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1 **Evaluation of seismic hazard for the assessment of historical elements at risk: description of**
2 **input and selection of intensity measures**

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10 **Abstract** The assessment of historical elements at risk from earthquake loading presents a
11 number of differences from the seismic evaluation of modern structures, for design or retrofitting
12 purposes, which is covered by existing building codes, and for the development of fragility curves,
13 procedures for which have been extensively developed in the past decade. This article briefly
14 discusses: the hazard framework for historical assets, including a consideration of the appropriate
15 return period to be used for such elements at risk; the intensity measures that could be used to describe
16 earthquake shaking for the analysis of historical assets; and available approaches for their assessment.
17 We then discuss various unique aspects of historical assets that mean the characterisation of
18 earthquake loading must be different from that for modern structures. For example, historical buildings
19 are often composed of heterogeneous materials (e.g. old masonry) and they are sometimes located
20 where strong local site effects occur due to: steep topography (e.g. hilltops), basin effects or
21 foundations built on the remains of previous structures. Standard seismic hazard assessment
22 undertaken for modern structures and the majority of sites is generally not appropriate. Within the
23 PERPETUATE project performance-based assessments, using nonlinear static and dynamic analyses
24 for the evaluation of structural response of historical assets, were undertaken. The steps outlined in
25 this article are important for input to these assessments.

26 **Keywords** Seismic hazard assessment, site effects, intensity measures, fragility curves, historical
27 buildings, cultural heritage assets, monuments

28 **1. Introduction**

29
30 As for modern structures, an evaluation of the seismic vulnerability of historical elements at risk¹
31 requires that earthquake loading be defined in an appropriate manner. Historical assets are generally
32 greatly different from the type of structures covered by current seismic design codes (e.g. Eurocode 8,
33 EC8) and they present great intra-group variation. Therefore, it is necessary to carefully consider what
34 description(s) of earthquake loading needs to be considered for which type of historical asset. The
35 hazard framework (e.g. in terms of return periods and deterministic or probabilistic approaches)
36 prescribed in current design codes may not be appropriate for historical assets. This is because of, for
37 example: their cultural and artistic importance and the acceptable level of retrofitting and
38 strengthening considering archaeological and architectural constraints. The purpose of this article is to
39 briefly summarise the assessment of hazard for historical assets. Because it is the only article in this
40 special issue specifically covering hazard assessment it seeks to provide a background for those

¹ This article (and the PERPETUATE project) concerns historical buildings, such as those found in the centres of many towns in Europe, monuments (e.g. statues) and building contents (e.g. art works). In the following we use the general term 'historical assets' to mean all these types of elements at risk.

41 readers not familiar with the state of the art in this field and for reference purposes for other articles in
42 this volume.

43 The following section discusses the hazard framework used for historical assets within performance-
44 based assessments. Hazard is commonly assessed in terms of intensity measures (IMs, also called
45 strong-/ground-motion parameters) – a brief overview of the most common IMs and their estimation is
46 given in the subsequent section. Section 4 is based on the classification of historical assets proposed in
47 the PERPETUATE project (Lagomarsino et al., 2010; Lagomarsino et al., 2011; Lagomarsino and
48 Cattari, 2013) and discusses the descriptions of seismic hazard that need to be considered for assets
49 falling into each class. Because of overlaps between the descriptions required for each class this
50 section is generally structured in terms of characteristics of earthquake actions (i.e. concerning the
51 hazard) rather than in terms of asset classes (i.e. concerning the vulnerability/exposure). This is done
52 by contrasting the requirements for the evaluation of historical assets to the requirements for modern
53 elements at risk.

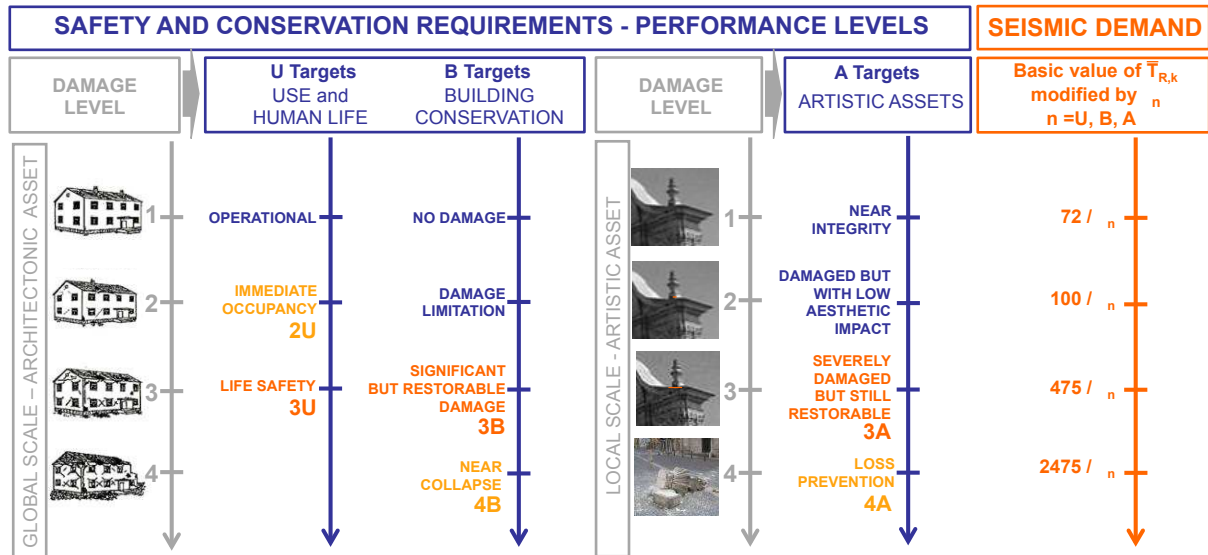
54 **2. Appropriate hazard framework for historical assets**

55
56 The current practice of selecting the appropriate seismic load with a given probability of exceedance
57 (e.g. 10%) in a certain period of time (e.g. 50 years) is not generally appropriate for historical assets.
58 Checking a structure with this principle may lead to the requirement to undertake substantial
59 retrofitting and strengthening, which could change its appearance and, consequently, its
60 archaeological, architectural, historical and artistic value. Many historical monuments are already
61 damaged and in many cases it is better to design retrofitting measures accepting a controllable degree
62 of damage, respecting mainly the serviceability and usability of the asset (perhaps simply preventing
63 collapse). On the other hand, historic buildings that are still in permanent use and occupancy (e.g.
64 historical city centres) should have the same level of safety as modern structures, which means that
65 their seismic resistance should follow the same principles as modern structures as defined by building
66 codes. Furthermore, for historical town centres, the spatial correlation of ground-motion fields and the
67 cross correlation of different scenarios must be taken into account. This is generally done by
68 stochastically simulating sets of earthquakes (Goda and Atkinson, 2009; Jayaram and Baker, 2009).
69 This procedure is still being developed and, hence, it is not discussed further. The hazard framework
70 should cover both types of structures with respect to their usability, i.e. monuments with no or limited
71 occupancy and historic buildings with permanent use and occupancy.

72 Hazard levels are associated with predefined probabilities of exceedance in a reference interval (e.g.
73 10% in 50 years) or, equivalently, predefined return periods (e.g. 475 years). Moreover, the seismic
74 hazard is associated with the reliability of existing buildings. Hazard levels thus defined are then
75 further modified through a coefficient depending on the importance of the building considered. In
76 particular, importance is mainly related to the requirement that the structure remains operational after
77 an earthquake.

78 By relating required performance targets and hazard levels, four considered return periods (T_R) are
79 proposed for each type of asset (Figure 1, Lagomarsino and Cattari, 2013). To take into account the
80 varying importance and significance of each asset, the use of the following three values, to modify the
81 return periods, is proposed: *Use coefficient* (γ_U), *Building coefficient* (γ_B) and *Artistic coefficient* (γ_A).
82 These factors are defined as a function of the building use, its cultural and historical value and the
83 presence of relevant artistic assets in the building (Lagomarsino and Cattari, 2013). Three independent
84 coefficients are required because of the great variety of cultural heritage assets. Some highly important
85 structures, from the historical and architectural point of view, are rarely used. In this case, it is

86 necessary to prevent collapse, while life safety and immediate occupancy are not priorities. On the
 87 contrary, less important architectural structures may be strategic or public buildings, for which use
 88 and safety performance are fundamental. Moreover, sometimes artistic assets are located in buildings
 89 that present no particular relevance from the architectural point of view; in these cases, the artistic
 90 coefficient can increase the seismic hazard for the verification of the artistic assets performance.



91

92 **Fig. 1** Damage levels, performances and related return periods in years
 93 (Lagomarsino and Cattari, 2013).

94 These coefficients can be lower than unity when a particular performance is considered less important
 95 than average. In general, these coefficients can range from 0.5 to 2.0. Table 1 shows the return periods
 96 and the corresponding probability of exceedance in the reference period of 50 years associated to the
 97 different hazard levels. It is worth noting that H1 is very rarely considered for historical assets.

98 **Table 1** Return periods in years (upper values) and probability of exceedance in 50 years (lower values) to be
 99 considered for the different earthquake hazard levels, as a function of the importance and significance coefficient
 100 (Lagomarsino and Cattari, 2013).

		Importance and significance coefficient				
		0.5	0.7	1	1.5	2
Earthquake Hazard Level	H1	25, 86%	35, 76%	50, 63%	75, 49%	100, 39%
	H2	36, 75%	50, 63%	72, 50%	108, 37%	144, 29%
	H3	238, 19%	333, 14%	475, 10%	713, 7%	950, 5%
	H4	1238, 4%	1733, 2.8%	2475, 2%	3713, 1.3%	4950, 1%

101 **3. IMs for historical assets**

102

103 The assessment of seismic hazard for a given site is often performed for one (or more) IMs, which
 104 seek to characterise earthquake ground motions as a single (or multiple) scalar value(s). Since
 105 earthquake shaking is a complex non-stationary (both in the time and frequency domains)

106 phenomenon, its characterisation as a single number is obviously a great simplification. This
107 simplification, however, makes seismic hazard assessment much more straightforward since the link
108 between earthquake (event) parameters, such as magnitude and location, and site parameters can be
109 expressed as a closed-form equation [ground-motion prediction equations (GMPE), also known as
110 attenuation relations] to estimate the probability of exceeding a given level of earthquake shaking.
111 These probabilities are a vital input to probabilistic seismic hazard assessment (PSHA). The
112 connection back to contributing earthquakes can be made through disaggregation of: PSHA results
113 (e.g. Bazzurro and Cornell, 1999) or macroseismic intensity observations (e.g. Pace et al., 2011), to
114 obtain the magnitude and distance of the most important scenarios and, thereafter, to choose
115 appropriate accelerograms from a databank or simulate ground motions consistent with these
116 scenarios. There are many techniques for the simulation of earthquake ground motions (see, e.g.,
117 Douglas and Aochi, 2008, for a review) but such techniques cannot be currently used within PSHA
118 without passing through the step of assessing the probability of exceeding a given level of a certain
119 IM. Therefore, IMs are fundamental within seismic hazard assessment and the definition of earthquake
120 scenarios.

121 3.1. Previously proposed IMs

122

123 Many dozens of IMs have been proposed in the literature to capture different aspects of earthquake
124 shaking, e.g.: amplitude, duration, frequency content, energy content and shape, or various
125 combinations of these characteristics. The most commonly-used IMs are introduced below.

126 The IMs that are most often used to characterise earthquake shaking are: peak ground acceleration
127 (PGA, the maximum absolute ground acceleration), peak ground velocity (PGV, the maximum
128 absolute ground velocity), peak ground displacement (PGD, the maximum absolute ground
129 displacement) and response spectral ordinates for a linear-elastic lightly-damped (often 5% of critical)
130 single-degree-of-freedom (SDOF) system. ‘Response spectral ordinate’ usually refers to spectral
131 acceleration (SA, or pseudo-spectral acceleration, PSA), which is the maximum absolute acceleration
132 that the mass of the SDOF system experiences during the earthquake shaking. Spectral displacement
133 (SD), defined as the maximum absolute relative displacement experienced by the mass during the
134 shaking, is, however, becoming increasingly employed due to the advent of displacement-based design
135 (e.g. Priestley et al., 2007). Response spectral velocity (SV, or pseudo-spectral velocity, PSV), the
136 maximum absolute relative velocity experienced by the mass during the shaking, is rarely used in
137 earthquake engineering because it is difficult to relate to design parameters. PSV is, however, used to
138 plot response spectra using the tripartite representation, where PSA, PSV and SD are all shown on a
139 single graph using special logarithmic axes. These different response spectral measures (SA, SV and
140 SD) are all highly correlated, except at very short and long periods, and PSA and PSV equal SD
141 multiplied by $(2\pi/T)^n$, where $n=1$ for PSV and 2 for PSA and T is the natural period of the SDOF
142 system. Response spectra provide a convenient means of summarizing the peak response of all
143 possible linear SDOF systems to a particular component of ground motion.

144 Response spectral ordinates can also be defined for different types of SDOF systems, such as those
145 with elastic-perfectly plastic force-deformation properties, which can be defined in terms of constant
146 strength or constant ductility. Other force-deformation functions (e.g. strain hardening and strain
147 softening) are also sometimes used. Inelastic spectral ordinates are often estimated from elastic
148 spectral ordinates by conversion formulae (e.g. Miranda and Bertero, 1994).

149 Standard response spectra in terms of SA and SD can be used to estimate the forces and deformations
150 that a given structure will be subjected to during shaking. Some researchers consider the maximum

151 absolute energy input into the system (e.g. Chapman, 1999). This property can also be plotted in terms
152 of spectra for different periods and damping. These IMs remain, however, solely a research topic
153 because of the difficulty of relating them to structural damage.

154 Spectral IMs defined in terms of systems with different periods, such as those described above,
155 provide information on the frequency content of the earthquake shaking, which is important because
156 structures have natural periods varying from a few tenths of a second (single-story building) to a few
157 seconds (mainly Class C in the PERPETUATE classification, e.g. tall towers) and, therefore, they will
158 feel the shaking differently. There are many IMs, however, that do not explicitly account for frequency
159 content. The most obvious of these is PGA, although this equals SA and PSA for an infinitely-stiff
160 structure ($T=0s$) and it is strongly correlated to short-period (roughly $<0.2s$) SA and PSA. To capture
161 the frequency content of ground motions in terms of a single IM many versions of the spectral
162 intensity (Housner, 1959), based on the integral of spectral ordinates over a given period range, have
163 been proposed.

164 Response spectra (or other types of spectra, e.g. Fourier amplitude) provide a good representation of
165 the frequency content of earthquake ground motions. For some engineering applications, however, it is
166 useful to capture the predominant period (or frequency) of ground motions in a single number. Rathje
167 et al. (2004) review different measures of the frequency content of ground motions, namely: mean
168 period, defined in terms of the Fourier amplitude spectrum; predominant spectral period, defined as
169 the period of the maximum spectral acceleration; smoothed predominant spectral period; and average
170 spectral period. They conclude that the mean period is the most appropriate measure of the frequency
171 content of the four IMs studied.

172 Although spectral energy parameters are not often computed, one IM that is related to the energy
173 content of ground motion and which is quite commonly used is Arias (1970) intensity (AI), equal to
174 the integral of the ground acceleration squared multiplied by $\pi/2g$. A number of related IMs measuring
175 the duration, amplitude and energy-input rate can be defined based on AI. Apart from those related to
176 the duration (discussed below) the other IMs derived from AI are not often used and, therefore, are not
177 detailed here. A similar IM to AI is cumulative absolute velocity, which is computed using the integral
178 of the absolute ground acceleration (sometimes only considering amplitudes over a certain threshold).

179 Many methods to characterise the duration of earthquake shaking have been proposed (Bommer and
180 Martinez-Pereira, 1999). These are either absolute (defined in terms of absolute thresholds of
181 acceleration or AI) or relative (defined in terms of thresholds relative to the PGA or AI) and are either
182 bracketed (interval between first and last exceedance of an acceleration threshold), uniform (total
183 length of time that absolute acceleration is above a threshold) or significant (interval between two
184 thresholds of AI). These various quantities seek to capture different aspects of the duration of shaking
185 and they often lead to greatly different values for the same accelerogram.

186 Earthquake ground motion features many cycles of motion that can have a damaging effect on
187 structures. However, since the cycles are highly inhomogeneous in terms of frequency, amplitude and
188 form there have been many proposals on how to count the number of effective cycles in a given
189 strong-motion record. Hancock and Bommer (2005) review the various methods (both absolute and
190 relative), including methods based on: rainflow counting, which counts both high- and low-frequency
191 cycles in broad-banded signals, and peak counting, including or excluding non-zero crossings.

192 For systems whose deformation involves restoring mechanisms of elastic linear nature, the viscoelastic
193 response spectra provide an efficient indication of an accelerogram's destructiveness. However, for
194 systems with strongly inelastic restoring mechanisms, elastic spectra are often inadequate IMs. This is

195 definitely the case with systems that rely solely on friction. The potential of a particular ground
196 accelerogram to inflict damage to such systems has been investigated by Garini and Gazetas (2013),
197 who studied two sliding systems (a rigid block on top of a horizontal or an inclined base) representing
198 a symmetric and an asymmetric rigid-plastic restoring-force–displacement mechanism. With the
199 supporting base of each system subjected to near-fault ground motions, the resulting slippage serves as
200 an index of the damage that this motion can inflict on the system.

201 The use of IMs to characterise ground motions has advantages over using accelerograms (e.g. derived
202 from the earthquake scenario) directly. Because of the infinite variety of possible earthquake ground
203 motions it is easier to understand the results of the structural modelling if these motions are
204 characterised by a small set of IMs. One IM (or a set of IMs) can then be used to evaluate the seismic
205 risk via fragility functions. This simplification comes at the cost of introducing uncontrolled factors
206 because the earthquake shaking cannot be fully characterised by only a few scalar parameters.

207 3.2. Ground-motion models for prediction of IMs

208
209 As noted above the availability of robust GMPEs for the prediction of the median value of an IM and
210 its variability (generally modelled by the standard deviation, σ) is necessary for PSHA. If a GMPE is
211 not available, the IM would have to be assessed through correlations with IMs that are themselves
212 predictable from GMPEs or from accelerograms for disaggregated earthquake scenarios.

213 The ubiquitous use of PGA and elastic response spectra is demonstrated by the large number of
214 published GMPEs for these IMs [Douglas (2011) identifies more than 200 models] for many types of
215 earthquakes (e.g. shallow crustal, stable continental, subduction and mining-induced) and many
216 regions. Therefore, for these parameters the problem is not a lack of GMPEs but which GMPEs to
217 choose. Epistemic uncertainty in the prediction of earthquake ground motions remains high (e.g.
218 Douglas, 2010) due to a lack of data (particularly in the near-source region) and knowledge on, for
219 example, the appropriate independent parameters and functional form. Therefore, it is common
220 practice to apply a number of GMPEs within PSHA and weight the results based on the degree of
221 belief in a certain GMPE providing the correct estimate of the median ground motion and its
222 variability (e.g. Delavaud et al., 2012).

223 Other IMs are less well served by robust GMPEs. Douglas (2012) identified 96 GMPEs for the
224 prediction of PGV, 19 for PGD, 33 for AI and 15 for relative significant duration. There are even
225 fewer published GMPEs for the remaining IMs (e.g. inelastic spectral ordinates, other measures of
226 duration, number of cycles and fundamental periods), which means that their estimation could be
227 problematic. For structures with large or long geographical footprints (e.g. city walls) an important IM
228 could be the maximum transient ground strain. The prediction of this parameter is discussed by
229 Paolucci and Smerzini (2008), who provide equations for its estimation based on correlations with
230 PGA and PGV.

231 When conducting seismic hazard analyses for the prediction of more than one IM it is important to
232 consider the correlations between the parameters otherwise the evaluated seismic hazard and scenarios
233 will not correspond to the desired probability of exceedance. To take account of correlations, vector-
234 valued PSHA (e.g. Bazzurro and Cornell, 2002) has been proposed but is rarely conducted in practice.

235 GMPE should generally be reserved for the estimation of ground motions at stiff soil or rock sites; for
236 soft soil site-specific analyses should be preferred (Baturay and Stewart, 2003). Site amplification
237 factors given in codes (e.g. EC8 or NEHRP) could also be used to define earthquake loading for non-

238 rock sites. Improved site amplification factors and site classification categories for EC8 have been
239 recently proposed by Pitilakis et al. (2012, 2013a, b).

240 3.3. Selection of the most appropriate IMs for historical buildings

241

242 Fragility functions are a key instrument in modern seismic risk assessments, such as undertaken in
243 PERPETUATE (see other articles in this special issue). A fragility function provides the conditional
244 probability that a considered asset equals or exceeds a certain damage level for a given level of
245 earthquake loading, represented by an IM or a set of IMs. As mentioned before, each IM represents
246 certain characteristics of the seismic action and, therefore, the choice of the appropriate IM depends on
247 the structural behaviour of the studied asset (e.g., Seyed et al., 2010; Gehl et al., 2013).

248 In the scope of seismic risk assessment of historical assets, it is important to select the IMs carefully to
249 reduce scatter in the final results. Often the most appropriate IM will vary with the type of asset. This
250 selection requires non-linear time-history analyses (unlike the capacity spectrum method). In this
251 view, Gehl et al. (2013) developed a procedure that relies on the statistical treatment of numerous
252 nonlinear dynamic analyses. At first, a structural model is considered and characterized within modal
253 and pushover analysis to identify dynamic properties and damage limit states of the structure,
254 respectively. The performance of each IM can then be evaluated based on calculation results using
255 data mining techniques, such as: the variable clustering method, comparison of standard deviations of
256 fragility functions and receiver-operating-characteristics analysis.

257 4. Appropriate descriptions of earthquake actions

258

259 Lagomarsino et al. (2011) provide a classification of cultural heritage assets by the type of damage
260 that can occur during earthquakes. Seven main asset classes are defined: A) assets subjected to
261 prevailing in-plane damage, B) assets subjected to prevailing out-of-plane damage, C) assets damaged
262 by high combined axial and bending loads, D) arched structures subjected to in-plane damage, E)
263 massive structures to which local failure of masonry prevails, F) blocky structures subjected to
264 overturning and sliding and G) built systems subjected to complex damage. The following discussions
265 are based on these seven asset classes.

266 4.1. Accounting for site effects

267

268 Local site conditions influence strong ground motion in several ways, which are collectively referred
269 to as site effects. These are related to the thickness and impedance contrast between soil layers, the
270 surface and the subsurface topography (lateral discontinuities, faults, valley and basin edges, inclined
271 soil layers or soil-bedrock interfaces) and soil non-linearity. Additional local effects may include
272 liquefaction, lateral spreading and landslides, which are briefly discussed below.

273 At small strains soil behaves linearly and hence amplitude-independent site amplification factors are
274 appropriate. However, at greater accelerations the soil stiffness degradation and nonlinearity reduce
275 substantially the amplitude of the propagating seismic waves resulting in significant attenuation of the
276 surface motion in comparison to the bedrock excitation. This reduction in acceleration is accompanied
277 by irreversible soil displacement. The reader is referred to Pitilakis (2004) for a review.

278 During earthquakes, the amplitude, frequency content and duration of shaking change as seismic
279 waves propagate through soil layers and reach the surface. This phenomenon, wherein the local soil
280 acts as a filter and modifies the ground motion characteristics, is known as site response. When shear
281 waves reach boundaries between different soil materials, they are reflected, refracted and converted

282 resulting in amplification or attenuation of motion. Under the idealisation of linear-elastic conditions,
283 the amplification/attenuation depends only on the relation of the natural frequency of the soil (its depth
284 and stiffness) to the frequency content of the excitation. However, reality is often more complicated
285 and unpredictable behaviour can take place because of nonlinear material behaviour, resulting in the
286 generation of higher-frequency waves and a decrease of its natural frequencies.

287 Soil layer effects have been investigated extensively, and much of the present knowledge is reflected
288 in modern seismic codes. The current understanding of site effects is that the first and most important
289 factor explaining the observed response is the impedance contrast and secondly the trapping of up-
290 going seismic waves. For example, EC8 and other seismic codes propose amplification factors and
291 response spectral shapes for specific soil categories, based essentially on the amplification produced
292 by the thickness and the impedance contrast between soil layers. Basin edge, valley and lateral
293 irregularity effects are not yet fully understood and they have not yet been introduced into building
294 codes as they are more difficult to understand and quantify in a simple manner.

295 A scalar factor, which alone accounts for the amplification of ground motion and its spatial
296 distribution, is insufficient. Modelling based on a one-dimensional approximation fails to reproduce
297 the long duration of strong motion, in particular in deep and large basins. This type of induced ground
298 motions could particularly affect long-period structures (e.g., building class C) or components. It is not
299 always valid to separate source and path effects from site effects. The latter is dependent on the first
300 two factors. This important fact will be demonstrated in the following sections discussing valley and
301 topographic effects as well as other aspects of ground motion.

302 The analysis of site effects on ground motion presupposes detailed and well-focussed geotechnical and
303 geological surveys including specialized laboratory and field tests. These surveys are even more
304 important when soil-foundation-structure effects are taken into consideration.

305 4.1.1. Accounting for valley and basin effects

306
307 The effects of subsurface geometry (e.g. valley and basin edges and lateral discontinuities) on ground
308 motion have been recognized for a long time and have been the topic of several instrumental and many
309 theoretical and numerical investigations in the past decades. The complexity of these phenomena,
310 combined with the limitations of both geophysical and geotechnical measurements and numerical
311 simulations, have not yet made it possible to include such effects in standard seismic hazard
312 assessments. Modern building codes, like EC8, do not include any prevision for basin edge and valley
313 effects.

314 . Alluvial basins may strongly influence the nature and intensity of ground shaking. Conventional 1D
315 modelling generally fails to reproduce the wave scattering phenomena introduced by the non-level
316 geometry. Such phenomena include: (1) generation of surface waves (Rayleigh and Love) at the basin-
317 edge lateral boundaries, which tend to increase both the amplitude away from the edges and the
318 duration of ground motion; (2) amplification and resonance enhanced by low-velocity non-horizontal
319 near-surface layers; and (3) multiple refractions and reflections of incoming waves at the sloping
320 boundaries and the ground surface (“entrapped” waves). All these phenomena are responsible not only
321 for aggravating the ground motions but also for producing a potentially destructive “parasitic” vertical
322 component (Gelagoti et al 2010). On the other hand, strongly nonlinear soil response tends to abate
323 some of these adverse effects (Gelagoti et al 2012). Source, path, and azimuthal effects may also
324 complicate the ground motions.

325 4.1.2. Accounting for steep topography

326

327 Historical city centres or monuments are often on the tops of steep hills because of defensive, cultural,
328 agricultural or religious reasons. It has been found that steep topography can significantly amplify
329 ground motions at certain frequencies and distances from the relief edge. Therefore, this should be
330 accounted for when assessing ground motions at such sites. Formulations in seismic design codes on
331 topographic amplification are unlikely to be sufficient because the slopes at the sites of historical
332 structures are sometimes steeper than those covered by such recommendations. For example, based on
333 ground-motion observations from aftershocks of the L'Aquila 2009 earthquake recorded at two sites,
334 Gallipoli et al. (2013) find that the Italian code provisions for the topographic amplification factor
335 underestimated the observed amplifications. EC8 incorporates an aggravation factor to the design
336 acceleration depending only on the geometry of the surface topographic relief. There are no provisions
337 accounting for the presence of lateral discontinuities or the spatial variation in the deposit thickness.
338 The incidence angle and the frequency content of the induced wave field are currently ignored. Also,
339 no discrimination is made as to whether the site is a slope or a hill, i.e. whether the ground behind the
340 slope is at the same level. Moreover the ground (geological and geotechnical) conditions do not play
341 any role in these code recommendations; the amplification is purely a geometrical effect.

342 4.1.3. Accounting for specific site conditions

343

344 Similarly to the situation for steep topography it is common to classify areas of very poor soils as 'not
345 fit for building unless specific soil improvement measures are taken'. However, these sites may
346 already be the location of historical buildings (e.g. the Tower of Pisa). In addition, in some cities more
347 recent historical structures are built on the debris of previous buildings and monuments, which may
348 date from centuries before (e.g. this is typical in ancient cities like Rome, Naples and Thessaloniki).
349 This means that the strong or peculiar amplifications that could occur at such sites need to be assessed.
350 The guidance provided by design codes is likely to be totally insufficient for such sites since they are
351 not a usual location for modern structures.

352 Monumental structures founded on very soft soil deposits or upon the debris of ancient cities may
353 undergo excessive deformations in case of an earthquake because of nonlinear soil response. This may
354 cause performance problems, especially for particularly heavy structures. Consequently, the effect of
355 soil on the seismic response of monuments can only be studied through the comprehensive prism of
356 soil-structure interaction.

357 Finally, while rare, there are several monuments that are founded above a strong geological and
358 geotechnical discontinuity, which may produce serious damage to the structure even under static
359 conditions. Examples include an ancient Greek temple in the Peloponnese, for which the discontinuity
360 may be the surface trace of an active fault, and the Colosseum in Rome, which straddles two types of
361 quaternary deposits.

362 4.1.4. Accounting for soil-foundation-structure interaction

363

364 Monumental buildings with masonry foundations are often characterized by massive and complex
365 structural systems. During an earthquake, such systems might be prone to soil-foundation-structure
366 interaction (SFSI), which could modify the response of the foundation. Kinematic SFSI may filter the
367 energy transferred to the structure, and alter significantly its seismic response. It is well known that for
368 heavy stiff structures resting on soft soil, linear and nonlinear SFSI play an important role on the
369 response of the foundation (Pitilakis et. al, 2013c). SFSI transfers stress fields from the structure to the
370 foundation, filtering high frequencies of the incoming seismic wave and thereby modifying the

371 resonance period of the building (Pitilakis and Clouteau, 2010). This, in turn, influences the design of
372 mitigation measures for the monument.

373 Conventional foundation models usually consider the foundation as a non-deformable rigid body. On
374 the contrary, historical masonry buildings have a foundation system that can transfer negligible tensile
375 stress and no bending moment. As a result, the response of flexible and brittle foundation systems
376 cannot support significant rocking movement, considerably affecting the overall structural response. In
377 this case, soil-foundation interaction should be accounted for by using models incorporating soil-
378 foundation compliance and damping increases (Pitilakis and Karatzetzou, 2013).

379 Modern seismic codes are based on performance-based assessment, which often employ static
380 pushover analyses for the evaluation of structural capacity. Traditionally, pushover analyses are
381 performed assuming a structure that is fixed at its base. Foundation compliance and the geometry of
382 the foundation system may, however, significantly modify the actual response in terms of both
383 capacity and demand, resulting in altered seismic performance.

384 4.2. Elastic approximation

385

386 Modern seismic design codes often reduce the elastic forces computed from earthquake response
387 spectra by a factor (often called R or q) that accounts for inelastic behaviour of the structure under
388 strong earthquake loading. According to EC8, the behaviour factor q is an approximation of the ratio
389 of the seismic forces that the structure would experience if its response was completely elastic with
390 5% viscous damping, to the seismic forces that may be used in design, with a conventional elastic
391 analysis model, still ensuring a satisfactory response of the structure. Moreover, the behaviour factor q
392 accounts for the influence of the viscous damping being different from 5%, which may also vary in
393 different horizontal directions of the structure.

394 Despite the merits of this simplified approach, several drawbacks are encountered in practice. Since
395 this reduction factor was initially conceived for systems assumed to have elastic-perfectly-plastic
396 behaviour, its use requires that the plastic mechanisms that develop in the structure under investigation
397 be of the same nature. However, this type of elastic-perfectly plastic material behaviour is not
398 normally desired for historical buildings and monuments nor is it possible for massive masonry
399 structures (class E). Historical buildings were not designed to sustain large plastic deformations, due to
400 brittle failure of the construction materials used in the past. Moreover, the energy dissipation
401 mechanism is different in historical buildings and monuments from modern structures, due to the
402 different shape of the hysteresis loop, which also depends on the material. In such situations, the use of
403 reduction factors with elastic spectra is inappropriate and elastic response spectral ordinates are more
404 useful than inelastic quantities.

405 4.3. Accounting for long-period motions

406

407 Class C structures (e.g. towers) may have natural periods longer than 2s. Structures with out-of-plane
408 mechanisms (class B) and blocky structures (class F) could also require accurate ground-motion
409 estimates for periods longer than 2s. Seismic codes have largely adopted smooth design acceleration
410 spectra, which may be applicable up to a period limit of 2 to 4 s. The response of monuments, some of
411 which may respond strongly inelastically (towers, columns and other particularly slender structures)
412 with periods significantly longer than 2s, also gives rise to the need for design response spectra that
413 extend to longer periods than have been traditionally considered. This need is even more pronounced

414 when the capacity spectrum method is applied, where the design demand spectra are strongly affected
415 by limited knowledge of long-period ground motions.

416 Until recently the prediction of response spectral ordinates for periods longer than about 2s was
417 difficult because of a lack of reliable GMPEs beyond this period, due to low signal-to-noise ratios for
418 records from analogue accelerograms particularly those from small earthquakes. However, the advent
419 of digital strong-motion networks and improvements in accelerogram processing procedures means
420 that robust GMPEs are now available to predict response spectral ordinates to 5s and beyond, e.g.
421 those produced in the Next Generation Attenuation (NGA) project (e.g. Abrahamson et al., 2008).

422

423 4.4. When elastic response spectra are not adequate descriptors of hazard

424

425 Class F and G assets, which include elements subjected to rocking or sliding, are strongly inelastic
426 systems. The analysis of idealized systems corresponding to this type of asset (e.g. a rigid block
427 rocking or sliding on a horizontal or sloping rigid base) shows that their response is extremely
428 sensitive to the presence, characteristics, sequence and direction of long duration pulses (Garini *et al*,
429 2011). Hence, forward-directivity and fling-step affected motions, which contain severe acceleration
430 pulses and/or velocity steps, may be particularly destructive for these systems. Sliding systems are
431 governed by the Coulomb friction law defined by a single parameter (the friction coefficient, μ), which
432 allows the introduction of “equivalent” sliding motions for a given displacement level. In this type of
433 analysis the block remains in full contact with its base while moving with the same acceleration and
434 velocity, until the triggering acceleration exceeds the critical yield acceleration. The response of the
435 block is fully inelastic once sliding starts. For the inclined system, the relative displacement between
436 the block and the base prevails in the downward direction and, therefore, slippage accumulates in
437 every sliding period and the block cannot end up in its initial position. In contrast for the horizontal
438 case, sliding occurs in both directions and, therefore, the block can theoretically return to its origin
439 (Gazetas *et al*, 2009).

440 Response spectra are not adequate descriptors of the purely plastic response of such systems. This has
441 been demonstrated by employing the concept of “equivalent” response spectra, where equivalent
442 means spectra (up or down-scaled versions of actual accelerograms), that have an identical effect on
443 the (strongly) inelastic idealized systems — just overturning of the rocking block, or a certain amount
444 of slippage of the sliding block (Gazetas, 2012). The resulting equivalent response spectra exhibit a
445 huge scatter, even within each type of system, with the largest and smallest spectra differing by a
446 factor of more than three, throughout the entire period range of interest. This type of analysis shows
447 that the use of elastic spectra as a representative index of destructiveness for all systems is limited and
448 it is necessary to use more appropriate IMs for certain asset classes.

449

450 4.5. Accounting for earthquake loading in more than one direction

451

452 Earthquake analysis of ordinary structures conducted within the framework of current seismic design
453 codes does not account for shaking in more than one horizontal direction at a time, or if it does then
454 this is often via simple rules. This is because modern structures in seismically-active regions are often
455 designed to be roughly symmetrical to avoid torsion effects and because the predominant failure mode
456 of modern structures is in-plane, for which earthquake shaking in the axis of the wall is the critical
457 parameter. In addition, vertical earthquake shaking is not often accounted for, or again it is accounted
458 by simple means, because modern buildings have a high factor of safety against gravity loads and long

459 horizontal spans, for which vertical loads could be important, are not common. Certain types of
460 historical buildings, however, may require careful consideration of all three components (two
461 horizontal and one vertical) of earthquake ground motions. This is even more important when the two
462 horizontal components have significantly different amplitudes, e.g. due to directivity near the
463 earthquake source.

464 Structures categorised into asset classes B (those subjected to prevailing out-of-plane damage) and A
465 (subject to torsion) would need both horizontal components of earthquake shaking to be considered.
466 Structures categorised into asset classes C (those damaged by high combined axial and bending loads)
467 and D (arched structures subjected to in-plane damage) would need a consideration of the vertical
468 component of motion to be made. For class C structures, this will contribute to the axial loads and will
469 affect the bending loads, and, for class D structures, the vertical shaking will affect the vertical loads
470 applied on the arch. Class F structures (blocky structures subjected to overturning) require a
471 consideration of vertical motions since this loading could reduce the apparent weight of its elements
472 and hence make them easier to overturn.

473 In many damaging earthquakes of the last twenty years, the vertical component was high relative to
474 the horizontal. Moreover, even if certain components of the earthquake acceleration seem insignificant
475 by themselves, they could increase the damage of the earthquake when they are combined with
476 shaking in other directions. Thus, certain locations and buildings may require careful consideration of
477 all three components of earthquake ground motions.

478 Historical buildings can present design (e.g. geometric or constitutive) weaknesses that could activate
479 complex 3D failure modes such as torsional and high vibration modes, which are usually not
480 considered in standard vulnerability assessments. Buildings that were not specifically designed under
481 seismic risk mitigation rules often present combined eccentricities in term of mass, stiffness and
482 strength. These weaknesses could be the source of significant damage, when the buildings are exposed
483 to motions in two or three dimensions. For instance, if a building presenting strength eccentricity is
484 exposed to large motions in both horizontal dimensions, a decrease in stiffness due to plastic
485 deformations in the weak direction will probably substantially influence its behaviour in the
486 orthogonal direction. The building cannot be realistically modelled by a 2D frame and, hence, a full
487 3D model must be used.

488 The consequences of simultaneous shaking in three directions to historical buildings can be increased
489 because of other specificities shared by these buildings. The structural complexity of many of these
490 buildings can notably increase their vulnerability: some 3D structures, such as vaults, can be seriously
491 affected by 3D accelerations, which could involve the loss of equilibrium of the structure. Moreover,
492 the weaknesses of some highly vulnerable historical buildings could be highlighted by such ground
493 motion: cracks will form at the weak connections among walls, or among walls and floors resulting in
494 blocks or in wall overturning.

495

496 4.6. Accounting for permanent displacements

497

498 Large magnitude earthquakes are often accompanied by permanent ground displacements at the
499 surface because of large fault movements, even when the rupture is blind. This may induce significant
500 distress to overlying structures. The mechanisms of fault-soil-structure interaction have been
501 addressed in the literature through analysis of historical case studies (e.g. Anastasopoulos and Gazetas,
502 2007; Faccioli et al., 2008), small and large scale experiments (e.g. Loli et al., 2011) and nonlinear

503 finite element analyses (e.g. Anastasopoulos et al., 2009). Furthermore, there have been recent
504 attempts to estimate permanent displacement associated damage and develop fragility curves (e.g.
505 Fotopoulou and Pitilakis, 2012, 2013; Negulescu and Foerster, 2010).

506 Massive large dimension structures categorized in asset class E cover large extended areas (e.g.
507 defensive city walls). For this type of asset it is important that permanent ground displacements, such
508 as those caused by proximity to the fault trace, seismically-induced landslides or on ground that
509 liquefied, are accurately assessed so that strains and deformations in the structure can be estimated.
510 For these situations the critical ground motions are not best described by (elastic) response spectra but
511 with time histories containing: strong one-sided pulses, long-period motions and, ideally, including the
512 permanent offset.

513 Unlike current buildings for which the seismic performance is usually expressed by the maximum
514 instantaneous displacement (e.g. the inter-storey drift) and by the ductility demand associated with this
515 displacement, for massive linear structures like the ones included in class D (e.g. triumphal arches,
516 aqueducts and bridges) the seismic performance is generally expressed in terms of permanent
517 displacements at the end of the earthquake. This class of structures undergoes permanent
518 displacements in only one direction and therefore the displacements increase monotonically and reach
519 a maximum at the end of the earthquake.

520 It is difficult to decide for which monument classes permanent displacement play an important role in
521 the occurrence of damage since the classes are defined mainly considering characteristics of the upper
522 structure while permanent displacements act at the foundation level. If the foundation system has the
523 ability to accommodate large deformations or to span soft spots then the damage that a structure may
524 suffer due to permanent displacement can be significantly reduced compared to a foundation system
525 that does not possess any ductility. The majority of historical monuments have shallow foundation
526 systems, but it is important to know if all foundation elements (perimeter and interior wall footings)
527 are tied together to enable them to bridge areas of local settlement and provide better resistance against
528 soil movements. The problem is that, unlike the upper structure, the foundation system cannot be
529 evaluated only by visual inspection of the monument. Moreover, the foundation system can be very
530 different from one monument to another even for structures belonging to the same class. Finally the
531 integrity of the foundation system may be seriously affected by, for example, aging and non-rigid-
532 body behaviour of the foundation. In this case the impedance functions used in the structural
533 modelling should be modified from those for the infinite rigidity case. Impedance functions have
534 recently been proposed for flexible masonry foundations by Pitilakis and Karatzetzou (2013).

535 The standard approach for the seismic design of shallow foundations is equivalent to ensuring that the
536 bearing strength factor of safety does not fall below a certain value. However, brief instances of
537 bearing failure (yielding) during an earthquake may not necessarily be destructive. A more important
538 consideration is the residual foundation displacements accumulated at the end of the earthquake (Toh
539 and Pender, 2008). This suggests that deformations caused by earthquakes can be developed in two
540 consecutive steps (governed by the soil deformation): the first step during the earthquake can bring
541 instability and generate a failure surface and the second step can follow immediately after the
542 earthquake if the residual shear strength on the failure surface is less than the one required to maintain
543 static equilibrium. In the first stage the damage of the structure is mostly due to the shaking while in
544 the second stage it is caused by the soil deformation. A series of earthquakes of different intensities
545 may also lead to accumulation of permanent deformations at the foundation level.

546 Few studies can be found in the literature where the effect of permanent ground displacements has
547 been addressed with regards to historic assets. Three recent articles can be cited: Karakhanian et al.
548 (2008), concerning the St. Simeon Monastery (Syria); Galli and Galadini (2001), reporting several
549 cases of surface faulting on archaeological relics in the Dead Sea Valley, Crete and central and
550 northern Italy; and Oliveira (2003), modelling various Portuguese structures. A comprehensive
551 numerical study on the effect of normal fault rupture interacting with masonry structures is presented
552 in Gazetas et al. (2013), which demonstrates the key role of the foundation continuity and stiffness.

553 5. Conclusions

554
555 This paper is an introduction to seismic hazard assessment for the analysis of historical assets. It
556 constitutes one of the principal steps of the performance-based assessments undertaken within the
557 PERPETUATE project. After a brief discussion on the appropriate hazard framework, the IMs that
558 could be used to describe earthquake shaking applied to historical assets were presented and available
559 models for their assessment were introduced. The importance of the choice of appropriate IMs for
560 each type of historical asset was emphasised. Based on the classification of historical assets proposed
561 in PERPETUATE, about a dozen characteristics of historical assets or their locations were discussed
562 with respect to their impact on how seismic hazard should be described. In particular, differences from
563 the approaches described in building codes for modern buildings were highlighted. Specific aspects,
564 such as, strong local site effects due to steep topography (e.g. hilltops), basin effects or foundations
565 built on the remains from previous structures were presented. The suitability of elastic response
566 spectra for different classes of historical assets was also discussed. The cases where more than one
567 direction in the earthquake loading must be considered were identified. A decision tree approach could
568 be useful for a preliminary screening of historical assets to identify critical aspects of the site and
569 monument but because historical assets are often special cases such tools are not appropriate for
570 detailed studies. What is required is expert analysis covering all relevant disciplines and not blind
571 obedience to guidelines. The themes and concepts introduced here are developed further and applied
572 to actual case studies in subsequent articles in this volume and also in PERPETUATE reports.

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577

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