] This is a requirement of Eurocode 8 for importance class III & IV structures. Amplification is only applicable to cliff/slope heights exceeding 30 metres and to slope angles exceeding 15°.

12.2 Return Period for Seismic Action

Eurocode 8 Part 4 for the earthquake resistance design of silos, tanks and pipelines uses an importance factor for reliability differentiation according to an importance class dictated by the potential consequences of structural failure.

The importance classes and factors are as follows:

Importance Class	Potential Consequences of Structural Failure	Importance Factor γ_I
I	Risk to life is low and the economic and social consequences of failure are small or negligible.	0.8
II	Medium risk to life and local economic or social consequences of failure.	1.0
III	High risk to life and large economic and social consequences of failure.	1.2
IV	Exceptional risk to life and extreme economic and social consequences of failure.	1.6

The importance factor is multiplied by the seismic action for the code reference return period of 475 years and this has the effect of decreasing or increasing the return period according to the importance class of the silo, tank or pipeline.

Major accident hazard pipelines and installations are subjectively anticipated to correspond to importance class III or IV structures.

PD 6698 recommends a return period of 2500 years for the selection of a design PGA for high consequence class CC3[********] structures.

The adoption of a return period of 2500 years is equivalent to an importance factor of at least 1.9 and on average ~2.5 according to the approach underlying the hazard maps presented in PD 6698. This implies additional caution (beyond the Eurocode 8 normal requirements) for UK seismic design of high consequence structures.

The indicative return periods in general usage for designing against gross structural collapse of a high consequence facility or structure are as follows:

[********] As defined in Table B1 of BS EN 1990:2002. Importance classes I, II and III/IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively.

Facility/Structure	Return Period (years)	Source
CC3 buildings e.g. schools, assembly halls, cultural institutions etc.	~800	BS EN 1998-1:2004 [Importance class III - $\gamma_I = 1.2$]
CC3 silos, tanks & pipelines. [high risk to life]	~800	BS EN 1998-4:2006 [Importance class III - $\gamma_I = 1.2$]
CC3 buildings e.g. hospitals, fire stations, power plants etc.	~1300	BS EN 1998-1:2004 [Importance class IV - $\gamma_I = 1.4$]
CC3 silos, tanks & pipelines. [exceptional risk to life]	~2000	BS EN 1998-4:2006 [Importance class IV - $\gamma_I = 1.6$]
CC3 structures.	2500	PD 6698
LNG facilities.	5000	BS EN 1473:2007
Category III dams.	10,000	Building Research Establishment
Buildings on chemical manufacturing sites.	10,000	Chemical Industries Association[††††††††]
Nuclear power stations.	10,000	ONR Safety Assessment Principles for Nuclear Facilities 2014.
Category IV dams.	30,000	Building Research Establishment

Major accident hazard pipelines and installations can be classified as CC3 structures to BS EN 1990:2002. This is the highest risk category assigned to structures with '**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**'.

The selection of a return period of 2500 years for most UK CC3 structures appears to be slightly more cautionary than the estimated standard values in Eurocode 8 for corresponding structures. However, a return period of 2500 years is not as cautionary as the recommended value for UK onshore LNG facilities (an example of major hazard sites regulated under the COMAH regulations). However, PD 6698 recommends site specific seismic hazard analysis for structures and facilities where failure would have very significant regional or national consequences. This analysis may involve the selection of an appropriate return period for the seismic action.

The flexibility offered by PD 6698 in the selection of a return period for the seismic hazard will necessitate a policy decision by UKOPA on the appropriate value for major accident hazard pipelines. UKOPA may

[††††††††] No specific mention or exclusion of seismic hazard.

consider that installations with on-site personnel or with normally occupied buildings within the hazard zone should be assigned a higher return period than the value chosen for buried pipelines. This would recognise the difference between a fixed point hazard from an installation and the variable hazard along a pipeline.

12.3 PD 6698 Hazard Map & Code Soil Amplification

The combination of the PD 6698 hazard map for a return period of 2500 years and the soil amplification factors to Eurocode 8 should provide a suitable ground motion seismic action for the evaluation of any seismic resistance needs for major accident hazard pipelines and associated installations in the UK. This evaluation would take the form of a screening approach to identify any need for specialist or site specific assessment.

12.4 Screening Criteria

There is compelling evidence from past earthquakes that modern welded steel pipelines with full penetration butt welds exhibit a very low vulnerability to seismic hazards. The onset of damage is at a seismic intensity level of IX on the MM or EMS-98 scales. Selecting a screening level of not exceeding VII for the chosen return period would offer a cautious approach to identifying those areas of the UK that can be excluded from any seismic hazard considerations. For a return period of 2500 years there are several areas that would require a seismic assessment for onshore pipelines; two principal zones in England and Wales (North-West Wales and a continuous zone from South Wales through the West Midlands and up into Cumbria) and two zones in Scotland (west coast & around Comrie). Extending the return period to 10,000 years would identify much of the mainland for more detailed seismic assessment. However, most of southern/south-eastern England and areas of southern/eastern Scotland would be excluded from seismic assessment. The intensity maps are shown in figure 15.

Alternatively, a screening threshold based on peak ground acceleration might be considered. This would enable local site amplification to be included which could be an important factor for above-ground installations.

ASCE guidance[69] on the seismic resilience of natural gas systems appears to identify a threshold PGA of 0.20 g as a criterion for judging if risk assessment of a seismic threat should be included within an integrity management plan. The criterion is sourced from Appendix A of ASME B31.8S[70] for managing the integrity of gas pipelines and associated installations. The basis of the B31.8S code is embodied within federal regulations applicable to gas transmission pipelines operating at a design factor of 0.2 or greater. However, B31.8S also identifies a need to identify geological fault crossings, unstable slopes and liquefaction susceptible soils.

ASCE guidance[71] for the seismic design of natural gas distribution systems identifies a criterion of 0.15 g for the 1 second spectral response acceleration for a return period of 2500 years to determine if seismic issues should be considered.

Hovius & Meunier[72] determined a horizontal acceleration threshold of 0.20 g for earthquake induced landsliding. However, Wang et. al.[73] identified a threshold PGA of ~0.07 g and a threshold PGV of ~500 mm/s for landsliding induced by the Wenchuan earthquake (M_s ~8.0). The PGV threshold was considered a more relevant indicator than PGA.

Indicative threshold PGA values for the liquefaction of clean sands and high fines (35%) sands is presented in figure 16 according to the work of Idriss & Boulanger[74]. The PGA is related to the corrected[‡‡‡‡‡‡‡‡] standard penetration test $(N_1)_{60}$ value over the moment magnitude range relevant to the UK. The results suggest that a PGA threshold of 0.20 g may be non-conservative for very loose or loose deposits of sand. A screening value of 0.10 g is more appropriate as suggested in Appendix IV.

Musson[75] demonstrates that disaggregation of the results of a probabilistic seismic hazard analysis can be used to reveal the earthquakes that are contributing most to the seismic hazard at a location. This has potential for screening for landslide and liquefaction potential without the need for site geotechnical data or consideration of the actual PGA level. An example for a site in Essex is presented in figure 17. The results indicate that there is the possibility of disrupted rock and soil landslides. However, this is dependent on susceptible materials and sufficiently steep natural slope angles. Criteria are provided in Appendix IV based on the work of Keefer[76]. The results indicate that the annual probability of liquefaction (assuming susceptible deposits exist) is less than $4\text{x}10^{-4}$ (return period of 2500 years).

The disaggregation approach could be used to zone the UK for landslide and liquefaction potential.

Pipeline damage due to seismic wave propagation is considered unlikely based on the seismic intensity levels for the probable choice of return period for hazard assessment. The indicative minimum factor of safety against gross section tensile and compressive yield according to operating temperature and PGV is presented in figure 18[§§§§§§§§]. This indicates that

[‡‡‡‡‡‡‡‡] The blow count from the penetration resistance is normalised to an effective overburden pressure of 100 kPa and corrected to 60% of free full energy.

[§§§§§§§§] This calculation is based on finding the lowest reserve axial stress (prior to yield) across a matrix of 10 values of D/t in the range 10-100, 4 values of internal pressure in tensile failure is unlikely based on the need for very high PGV levels. However, compressive failure may be a possibly in higher temperature pipelines for the seismic hazard associated with very long return periods. This type of failure is associated with axial buckling and may lead to a product leak.

12.5 National Grid Seismic Policy Development

The Jacobs study and policy development for National Grid represents a comprehensive review of seismic hazard in the UK in relation to gas transmission pipelines, above-ground piping, building structures, mechanical plant and electrical equipment. The strategy for buried pipelines and associated above-ground installations is to assign an importance class according to the hazard range of the pipeline/installation and the potential outage time. The importance class dictates an importance factor which contributes to the calculated level of seismic hazard. The level of seismic hazard is used to determine the extent of seismic design requirements. This approach has similarities with the categorisation scheme used by the Building Research Establishment for seismic safety evaluation of large reservoir dams.

An importance class for fixed installations may be a basis for assessing the seismic design requirements (if any) including the selection of an appropriate return period for the seismic action. In the case of a fixed installation the potential point of system structural failure is known relative to the population within the hazard zone.

An importance class may be more difficult to apply to buried pipelines because although the hazard zone may be constant, the population exposure will vary. Also, the potential point of structural failure is unknown.

For importance class IV[*********] pipelines and installations Jacobs recommend calculating the design seismic hazard on rock as the PGA value from the PD 6698 hazard map for a return period of 2500 years multiplied by 3.0. This value is then increased by the code soil amplification factor appropriate to the location or site.

The multiplication factor applied to map values for importance class III[†††††††††] pipelines and installations is 1.5.

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the range 7-85 bar & 4 values of the design factor in the range 0.3-0.8. The factor of safety is the ratio between an effective seismic wave propagation velocity of 2000 m/s and the value required to achieve the reserve axial stress. The reference propagation velocity is reported to be a conservative value according to '*Guidelines for the Design of Buried Steel Pipe*' American Lifelines Alliance. July 2001.

[*********] High consequence with the potential for up to 1000 casualties.

[†††††††††] Potential for up to 100 casualties.

The selected uplift values appear to be potentially overly cautious considering the Eurocode 8 guidance for high consequence structures and the inherent caution in the code soil amplification factors.

The indicative areas of the UK mainland requiring a full seismic design for buried gas transmission pipelines and installations is shown in figure 19 for importance class III systems and in figure 20 for importance class IV systems.

However, the PD 6698 conservatisms do not extend to the choice of PGA value (geometric mean rather than maximum component) or to the ground motion attenuation model (older attenuation models used in the UK nuclear industry give higher acceleration levels).

13. Conclusions

Earthquakes

13.1 Earthquakes are a release of strain energy due to the fracture of rock and the transmission of some of that energy as seismic waves.

Natural earthquakes in the UK are attributed to the failure and movement of existing geological fault structures primarily due to crustal compression associated with tectonic plate movements.

Induced earthquakes in the UK have been attributed to underground mine collapses, rock blasting and hydraulic fracturing in deep boreholes. Hydraulic fracturing includes oil and gas reservoir stimulation, geothermal reservoir stimulation and shale gas extraction (fracking).

13.2 Earthquakes can be quantified in terms of a magnitude scale according to the amount of energy release. Several magnitude scales exist according to the determination method. The cumulative number of earthquakes in a region above a particular magnitude is found to decrease with magnitude according to the Gutenberg-Richter log-linear relationship. On average, the UK may expect a local magnitude 3.7 earthquake every year, a magnitude 4.7 earthquake every 10 years and a magnitude 5.6 earthquake every 100 years.

The threshold earthquake magnitude of engineering significance is a moment magnitude of ~4.5. There have been ~27 earthquakes across the UK of this magnitude or greater in a ~300 year period.

13.3 The anticipated maximum magnitudes for natural and induced earthquakes in the UK are:

- Natural 5.5-6.5
- Mining 3.2-3.5
- Geothermal 1.9
- Fracking ~3.0

13.4 Earthquake effects at surface can be measured quantitatively in terms of ground motions or assessed qualitatively through the observed damage. The normal ground motion measurement is the variation of horizontal and vertical ground acceleration with time which can be integrated to obtain velocity and displacement ground motions. Damage is assessed and assigned to a seismic intensity scale. Several intensity scales exist although the scales in most common usage are the 12-point Modified Mercalli Scale and the 12-point European Macroseismic Scale.

13.5 Natural earthquake activity is probably due to the release of localised crustal stress build-up associated with the interaction of the current stress field with major crustal inhomogeneities and pre-existing faults. Earthquakes are positively correlated with geological faults within crystalline basement rocks particularly on strike-slip faults with a favourable orientation in the current stress field.

13.6 Natural earthquake activity is variable across the UK mainland showing higher levels of activity in the west than in the south and east of the country.

Seismic Hazard

13.7 Seismic hazard to buried pipelines and above-ground installations include:

- Transient loading due to seismic wave propagation (ground shaking).
- Permanent ground movement due to:

    - Geological fault displacement at surface.
    - Ground movement due to slope instability.
    - Ground movement due to liquefaction.

13.8 Seismic hazard assessment in the UK has been under development since the early 1970's with particular motivation from the nuclear industry. Earthquake instrumental monitoring in the UK has expanded since the early 1970's to reach a current network of over 100 stations operated by the British Geological Survey.

13.9 A seismic hazard study completed in 1991 for the Department of Environment concluded that there was no justification for the inclusion of earthquake resistance into the design of conventional structures. However, certain structures whose failure could pose a hazard to a significant number of people should incorporate earthquake loading in their design. The study identified an outstanding need for guidance on how to decide if a structure or installation was sufficiently hazardous to justify a seismic design.

13.10 Eurocode 8 (BS EN 1998 Parts 1-6) for the seismic design of structures in Europe was issued in 2005 and 2006. The national annexes for the UK decisions on nationally determined parameters for Eurocode 8 application in the UK were issued in 2008 and 2009. The UK National Annexes are supported by PD 6698 issued in 2009.

Special structures, such as nuclear power plants, offshore structures, large dams and long span suspension bridges, are excluded from the scope of Eurocode 8. This is due to particular aspects of regulatory and detailed design performance for these categories of structure.

The UK decision on seismic design is that there are no requirements for most structures but certain types of structure may warrant an explicit consideration of seismic actions. PD 6698 identifies major hazard sites and major accident hazard pipelines as categories of structure that should be designed to withstand very low probability events, including earthquakes.

PD 6698 recommends that consequence class CC3 (defined in BS EN 1990:2002 as 'High consequence for loss of human life, *or* economic, social or environmental consequences very great') pipelines (and installations) are assessed on a project specific basis to determine if an explicit consideration of seismic actions is required. The structural vulnerability, location and consequence of failure are factors to be considered in deciding on any need for seismic design.

Facilities assessed as posing a large risk to the population or to the environment require a site specific hazard assessment to establish appropriate design ground motions. Other high consequence facilities may use the peak ground acceleration according to a seismic hazard map in PD 6698 for a return period of 2500 years (annual probability of exceedance of $4x10^{-4}$).

13.11 A particular challenge for major accident hazard pipeline operators is to interpret which pipelines and installations should be considered for seismic design and which seismic hazard assessment approach is appropriate (site specific or hazard map).

13.12 Return period selection for the determination of a seismic action for the design of structures to prevent local or global structural collapse varies as follows:

Facility/Structure	Return Period (years)
Eurocode 8: CC3 buildings e.g. schools, assembly halls, cultural institutions etc. & CC3 silos, tanks & pipelines. [high risk to life]	~800
Eurocode 8: CC3 buildings e.g. hospitals, fire stations, power plants etc.	~1300
Eurocode 8: CC3 silos, tanks & pipelines. [exceptional risk to life]	~2000
PD 6698: CC3 structures.	2500
BS EN 1473: LNG facilities.	5000
Category III dams, Buildings on chemical manufacturing sites, Nuclear power stations.	10,000
Category IV dams[‡‡‡‡‡‡‡‡‡].	30,000

This summary appears to indicate that a suitable return period for major accident hazard pipelines is ~2500 years and for major hazard sites is 5000 or 10,000 years. The differing return periods for pipelines and installations reflects the difference in the potential failure point and consequence (variable for pipelines but fixed for installations).

<u>Soil Amplification</u>

13.13 PD 6698 provides seismic hazard maps for peak ground acceleration (PGA) on rock for return periods of 475 years and 2500 years. The PGA values for local site conditions can be determined using soil amplification factors according to Eurocode 8.

<u>Pipeline Vulnerability</u>

13.14 A review of the performance in past earthquakes of modern welded steel pipelines with full penetration butt welds indicates a low vulnerability to damage. A seismic intensity level of ~IX appears to be the threshold level for damage. This intensity level is associated with the collapse of weak structures, very heavy damage to ordinary buildings and damage to some specially designed structures.

A seismic hazard analysis suggests that the annual probability of seismic intensity level IX on the UK mainland is less than 1x10^{-4} per annum

[‡‡‡‡‡‡‡‡‡] Category IV dams are structures with a downstream population at risk of exceeding 1000. The International Commission for Large Dams recommends a return period of 10,000 years or a safety evaluation based on the maximum credible earthquake for the region.

(equivalent to a return period of greater than 10,000 years). A maximum credible earthquake for the UK would need to occur within ~40 km of a site for any credible prospect of achieving intensity level IX. The chance of intensity level IX at the epicentre is estimated at 1 in 6.

13.15 Pipeline stress state calculations for the effect of propagating seismic waves suggest that tensile failure is highly improbable in the UK. Compressive failure in the form of longitudinal buckling is also unlikely unless pipelines are operating at elevated temperatures.

13.16 Pipelines may be vulnerable to damage due to slope instability or to ground movement associated with liquefaction. However, this would require a pipeline to be routed across unstable or metastable slopes or to traverse very loose or loose susceptible granular soils.

Criteria are available to assess landslip and liquefaction potential in susceptible deposits.

<u>Above-Ground Installation Vulnerability</u>

13.17 Above-ground installations are susceptible to dynamic amplification of earthquake forcing vibrations at ground supports or via supporting structures. This depends on the natural period of vibration of the above-ground piping relative to the forcing frequency of the input motions.

Dynamic amplification can be quantified by a response spectrum which varies according to the local site conditions and the frequency content of the input motions. Design spectra are provided in Eurocode 8.

<u>Screening Criteria</u>

13.18 The ASME B31.8S code 'Managing System Integrity for Gas Pipelines' indicates that pipelines may be susceptible to extreme loading at locations where the pipeline crosses a fault line, the soil is subject to liquefaction or the ground acceleration exceeds 0.20 g.

13.19 ASCE guidance for the seismic design of natural gas distribution systems identifies a criterion threshold of 0.15 g for the 1 second spectral response acceleration for a return period of 2500 years. Seismic design or mitigation should be considered when the seismic hazard exceeds the threshold level.

13.20 A seismic intensity threshold of VII for a return period of 2500 years or a threshold of VIII for a return period of 10,000 years would limit seismic design for major accident hazard pipelines and installations to the more seismically active areas of the UK. This includes; north-west Wales, South Wales through Herefordshire to the Midlands and north along the Pennines to Cumbria, the west coast of Scotland and the area around Comrie.

Building Research Establishment Guide

13.21 An engineering guide to the seismic risk to dams in the UK may provide a suitable model framework for a similar guide for major accident hazard pipelines and installations.

National Grid Policy Direction [January 2014]

13.22 Consultants working for National Grid have developed a comprehensive strategy for seismic design considerations associated with gas transmission pipelines, above-ground installations, buildings, mechanical plant and electrical equipment.

Pipelines and installations are assigned to an importance class according to safety (potential casualties within the hazard zone) and security of supply (outage time). Safety is considered in terms of the number of casualties within the hazard zone: none, up to 10, up to 100, up to 1000 & over 1000. Security of supply is considered in terms of repair time: ~1 day, ~1 week & more than 2 weeks.

The potential for casualties of over 1000 dictates a site specific seismic assessment.

Each importance class is assigned an importance factor. The importance factor is used to determine the appropriate return period for the seismic action.

The importance factors are as follows:

Importance Class	I	II	III	IV
Importance Factor γ_I	n/a	0.45	1.0	2.0

The approach anticipates the usage of the PD 6698 seismic hazard map for a return period of 2500 years. However, the PGA inferred from the map is increased by a factor of 1.5 to convert from geometric mean value to maximum component value and to introduce an allowance for the possibly that a site specific study might determine a higher hazard level.

The design ground acceleration on soil is determined as,

$$a_g.S = \gamma_f.\gamma_I.a_{gR}.S$$

where

a_g is the design ground acceleration on rock.
γ_f is a load factor (= 1.5).
γ_I is the importance factor.
a_{gR} is the reference ground acceleration on rock for a return period of 2500 years according to PD 6698.
S is the soil amplification factor according to Eurocode 8.

The seismic design requirements are determined according to the importance class and the value $a_g.S$ as follows:

Importance Class	$a_g.S \leq 0.1\ g$		$a_g.S > 0.1\ g$	
	Buried Pipelines	Above-Ground Installations	Buried Pipelines	Above-Ground Installations
I	None			
II	None			Simplified
III & IV	None	Simplified	Full	

14. Recommendations

14.1 UKOPA should confirm the need for an engineering guide to the seismic risk to major accident hazard pipelines and installations in the United Kingdom (UK). The guide should take account of the model framework developed by the Building Research Establishment for the seismic risk assessment of large reservoir dams in the UK.

Key decisions include:

- Any need to differentiate major accident hazard pipelines or installations in terms of population density within the hazard zone.
- Any need to differentiate between major accident hazard pipelines and installations in the selection of return periods for the seismic action.
- The choice of actual return periods.
- The criterion to be applied in the selection of the appropriate level of seismic assessment.

14.2 As far as possible any guidance should utilise the seismic hazard map for a return period of 2500 years as published in PD 6698. Any new hazard maps should be sourced from the British Geological Survey e.g. seismic intensity if this is deemed to be the basis for regional screening.

14.3 UKOPA should engage with other pipeline operators in countries of Northern Europe with regions of similar seismicity to the UK e.g. France, Netherlands, Germany & Denmark, to determine their seismic design practices for major accident hazard pipelines and installations. This should assist in developing and benchmarking a proportionate approach to the seismic risk issue in the UK.

14.4 UKOPA should seek to clarify seismic hazard screening criterion for gas pipelines in the USA according to ASME B31.8S and ASCE TCLEE[§§§§§§§§§] monograph no. 8. This may assist in the development of seismic screening criteria for use in the UK.

15. References

1. Musson, R.M.W. *UK seismic hazard assessments for strategic facilities: a short history.* Bollettino di Geofisca Teorica ed Applicata. 2013.

2. Irving, J. *Earthquake hazard in Britain.* Earthquake Engineering in Britain. Thomas Telford. 1985.

3. Crampin, S., Fyfe, C.J., Bickmore, D.P., & Linton, R.H.W. *Atlas of seismic activity 1909 to 1968.* Seismological Bulletin No. 5. Institute of Geological Sciences. HMSO. 1976.

4. Lilwall, R.C. *Seismicity and seismic hazard in Britain.* Seismological Bulletin No. 4. Institute of Geological Sciences. HMSO. 1976.

5. Irving, J. *Seismic hazard in the United Kingdom.* Proceedings of the Institution of Mechanical Engineers Seminar on Seismic Qualification of Safety Related Nuclear Plant and Equipment. Institution of Mechanical Engineers. 1984.

6. Principia Mechanica Ltd. *British Earthquakes.* Report for CEGB. Report No. 115/82. 1982.

7. Soil Mechanics Ltd. *Re-assessment of UK seismicity data*. Report to CEGB. Report No. 7984. 1982.

8. Musson, R.M.W. *British Earthquakes*. Proceedings of the Geologists' Association. Volume 118. Issue 4. 2007.

9. Marrow, P.C. *Seismic Monitoring of the North Sea.* Offshore Technology Report OTH 90 323. Health and Safety Executive. HMSO 1992.

[§§§§§§§§§] Technical Council on Lifeline Earthquake Engineering.

10. *Earthquake hazard and risk in the UK.* Ove Arup and Partners for the Department of the Environment. HMSO. 1993.

11. Musson, R.M.W. & Winter, P.W. *Seismic Hazard Maps for the U.K.* Natural Hazards 14: 141-154. 1997.

12. Musson, R.M.W. *The use of Monte Carlo simulations for seismic hazard assessment in the U.K.* Annali Di Geofisica, Vol. 43, N. 1, February 2000.

13. Musson, R.M.W., & Sargeant, S.L. *Eurocode 8 seismic hazard zoning maps for the UK.* CR/07/125. British Geological Survey. NERC. 2007.

14. Booth, E.D., Skipp, B., & Watt, P. *Establishing the Need for Seismic Design in the UK.* Report ICE-02. Issue 2. Institution of Civil Engineers. 31 January 2008.

15. PD 6698:2009. *Recommendations for the design of structures for earthquake resistance to BS EN 1998*. BSI. July 2009.

16. IGEM/TD/1 Edition 5. *Steel pipelines and associated installations for high pressure gas transmission*. Communication 1735. Institution of Gas Engineers and Managers. 2008.

17. Gangopadhyay, A., & Talwani, P. *Symptomatic Features of Intraplate Earthquakes*. Seismological Research Letters. Eastern Section. Volume 74. Number 6. November/December 2003.

18. Shedlock, K.M. *Intraplate Earthquakes*. In AccessScience@McGraw-Hill, http://www.accessscience.com, DOI 10.1036/1097-8542.YB010921.

19. Chadwick, R.A., Pharaoh, T.C., Williamson, J.P., & Musson, R.M.W. *Seismotectonics of the UK*. BGS Technical report WA/96/3C. British Geological Survey. March 1996.

20. Esparza, E.D. *Pipeline Response to Blasting in Rock*. American Gas Association. 1991.

21. Corder, I., & Osborne, G. *The Effects of Blasting on Pipelines – Steetley Quarry Tadcaster*. ERS R.3988. British Gas Engineering Research Station. July 1988.

22. T/SP/GM/4. *Specification for The Protection of Steel Pipelines Operating at Pressures Above 7 bar Subjected to Vibrations Caused by Blasting, Piling or Demolition*. National Grid Gas. October 2005.

23. Dowding, C.H. *Construction Vibrations*. Prentice-Hall. 1996.

24. Haszeldine, S. *Shale gas, NW England earthquakes, and UK regulation.* Briefing note for DECC Science Advisory Group. 21 November 2011 with updates in February 2012 & April 2012.

25. *Shale gas extraction in the UK: a review of hydraulic fracturing*. The Royal Society and The Royal Academy of Engineering. June 2012.

26. Green, C.A., Styles, P., & Baptie, B.J. *Preese Hall Shale Gas Fracturing. Review & Recommendations for Induced Seismic Mitigation.* Report to DECC. April 2012.

27. *UK Onshore Shale Gas Wells Guidelines. Exploration and appraisal phase.* United Kingdom Onshore Operators Group. Issue 1. February 2013.

28. Styles, P.P., & Baptie, B. *Induced Seismicity in the UK and its Relevenace to Hydraulic Stimulation for Exploration of Shale Gas*. Briefing Note. 2012.

29. Kusznir, N.J., Blenkinsop, T.G., Long, R.E., & Smith, M.J. *Seismicity associated with Longwall coal mining.* Earthquake Engineering in Britain. Thomas Telford Ltd. London. 1985.

30. Bromley, C.J. & Mongillo, M.A. *Geothermal Energy from Fractured Reservoirs – Dealing with Induced Seismicity*. IEA Open Technology Bulletin. Issue No. 48. February 2008.

31. Fact Sheet: *The History of Onshore Oil and Gas*. United Kingdom Onshore Operators Group. 2014.

32. BS EN 1998-1:2004. *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.* BSI. 8 April 2005.

33. BS EN 1998-2:2005. *Eurocode 8: Design of structures for earthquake resistance - Part 2: Bridges.* BSI. December 2005.

34. BS EN 1998-3:2005. *Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings.* BSI. 11 January 2006.

35. BS EN 1998-4:2006. *Eurocode 8: Design of structures for earthquakes resistance – Part 4: Silos, tanks and pipelines.* BSI. 29 September 2006.

36. BS EN 1998-5:2004. *Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects.* BSI. 8 April 2005.

37. BS EN 1998-6:2005. *Eurocode 8: Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys.* BSI. January 2006.

38. NA to BS EN 1998-1:2004. *UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.* BSI. 29 August 2008.

39. NA to BS EN 1998-2:2005. *UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 2: Bridges*. BSI. 30 June 2009.

40. NA to BS EN 1998-4:2006. *UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines*. BSI. 29 August 2008.

41. NA to BS EN 1998-5:2004. *UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects.* BSI. 29 August 2008.

42. NA to BS EN 1998-6:2005. *UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys.* BSI. 29 August 2008.

43. Charles, J.A., Abbiss, C.P., Gosschalk, E.M., & Hinks, J.L. *An engineering guide to seismic risk to dams in the United Kingdom.* Building Research Establishment Report BR 210. 1991.

44. Wieland, M. *ICOLD's revised seismic design and performance criteria for large storage dams*. International Commission on Large Dams. 2013.

45. *A guide to nuclear regulation in the UK*. Office for Nuclear Regulation. Health and Safety Executive. October 2013.

46. *License condition handbook*. Office for Nuclear Regulation. October 2014.

47. *Safety Assessment Principles for Nuclear Facilities*. Office for Nuclear Regulation. Revision 0. 2014 Edition.

48. Newell, J.P., & Roberts, G.P. *Seismic analysis and design practice for UK nuclear installations.* Civil engineering in the nuclear industry. Thomas Telford. 1991.

49. *Control of Major Accident Hazards Regulations 1999*. SI 1999/743. The Stationery Office. 1999. ISBN 0110821920.

50. BS EN 1473:2007. *Installation and equipment for liquefied natural gas – Design of onshore installations.* BSI. February 2007.

51. *Guidance for the location and design of occupied buildings on chemical manufacturing sites.* Chemical Industries Association. February 1998.

52. PD 8010-1:2004. *Code of practice for pipelines – Part 1: Steel pipelines on land.* Published Document. British Standards Institution. 31 July 2004.

53. IGE/TD/12. *Pipework stress analysis for gas industry plant.* Recommendations on Transmission and Distribution Practice. Edition 2. Communication 1681. The Institution of Gas Engineers and Managers. 2003.

54. BE EN 1594:2013. *Gas infrastructure – Pipelines for maximum operating pressure over 16 bar – Functional requirements.* BSI. September 2013.

55. BS EN 1993-4-3:2007. *Eurocode 3. Design of steel structures. Pipelines.* BSI. May 2007.

56. BS EN 14161:2003. *Petroleum and natural gas industries – Pipeline transportation systems.* BSI. December 2003.

57. Mallard, D.J., Skipp, B.O., & Aspinall, W.P. *An appraisal of existing seismic hazard estimates for the UK Continental Shelf.* Research Report 166. Health and Safety Executive. HMSO. 2003.

58. Principia Mechanica Ltd. *Seismological Studies for UK Hazard Analysis.* Report for CEGB. Report No. 346/85. 1985.

59. Douglas, J. *A comprehensive worldwide summary of strong-motion attenuation relationships for peak ground acceleration and spectral ordinates (1969 to 2000).* ESEE Report No. 01-1. Engineering Seismology

and Earthquake Engineering. Imperial College of Science, Technology and Medicine. January 2001.

60. Douglas, J. *Errata of and additions to ESEE Report No. 01-1: 'A comprehensive worldwide summary of strong-motion attenuation relationships for peak ground acceleration and spectral ordinates (1969 to 2000)'.* Civil and Environmental Engineering. Imperial College of Science, Technology and Medicine. October 2002.

61. Campbell, K.W., & Bozorgnia, Y. *Campbell-Bozorgnia NGA Empirical Ground Motion Model for the Average Horizontal Component of PGA, PGV, PGD and SA at Selected Spectral Periods Ranging from 0.01-10.0 Seconds.* NGA Report (Version 1.0). October 2006.

62. Bommer, J.J., Stafford, P.J., Alarcón, J.E., & Akkar, S. *The Influence of Magnitude Range on Empirical Ground-Motion Prediction.* Bulletin of the Seismological Society of America. Vol. 97, No. 6, 2152-2170. December 2007.

63. *Guidelines for the Design of Buried Steel Pipe.* American Lifelines Alliance. ASCE. July 2001 (with addenda through to February 2005).

64. Akka, S., & Bommer, J.J. *Empirical Prediction Equations for Peak Ground Velocity Derived from Strong-Motion Records from Europe and the Middle East.* Bulletin of the Seismological Society of America. Vol. 97, No. 2, 511-530. April 2007.

65. Musson, R.M.W. *Intensity attenuation in the UK*. Journal of Seismology. Vol. 9, 73-86. 2005.

66. Musson, R.M.W. *Updated intensity attenuation for the UK.* Open Report OR/13/09. British Geological Survey. 2013.

67. Jones, T., Dhu, T., Fulford, G., Corby, N., & Sinadinovski, C. *Earthquake risk in Newcastle and Lake Macquarie*. Geoscience Australia. 2002.

68. Nielson, A.H., & Boyd, S.A. *Phase 3 Report – Recommendations for a Seismic Policy. Seismic Policy Development.* Report to National Grid. Jacobs U.K. Limited. 6th January 2014.

69. McDonough, P.W. *Seismic Resilience of Natural Gas Systems. Improving Performance.* Technical Council on Lifeline Earthquake Engineering. Monograph No. 34. ASCE. 2012.

70. ASME B31.8S-2004. *Managing System Integrity of Gas Pipelines.* ASME Code for Pressure Piping, B31. Supplement to ASME B31.8. ASME. 14 January 2005.

71. McDonough, P.W. *Seismic Design for Natural Gas Distributors.* Technical Council on Lifeline Earthquake Engineering. Monograph No. 8. ASCE. August 1995.

72. Hovius, N., & Meunier, P. *Earthquake ground motion and patterns of seismically induced landsliding.* Landslides: Types, Mechanisms and Modelling. Cambridge University Press. 2012.

73. Wang, X., Nie, G., & Wang, D. *Relationship between ground motion parameters and landslides induced by Wenchuan earthquake*. Earthquake Science 23, 233-242. 2010.

74. Idriss, I.M., & Boulanger, R.W. Soil *Liquefaction During Earthquakes.* MNO-12. Earthquake Engineering Research Institute. 2008.

75. Musson, R.M.W. *Design Earthquakes in the U.K.* Bulletin of Earthquake Engineering 2: 101-112. 2004

76. Keefer, D.F. *Landslide caused by earthquakes*. Geological Society of America Bulletin. Vol. 95. Issue 2. 1984.

Table 1 – Indicative Damage Thresholds for Welded Steel Pipelines

Ground Shaking											
Fault Rupture											
Liquefaction											
Landsliding											
MMI scale	I	II	III	IV	V	VI	VII	VIII	IX	X	XI
~PGV (mm/s)	< 1	1-10		10-30	30-80	80-160	160-300	300-600	600-1200	> 1200	
~PGA (g)	<0.002	0.002-0.014		0.014-0.04	0.04-0.09	0.09-0.18	0.18-0.34	0.34-0.65	0.65-1.24	>1.24	

	No damage to steel pipelines
	Damage to steel pipelines with poor quality welds. No damage to pipelines with good quality welds.
	Damage to steel pipelines with modern full penetration butt welds.

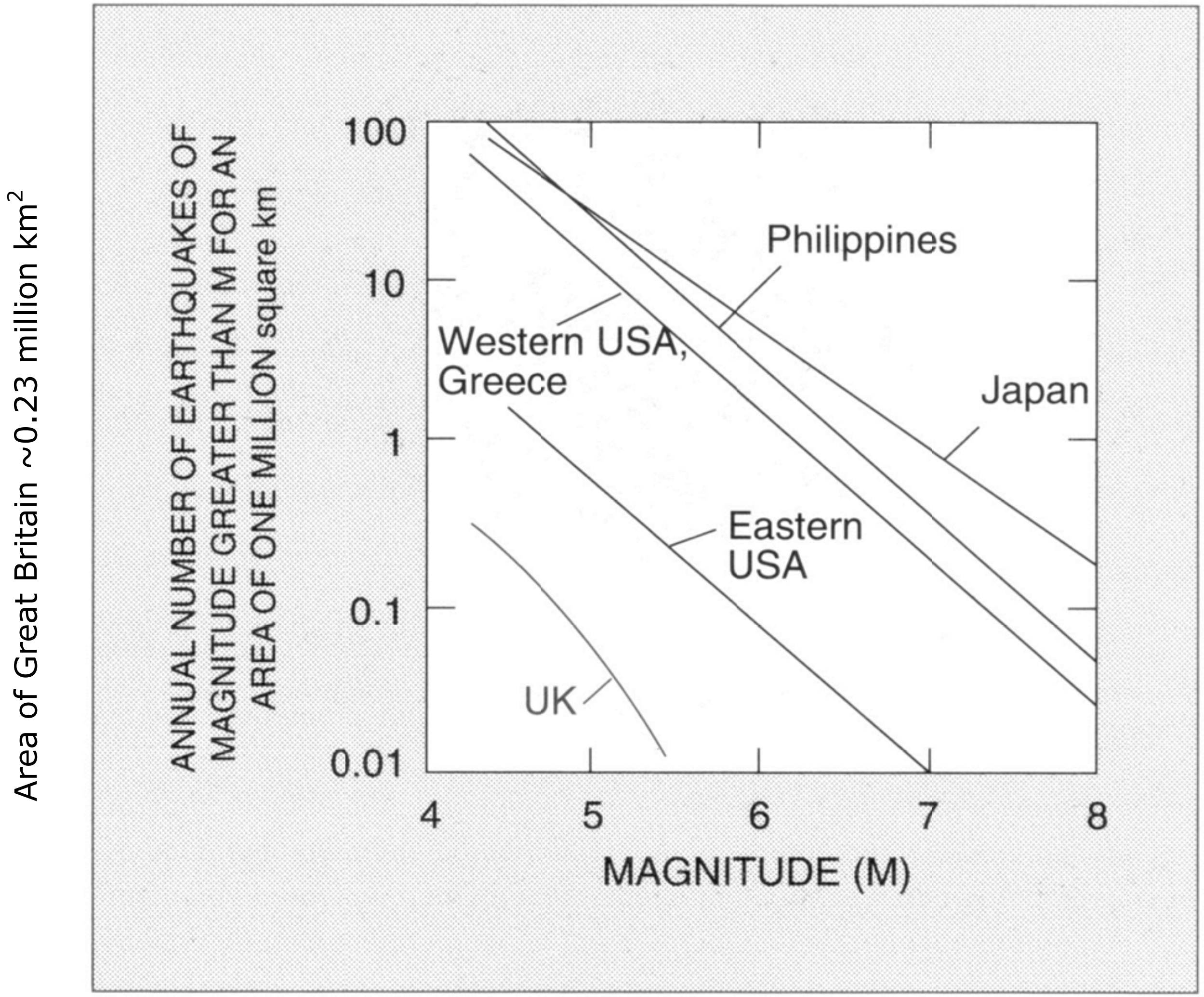


Figure 1	Relative seismic activity rates.

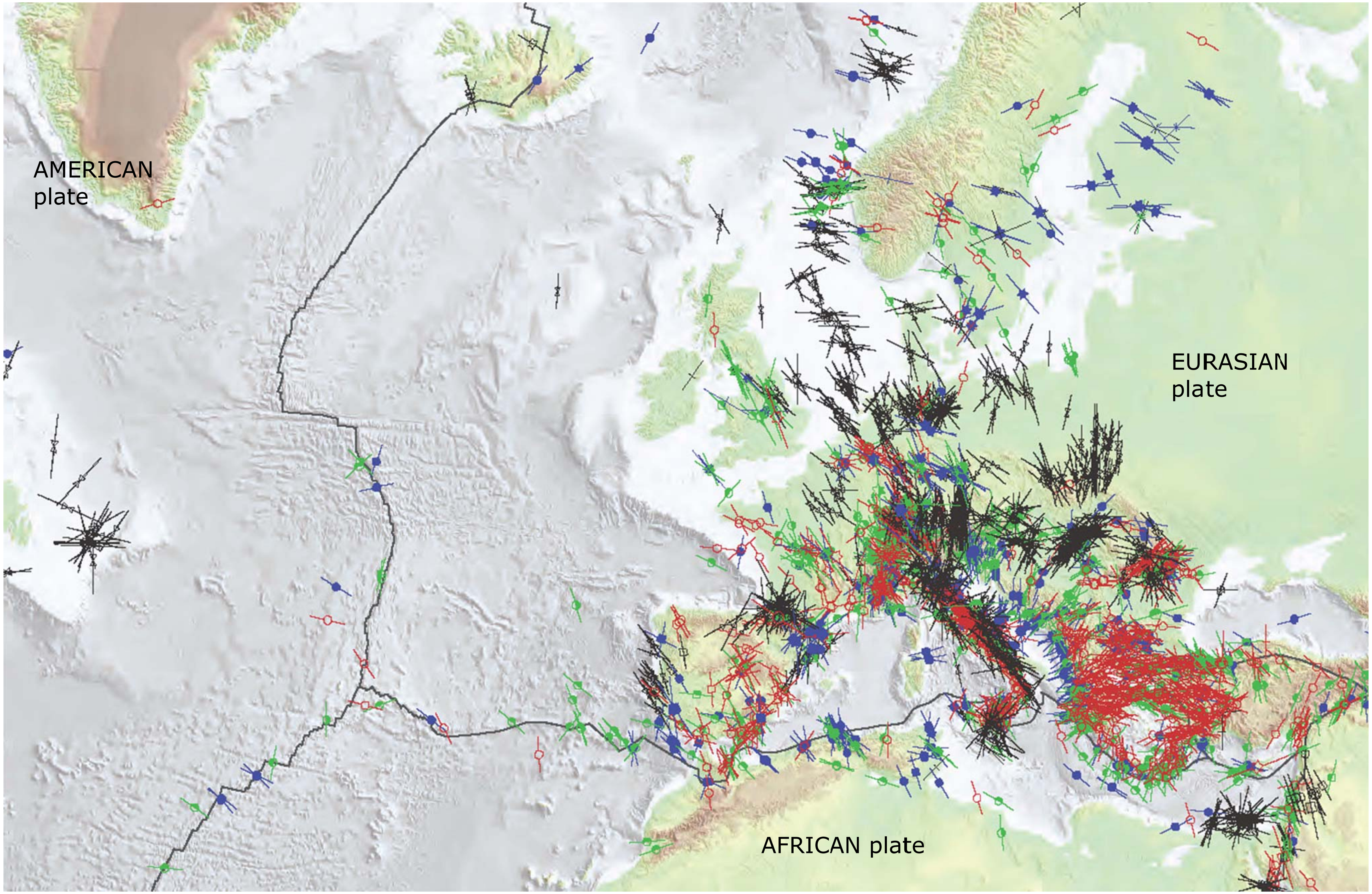


Figure 2	A stress map of the maximum horizontal compressive stress in the depth range 0-40 km. [Green stress vectors indicate that the vertical stress is intermediate i.e. the maximum & minimum principal stresses are horizontal]

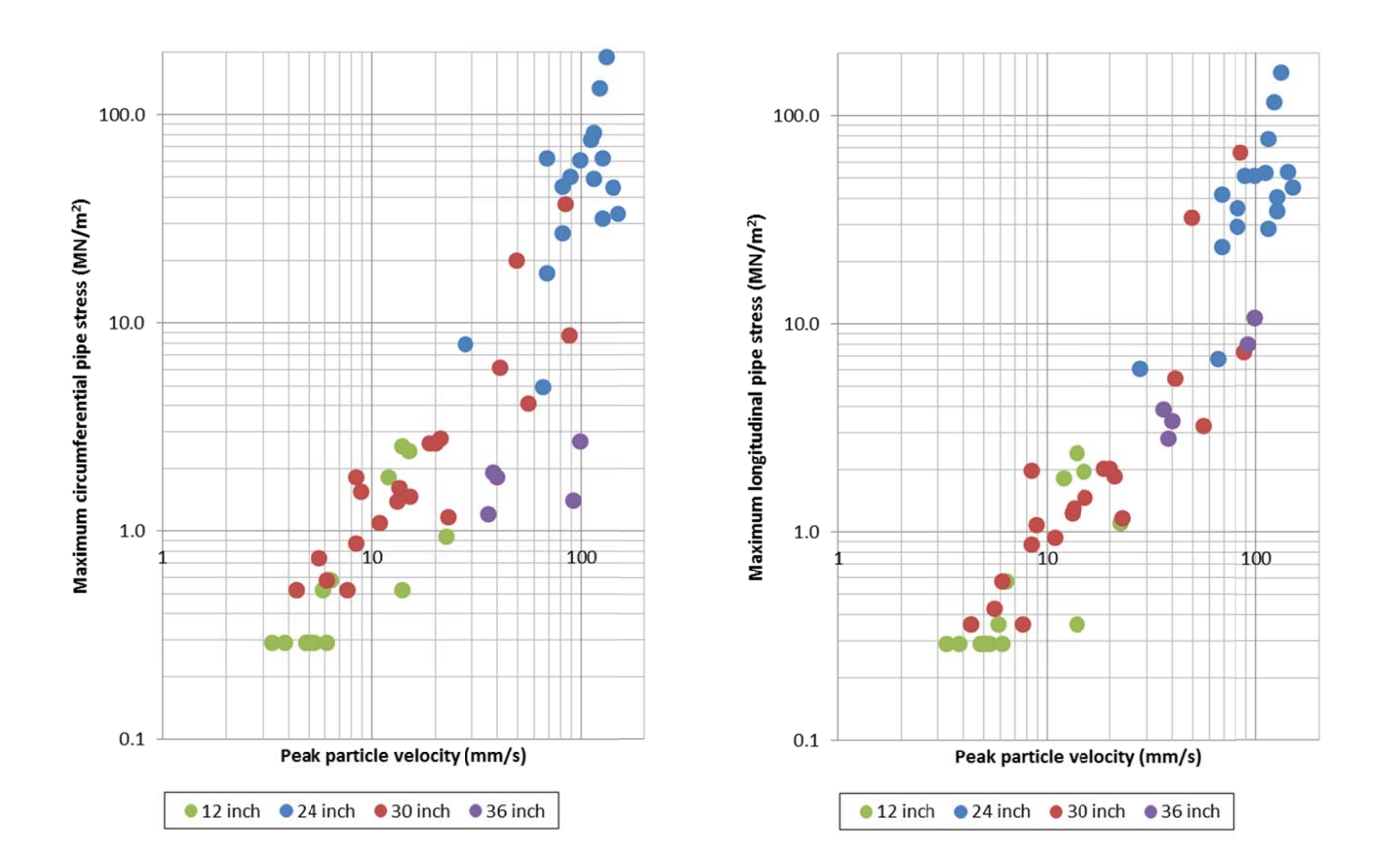
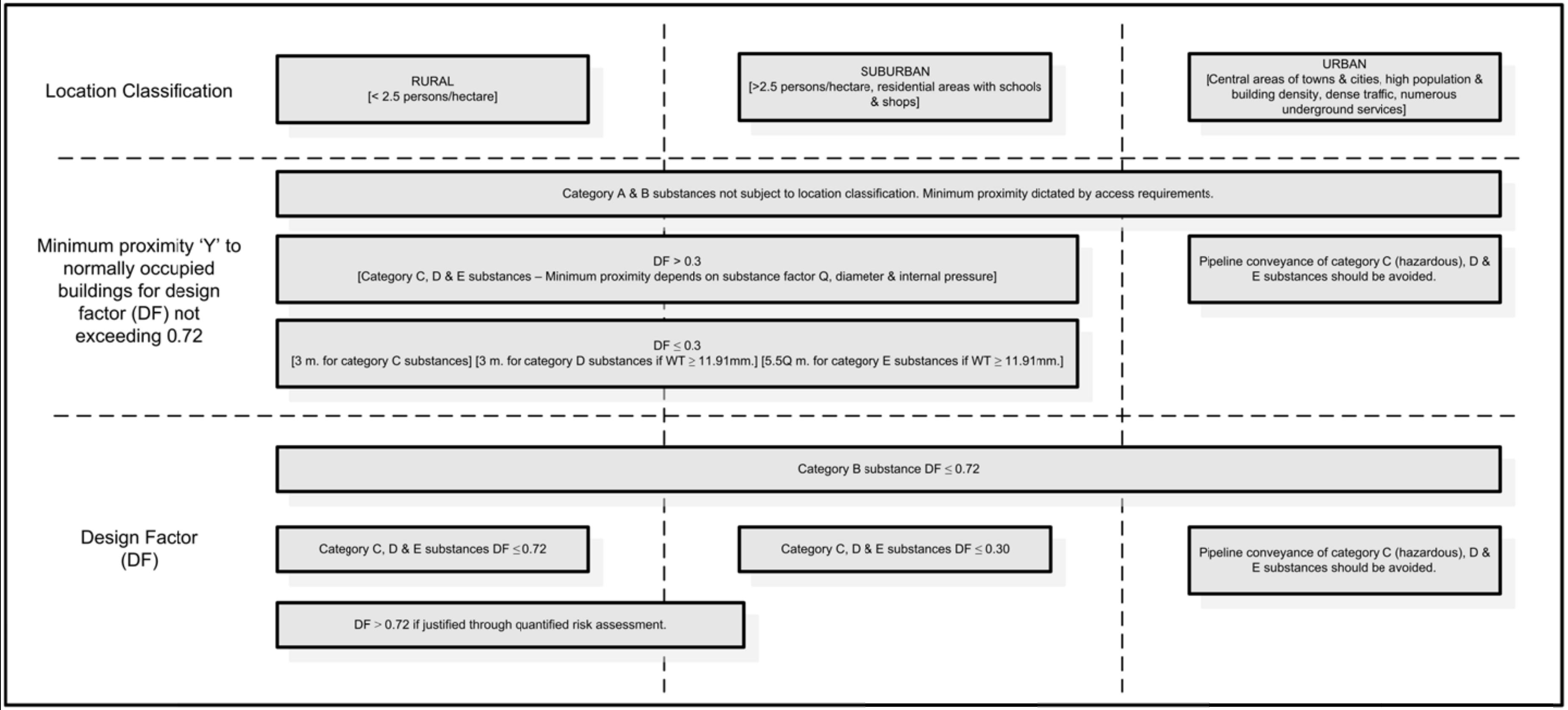


Figure 3	Transient pipe stress and peak radial particle velocity data from blasting adjacent to buried pipelines.



Category	Description
A	Non-flammable water-based fluids
B	Flammable and/or toxic liquids
C	Non-flammable, non-toxic gases
D	Non-toxic single-phase natural gas
E	Flammable and/or toxic gases

Substance	Substance factor, Q
Ammonia	2.5
Ethylene	0.8
Hydrogen	0.45
Liquid Petroleum Gas	1.0
Natural gas liquids	1.25
Category C	0.3

$$Y = Q.\left(\frac{D_o^2}{32000} + \frac{D_o}{160} + 11\right).\left(\frac{p}{32} + 1.4\right)$$

Y *in metres,* D_o *is external diameter in millimetres,* p *is pressure in barg*

V0.0 24 January 2015

Figure 4	Standard stress level and minimum proximity limits according to PD 8010-1:2004.

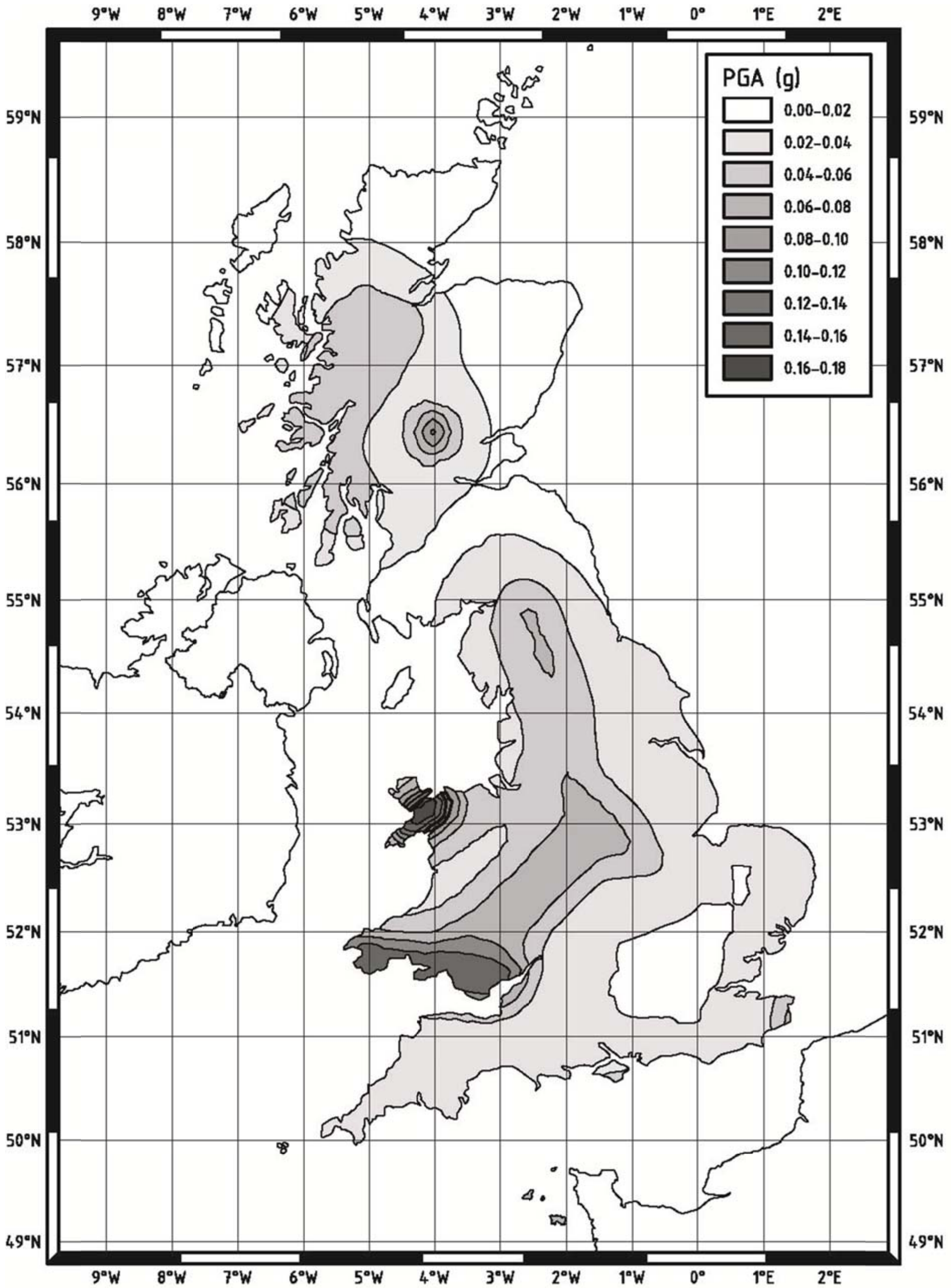
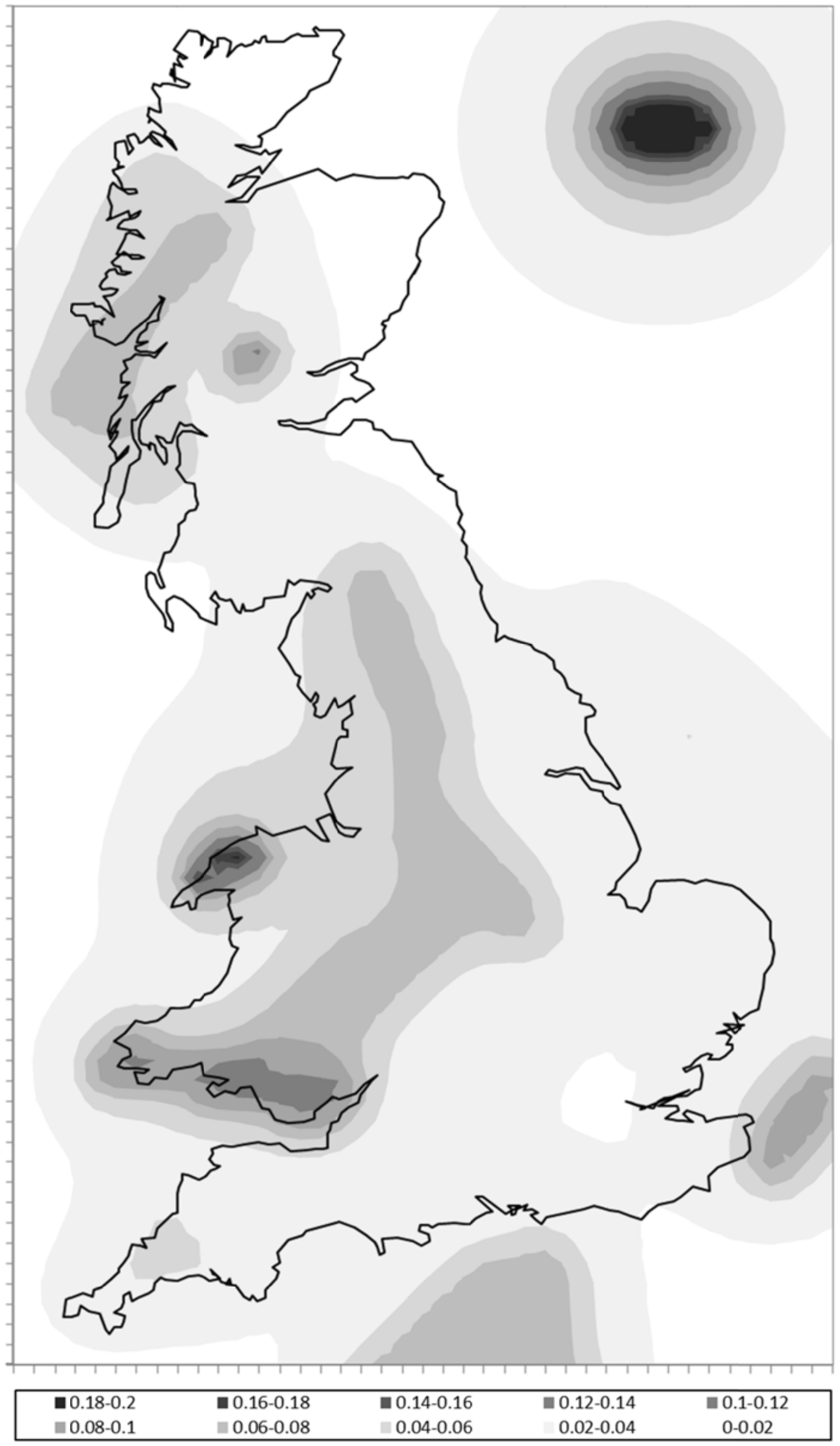
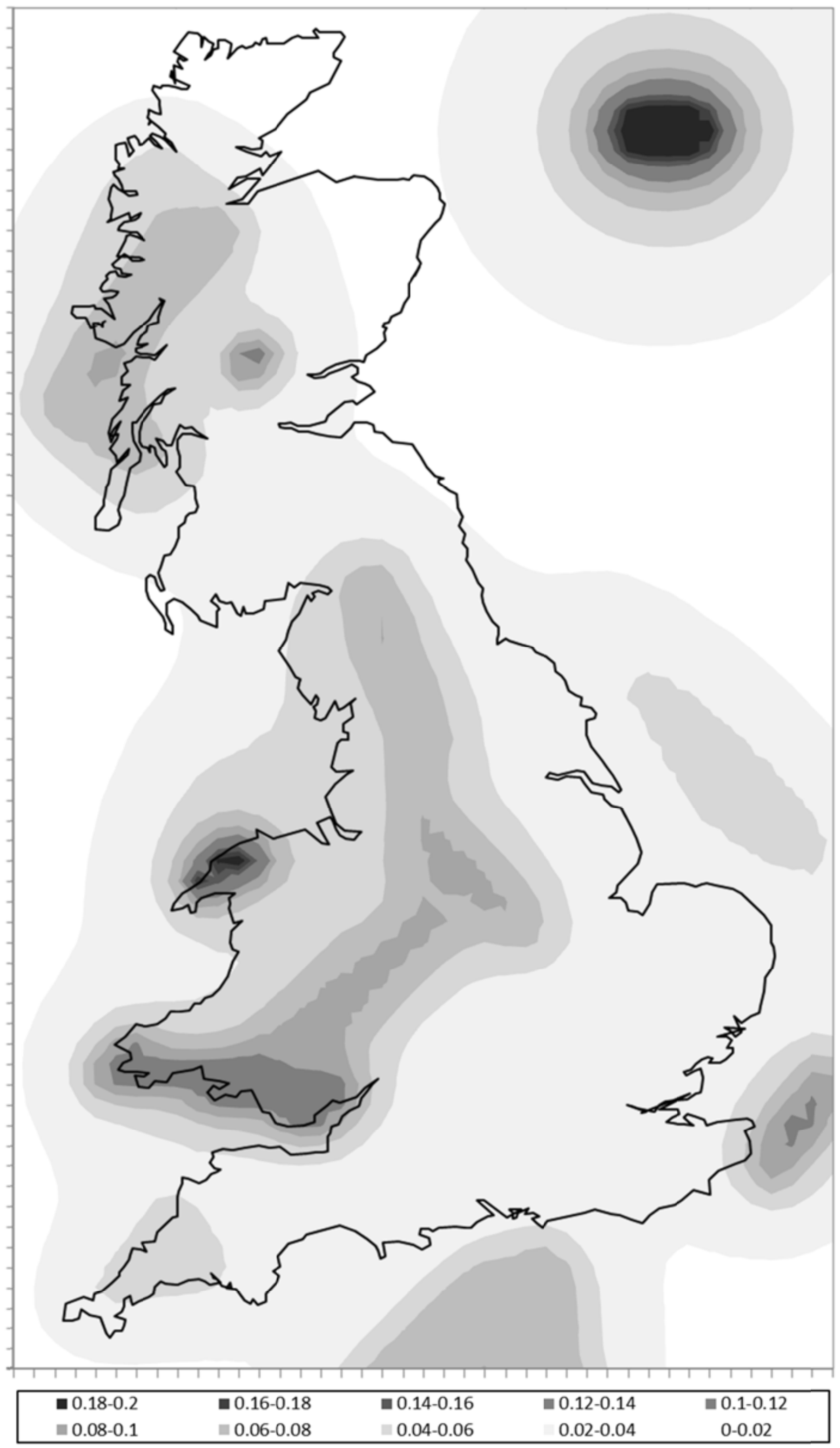


Figure 5	Seismic hazard map for PGA on rock from PD 6698 for a return period of 2500 years.



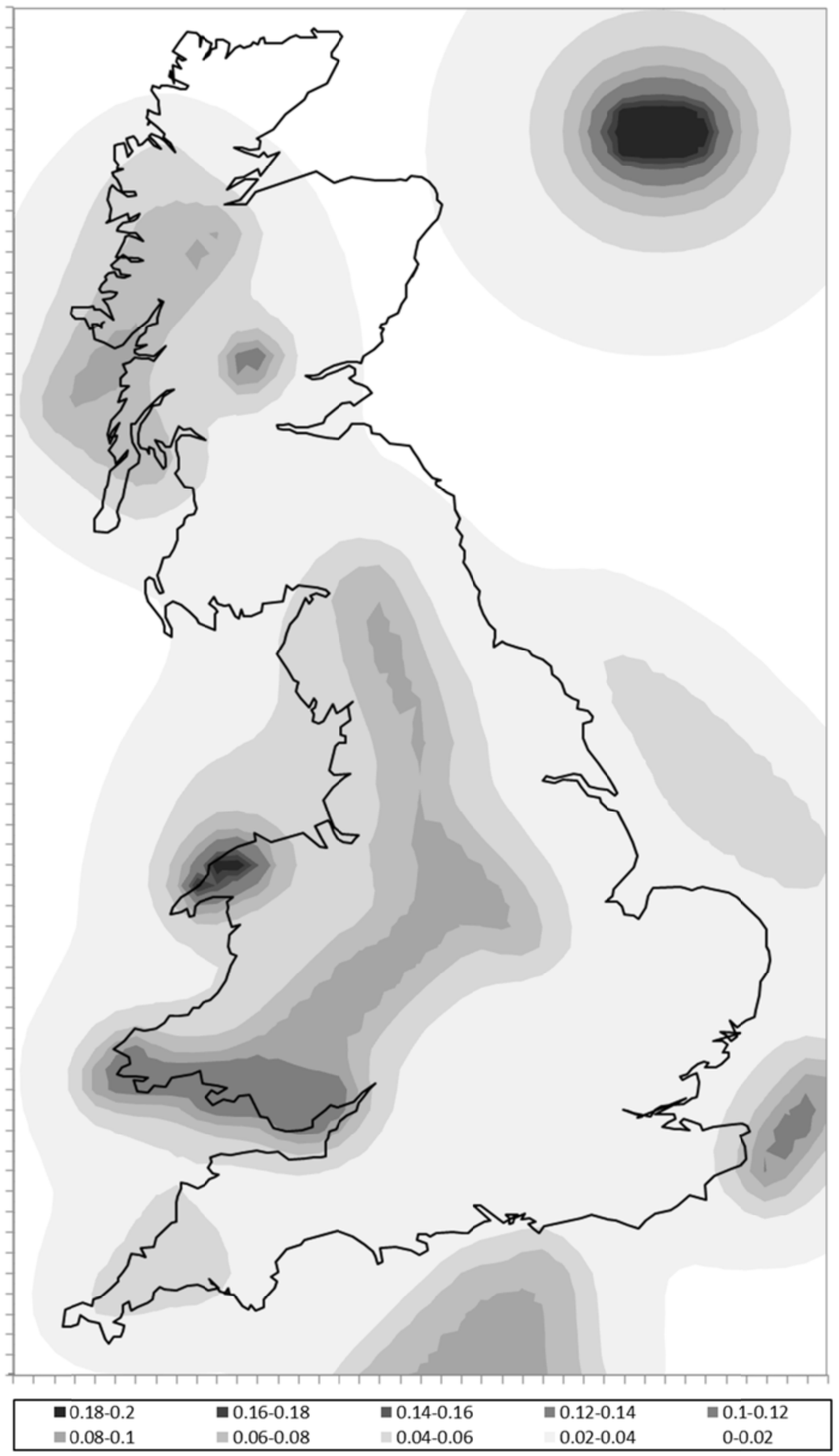
'ROCK'

$V_{s30} > 750\ m/s$



'STIFF'

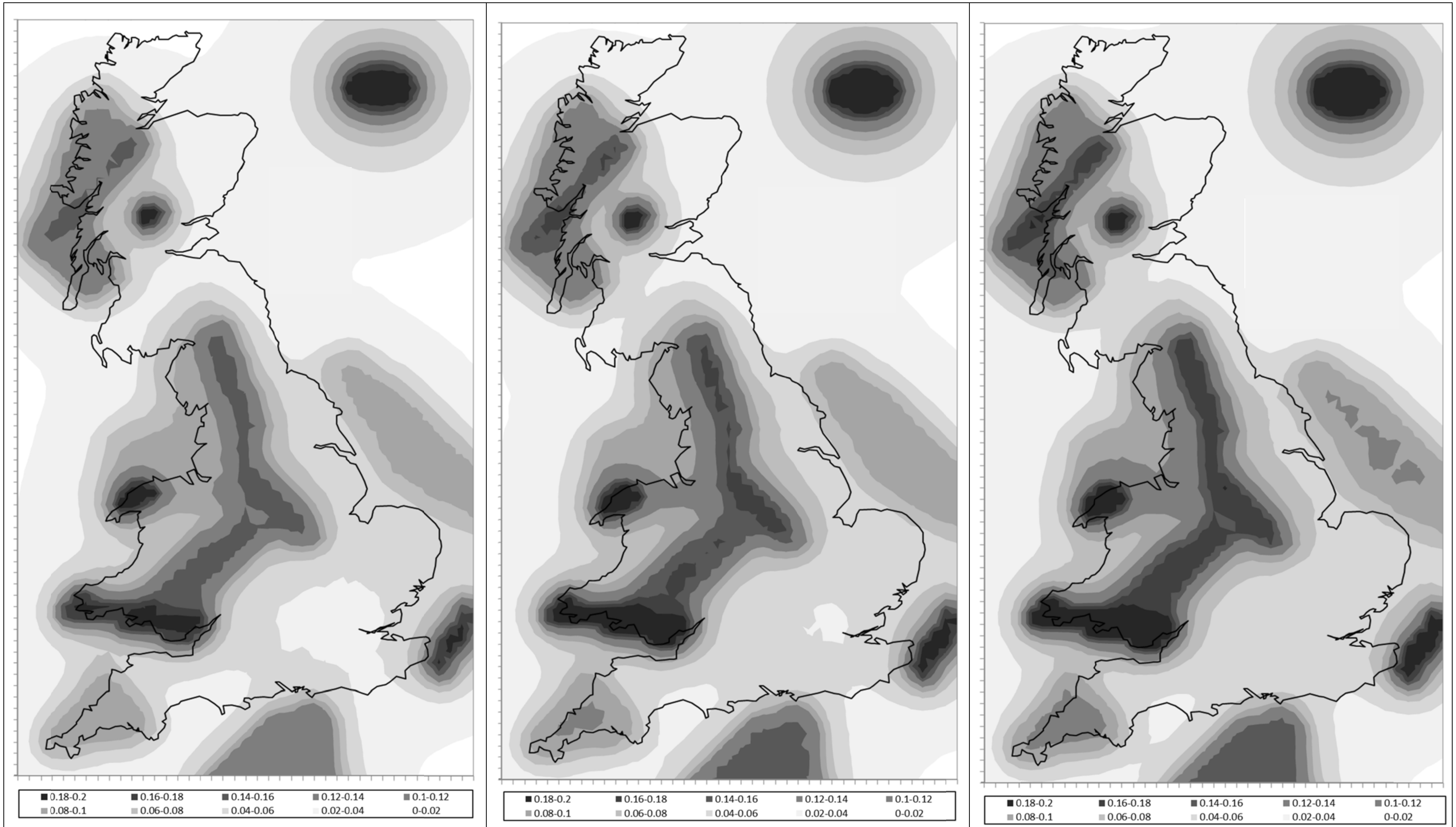
$360 < V_{s30} > 750\ m/s$



'SOFT'

$V_{s30} < 360\ m/s$

Figure 6	Indicative PGA (g) hazard maps for a return period of 2500 years.



'ROCK'	'STIFF'	'SOFT'
$V_{s30} > 750\ m/s$	$360 < V_{s30} > 750\ m/s$	$V_{s30} < 360\ m/s$

Figure 7 Indicative PGA (g) hazard maps for a return period of 10,000 years.

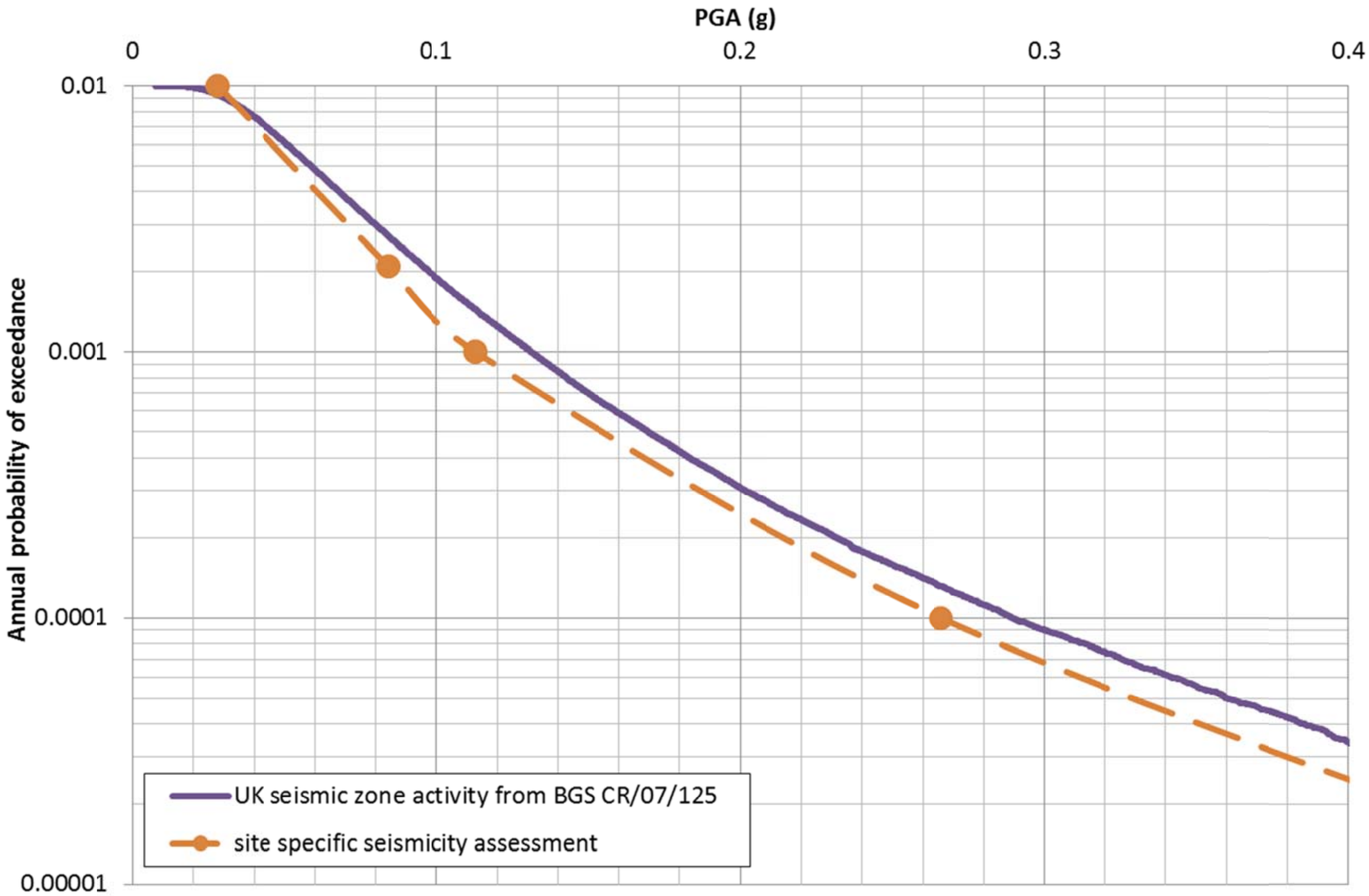


Figure 8	Comparison of PGA hazard results at grid reference SO503269 from 'hazard map' uniform seismicity zones and more detailed seismicity from site specific study.

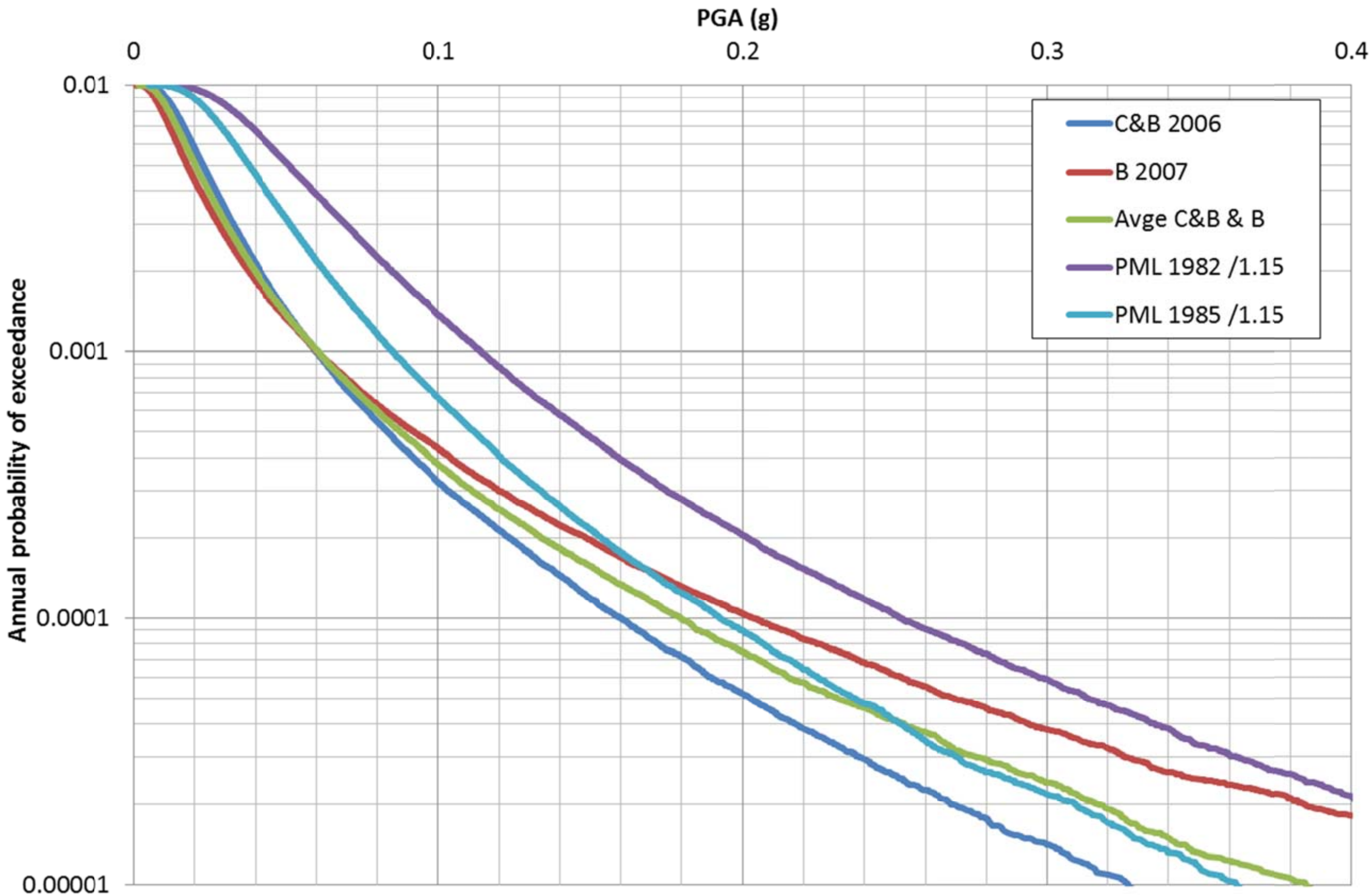
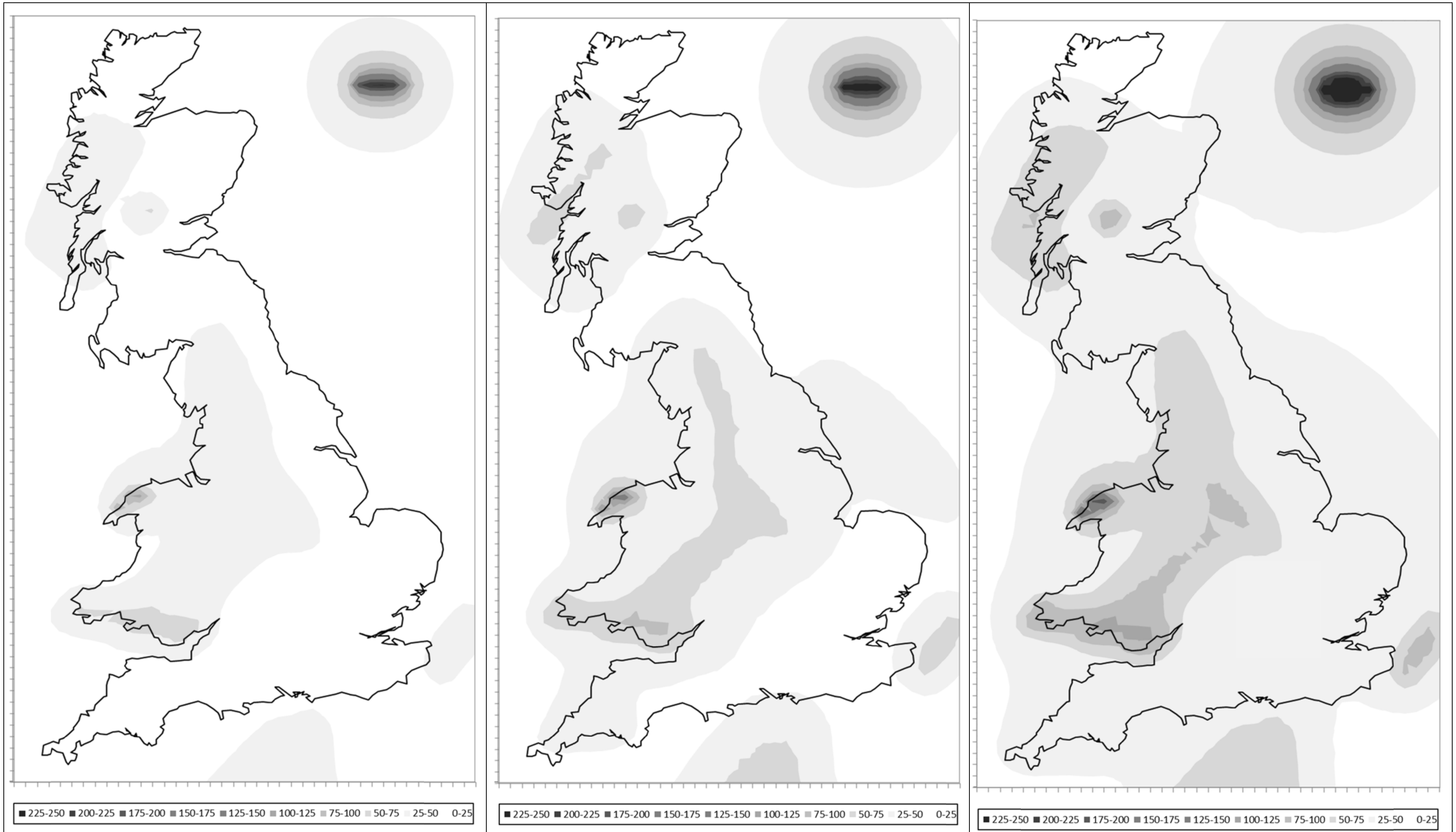
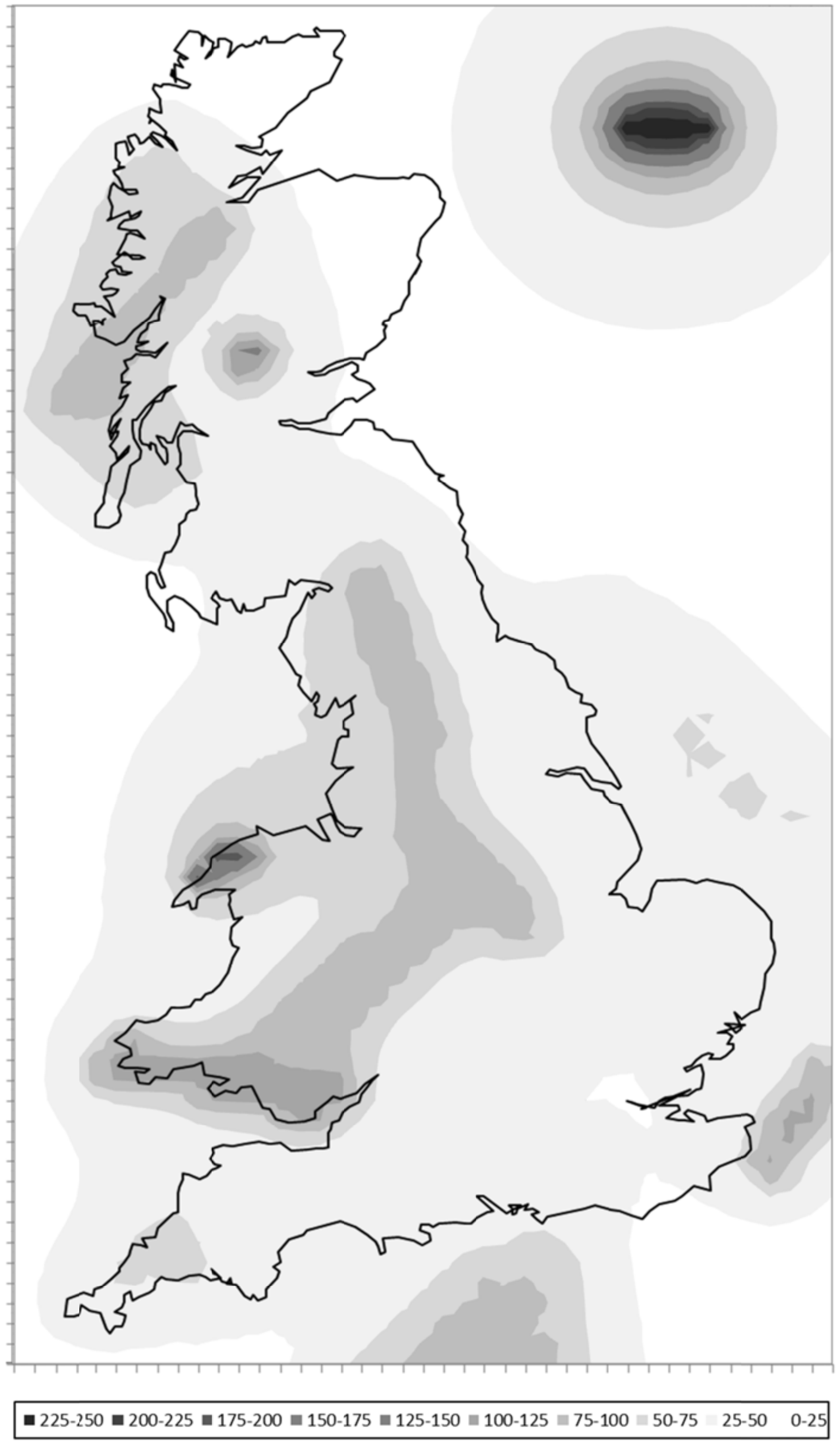


Figure 9	Comparison of PGA hazard results at grid reference SO503269 according to choice of attenuation model.



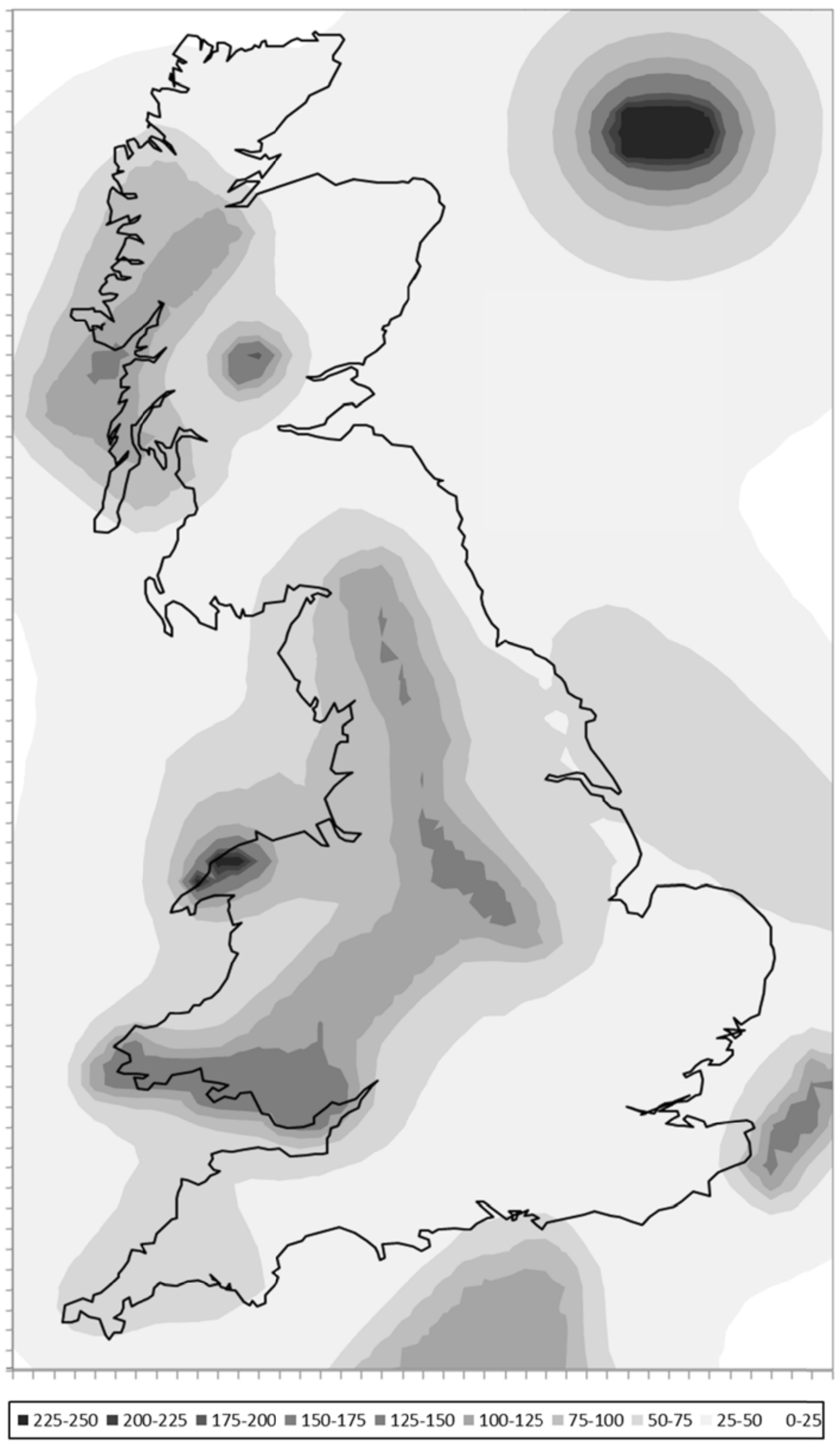
'ROCK'	'STIFF'	'SOFT'
$V_{s30} > 750\ m/s$	$360 < V_{s30} > 750\ m/s$	$V_{s30} < 360\ m/s$

Figure 10	Indicative PGV (mm/s) hazard maps for a return period of 2500 years.



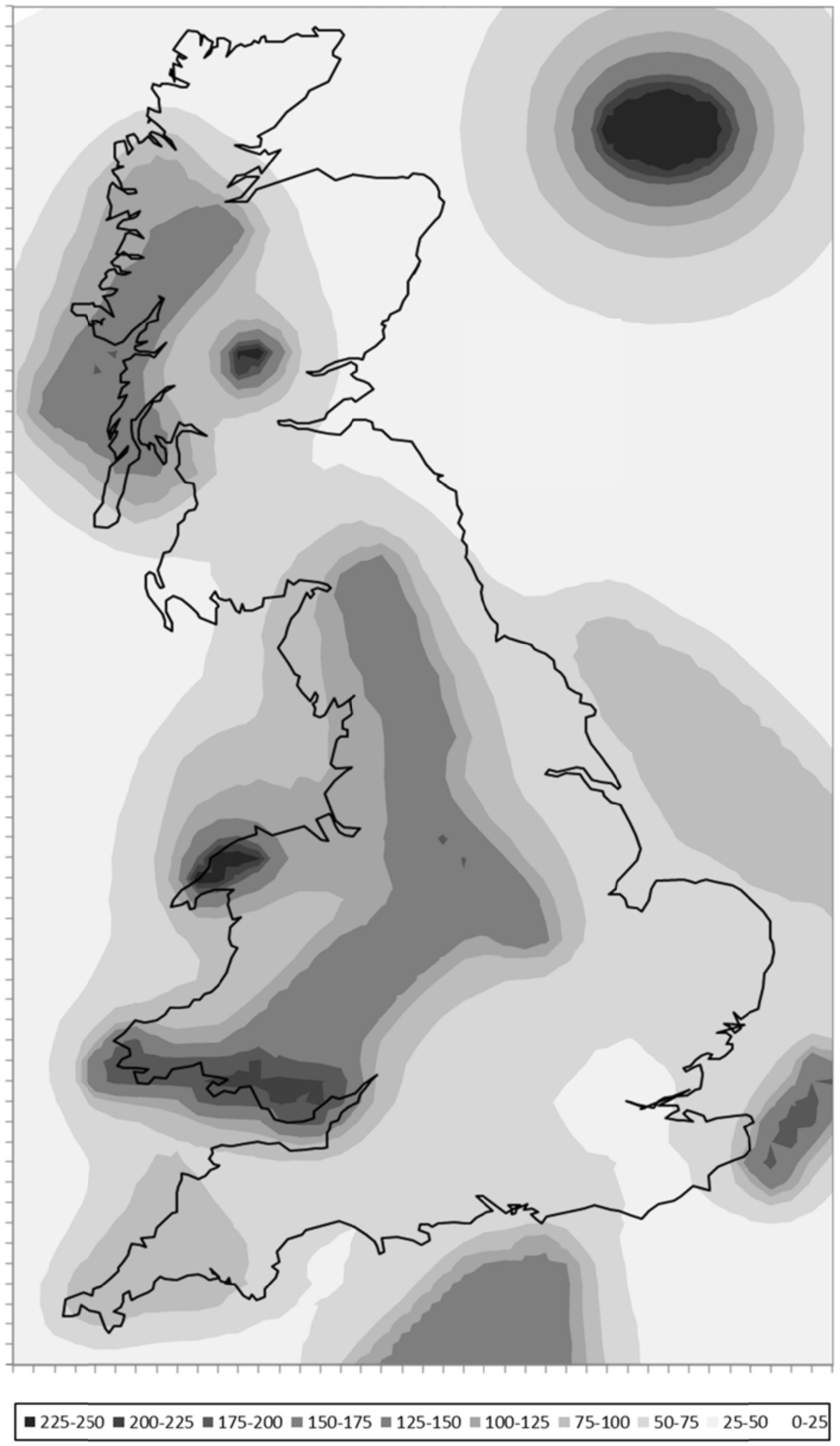
'ROCK'

$V_{s30} > 750\ m/s$



'STIFF'

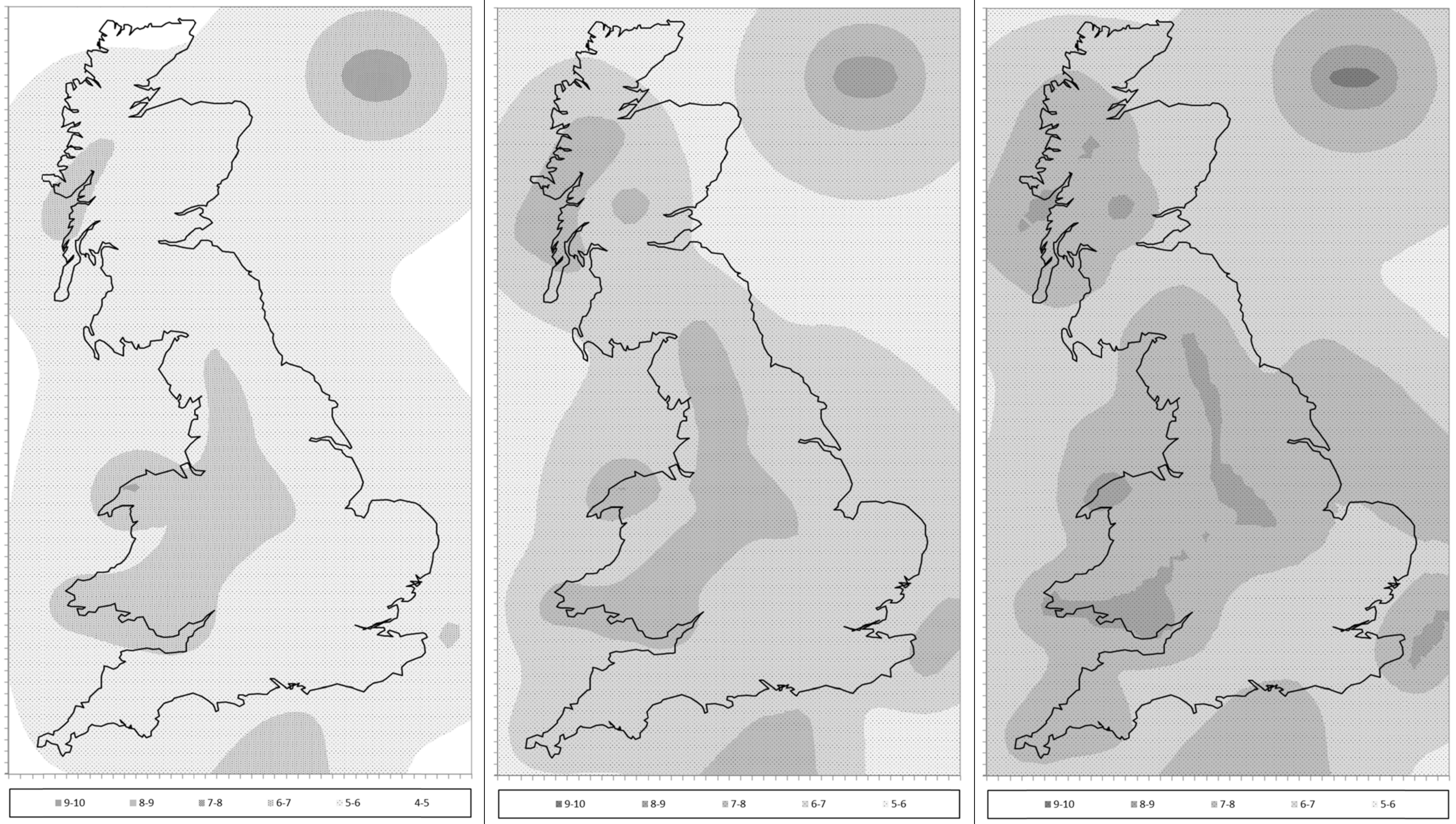
$360 < V_{s30} > 750\ m/s$



'SOFT'

$V_{s30} < 360\ m/s$

Figure 11 Indicative PGV (mm/s) hazard maps for a return period of 10,000 years.



475 years

2500 years

10,000 years

Figure 12	Indicative seismic intensity hazard maps for return periods of 475, 2500 and 10,000 years.

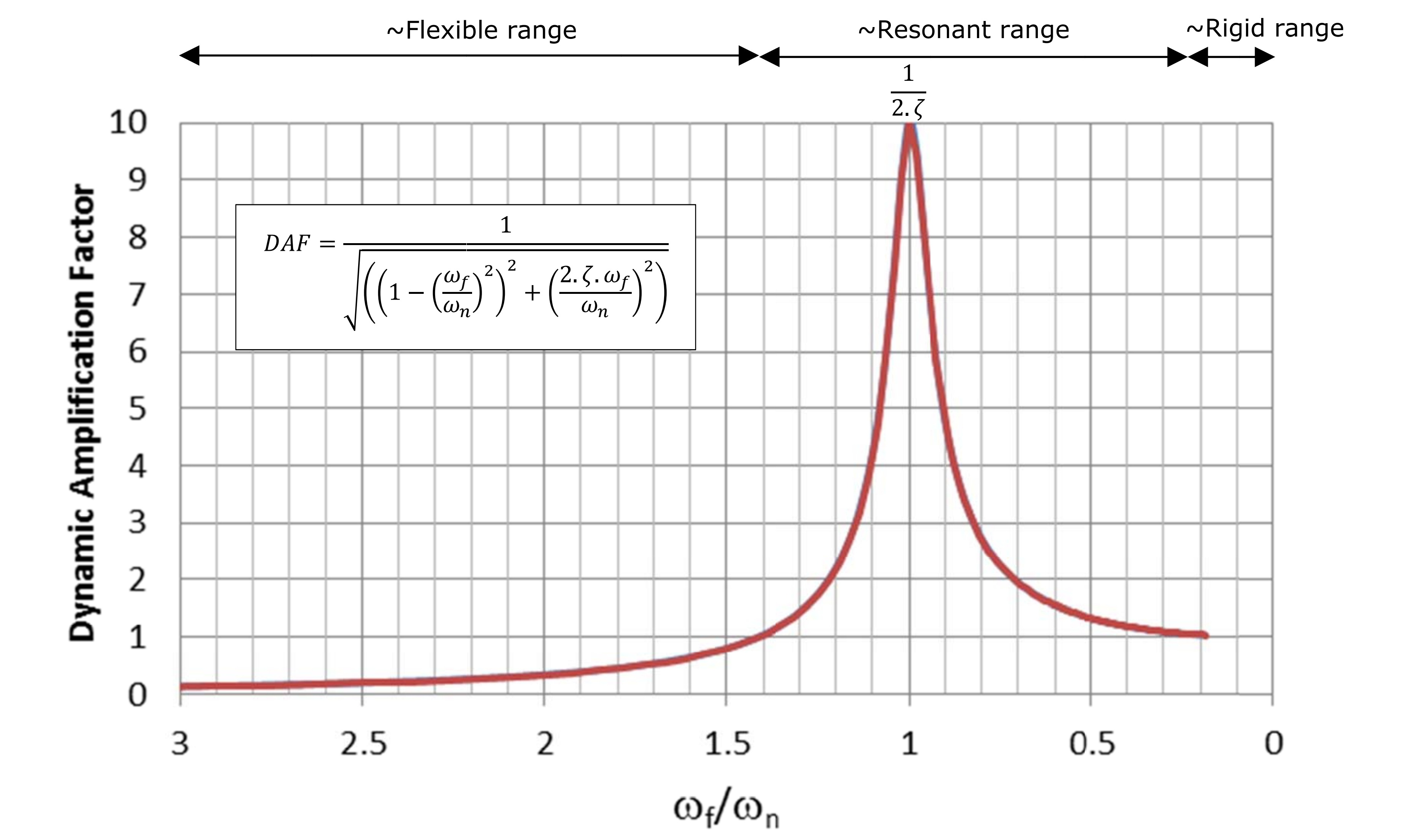


Figure 13	Dynamic amplification factor of SDF oscillator according to ratio of harmonic forcing frequency to the natural vibration frequency of the system [ζ=5%].

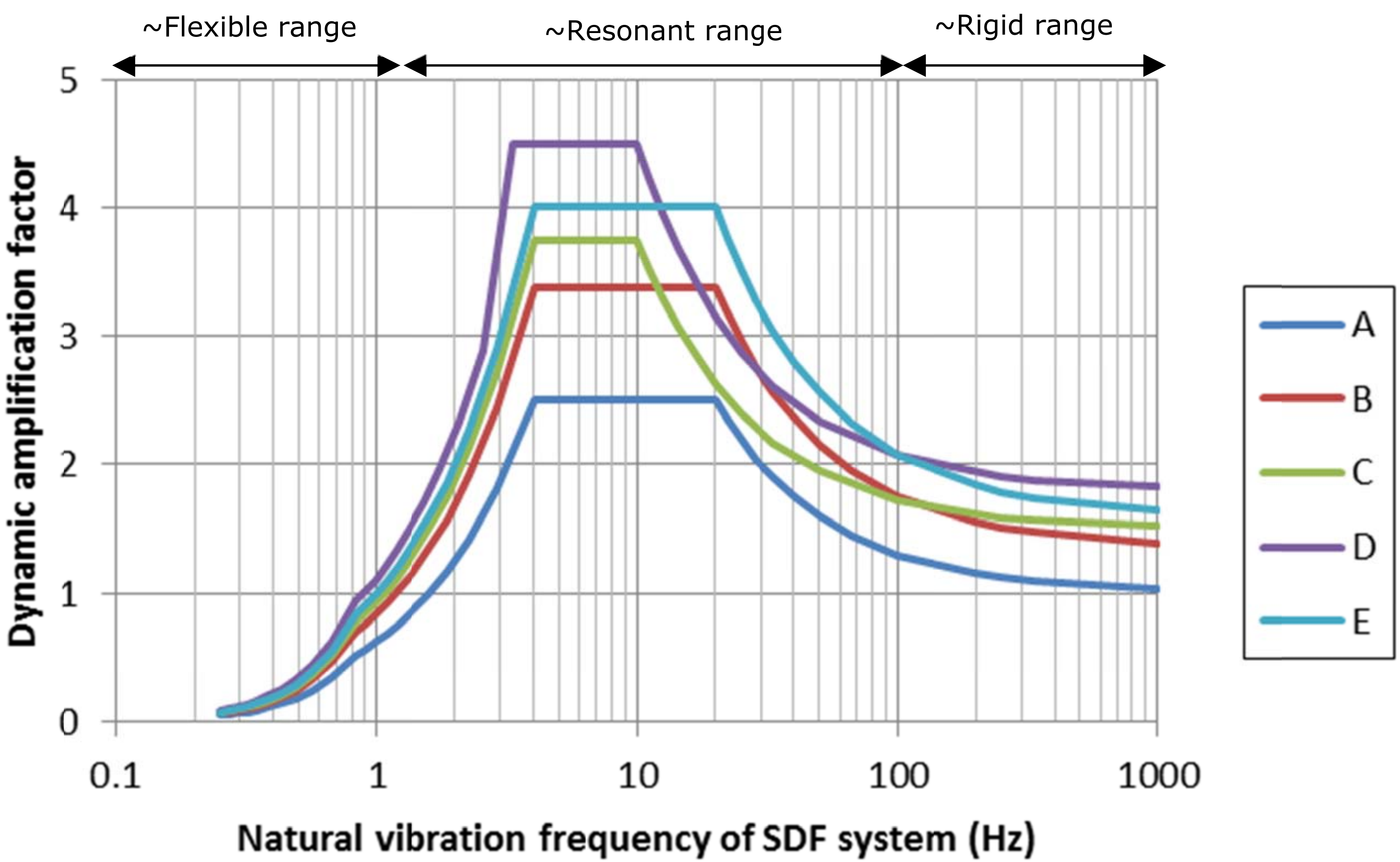


Figure 14	Eurocode 8 Type 2 elastic dynamic amplification factors for SDF oscillator according to ground types A to E [5% damping].

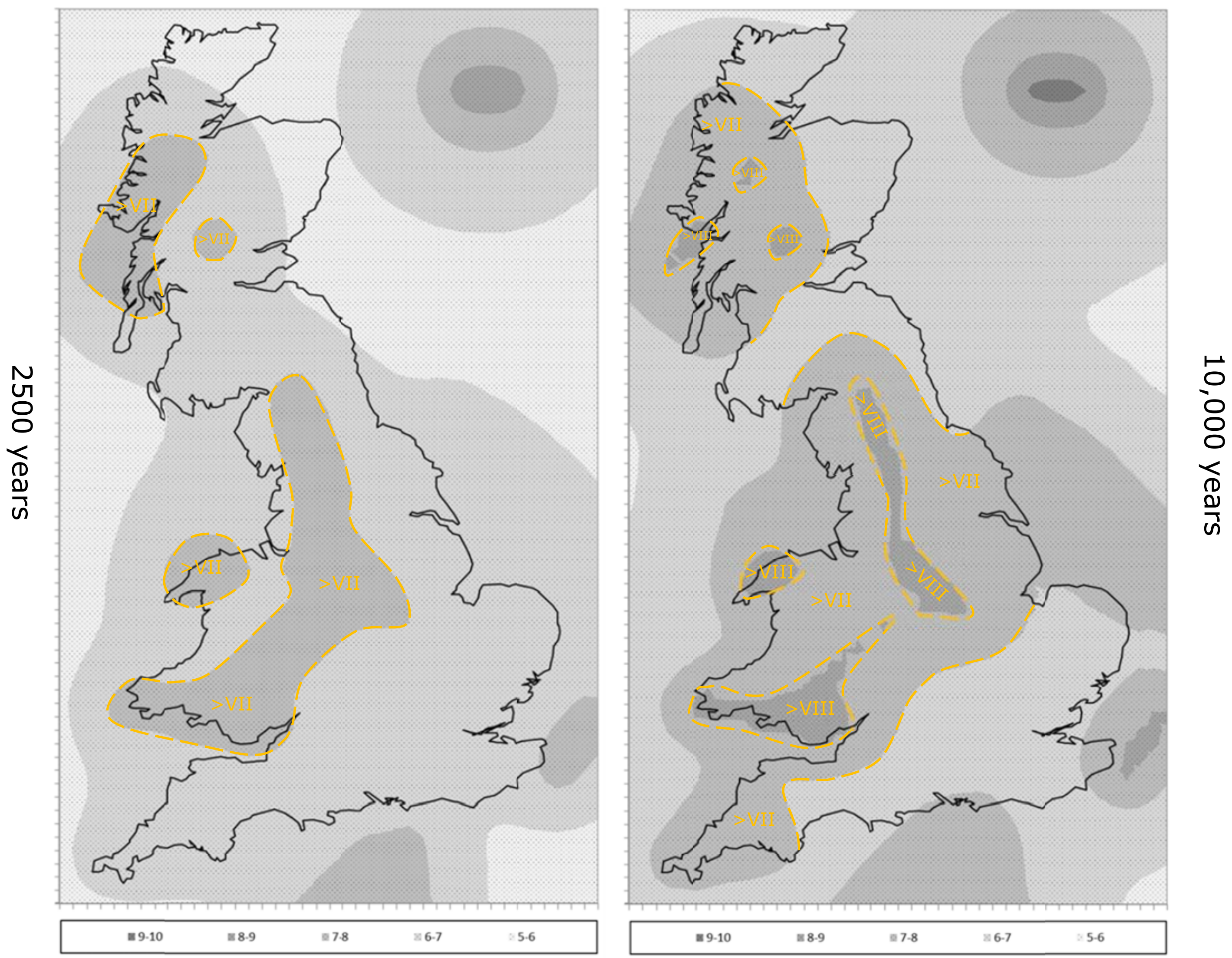


Figure 15	Areas with a seismic intensity exceeding VII.

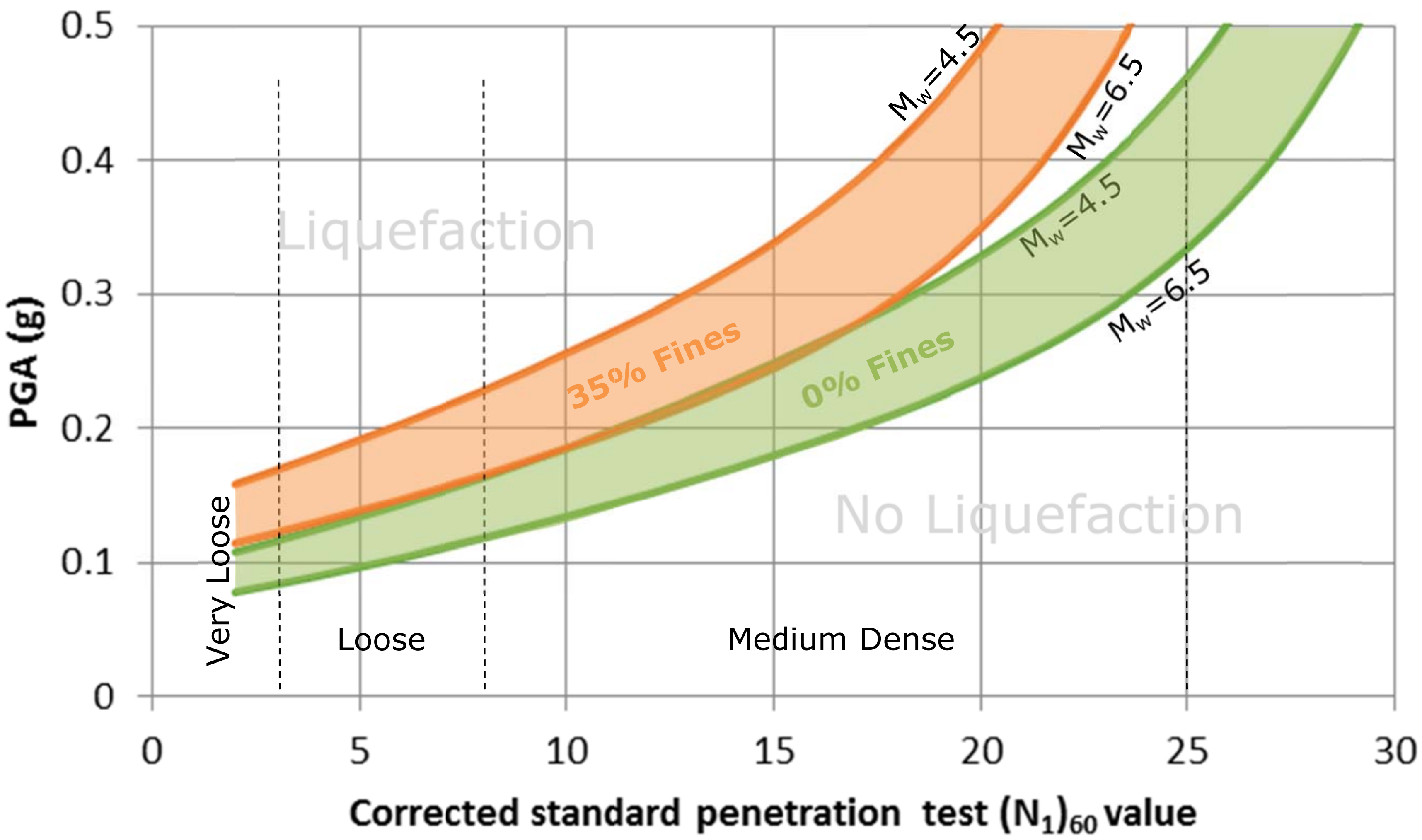


Figure 16	Indicative threshold PGA levels for liquefaction in clean and high fines content sands.

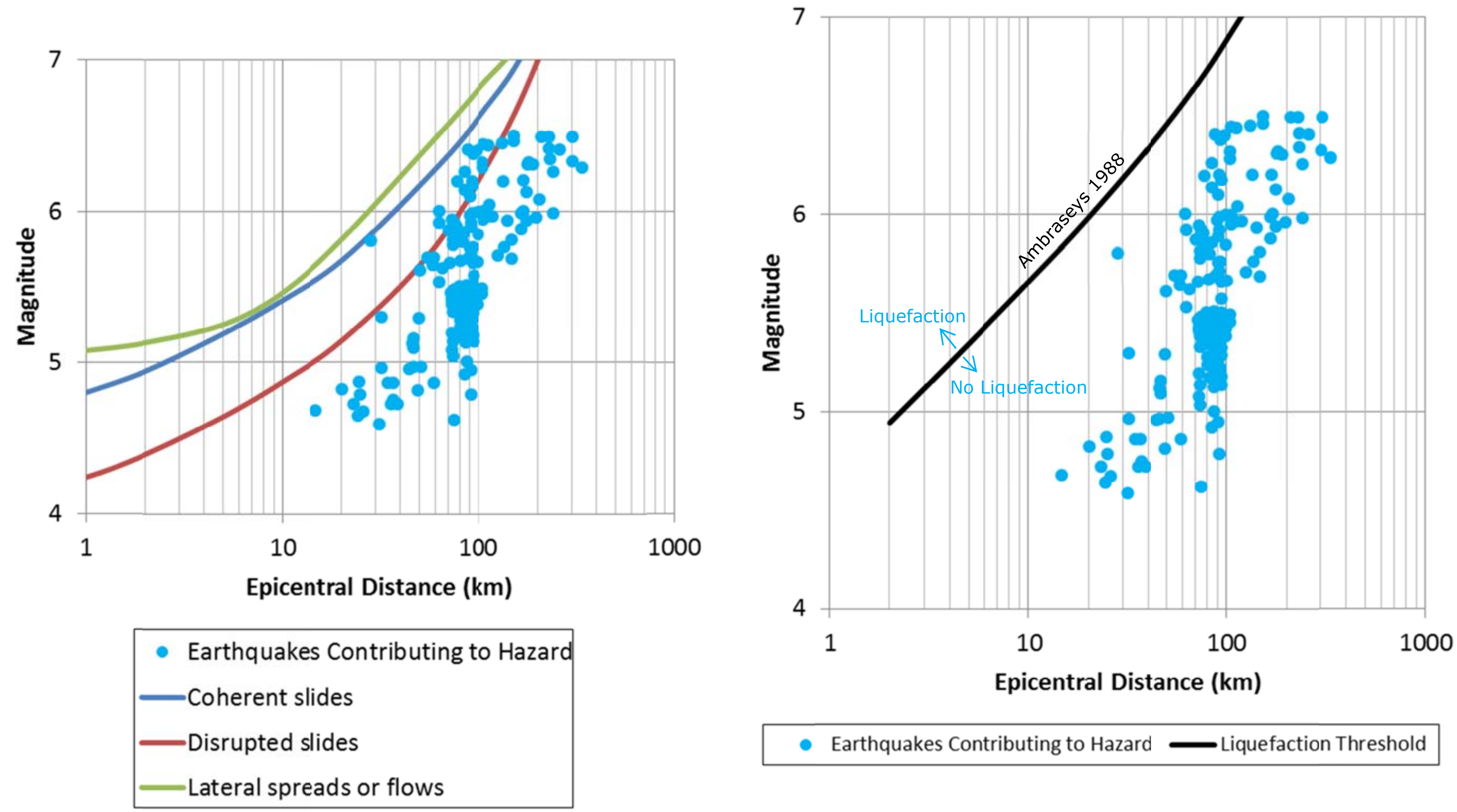
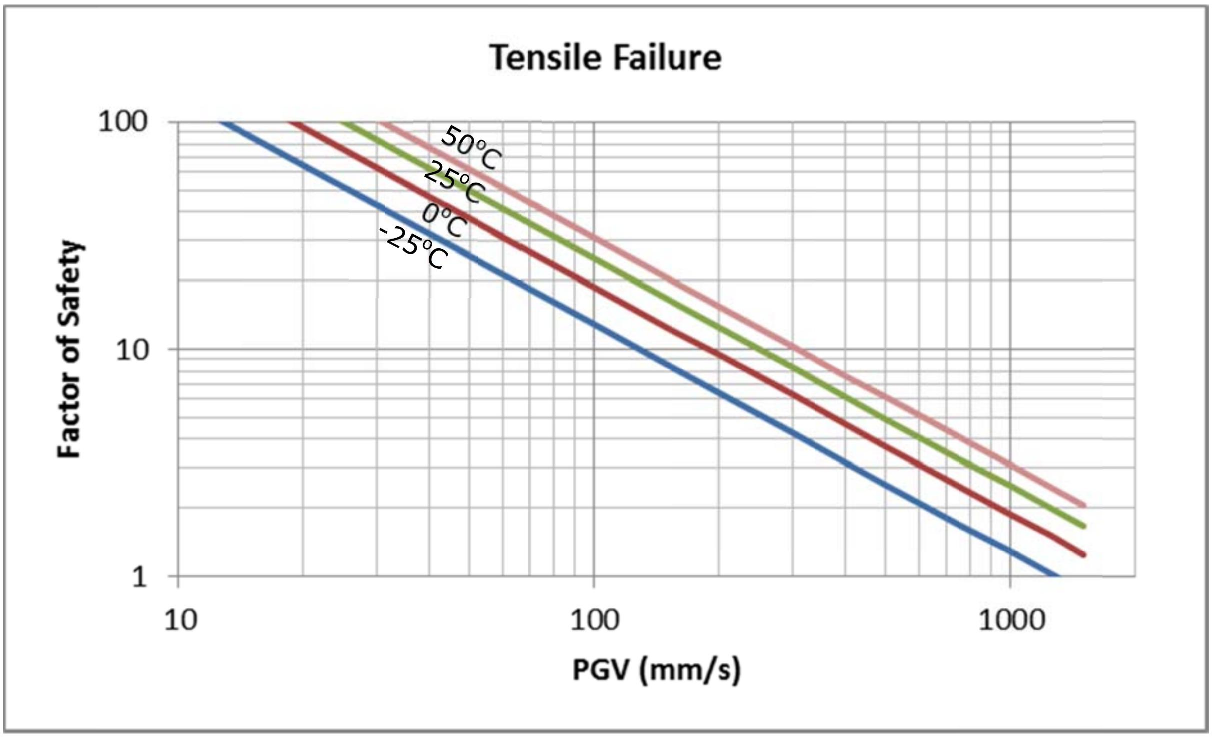


Figure 17	Earthquakes contributing to seismic hazard for a return period of 2500 years. [Bradwell, Essex]



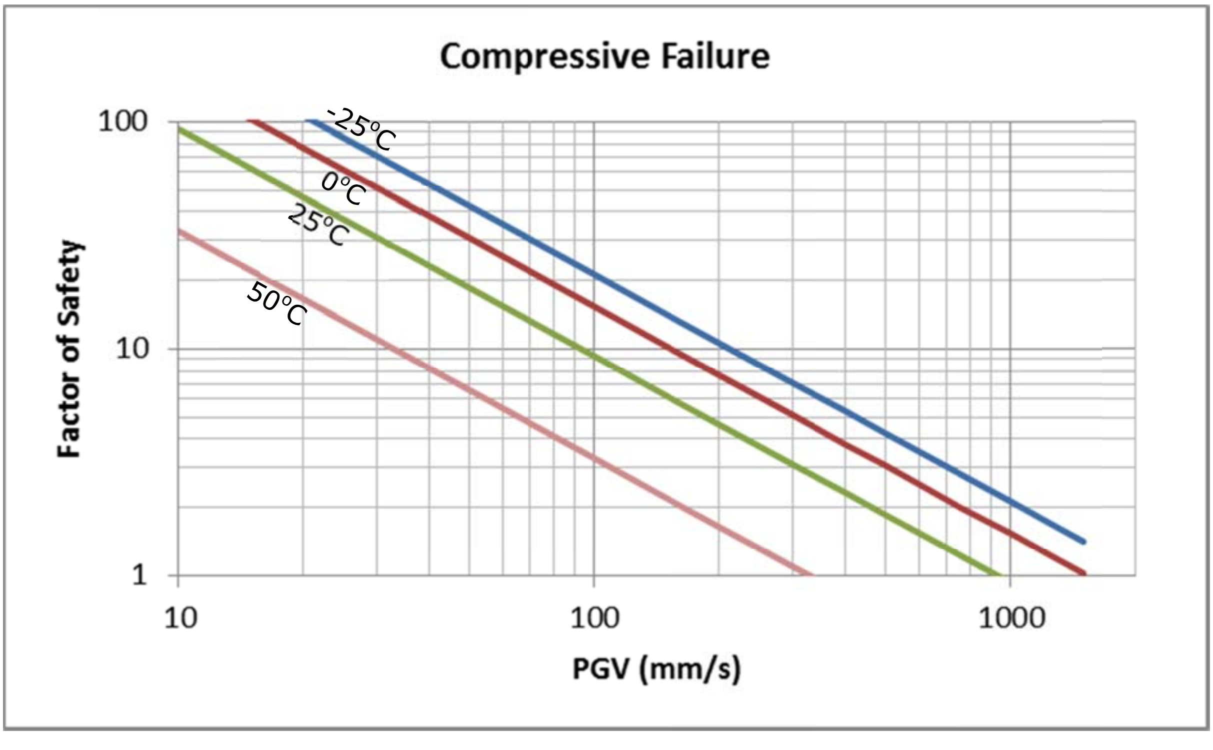


Figure 18	Indicative minimum factor of safety against gross section tensile and compressive yield due to seismic wave propagation. [Lines marked with operating temperature].

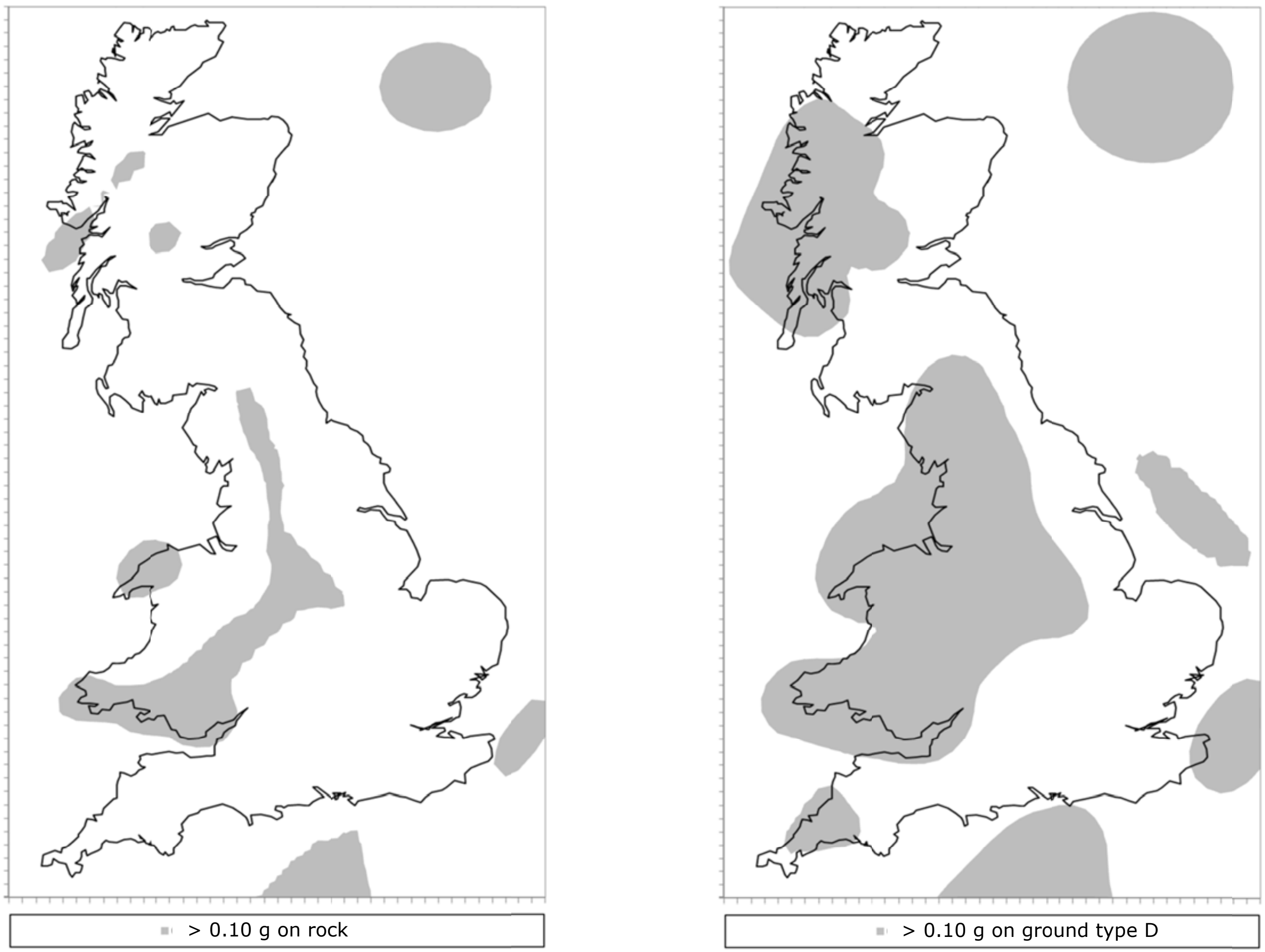


Figure 19	Areas requiring a full seismic design for importance class III natural gas pipelines and installations.

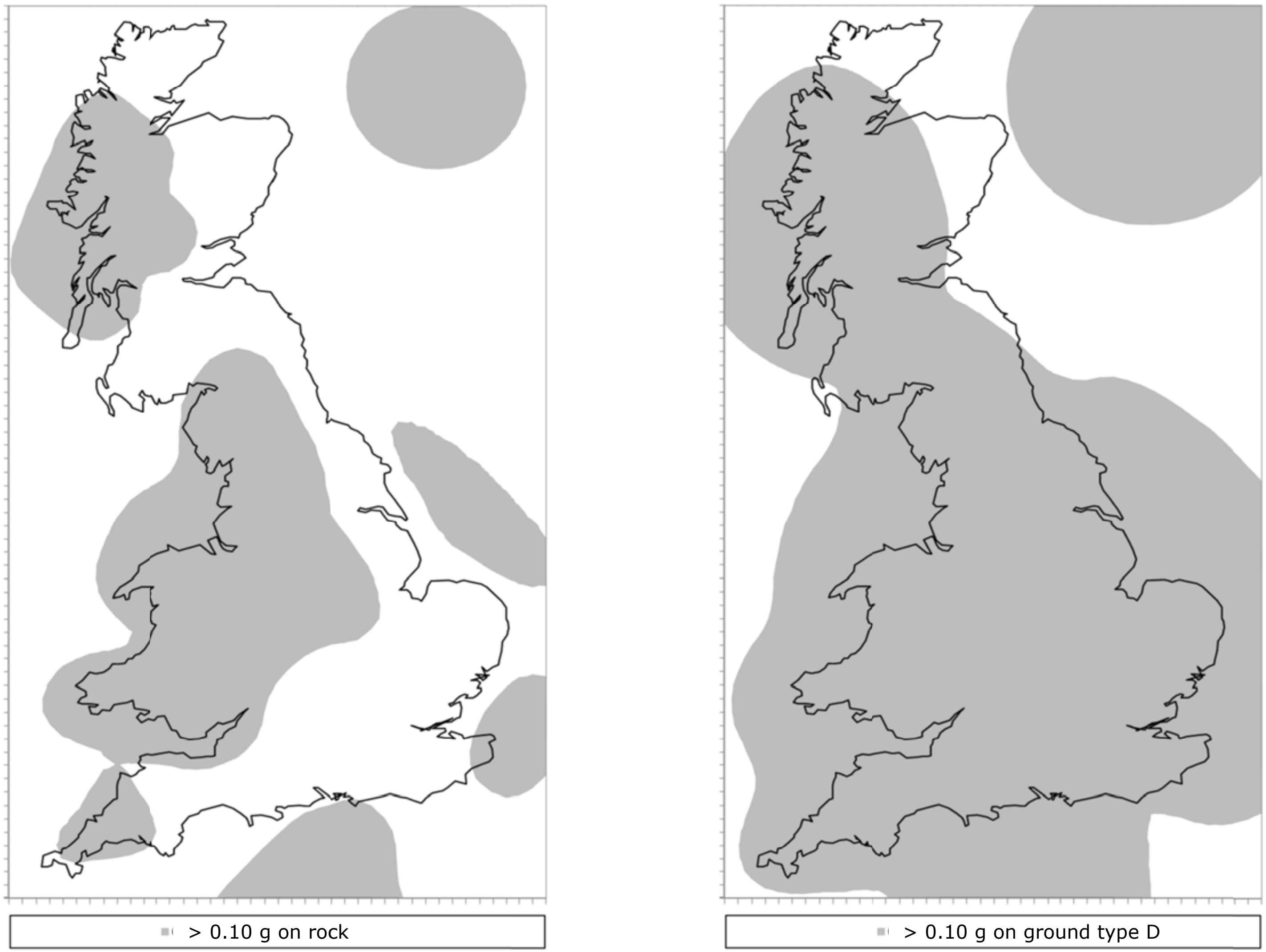


Figure 20	Areas requiring a full seismic design for importance class IV natural gas pipelines and installations.

Appendix I

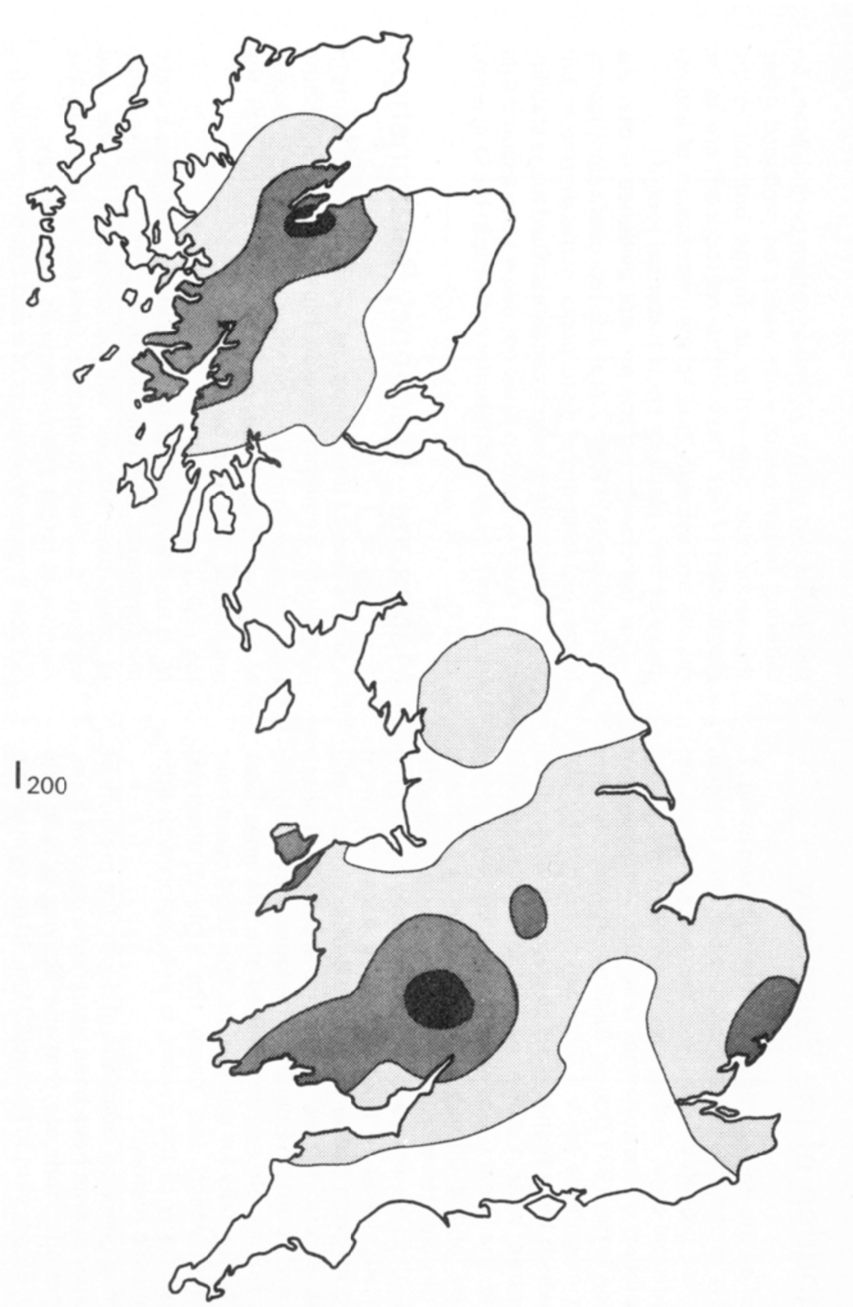
Comparison of Intensity Scales

JMA Intensity	MM Intensity	Description	EMS-98 Intensity	Description	Peak Ground Acceleration (g)	Peak Ground Velocity (mm/s)
0 0.001g	I	Too small to be felt by people. Detected by instruments.	I	Not felt.	< 0.002	< 1
I	II	Felt only by persons at rest. People at the top of buildings may notice slight shaking. Delicately suspended items may begin to swing.	II	Felt only by very few individual people at rest in houses.	0.002-0.014	1-11
	III	Felt quite noticeably by persons indoors. Similar to the passing of a small truck. Hanging objects set in motion.	III	Felt indoors by a few people. People at rest feel a swaying or light trembling.		
II 0.01g	IV	Felt indoors by many, outdoors by few during the day. At night, some awakened. Similar to the passing of a heavy truck. Cars may rock, cups and plates rattle.	IV	Felt indoors by many people, outdoors by very few. A few people are awakened. Windows, doors and dishes rattle.	0.014-0.039	11-34
III	V	Felt by nearly everyone; many awakened. Liquids slop out of cups and glasses. Small objects may fall over. Some windows broken.	V	Felt indoors by most, outdoors by few. Many sleeping people awake. A few are frightened. Buildings tremble throughout. Hanging objects swing considerably. Small objects are shifted. Doors and windows swing open or shut.	0.039-0.092	34-81
IV 0.1g	VI	Felt by all. People likely to be frightened and run out of their houses, people walk unsteadily as if on a moving ship, pictures fall off walls. Some heavy furniture moved. A few instances of fallen plaster. Damage slight.	VI	Many people are frightened and run outdoors. Some objects fall. Many houses suffer slight non-structural damage like hair-line cracks and fall of small pieces of plaster.	0.092-0.18	81-160
V	VII	Difficult to remain standing, furniture breaks. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures. Some chimneys broken. Small slides and caving along sand or gravel banks.	VII	Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many well-built ordinary buildings suffer moderate damage: small cracks in walls, fall of plaster, parts of chimneys fall down; older buildings may show large cracks in walls and failure of fill-in walls.	0.18-0.34	160-310
	VIII	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.	VIII	Many people find it difficult to stand. Many houses have large cracks in walls. A few well-built ordinary buildings show serious failure of walls, while weak older structures may collapse.	0.34-0.65	310-600
VI	IX	People likely to panic, animals run around. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Underground pipes broken.	IX	General panic. Many weak constructions collapse. Even well-built ordinary buildings show very heavy damage: serious failure of walls and partial structural failure.	0.65-1.24	600-1160
	X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent. Large landslides, water thrown from rivers.	X	Many ordinary well-built buildings collapse.	>1.24	>1160
VII	XI	Roads, railway lines and underground services destroyed. Few, if any (masonry) structures remain standing. Bridges destroyed. Large cracks in the ground. Rock falls.	XI	Most ordinary well-built buildings collapse, even some with good earthquake resistant design are destroyed.		
1g	XII	Damage total. Lines of sight and level are distorted. Objects thrown into the air.	XII	Almost all buildings are destroyed.		

Appendix II

Seismicity and seismic hazard in Britain.
[R.C. Lilwall. Institute of Geological Sciences. 1976]

Lilwall's seismic hazard map for a return period of 200 years:



The depths of shading on the hazard map give MM intensities up to 5, 5-6, 6-7, and over 7.

Lilwall gives examples of seismic risk for differing situations of seismic activity in Britain as follows:

- ‘Background’ UK seismic risk (away from Great Glen, Comrie, Menstrie, South Wales and Herefordshire).
- Seismic risk near an ‘active’ centre (South Wales-Herefordshire area).
- Seismic risk along the Great Glen Fault.

The reported values of Modified Mercalli intensity (MMI), maximum ground acceleration (PGA) and maximum ground velocity (PGV) for a return period of 10,000 years are:

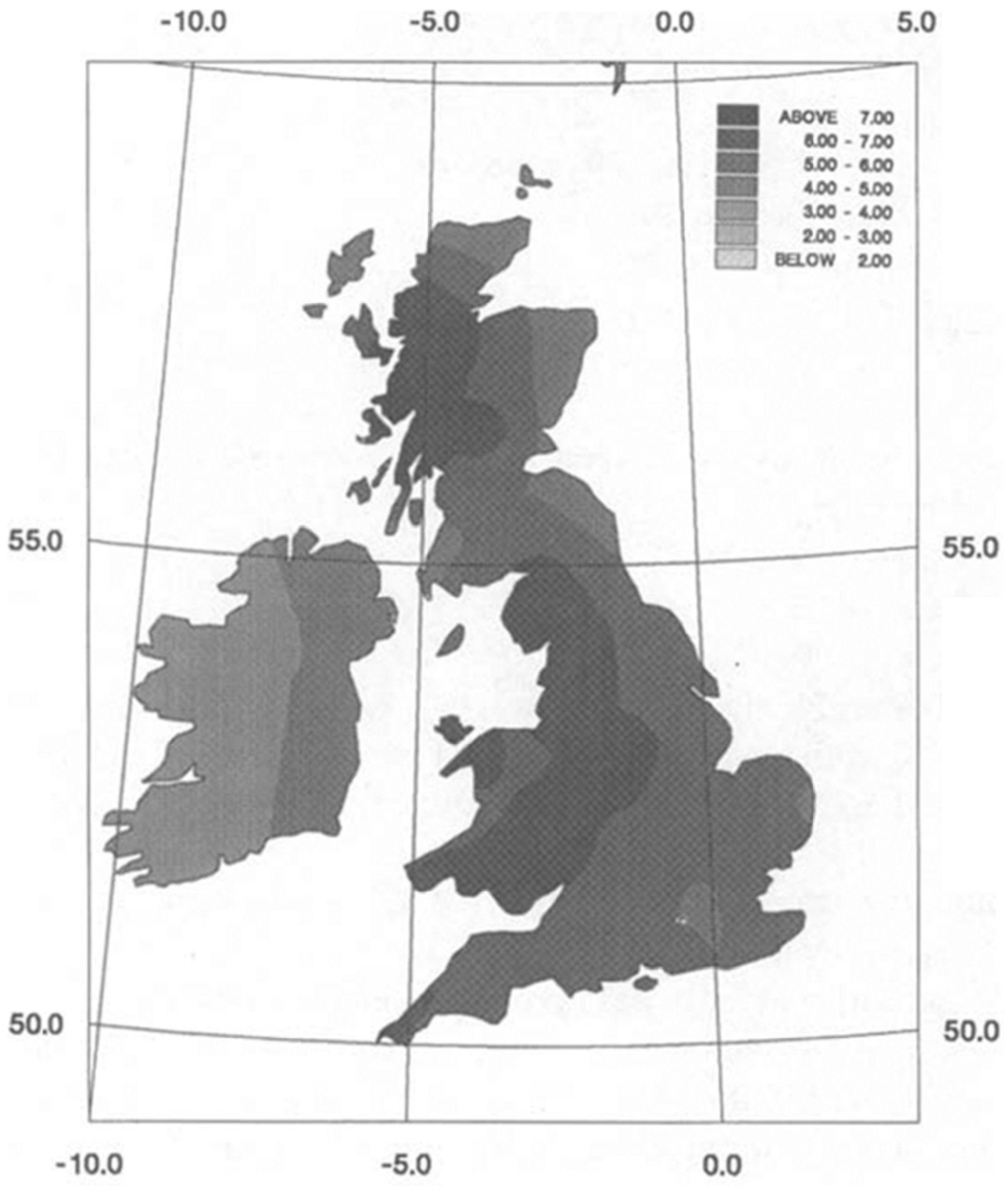
Hazard Parameter	‘Background’	South Wales & Herefordshire	Great Glen Fault
MMI	8	9	~10#
PGA (g)	0.2	>0.2 [0.2 for 2500 years]	>>0.2 [0.2 for 400 years]
PGV (mm/s)	150	>200 [200 for 2500 years]	~500 [500 for 7500 years]

indicative [MM intensity=9 for 1,500 years and 10 for 20,000 years]

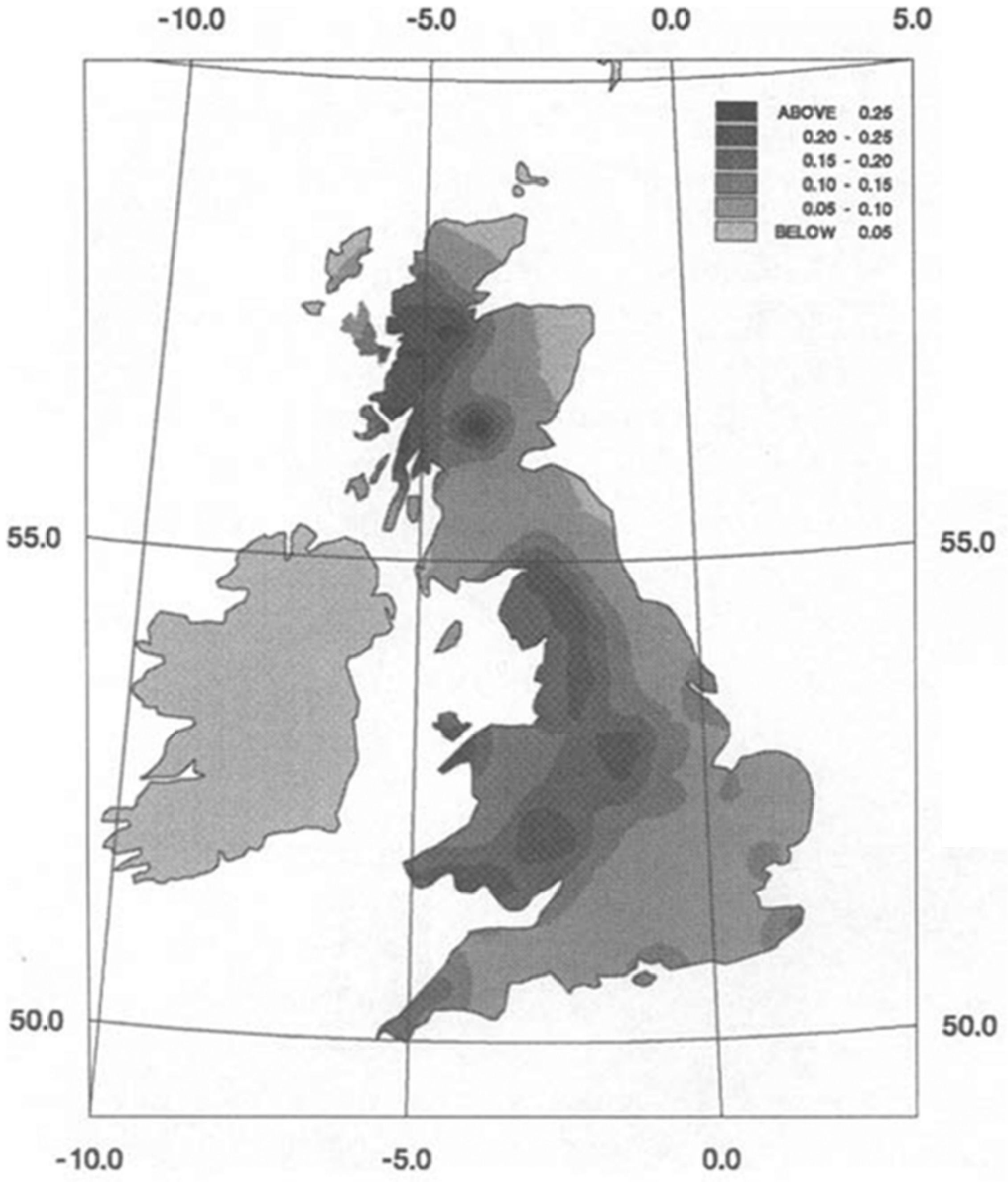
Appendix III

Seismic Hazard Maps for the U.K.
[R.M.W. Musson and P.W. Winter. Natural Hazards 14: 141-154, 1997. Kluwer Academic Publishers]

Seismic hazard map for the UK showing EMS-98 intensity for a 10% probability of exceedance in 50 years [equivalent to a return period of 475 years].



Seismic hazard map for the UK showing peak ground acceleration values for a 10^{-4} annual probability of exceedance [equivalent to a return period of 10,000 years].



Appendix IV

Seismic Hazard to Steel Pipelines

IV.1 Ground Shaking

Ground shaking is produced by a combination of elastic seismic waves radiating out from the strain energy release at the earthquake source. The shaking is produced by two types of elastic seismic wave: body and surface waves.

Body waves are generated at the earthquake source and travel outwards through interior layers within the Earth's crust. Surface waves are produced by constructive interference of body waves and propagate across the outer layers of the Earth's crust.

Body waves include longitudinal P-waves and transverse S-waves. The P-waves exhibit displacements along the direction of travel of the wave due to alternative compression and dilation of the propagating medium. The P-wave is the fastest seismic wave and can travel through both solids and liquids. The S-waves exhibit particle displacement that is transverse to the direction of travel. The transverse displacement of S-waves is associated with an induced shear stress into the propagating medium. The S-wave cannot propagate through a liquid. S-waves travel through soils and rocks at about half the speed of P-waves.

Surface waves include Rayleigh waves and Love waves. Rayleigh waves travel as a surface ripple by vertical elliptical rolling of particles along the direction of travel. Love waves exhibit a horizontal particle displacement in a direction transverse to the direction of propagation. Surface waves travel at a speed that is about 10% less than S-waves. Surface waves are of low frequency (long wavelength) and may be of long duration and large amplitude.

Raleigh waves originate from P-waves at a distance of ~0.63 of the earthquake focal depth and from S-waves at a distance of ~2.33 of the focal depth[1].

Peak ground motions tend to be dominated by surface waves at distances exceeding about twice the thickness of the earth's crust from the earthquake source[2]. The base of the crust is at an average depth of 30 ± 5 km beneath the British Isles[3].

Seismic waves produce dynamic ground strains which must be accommodated by the pipeline structure.

BS EN 1998-4:2006[4] Annex B suggests a simplified approach to dynamic strain estimation in pipelines due to seismic waves. This is based on the method suggested by Newmark[5]. The maximum longitudinal axial strain in a pipeline is $-\frac{v_{max}}{c}$ and the maximum longitudinal curvature is $\frac{1}{\mathrm{c}^2}.\mathrm{a}_{\max}$, where v_{max} is the peak ground velocity, a_{max} is the peak ground acceleration and c is the apparent seismic wave speed.

A derivation from first principles of the strain equations is presented in Appendix V. This extends the treatment to consider the wave type and the wave propagation direction relative to the pipeline. The influence of soil friction on limiting the maximum level of longitudinal direct dynamic strain is also included.

Assuming coincident propagation of compression and shear waves along the pipeline axis, the maximum combined longitudinal strain is,

$$\varepsilon_{combined} = \epsilon_{axial} + \epsilon_{flexural} = \frac{v_{max}}{c_p} + \frac{a_{max}}{c_s^2}.\frac{D_o}{2}$$

where c_p is the compressional wave velocity
c_s is the shear wave velocity
D_o is the pipe external diameter

Alternatively, O'Rourke & Liu[6] suggest the maximum axial pipe strain may be determined using the ground strain calculated from the S-waves (at 45^{o} to the direction of propagation) or from the R-waves (using the effective propagation velocity).

IV.2 Permanent Ground Movement

IV.2.1 Fault Surface Rupture

Surface rupture may occur when an earthquake event happens on a geological fault with a surface outcrop.

The potential seismic activity of an existing fault depends on the current regional tectonic processes. Geological faults in interplate regions (close to tectonic plate boundaries) are more likely to be active than geological faults within intraplate regions (remote from tectonic plate boundaries). Less than 10% of earthquakes occur in intraplate regions.

The orientation of a pipeline to a geological fault and the type of fault is important in relation to the structural response of a pipeline to fault rupture. Normal faults produce tensile loading due to ground extension and induce longitudinal bending due to overburden loading associated with vertical differential downward movement across the fault trace. Reverse faults produce compressive loading due to ground compression and also induce longitudinal bending due to differential vertical upward displacement across the fault. Strike-slip faults may impose tension or compression into a pipeline depending on the orientation of the fault crossing.

Empirical correlations between fault displacement, rupture length and earthquake magnitude have been developed by Wells & Coppersmith[7] for interplate earthquakes and by Nuttli[8] for intraplate earthquakes.

IV.2.2 Liquefaction[9]

Liquefaction is the loss of shear strength in a material due to porewater pressure levels producing a loss of interparticle stress. The porewater pressure increase is attributed to soil structure collapse under ground shaking.

The types of sediment most prone to liquefaction are saturated loosely compacted clay-free deposits of sand and silts.

Natural sediments most susceptible to liquefaction include recent (less than 10,000 years old):

- Fluvial (water borne in delta, river channel & flood-plains) deposits.
- Colluvial (gravity borne loose debris) deposits.
- Aeolian (wind borne) deposits.

Poorly compacted artificial fills may also be susceptible.

Liquefaction may have the following consequences:

Consequence	Description
Flow failures	These occur on slopes and may involve a flow of liquefied soil at surface or a disrupted slide of intact blocks over a liquefied layer. Flow failures usually involve slope gradients exceeding 3 degrees.
Lateral spreads	These involve the lateral displacement of surficial blocks of soil over a liquefied subsurface layer. Lateral spreads are generally associated with shallow slopes (typically less than 3 degrees).
Ground oscillation	This is associated with liquefaction within a subsurface layer on a flat site. The decoupling of surface strata from the liquefied layer leads to ground fissuring and disruption.
Loss of Bearing Strength	The loss of shear strength in liquefied ground can lead to the bearing capacity failure of structures.
Settlement	This has been attributed to dissipation of excess porewater pressure and loss of material due to sand

	boils (eruption of liquefied sand at surface).
Flotation	Pipelines may be subject to flotation due to buoyancy uplift and negligible overburden resistance in liquefied soil.
Lateral pressure increase on retaining structures	This is attributed to the loss of shear strength in the material behind the retaining structure and the resulting loss of the favourable active earth pressure loading on the structure.

Liquefaction potential depends on two factors:

- Soil susceptibility [soil type, density, watertable level].
- Intensity of seismic ground shaking.

Observational results and assessment[10] suggest that a quantity referred to as the cyclic stress ratio (CSR) must exceed 0.05 for liquefaction to occur regardless of the relative density or fine grained structure of the deposit. The CSR for liquefaction increases with the relative density of the deposit and the fines content. CSR is defined as,

$$CSR = 0.65.PGA.\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right).r_d$$

where PGA is the peak ground acceleration in g
r_d is a stress reduction factor [=1 at surface, 0.9 at 10m]
σ_{vo} is the total vertical overburden pressure
σ'_{vo} is the effective vertical overburden pressure

The critical CSR increases to ~0.1 for sands with a corrected standard penetration test N_{60} value of 10 for earthquakes of magnitude <7.5.

Ambraseys[11] compiled worldwide data from earthquakes to examine the influence of earthquake magnitude and epicentral distance on the development of liquefaction. The data for liquefaction sites associated with earthquakes at a depth not exceeding 30 km is shown in figure IV.1. Ambrasey[11] suggested a bounding curve to the data given by,

$$M_w = -0.31 + 2.65x10^{-8}.R_e + 0.99.\log(R_e)$$

where

R_e is the epicentral distance in cm.

Towhata[12] provides critical grading limits for liquefaction prone sands based on the Japanese Code for Harbour Structures:

- Uniformly graded sands with a D_{50} (50% of particles finer than this diameter) in the range 0.07-0.7 mm.
- Well graded sands with a D_{50} in the range 0.02-1.0 mm.

Sands with critical gradings are not liquefaction prone if the normalised standard penetration blow count is greater than 30[12] (relative density is 'Dense').

Criteria for the identification of liquefaction prone clayey soils are given in PRCI report L51927[10] as follows:

- Grain size finer than 0.005 mm is less than 15%.
- Liquid Limit is less than 35%.
- Water content is greater than 90% of the Liquid Limit.

Wakamatsu[13] found that liquefaction is generally induced by seismic shaking with intensity in excess of V on the Japan Meteorological Agency (JMA) Scale. This is almost equivalent to VIII on the Modified Mercalli (MM) and EMS-98 Intensity Scales. These scales are compared in Appendix II including indicative peak acceleration levels according to JMA[14] and Wald et. al.[15]. These scales would suggest that the peak ground acceleration needs to be above ~0.10 g before liquefaction will be observed.

Lateral spread movement due to liquefaction can be estimated using expressions provided in PRCI report L51927[8].

IV.2.3 Landsliding

Keefer[16] studied data from 40 world-wide earthquakes to determine the characteristics, geologic environments, and hazards of landslides caused by seismic events.

Above a threshold earthquake magnitude for differing types of landslide there are bounds on the distance from the epicentre at which an earthquake of a given magnitude is likely to cause a landslide[16,2], figure IV.2.

The classification of landslides used by Keefer[16] and the identified constraints on minimum slope steepness, earthquake magnitude and seismic intensity level are summarised in Table IV.1.

Using the intensity attenuation model of Musson[17], the implied minimum intensity levels at a 95% confidence level from the Keefer[16] results are:

- ~V for disrupted slides and falls.
- ~VI for coherent slides.

- ~VI for lateral spreads and flows.

PRCI report L51927[10] provides a slope stability chart for 3 soil categories to enable the critical ground acceleration to be determined according to the inclination angle of a slope. The expression for critical acceleration is,

$$A_c = \frac{c'}{\gamma.h} + (1-\lambda).tan\phi'.cos\theta - sin\theta$$

where h is the thickness of the potential landslide
θ is the slope angle
c' is the effective cohesion of the slope material
ϕ' is the effective friction angle of the slope material
$\lambda = \frac{\gamma_w.(h-D_w)}{\gamma.h}$
D_w is the depth to the watertable
γ is the unit weight of the slope material
γ_w is the unit weight of water

An alternative formulation based on a horizontal ground acceleration applied to an infinite slope with seepage, figure IV.3, gives the following expression for critical acceleration,

$$A_c = \frac{\frac{c'}{\gamma.h} + (1-\lambda.cos^2\theta).cos^2\theta.tan\varphi' - sin\theta.cos\theta}{\left(cos^2\theta + sin\theta.cos\theta.\left((1-\lambda.cos^2\theta).tan\varphi'\right)\right)}$$

A conservative estimate of the ground movement level due to an earthquake triggered landslide can be obtained from expressions provided by PRCI report L51927[10].

IV.3 References

1. Okamoto, S. *Introduction to Earthquake Engineering*. 2nd edition. University of Tokyo Press. 1984.

2. Kramer, S.L. *Geotechnical Earthquake Engineering.* Prentice Hall. 1996.

3. Cameron, T.D.J., Crosby, A., Balson, P.S., Jeffery, D.H., Lott, G.K., Bulat, J., & Harrison, D.J. *The geology of the southern North Sea.* United Kingdom Offshore Regional Report. British Geological Survey. 1992.

4. BS EN 1998-4:2006. *Eurocode 8: Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines.* BSI. 29 September 2006.

5. Newmark, N.M. Problems in *Wave Propagation in Soil and Rock.* Proceedings of the International Symposium On Wave Propagation and Dynamic Properties of Earth Materials. University of New Mexico. Albuquergue. New Mexico. 1967.

6. O'Rourke, M.J., & Liu, X. *Seismic Design of Buried and Offshore Pipelines.* Monograph No. 4. MCEER. 2012.

7. Wells, D.L., & Coppersmith, K.J. *New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement.* Bulletin of the Seismological Society of America. 84(4). 1994.

8. Nuttli, O.W. *Average seismic source-parameter relations for mid-plate earthquakes.* Bulletin of the Seismological Society of America. 73(2). 1983.

9. Earthquake Basics Brief No. 1. *Liquefaction. What it is and what to do about it.* Earthquake Engineering Research Institute. January 1994.

10. Honnegger, D.G. & Nyman, D.J. *Guidelines for the Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines*. L51927. Pipeline Research Council International. 1 October 2004.

11. Ambraseys, N.N. *Engineering seismology*. Earthquake Engineering and Structural Dynamics. Vol. 17. Pp. 1-105. 1988.

12. Towhata, I. *Geotechnical Earthquake Engineering*. Springer. 2008.

13. Wakamatsu, K. *Liquefaction History, 416-1990, in Japan*. Fourth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction. Volume 1. NCEER-92-0019. NCEER. 12 August 1992.

14. Japan Meteorological Agency. *The Seismological Bulletin of the Japan Meteorological Agency for January 1992*. Pp. i-ii. 1992.

15. Wald, D.J., Quitoriano, V., Heaton, T.H., & Kanamori, H. *Relationship between Peak Ground Acceleration, Peak Ground Velocity and Modified Mercalli Intensity in California.* Earthquake Spectra. Vol 15. No. 3. 1999.

16. Keefer, D.F. *Landslide caused by earthquakes*. Geological Society of America Bulletin. Vol. 95. Issue 2. 1984.

17. Musson, R.M.W. *Intensity attenuation in the UK*. Journal of Seismology. Vol. 9, 73-86. 2005.

Table IV.1 – Landslide Classification Used by Keefer[16]

<table>
<tr><th>Type</th><th>Material</th><th>Style</th><th>Internal Disruption</th><th>Depth[&]</th><th>Critical Slope Angle</th><th>Minimum Earthquake Magnitude</th><th>Minimum Earthquake Intensity[~]</th></tr>
<tr><td rowspan="6">Disrupted</td><td rowspan="3">Rock</td><td>Falls</td><td rowspan="6">High to Very High</td><td rowspan="2">Shallow</td><td>$>40^{o}$</td><td rowspan="2">4.0 M_L</td><td rowspan="6">VI (P)
IV (L)</td></tr>
<tr><td>Slides[#]</td><td>$>35^{o}$</td></tr>
<tr><td>Avalanches</td><td>Deep</td><td>$>25^{o}$</td><td>6.0 M_s</td></tr>
<tr><td rowspan="3">Soil</td><td>Falls</td><td rowspan="3">Shallow</td><td>$>40^{o}$ $</td><td rowspan="2">4.0 M_L</td></tr>
<tr><td>Slides[#]</td><td>$>15^{o}$</td></tr>
<tr><td>Avalanches</td><td>$>25^{o}$</td><td>6.5 M_s</td></tr>
<tr><td rowspan="5">Coherent</td><td rowspan="2">Rock</td><td>Slumps[*]</td><td rowspan="5">Slight to Moderate</td><td rowspan="2">Deep</td><td>$>15^{o}$</td><td rowspan="2">5.0 M_L</td><td rowspan="6">VII (P)
V (L)</td></tr>
<tr><td>Block slides[#]</td><td>$>15^{o}$</td></tr>
<tr><td rowspan="3">Soil</td><td>Slumps[*]</td><td rowspan="2">Deep</td><td>$>10^{o}$</td><td rowspan="2">4.5 M_L</td></tr>
<tr><td>Block slides[#]</td><td>$>5^{o}$</td></tr>
<tr><td>Earth flows[#]</td><td>Shallow</td><td>$>10^{o}$</td><td>5.0 M_L</td></tr>
<tr><td rowspan="2">Lateral Spreads & Flows</td><td rowspan="2">Soil</td><td>Spread</td><td>Moderate</td><td>Variable</td><td>$>0.3^{o}$</td><td rowspan="2">5.0 M_L</td></tr>
<tr><td>Flow</td><td>Very High</td><td>Shallow</td><td>$>2^{o}$</td><td>VII (P)
V (L)</td></tr>
</table>

[&] shallow is ~<3 metres.
[#] translational on basal shear surface.
[*] rotational on basal shear surface.
$ suggested criterion value [actual data suggested $>63^{o}$].
~ Modified Mercalli Scale. (P) indicates predominant minimum, (L) indicates lowest minimum.

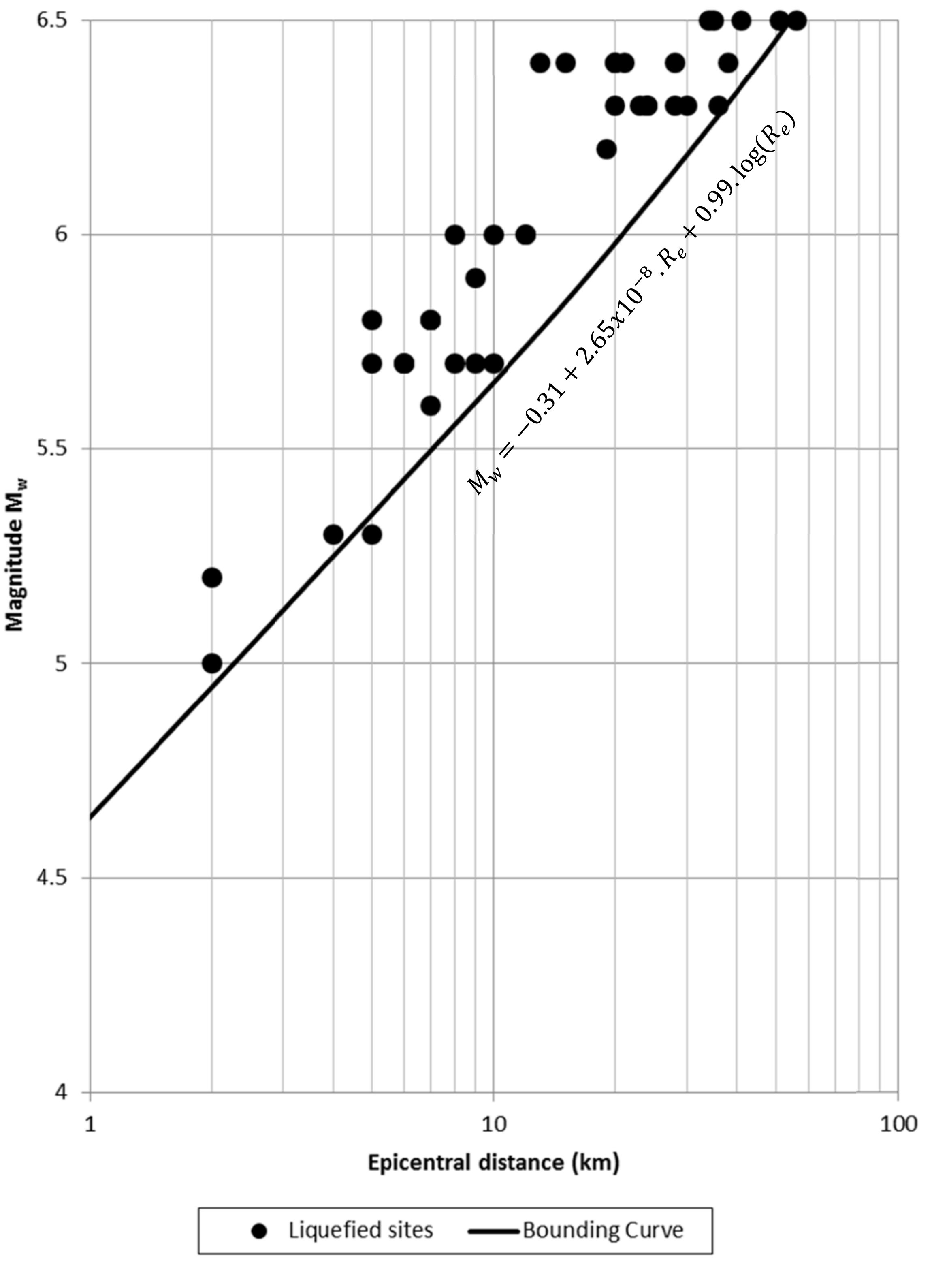


Figure IV.1	Maximum epicentral distance to liquefaction sites according to Ambraseys[11]. [note: R_e in cm. for bounding curve].

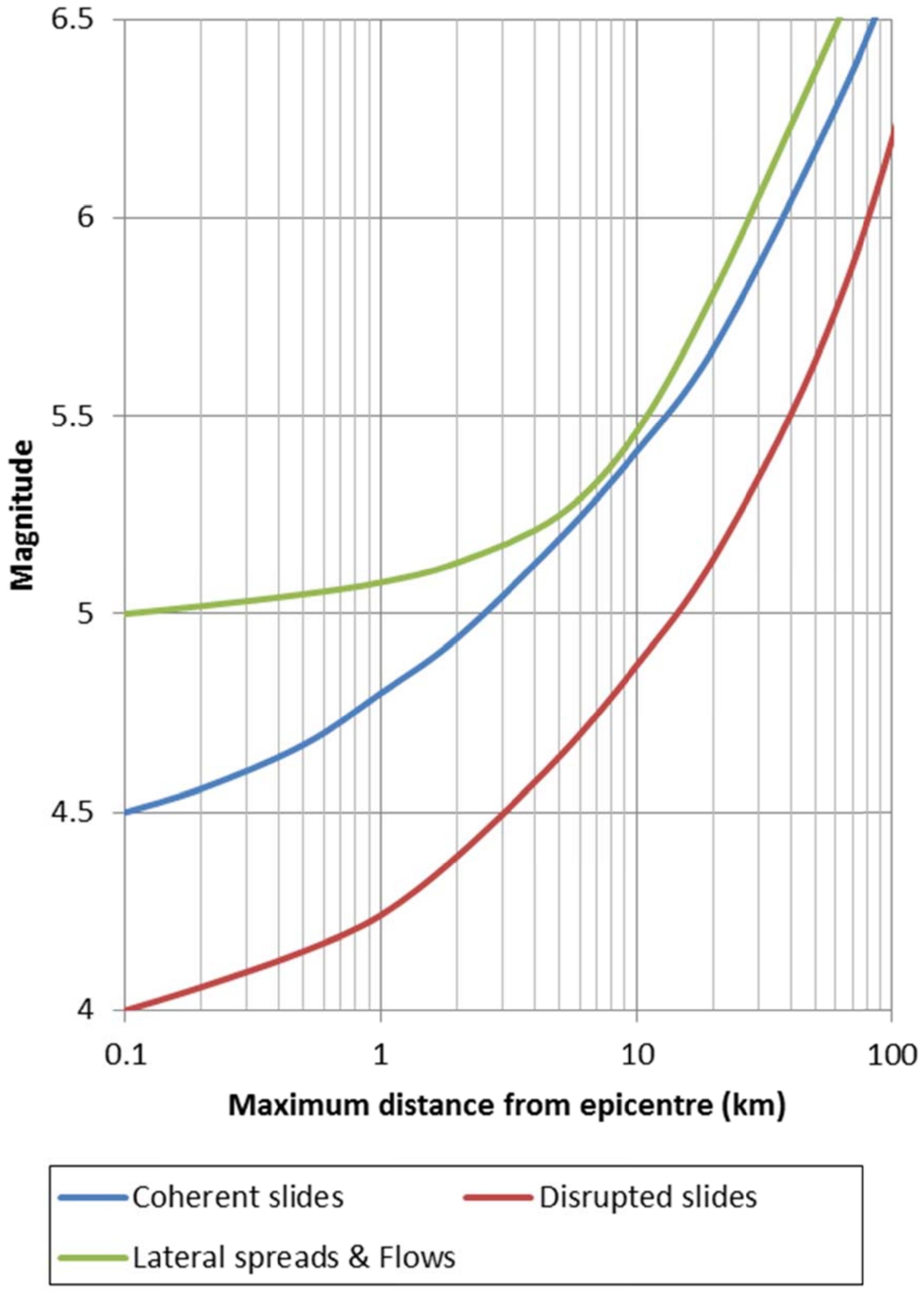


Figure IV.2	Maximum epicentral distance to different types of landslide according to Keefer[16]. [see Table IV.1 for landslide types]

Tangential force down slope is,

$$T = W.sin\theta + a.W.cos\theta = \gamma.h.1.sin\theta + a.\gamma.h.1.cos\theta$$

Tangential stress down slope is,

$$\tau = \gamma.h.sin\theta.cos\theta + a.\gamma.h.cos^2\theta$$

Normal total force on slip surface,

$$N = W.cos\theta - a.W.sin\theta$$

Normal total stress on slip surface,

$$\sigma = \gamma.h.cos^2\theta - a.\gamma.h.sin\theta.cos\theta$$

Porewater pressure on slip surface,

$$u = (h - D_w).\gamma_w.cos^2\theta$$

Normal effective stress on slip surface,

$$\sigma' = (\gamma.h - (h - D_w).\gamma_w.cos^2\theta).(cos^2\theta - a.sin\theta.cos\theta)$$

Shearing resistance on slip surface,

$$\tau_f = c' + \sigma'.tan\varphi'$$

Factor of safety,

$$F = \frac{\tau_f}{\tau} = \frac{c' + (\gamma.h - (h - D_w).\gamma_w.cos^2\theta).(cos^2\theta - a.sin\theta.cos\theta).tan\varphi'}{\gamma.h.sin\theta.cos\theta + a.\gamma.h.cos^2\theta}$$

For F=1, a=A_c,

$$A_c = \frac{\frac{c'}{\gamma.h} + (1 - \lambda.cos^2\theta).cos^2\theta.tan\varphi' - sin\theta.cos\theta}{\left(cos^2\theta + sin\theta.cos\theta.\left((1 - \lambda.cos^2\theta).tan\varphi'\right)\right)}$$

where $\lambda = \frac{(h-D_w).\gamma_w}{\gamma.h}$

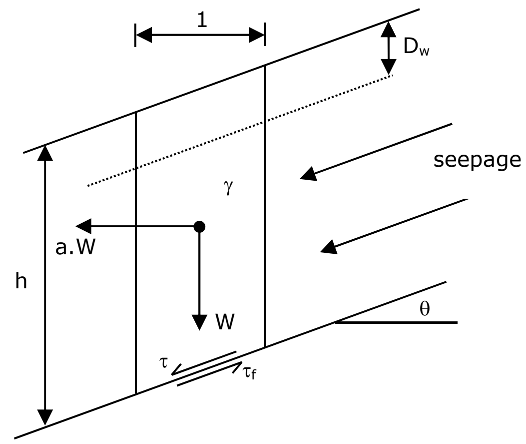


Figure IV.3 Infinite slope with seepage

Appendix V

V.1. Ground Strain & Curvature

A simple harmonic progressive wave, figure V.1, can be represented by the expression for particle displacement by:

$$\mathrm{u} = \mathrm{u}_{\mathrm{max}}.\sin 2\pi\left(\frac{\mathrm{t}}{\mathrm{T}} - \frac{\mathrm{x}}{\lambda}\right)$$

where $\mathrm{u}_{\mathrm{max}}$ is the maximum particle displacement
t is the time
T is the wave period [time for one complete cycle]
x is the distance in the direction of wave propagation
λ is the wavelength of the wave

The particle velocity is given by the following expression:

$$\frac{du}{dt} = \frac{2\pi.\mathrm{u}_{\mathrm{max}}}{T}.\cos 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right)$$

The peak particle velocity is obtained when: $\cos 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right) = 1$ giving,

$$v_{max} = \frac{2\pi.\mathrm{u}_{\mathrm{max}}}{T}$$

Since the wave takes T seconds (the period) to cover a distance of one wavelength (λ) the propagation velocity of the wave is,

$$c = \frac{\lambda}{T}$$

The particle acceleration is given by the following expression:

$$\frac{d^2u}{dt^2} = \left(\frac{2\pi}{T}\right)^2.\mathrm{u}_{\mathrm{max}}.\sin 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right)$$

The peak particle acceleration is obtained when: $\sin 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right) = 1$ giving,

$$a_{max} = \left(\frac{2\pi}{T}\right)^2.\mathrm{u}_{\mathrm{max}}$$

For the case where the particle displacement is in the direction of wave propagation (e.g. a dilatational/compressive body wave [P wave]) the longitudinal ground strain in the direction of wave propagation is:

$$\frac{du}{dx} = -\frac{2\pi.\mathrm{u_{max}}}{\lambda}.\cos 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right)$$

The maximum longitudinal ground strain is obtained when: $\cos 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right) = 1$ giving,

$$\varepsilon_{g_{max}} = -\frac{2\pi.\mathrm{u_{max}}}{\lambda}$$

Noting that $c_c = \frac{\lambda}{T}$, where c_c is the compressive-wave velocity,

$$\varepsilon_{g_{max}} = -\frac{2\pi.\mathrm{u_{max}}}{c_c.T} = -\frac{v_{max}}{c_c} \qquad 1$$

The compressive-wave velocity is obtained from the constrained modulus, M_s, and the mass density, ρ, of the elastic medium,

$$c_c = \sqrt{\frac{E_s.(1-\nu)}{\rho.(1+\nu).(1-2.\nu)}}$$

where

E_s is the Young's modulus of the elastic medium [$M_s = \frac{E_s.(1-\nu)}{(1+\nu).(1-2.\nu)}$]
ν is the Poisson's ratio for the elastic medium

For the case where the particle displacement is perpendicular to the direction of wave propagation (e.g. a shear body wave [S-wave]) the longitudinal ground curvature in the direction of wave propagation is:

$$\frac{d^2u}{dx^2} = \kappa = \left(\frac{2\pi}{\lambda}\right)^2.\mathrm{u_{max}}.\sin 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right)$$

The maximum longitudinal ground curvature is obtained when: $\sin 2\pi\left(\frac{t}{T} - \frac{x}{\lambda}\right) = 1$ giving,

$$\kappa = \left(\frac{2\pi}{\lambda}\right)^2.\mathrm{u_{max}}$$

Noting that $c_s = \frac{\lambda}{T}$, where c_s is the shear-wave velocity,

$$\kappa = \frac{1}{c_s^2}.\left(\frac{2\pi}{\mathrm{T}}\right)^2.\mathrm{u_{max}} = \frac{1}{c_s^2}.\mathrm{a_{max}} \qquad 2$$

The shear-wave velocity is obtained from the shear modulus, G_s $[= \frac{E_s}{2.(1+\nu)}]$, and the mass density, ρ, of the elastic medium,

$$c_s = \sqrt{\frac{E_s}{\rho.2.(1+\nu)}}$$

Equations 1 & 2 were derived by Newmark[1] and form the basis for strain determination in buried pipelines due to ground shaking associated with seismic waves.

Estimated seismic wave propagation velocities according to Dowding[2] are:

Ground Type	Wave Velocity (m/s)	
	Compressive c_c	Shear c_s
Limestone	2000-5900	1000-3100
Metamorphic rocks	2100-3500	1000-1700
Basalt	2300-4500	1100-2200
Granite	2400-5000	1200-2500
Sand	500-2000	250-850
Clay	400-1700	200-800

O'Rourke & Liu[3,4] indicate that the horizontal propagation velocity of seismic S-waves must be considered in estimating the horizontal ground strain along a pipeline. For inclined waves arriving from the focal point of the earthquake the apparent horizontal propagation velocity is,

$$c_h = \frac{c_s}{\sin \Upsilon}$$

where Υ is the incidence angle from the vertical, figure V.2.

O'Rourke & Liu[3,4] report observed apparent horizontal propagation velocities for S-waves of 2100 to 5300 m/s (average of ~3400 m/s) in generally soft sediments.

The direction of the seismic wave propagation in the horizontal plane relative to the alignment of the pipeline is also important as determined by Yeh[5] and CGLFL[6].

The longitudinal ground strain along the direction of a pipeline due to a seismic dilatational/compressive body wave (P-wave) with an incidence angle of α with the pipeline, figure V.3, is given by the following expression,

$$\varepsilon_g = -\frac{v_{max}.\cos^2\alpha}{c_h}$$

The longitudinal ground strain along the alignment of the pipeline is a maximum when the P-wave is propagating in a direction parallel to the pipeline i.e. α=0, giving,

$$\varepsilon_{g_{max}} = -\frac{v_{max}}{c_h}$$

The ground curvature along the direction of a pipeline due to a seismic dilatational/compressive body wave (P-wave) with an incidence angle of α with the pipeline, figure V.3, is given by the following expression,

$$\kappa = \frac{1}{\left(\frac{\mathrm{c_h}}{\cos\alpha}\right)^2}.\,\mathrm{a_{max}}.\sin\alpha = \frac{\mathrm{a_{max}}.\cos^2\alpha.\sin\alpha}{\mathrm{c_h}^2}$$

The maximum ground curvature occurs for a P-wave incidence angle of 35 degrees giving,

$$\kappa_{max} = \frac{\mathrm{a_{max}}}{2.6.\,\mathrm{c_h}^2}$$

The longitudinal ground strain along the direction of a pipeline due to a seismic translational body wave (S-wave) with an incidence angle of α with the pipeline, figure V.4, is given by the following expression,

$$\varepsilon_g = -\frac{v_{max}.\sin\alpha.\cos\alpha}{c_h}$$

The longitudinal ground strain along the alignment of the pipeline is a maximum when the S wave is propagating at an incidence angle of 45 degrees i.e. α=45^{o}.

The maximum longitudinal ground strain for an S wave is,

$$\varepsilon_{g_{max}} = -\frac{v_{max}}{2.\,c_h}$$

The ground curvature along the direction of a pipeline due to a seismic transverse body wave (S-wave) with an incidence angle of α with the pipeline, figure V.4, is given by the following expression,

$$\kappa = \frac{1}{\left(\frac{\mathrm{c_h}}{\cos\alpha}\right)^2}.\,\mathrm{a_{max}}.\cos\alpha = \frac{\mathrm{a_{max}}.\cos^3\alpha}{\mathrm{c_h}^2}$$

The maximum ground curvature occurs for an S-wave incidence angle of 0 degrees giving,

$$\kappa_{max} = \frac{\mathrm{a_{max}}}{\mathrm{c_h}^2}$$

Surface seismic waves e.g. Raleigh (R-waves) exhibit components of particle motion both in the direction of and transverse to the direction of wave propagation.

O'Rourke & Liu[3,4] demonstrate that the apparent propagation phase velocity of R-waves is dependent on the wave frequency and the shear wave velocity increase with depth. In the simple case of a uniform weaker surface layer, the R-wave propagation velocity is unaffected by the weaker layer if the R-wave wavelength is large (low frequency) relative to the layer thickness. The R-wave phase velocity is slightly less than the shear wave velocity of the stiffer ground (below the weaker surface layer). However, if the wavelength of the R-wave is comparable to or smaller than the thickness of the surface layer then the phase velocity is slightly less than the shear wave velocity of the layer.

O'Rourke & Liu[3,4] indicate that the longitudinal ground strain due to an R-wave is determined from the expression,

$$\varepsilon_{g_{max}} = -\frac{v_{max}}{c_R}$$

where c_R is the propagation velocity of the R-wave.

The propagation velocity, c_R, is taken as the phase velocity corresponding to a wavelength equal to two to four times the separation distance between the stations of interest.

The ground strain and curvature for surface waves is estimated based on the most adverse expressions derived for body waves.

A summary of the expressions for maximum ground strain and curvature for each wave type is presented below:

Wave Type	Direct Strain	Curvature
Dilatational/Compressive [P wave]	$\varepsilon_{g_{max}} = -\frac{v_{max}}{c_h}$	$\kappa_{max} = \frac{\mathrm{a_{max}}}{2.6.\mathrm{c_h}^2}$
Transverse [S wave]	$\varepsilon_{g_{max}} = -\frac{v_{max}}{2.c_h}$	$\kappa_{max} = \frac{\mathrm{a_{max}}}{\mathrm{c_h}^2}$
Surface [Raleigh]	$\varepsilon_{g_{max}} = -\frac{v_{max}}{c_R}$	$\kappa_{max} = \frac{\mathrm{a_{max}}}{\mathrm{c_R}^2}$

Note that c_R is the phase velocity

V.2. Pipe Strain & Curvature

The imposed direct strain in a pipeline is limited by the load transfer capability between the ground and the pipe through soil friction. The maximum longitudinal pipe strain is,

$$\varepsilon_{p_{max}} = -\frac{\pi . D_o . \tau . \lambda^*}{4. A_o . E_p}$$

where D_o is the external diameter of the pipe
A_o is the cross sectional area of the pipe
E_p is the Young's modulus of the pipe material
τ is the maximum soil frictional shear stress
λ^* is the apparent wavelength of the seismic wave

The minimum soil frictional shear stress to impose a pipe strain equal to the ground strain is obtained from,

$$\pi . D_o . \tau = A_o . E_p . \frac{d^2 u}{dx^2}$$

$$\tau = t . E_p . \frac{a_{max}}{c_h^2}$$

where t is the wall thickness of the pipe

V.3. References

1. Newmark, N.M. *Problems in Wave Propagation in Soil and Rock.* Proceedings of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials. University of Mexico. 23-25 August 1967.

2. Dowding, C.H. *Construction Vibrations*. Prentice Hall. 1996.

3. O'Rourke, M.J., & Liu, X. *Response of Buried Pipelines Subject to Earthquake Effects.* Monograph No. 3. MCEER. 1999.

4. O'Rourke, M.J., & Liu, X. *Seismic Design of Buried and Offshore Pipeline.* Monograph No. 4. MCEER. 2012.

5. Yeh, G. *Seismic Analysis of Slender Buried Beams.* Bulletin of the Seismological Society of America. Volume 64. No. 5. 1974.

6. *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems.* Committee on Gas and Liquid Fuel Lifelines. ASCE. 1984.

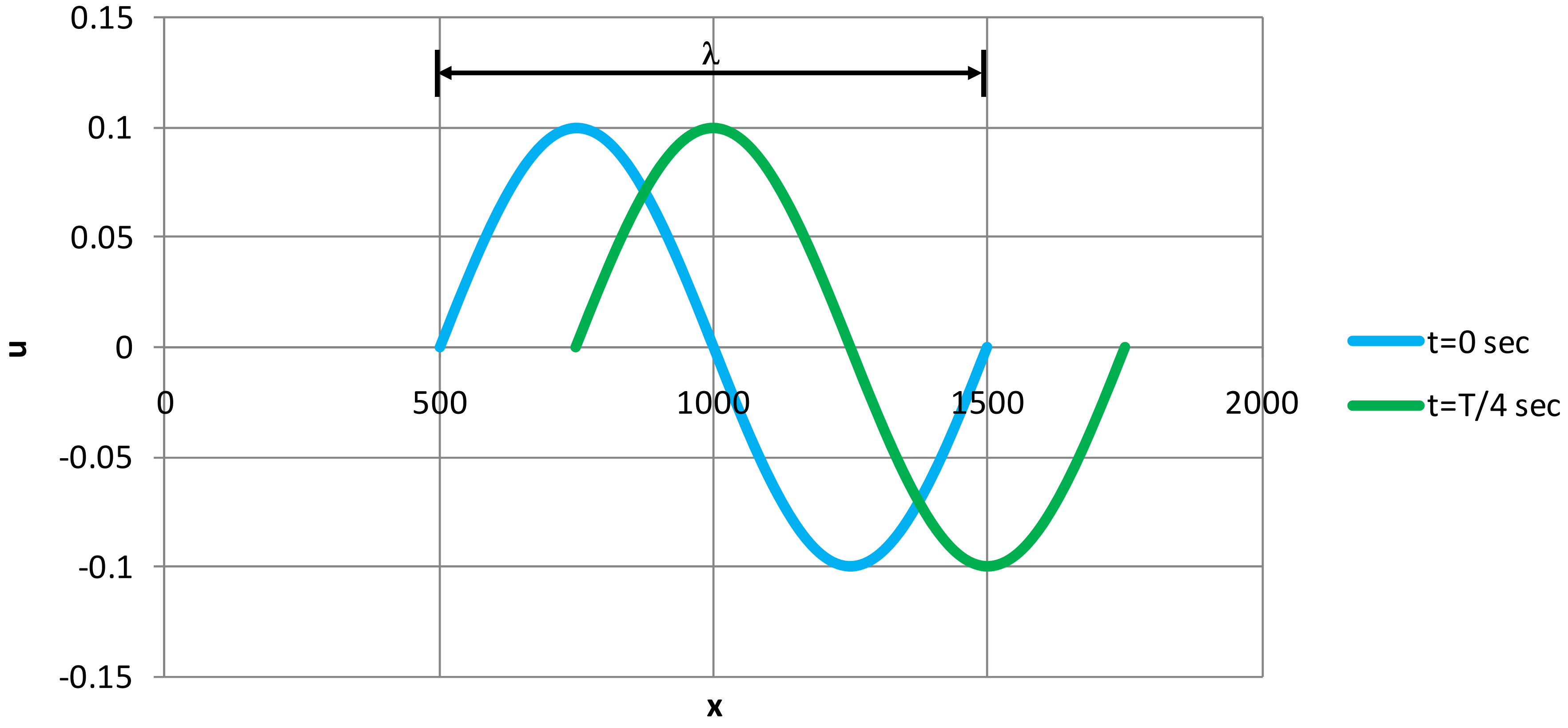


Figure V.1	Simple harmonic wave at two time points

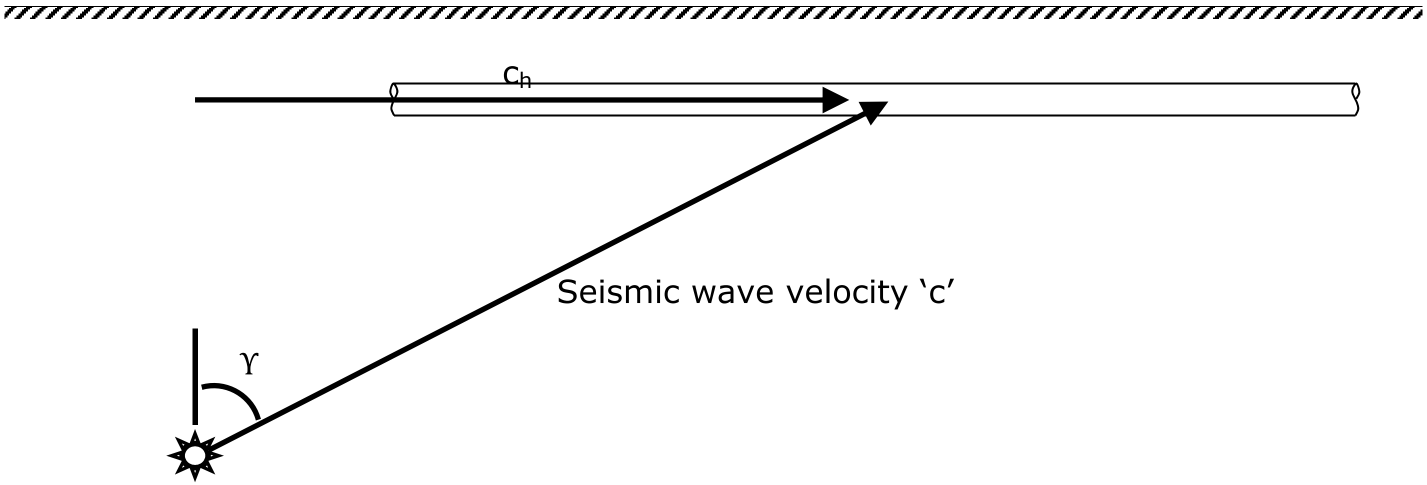


Figure V.2	Apparent horizontal velocity of seismic waves

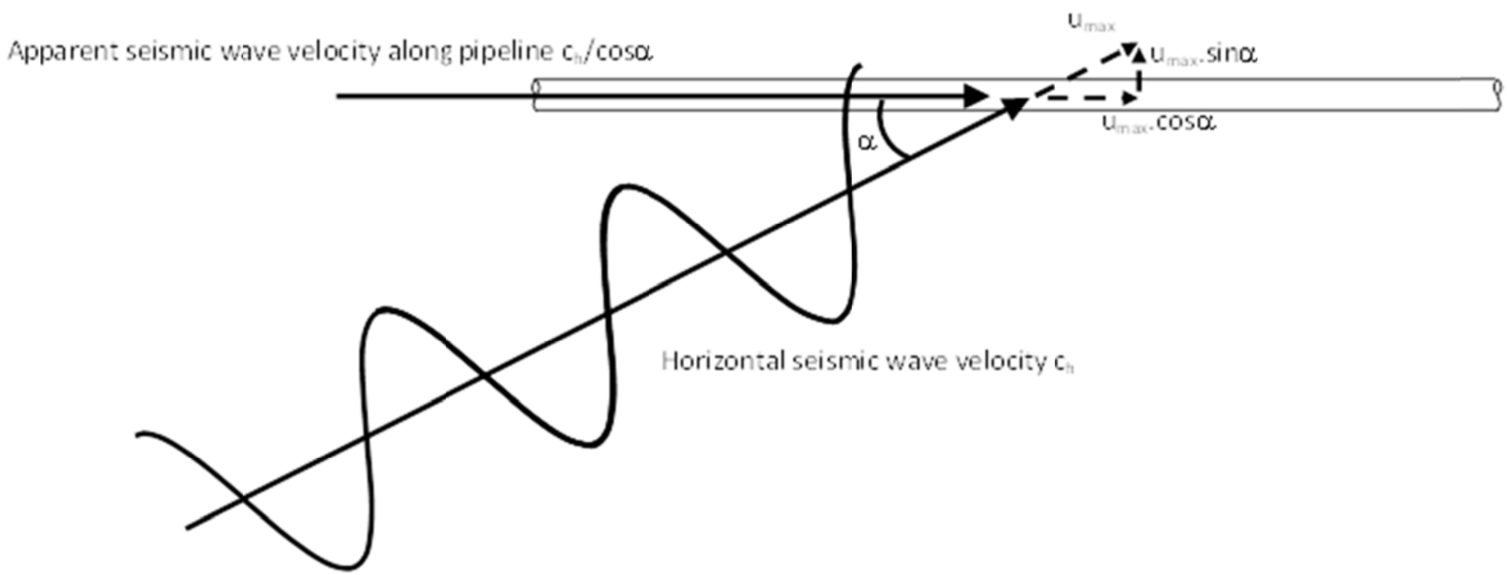


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Figure V.3	Apparent horizontal propagating velocity of seismic waves and reduced particle movement for P-waves

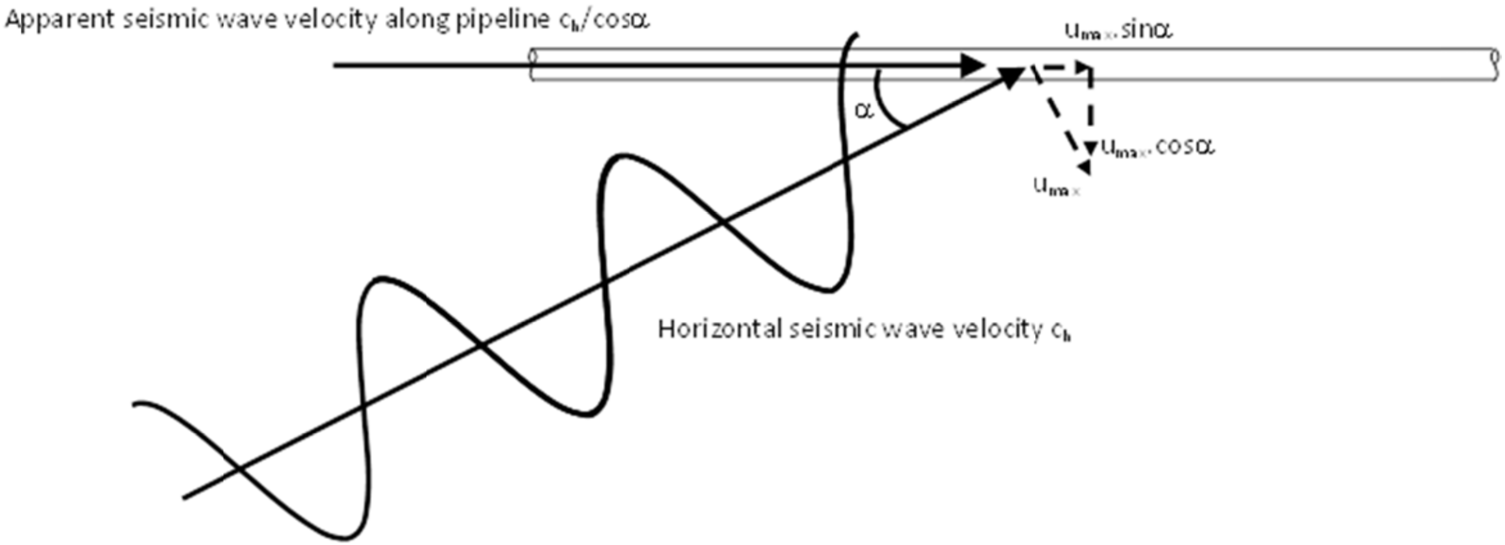

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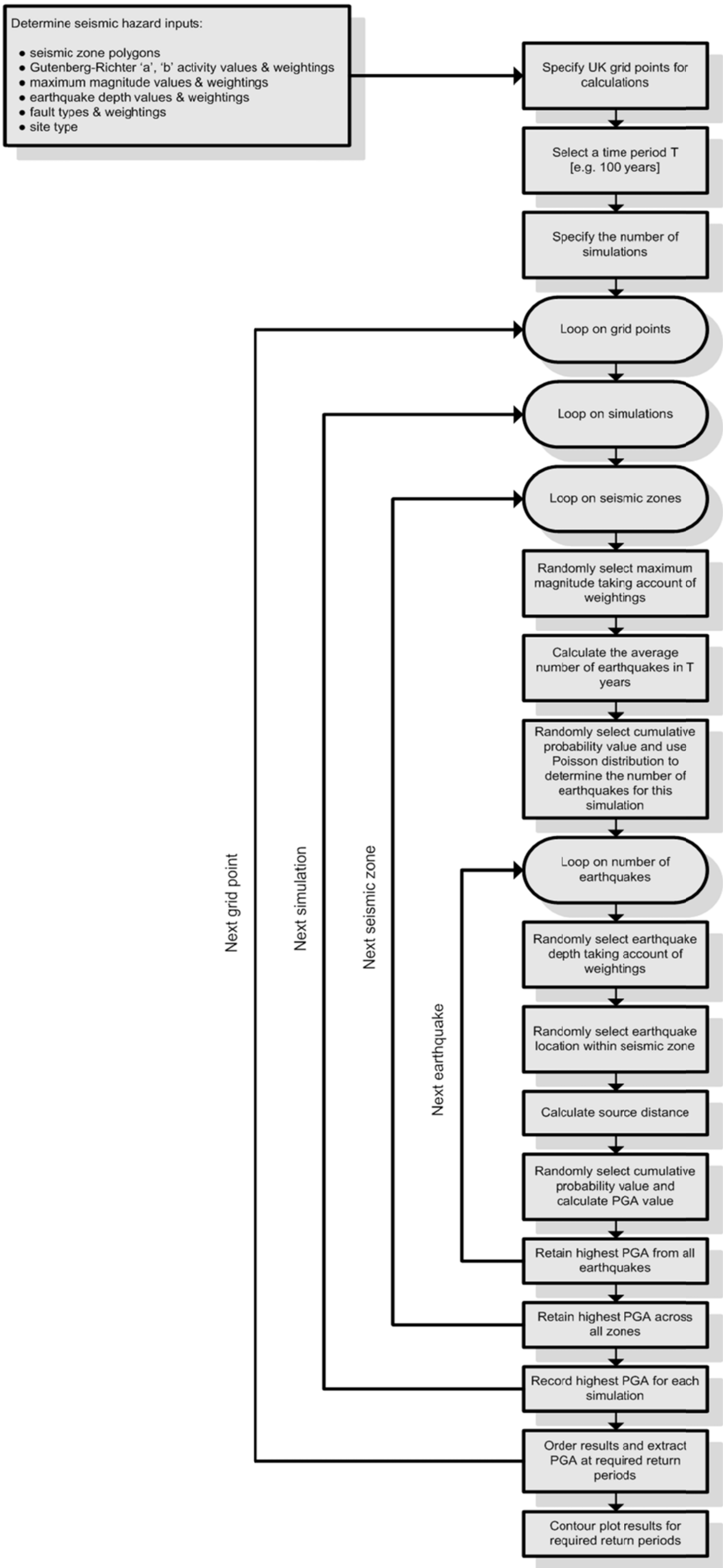
Figure V.4	Apparent horizontal propagating velocity of seismic waves and reduced particle movement for S-waves

Appendix VI

Flow chart for probabilistic seismic hazard analysis using a Monte Carlo approach



Appendix VII

Performance of Steel Pipelines in Past Earthquakes

Earthquake	Year	Magnitude	Focal depth (km)	Maximum Intensity	Pipeline Performance: Damage & Survival	Pipeline Performance: Welds	References
Long Beach	1933	6.3 M_L	~9.7	VII/VIII [MM]	5 minor & 10 major breaks on welded steel transmission pipelines.	Unknown	O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994.
					46 breaks on large diameter (460-510mm) steel distribution mains.	poor	
Kern County	1952	7.3 M_W	~16	VIII	3 weld failures on 12 inch pipeline.	Oxy-acetylene.	O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994.
					1 weld failure on 12 inch pipeline.	Reinforced oxy-acetylene.	
					No damage to 26 inch pipeline.	Oxy-acetylene & reinforced oxy-acetylene.	
					No damage to 22 inch pipeline.	Unknown	
					No damage to 34 inch post WWII pipeline at ~1 metre fault offset.	Electric arc.	
					2 upheaval buckles on 6 inch pipeline but no weld failures.	Oxy-acetylene.	
Alaska	1964	9.2 M_W	~25	VII	1 leak in 12 inch pipeline due to crack at weld	Unknown	Eckel, E.B. *Effects of the Earthquake Of March 27, 1964, on Air and Water Transport, Communications, and Utilities Systems in South-Central Alaska.* Paper 545-B. USGA. 1967. USGS Isoseismal Map: http://earthquake.usgs.gov/earthquakes/states/events/1964_03_28_iso.php
				~VIII	Two breaks in 12 inch gas pipeline due to landslide.	Unknown	
				VII	No damage to a crude oil pipeline.	Unknown	
				VI	No damage to a multi-product pipeline.	Unknown	
Niigata	1964	7.5 M_L	~40	VII/VIII [MM] V [JMA]	No damage to high pressure steel natural gas pipeline.	Unknown	Okamoto, S. *Introduction to Earthquake Engineering*. University of Tokyo Press. 1984.
San Fernando	1971	6.4 M_L 6.6 M_W	~8.4	X/XI [MM]	Numerous breaks at welds on 12 inch pipeline.	Oxy-acetylene.	McDonough, P.W. (editor). *Seismic Design Guide for Natural Gas Distributors.* Monograph No. 9. ASCE. August 1995. Ballantyne, D. *Oil and Gas Pipelines*. USGS Open File Report 2008-1150. O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994. Ariman, T. & Muleski, G.E. *A Review of the Response of Buried Pipelines Under Seismic Excitations.* Lifeline Earthquake Engineering – Buried Pipelines, Seismic Risk, and Instrumentation. PVP-34. ASME. 1979. O'Rourke, T.D., Roth, B.L., & Hamada, M. *Large Ground Deformations and Their Effects on Lifeline Facilities: 1971 San Fernando Earthquake.* Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes. Volume 2. United States Case Studies. NCEER-92-0002. 17 February 1992.
					52 breaks in 16 inch pipeline and shell buckling in vicinity of fault crossing.	Oxy-acetylene.	
					Damage at 7 locations on 26 inch pipeline within zone of lateral spreading.	Electric arc.	
					Damage at 3 locations on 26 inch pipeline.	Oxy-acetylene.	
					No damage on 22 inch and 30 inch pipelines due to ~3 metres of lateral deformation.	Modern.	
					Breaks on 12 inch pipeline.	Oxy-acetylene.	
					1 break adjacent to 19 degree	Electric arc	

					elbow on 12 inch pipeline.	welds on double bell joints.	
					1 break on 22 inch pipeline	Electric arc welds on double bell joints.	
					Damage at bend, slip joint and mechanical coupling in 49 inch pipeline due to liquefaction induced ground movement	Electric arc welded slip joints.	
					Tensile girth weld failure in 6 inch pipeline at edge of landslide.	Electric arc.	
					No reported damage to 15 inch pipeline due to ~3 metres of lateral deformation.	Electric arc.	
					Damage to 59 inch pipeline at two mechanical couplings due to liquefaction related ground movement.	Electric arc welded slip joints.	
Imperial Valley	1979	6.6 M_L 6.5 M_W	~9.7	VII [MM]	4 inch pipeline survived ~0.6m. horizontal fault offset.	Oxy-acetylene.	Ballantyne, D. *Oil and Gas Pipelines*. USGS Open File Report 2008-1150. O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994.
					8 inch pipeline survived a ~0.4m. horizontal and ~0.05m. vertical fault offset.	Electric arc.	
					10 inch pipeline survived a ~0.3m. horizontal and ~0.05m. vertical fault offset.	Electric arc.	
Coalinga	1983	6.5 M_W	~8	VIII [MM]	Numerous gas leaks associated with building collapse. One downtown gas main failed to hold pressure during a post-earthquake pressure test.	Unknown	McDonough, P.W. (editor). *Seismic Design Guide for Natural Gas Distributors.* Monograph No. 9. ASCE. August 1995.
Michoacán	1985	8.0 M_W	~28	IX [MM]	Multiple axial buckles due to seismic waves on 42 inch welded steel water main.	Unknown	Ayala, A.G., & O'Rourke, M.J. *Effects of the 1985 Michoacan Earthquake on Water Systems and Other Buried Lifelines in Mexico.* NCEER-89-0009. 1989.
North Palm Springs	1986	5.9 M_L 6.1 M_W	~12	VII [MM]	No damage or disruption occurred on three 30-36 inch gas transmission pipelines. Some breaks on other water and gas lines.	Unknown	O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994. USGS: http://earthquake.usgs.gov/earthquakes/states/events/1986_07_08.php
Whittier Narrows	1987	5.9 M_L	10-15	VIII [MM]	No damage to the high pressure gas transmission system.	Unknown	McDonough, P.W. (editor). *Seismic Design Guide for Natural Gas Distributors.* Monograph No. 9. ASCE. August 1995.
Loma Prieta	1989	7.1 M_L 6.9 M_W	~18	IX [MM]	3 leaks on transmission pipelines.	Unknown	Ballantyne, D. *Oil and Gas Pipelines*. USGS Open File Report 2008-1150. McDonough, P.W. (editor). *Seismic Design Guide for Natural Gas Distributors.* Monograph No. 9. ASCE. August 1995. Eguchi, R.T., & Seligson, H.A. *Lifeline Perspective. Practical Lessons from the Loma Prieta Earthquake.* CETS. 1994.
Sierra Madre	1991	5.8 M_L 5.6 M_W	~11	VII [MM]	No reported damage to natural gas or oil pipelines.	Unknown	*Sierra Madre Earthquake of June 28, 1991.* EERI Special Earthquake Report. 1991.
Landers	1992	7.3 M_W	~1	VIII/IX [MM]	No damage occurred to 6 inch and 36 inch gas transmission pipelines.	Unknown	O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994.

Northridge	1994	6.7 M_W	~19	IX/X [MM]	24 breaks at girth welds and 1 buckle on 12 inch pipeline.	Oxy-acetylene.	Ballantyne, D. *Oil and Gas Pipelines*. USGS Open File Report 2008-1150. McDonough, P.W. (editor). *Seismic Design Guide for Natural Gas Distributors.* Monograph No. 9. ASCE. August 1995. O'Rourke, T.D., & Palmer, M.C. *The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines.* NCEER-94-0011. 1994. O'Rourke, M.J., & Liu, X. *Response of Buried Pipelines Subject to Earthquake Effects.* Monograph No. 3. MCEER. 1999.
					1 rupture at a weld on a 26 inch pipeline.	Oxy-acetylene.	
					1 fractured girth weld on a 15 inch pipeline.	Oxy-acetylene.	
					No failures on a 22 inch pipeline.	Unknown	
					1 break at a 19 degree vertical bend on a 10 inch pipeline due to slope movement.	Electric arc.	
					No damage to a 26 inch pipeline.	Oxy-acetylene.	
					Tensile failure of a 22 inch pipeline.	Electric arc.	
					No damage to a 24 inch pipeline.	Electric arc.	
					No damage to two 30 inch pipelines.	Unknown	
					No damage to a 16 inch pipeline.	Unknown	
					Failures in tension and compression on 49 inch pipeline.	Welded slip joints.	
					Failures in tension and compression on 68 inch pipeline.	Welded slip joints.	
					Failures in tension and compression on 6 inch pipeline.	Unknown	
Kobe	1995	6.8 M_W	~16	~XI/XII [MM] VII [JMA]	No failures on gas transmission pipelines.	Electric arc.	Schiff, A.J. *Hyogoken-Nanbu (Kobe) earthquake of January 17, 1995: Lifeline Performance.* ASCE. 1998. Shinozuka, M. (editor). *The Hanshin-Awaji Earthquake of January 17, 1995. Performance of Lifelines.* NCEER-95-0015. 1995.
					14 failures of girth welds on pre-1962 pipelines.	Poor quality.	
					No failures on post 1962 pipelines.	Modern.	
Kocaeli	1999	7.4 M_W	~17	~IX/X [MM] X [MSK]	No reported damage to gas and oil pipelines or gas distribution mains.	Unknown	Erdik, M. *Report on 1999 Kocaeli and Duzce (Turkey) Earthquakes.* BOĞAZİÇİ UNIVERSITY. 2000.
San Simeon	2003	6.5 M_W	~8	VIII [MM]	An 8 inch pipeline cracked in a weld at a mitre joint.	Gas welded.	Lund, Le Val. *Lifeline Performance, San Simeon Earthquake, December 22, 2003*. ASCE. 2004.
					A 2 inch distribution main suffered damage.	Unknown	
					No damage to 4 inch, 6 inch and 10/12 inch butt-welded pipelines.	Unknown	