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Response modification factors for concrete bridges in Europe

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Abstract

The paper presents a methodology for evaluating the ‘actual’ response modification factors (q or R) of bridges, and applies it to seven concrete bridges typical of the stock found in Southern Europe. The usual procedure for analytically estimating the q -factor is through pushover curves derived for the bridge in (at least) its longitudinal and transverse direction. The shape of such curves depends on the seismic energy dissipation mechanism of the bridge; hence, bridges are assigned to two categories, those with inelastically responding piers and those whose deck is supported through bearings on strong, elastically responding, piers. For bridges with yielding piers the final value of the q -factor is found as the product of the overstrength-dependent component (q_s) and the ductility dependent component (q_μ), both estimated from the pertinent pushover curve; for bridges with bearings and non-yielding piers of the wall type an equivalent q -factor is proposed, based on spectral accelerations at failure and at design level. In this paper pushover curves are also derived for an arbitrary angle of incidence of the seismic action using a procedure recently developed by the authors, to investigate the influence of the shape of the pushover curve on the estimation of q -factors. It is found that in all cases the available force reduction factors were higher than those used for design either to Eurocode 8 or to AASHTO.

Keywords: concrete bridges; behavior factor; response modification factor; pushover curve

Introduction

This study focuses on the estimation of ductility and overstrength factors, i.e. the two components of the available force reduction factor (Kappos 1999), for concrete bridges. This factor, which is the ratio of the force that the bridge would develop if it responded elastically to the design seismic action to the design base shear (V_e/V_d), is called response modification factor (R) in the US (AASHTO 2010) and behavior factor (q) in Europe (CEN 2005), and is an important design parameter. The maximum available value of q -factor for an (already designed) structure can be defined as the ratio of the maximum horizontal force developed by the structure prior to failure to the design base shear (V_u/V_d), and provides a meaningful measure of its safety. Evaluating this ratio is a problem of particular relevance for practice, especially in the case of important bridges or bridges with irregular and/or unconventional configuration, and also in the verification and calibration of code provisions.

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31 The procedure for analytically estimating the aforementioned components of the q -factor is usually
32 based on nonlinear static (pushover) analysis of the entire bridge, wherein pushover curves are derived
33 for the bridge in its longitudinal and transverse direction. Although a number of previous studies
34 include pushover curves for bridges, derived using single mode or multi-mode procedures (Kappos et
35 al. 2012), studies specifically addressing the derivation of q -factors for bridges are scarce and use
36 different procedures; moreover, they all concern either a single actual bridge or a single bridge
37 typology, and they all correlate q to the ductility of the critical piers. Some studies, like that of Itani et
38 al. (1997) analyse single columns only, taking into account both overstrength and ductility. Others like
39 that of Abeysinghe et al. (2002), Meimari et al. (2005), and Mackie & Stojadinović (2007), address
40 entire bridges but estimate q (R) as a function of the column ductility only (ignoring overstrength); it
41 is important to note that, as a result of ignoring the effect of overstrength, studies like that of
42 Abeysinghe et al. result in unrealistic (over-conservative) estimates of the q -factor that should be used
43 in design. For the case of bridges with bearings, Constantinou and Quarshie (1999) propose R -factors
44 for their inelastically responding piers with bearings (modelled as two-degree-of-freedom systems)
45 addressing both components of the behaviour factor in a way similar to that for bridges without
46 bearings.

47 In the present study pushover curves are derived for a number of typical bridge typologies not only
48 for their longitudinal and transverse direction but also for an arbitrary angle of incidence of the seismic
49 action using a procedure recently developed by Moschonas & Kappos (2012). Noting that the shape of
50 a pushover curve depends on the seismic energy dissipation mechanism of the bridge, bridges are
51 classified into two main categories according to their seismic energy dissipation mechanism: bridges
52 with yielding piers of the column type, and bridges with bearings and non-yielding piers of the wall
53 type. The method proposed herein differentiates the way of defining the aforementioned factors
54 according to the category of the bridge.

55 For bridges of the first category, the derived pushover curves are idealized as bilinear ones and the
56 available q -factor is estimated as the product of two components, a ductility-based one, and an
57 overstrength-based one ($q = q_{\mu} \cdot q_s$). The overstrength factor (q_s) is defined as the ratio of yield strength
58 to the design base shear, while the ductility factor (q_{μ}) is derived as a function of the available
59 displacement ductility of the bridge. For bridges of the second category, wherein the deck rests on
60 elastically responding piers through elastomeric bearings, a different procedure is proposed herein,
61 since no meaningful bilinear pushover curves can be derived. Hence the concept of equivalent q -factor
62 (q_{eq}) is introduced; this factor is defined as the ratio of the spectral acceleration (corresponding to the
63 pertinent predominant period of the bridge) for which failure occurs, to the design spectral
64 acceleration.

65 The foregoing methodology is then used to answer the very legitimate (and relevant to practicing
66 engineers) question ‘what are the actual q -factors of modern bridges?’ More specifically, the available
67 q -factors (or q_{eq} -factors) are estimated for seven actual bridges, typical of those used in European

68 motorways, in particular in Southern Europe, which is a high seismicity region. They include
69 typologies of both the first (inelastically responding piers) and the second category (bearings on elastic
70 piers), as well as a ‘mixed’ type of structure, combining features of both categories. The available
71 force reduction factors calculated for these bridges are then compared with the values specified in the
72 European (Eurocode 8) and North American (AASHTO) codes for seismic design of bridges.

73 **Methodology**

74 The methodology for evaluating the available force reduction factors (the actual q -factors) for concrete
75 bridges (the same procedure can be used for steel or composite bridges), is based on nonlinear static
76 (pushover) analysis of the entire bridge, wherein pushover curves are derived for the structure in (at
77 least) its longitudinal and transverse directions. A critical issue that differentiates the way of
78 evaluating the aforementioned factors is the seismic energy dissipation mechanism of the bridge.
79 According to this mechanism, bridges are classified into two main categories:

- 80 • Bridges with yielding piers of the column type: Piers are connected to the deck either
81 monolithically or through a combination of bearings and monolithic connections, which is fairly
82 common in modern ravine bridges in Europe. Inelastic behavior is developed due to the
83 formation of plastic hinges at the pier base, and possibly also the top, if the pier-to-deck
84 connection allows the development of substantial bending moment.
- 85 • Bridges with bearings (with or without seismic links, like stoppers) and non-yielding piers of
86 the wall type: In these bridges the inelastic behavior is developed due to the inelastic behavior
87 of bearings and seismic links. In most cases the deck is supported by wall-type piers which
88 remain in the elastic range even for earthquakes much stronger than the design event.

89 A key difference between the two main categories is the shape of the pushover curve, which is
90 clearly bilinear in the first category and essentially linear in the second one, wherein the slope of the
91 curve is defined by the effective stiffness of the bearings. Reinforced concrete members are modeled
92 using the lumped plasticity (point hinge) model of SAP2000 (CSI 2005) with multilinear moment –
93 rotation law for each hinge, accounting for residual strength after exceeding the rotational capacity;
94 elastic parts of the piers were modeled with cracked stiffness properties allowing for moderate tension
95 stiffening, as per the Eurocode 8 recommendations. Foundation compliance was modeled using
96 systems of translational and rotational springs at the bases of the piers and abutments. Relevant details
97 are given in Kappos & Sextos (2009) and Kappos et al. (2012). P- Δ effects were taken into account for
98 piers, but in most cases their effect was found to be very small.

99 *Bridges with inelastically responding piers*

100 In bridges with yielding piers of the column type, pushover curves, i.e. plots of base shear vs.
101 displacement of the ‘monitoring’ point on the deck (taken as the one above the critical pier or
102 abutment) are derived by performing a standard (fundamental mode based) pushover analysis. Some

103 of the bridges have also been analyzed using a modal pushover analysis for each mode independently
 104 (Paraskeva et al. 2006). When the modal pushover method is used, a “multi-modal” curve can be
 105 constructed by an appropriate combination of the values from individual curves (Kappos and
 106 Paraskeva 2008, Kappos et al. 2012). Alternatively, for bridges where the higher modes are significant
 107 (for the transverse response of the bridge) non-linear response history analysis may also be applied to
 108 derive dynamic pushover curves. The derived (through any of these procedures) pushover curve is
 109 then idealized as a bilinear one in order to define a conventional yield displacement, δ_y , and ultimate
 110 displacement $\delta_u = \mu_u \cdot \delta_y$, both referring to the entire bridge, not to a single pier (δ_u is taken here to
 111 correspond to a 20% drop in the base shear capacity, see Figure 1).

112 By definition, the value of the q -factor for a specific structure is given by the ratio of elastic force
 113 demand (V_{el}) to the design force (V_d), i.e. (see Figure 2)

$$114 \quad q = (S_a)_d^{el} / (S_a)_d^{in} = V_{el} / V_d = (V_{el} / V_y) / (V_y / V_d) = q_\mu \cdot q_s \quad (1)$$

115 where $(S_a)_d$ is the design spectral acceleration corresponding to the fundamental period of the structure
 116 and the indices ‘el’ and ‘in’ refer to the elastic spectrum and the corresponding inelastic spectrum,
 117 according to which the design seismic actions are determined (Kappos 1991, 1999). The two
 118 components of q can be estimated as discussed in the following.

119 The overstrength factor (q_s) is usually defined as the ratio of the yield strength to the design base
 120 shear of the structure

$$121 \quad q_s = V_y / V_d \quad (2)$$

122 where V_y is the (conventional) yield strength and V_d is the design base shear of the structure. In the
 123 absence of details of the design of the bridge (which in most cases addressed here was carried out
 124 using response spectrum modal analysis) the design shear can be estimated from

$$125 \quad V_d = m_{tot} \cdot S_{ad}(T) \quad (3)$$

126 where m_{tot} the total mass of the bridge and $S_{ad}(T)$ the pseudo-acceleration corresponding to the
 127 fundamental period of the bridge, taken from the design spectrum (that includes q); equation (3) is
 128 adopted by Eurocode 8 (CEN 2005) when the ‘fundamental mode method’ is used.

129 The overstrength factor (upper limit) can also be defined as the ratio of the ultimate strength (the
 130 maximum shear, V_u , corresponding to the last point of the second branch of the idealized bilinear
 131 curve, see Fig. 1) to the design base shear of the structure

$$132 \quad q_{s(\max)} = V_u / V_d \quad (4)$$

133 Obviously, when the pushover curve is idealized as elastic-perfectly-plastic, the two definitions of
 134 equations (2) and (4) coincide. A minimum value of the overstrength factor can be defined as the ratio

135 of the strength of the structure at the time where the first plastic hinge takes place to the design base
 136 shear

$$137 \quad q_{s(\min)} = V_{SLS} / V_d \quad (5)$$

138 where V_{SLS} is the strength of the structure when the first plastic hinge formation occurs. It is noted that
 139 for deterministic assessment purposes, mean values of material strengths must be introduced for
 140 calculating V_u , V_y and V_{SLS} . In the longitudinal direction of the bridge, the activation of the abutment-
 141 backfill system due to closure of the gap between the deck and the abutments strongly affects the
 142 damage mechanism (see Fig. 1(b)). In any case, the evaluation of the overstrength factor is not
 143 affected by the new seismic energy dissipation mechanism of the bridge. Furthermore, the activation
 144 of the abutment-backfill system increases the total strength of the bridge.

145 The ductility factor, q_μ , is derived as a function of the available ductility of the bridge, which is
 146 defined as the ratio of the ultimate limit state displacement (δ_u) to the yield displacement (δ_y),
 147 depending on the prevailing period. Veletsos and Newmark (1960) related q_μ to the kinematic ductility
 148 demand μ by the following expressions:

$$149 \quad q_\mu = \begin{cases} \sqrt{(2\mu-1)}, & T < 0.5s \\ \mu & , T > 0.5s \end{cases} \quad (6)$$

150 which are based on the familiar equal energy absorption and equal displacement approximations,
 151 respectively. It is noted that several other expressions for q_μ have been proposed in the literature, some
 152 of them accounting for additional factors such as the ground conditions or the peak ground
 153 displacement. Equations (6) were selected here due to their simplicity; it is noted, though, that in most
 154 concrete bridges the fundamental period T is longer than 0.5s and for this range most of the available
 155 relationships predict $q_\mu = \mu$ (or very nearly so).

156 As noted previously, the activation of the abutment-backfill system due to closure of the gap
 157 between the deck and the abutments may strongly affect the damage mechanism. So, a “full-range”
 158 analysis of the bridge is suggested in order to model the response of the bridge subsequent to gap
 159 closure. A detailed finite element modeling of the abutment-backfill system (in both the longitudinal
 160 and transverse direction), including soil flexibility (nonlinear behavior and consideration of both stiff
 161 and soft soils) and pile non-linearity (in flexure and shear), was made in the case of a typical overpass
 162 bridge (Pedini bridge in Figure 3). In such an analysis, all stages of the bridge seismic response are
 163 studied, i.e. the initial stage when the joint is still open, during which the contribution of the abutment-
 164 backfill system is small, and the second stage after closure, during which a significant redistribution of
 165 seismic forces between the piers and the abutment-backfill system takes place. In this case the
 166 pushover curve has a quadrilinear shape (Fig. 1(b)) and the additional parameter that has to be defined
 167 is the displacement at failure of the abutment-backfill system, δ_u' . Since it is common, especially in

168 design practice, to carry out the analysis of the bridge ignoring the abutment-backfill effect, failure of
 169 the abutment-backfill system can be approximated by estimating δ_u' from the following relationship

$$170 \quad \delta_u' = \alpha \cdot \delta_u \quad (7)$$

171 where δ_u is the ultimate displacement of the bridge without the abutment- backfill effect. The value for
 172 α was found to be about 0.6 for the analyzed overpass (Kappos & Sextos 2009); this approximate
 173 value of the δ_u' was used for bridges where the “full-range” analysis is not performed.

174 For bridges wherein higher modes are significant (for the transverse response of the bridge), a
 175 modal pushover analysis was also applied, as proposed for bridges by Paraskeva et al. (2006).
 176 Alternatively, for these bridges, non-linear response history analysis can also be applied to derive
 177 dynamic pushover curves. Regarding the use of multi-modal pushover curves it was found that they
 178 are much better suited to studying the ductility and overstrength characteristics of a bridge compared
 179 to standard pushover curves, especially for bridge structures where higher modes are significant
 180 (Paraskeva and Kappos 2009, Kappos et al. 2012). Figure 4 shows such static and dynamic pushover
 181 curves for a typical overpass (T7 in Fig. 3), while Figure 5 shows the corresponding static and
 182 dynamic curves for a bridge whose response is dominated by the first mode (G11 bridge in Fig. 3). It
 183 is noted that in these figures the dynamic curves, obtained from response history analysis for a number
 184 of records, correspond to combinations of the maximum displacement (δ_{\max}) with the simultaneous
 185 base shear, $V(t)$, or the base shear one time step before or after $V(t)$, or the maximum base shear V_{\max} ,
 186 which is not simultaneous with δ_{\max} . It is observed that in all cases the dynamic and multimodal
 187 pushover curves show both higher strength and higher ultimate displacement than the corresponding
 188 single-mode pushover curves; hence, the use of the standard pushover curve for the estimation of the
 189 available q-factor leads to more conservative results. To retain uniformity along all typologies studied,
 190 the estimated q-factors reported in the remainder of the paper are those derived from ‘standard’
 191 (single-mode) pushover analysis.

192 *Bridges with bearing-supported deck and elastically responding piers*

193 In the case of bridges with elastomeric bearings (with or without seismic links) and non-yielding
 194 piers of the wall type, pushover curves are derived by performing a standard pushover analysis given
 195 that the first (fundamental) mode of the bridge is similar to the first (fundamental) mode of the deck
 196 since the wall-type piers are much stiffer than the bearings, and as a consequence this mode has a very
 197 high participating mass ratio. In the longitudinal direction the first mode of the deck is a rigid-body
 198 displacement, while in the transverse direction it has a sinusoidal shape or it consists of a quasi-rigid-
 199 body displacement and rotation, depending on whether the transverse displacement of the deck at the
 200 abutments is restrained or free. In addition, the derived pushover curve has a bilinear shape because of
 201 the corresponding bilinear behavior of the bearings (Figures 6(a) and 6(b)). Note that in the usual case
 202 that common (low-damping ratio, $\zeta \approx 5\%$) bearings are used, the pushover curve is essentially a straight

203 line, whose slope is defined by the effective shear stiffness that does not change substantially (the
 204 hysteresis loop of these bearings is very thin). The choice of this linear approximation is advisable for
 205 both the economy of the analysis procedure and the more accurate assessment of the target
 206 displacement, since the definition of the first branch of the bilinear diagram of the bearings is subject
 207 to substantial uncertainty. Whenever seismic links (stoppers) are present, the pushover curve has a
 208 similar shape but an apparent hardening/softening is noticed, due to the successive activation and
 209 failure, respectively, of seismic links (Fig. 6(b)).

210 For bridges whose deck rests on elastic piers through bearings, a different procedure for evaluating
 211 the force reduction factor is proposed herein, since no meaningful bilinear pushover curves or ductility
 212 factors can be derived in this case. Hence the concept of equivalent q -factor (q_{eq}) is invoked, first
 213 introduced in Kappos (1991), which involves scaling the design q -factor (q_d) by the ratio of the
 214 spectral acceleration (corresponding to the pertinent prevailing period of the bridge, T) for which
 215 failure occurs, $S_{au}(T)$, to the design spectral acceleration, $S_{ad}(T)$ (see also Eq. (7))

$$216 \quad q_{eq} = (S_{au}(T)/S_{ad}(T)) \cdot q_d \Rightarrow q_{eq} = S_{au}(T)/S_{ad}(T) \quad (8)$$

217 where q_d is the design behavior factor which is equal to unity ($q_d \approx 1.0$) for bridges with non-yielding
 218 piers of the wall type (CEN 2005).

219 Available behavior factors for concrete bridges

220 To evaluate the force reduction factors of concrete bridges at the ultimate limit state, seven, more or
 221 less typical, bridges along the 670 km Egnatia Highway, which crosses the three regions of the
 222 northern part of Greece, Epirus, Macedonia, and Thrace, were selected. A comprehensive
 223 classification system for modern bridges in Europe, with emphasis on the Egnatia Highway stock, can
 224 be found in Moschonas et al (2009); the basic characteristics considered in the classification were the
 225 type of deck, type of piers, and type of pier-to-deck connections.

226 Four of the selected structures belong to the first category defined in the previous section
 227 (inelastically responding piers), two to the second one (deck supported through elastomeric bearings
 228 on elastically responding piers) and one is a ‘mixed’ type of structure, combining features of both
 229 categories. The main characteristics of the selected bridges are given in Fig. 3.

230 The pushover curves derived using analysis with SAP point hinge models as mentioned in the
 231 previous section, were idealized as bilinear curves (Fig. 1) in order to define a conventional yield
 232 displacement, δ_y , and ultimate displacement, δ_u . The derived overstrength factors for bridges with
 233 yielding piers, as well as the ductility factors for the same bridges, are given in Table 1; for q_{μ} in the
 234 longitudinal direction two values are reported, the one in parentheses corresponding to the case that
 235 eqn (7) is disregarded (i.e. possible failure of the abutment-backfill system is not taken into account).
 236 It is noted that both q_s and q_{μ} range within a rather broad range; taking the lowest among the values
 237 calculated for the longitudinal and the transverse direction in each bridge, q_s varies from 1.2 to 2.7, and

238 q_{μ} from 1.2 to 5.5. It should also be pointed out that high q_{μ} values do not necessarily correspond to
 239 high q_s values. Furthermore, it is noted that some unexpectedly high values of overstrength, notably
 240 the $q_s=5.8$ for Pedini bridge, are simply due to the fact that the contribution of the abutment – backfill
 241 system was modeled (‘full-range’ analysis) and substantial force was carried by this system
 242 subsequent to yielding of the piers; of course, for this and other bridges this was not the critical
 243 direction of the bridge.

244 Static pushover curves for some of the bridges were also derived for various angles of incidence of
 245 the seismic action (angles of 15°, 30°, 45°, 60° and 75°), using a procedure recently developed by
 246 Moschonas and Kappos (2012) with a view to investigating the influence of the characteristics of the
 247 ‘multidirectional’ pushover curves on the estimation of both the ductility and overstrength factors. All
 248 pushover curves derived for Pedini Bridge are plotted on the same diagram in Figure 7; note that in
 249 this case a simpler model, neglecting foundation compliance was used. A rather smooth and gradual
 250 transition from the pushover curve for the longitudinal direction to the corresponding one for the
 251 transverse direction is observed, as expected for a symmetric bridge such as this overpass. The
 252 conventional yield displacement, δ_y , ultimate displacement, δ_u , the corresponding available
 253 displacement ductility ratio μ_u , the ductility factor and the overstrength factor for all angles of
 254 incidence are given in Table 2. The ductility-related factor q_{μ} was calculated using Eq. (6), without
 255 taking into account the displacement at gap closure (eqn. 7) that is valid for the longitudinal direction
 256 only. It is noted that the angle of incidence of the seismic action affects the results of both the
 257 available overstrength and ductility factor; nevertheless, the values estimated for the transverse and
 258 longitudinal direction seem to bound the estimated values.

259 For bridges of the first category (yielding piers), the available q -factor (in each direction) was
 260 estimated as the product $q_{\mu} \cdot q_s$, whereas for bridges of the second category the previously described
 261 concept of the equivalent q -factor is utilized, defined from equation (8). All q -factor values are
 262 reported in Table 3; recall that one bridge (G2) belongs to both categories in its longitudinal direction.

263 *Some comparisons with code-specified values*

264 The estimated available force reduction factors for the typical bridges studied here can be compared
 265 with values prescribed by modern seismic codes. Eurocode 8 – Part 2 (CEN 2005) qualifies for the
 266 most direct comparison, since the studied bridges were designed according to provisions that are
 267 similar, albeit not identical, to those of this code. For concrete bridges with piers expected to yield
 268 under the design earthquake the Eurocode specifies a behavior factor equal to $3.5\lambda(\alpha_s)$ for ductile
 269 bridges, where $\lambda(\alpha_s)=1.0$ when the shear span ratio of the pier $\alpha_s \geq 3$ ($\alpha_s = L_s/h$, where L_s is the shear
 270 span of the pier columns and h the depth of their cross-section in the direction of flexure of the plastic
 271 hinge), which implies that its response is predominantly flexural, whereas for $3 > \alpha_s > 1$,
 272 $\lambda(\alpha_s) = \sqrt{\alpha_s/3}$. For the studied bridges in this category a value of 3.5 would be appropriate ($\alpha_s \geq 3$ for

273 most columns); this does not necessarily mean that this was indeed the q -factor used in their design,
274 since minor discrepancies exist between Eurocode 8 and the previous Greek Code (for instance, $q=3.5$
275 applied for $\alpha_s \geq 3.5$, in lieu of 3). Notwithstanding the aforementioned minor discrepancies, the fact that
276 the estimated q -factors (Table 3) vary from 4.2 to 10.1 in the longitudinal direction and from 3.7 to
277 11.6 in the transverse direction, is a clear indication that the code-prescribed value is not only feasible
278 but in several cases is actually an underestimation of the actual energy dissipation capacity of the
279 bridge, which is the result primarily of its ductility, but also of its overstrength.

280 For the bridges on elastomeric bearings $q=1$ was used in their design, hence the values reported in
281 the lower part of Table 3 simply indicate that the studied bridges were capable of resisting without
282 failure earthquake actions about four times higher than the design one.

283 Comparisons with other codes should be made with caution, as several differences exist in the
284 ‘philosophy’ of international codes. For instance, the American AASHTO (2010) LRFD Code adopts
285 a different level of design earthquake, i.e. the one having a return period of 1000 yr, whereas Eurocode
286 8 bases the design of bridges in motorways and national roads, on the 475 yr earthquake. There are
287 also differences in the detailing provisions and the material safety factors for concrete and steel
288 between the American and the European codes, although these are not deemed particularly significant.
289 In any case, AASHTO specifies values of the ‘response modification’ factor R equal to 1.5, 2.0, and
290 3.0 for single-column bents, and 1.5, 3.5 and 5.0 for multi-column bents, for ‘Operational Category’
291 Critical, Essential, or ‘Other’, respectively. The R -values for essential bridges are in the authors’
292 opinion the ones that correspond to the Eurocode values, since the latter are meant for highway
293 bridges. In fact the Eurocode treats importance of the bridge (‘critical’ etc.) in a different way, i.e. not
294 through q , but through the importance factor (γ_I), which varies from 0.85 to 1.3 (the upper limit is for
295 critical bridges). The different ‘philosophy’ of these two leading codes is clear here, since the
296 difference in the design seismic action between the highest and the lowest importance category is
297 $1.3/0.85=1.53$ in the Eurocode, while in AASHTO it varies between $3.0/1.5=2.0$ and $5.0/1.5=3.33$,
298 depending on the number of columns in the bents. If one ignores these and other differences among
299 the codes under consideration, the AASHTO-specified factors for essential bridges can be evaluated in
300 the light of the analyses presented herein. For the four bridges with single-column bents (Pedini, T7,
301 G11, and Krystallopigi in Fig. 3), it is clear that the value $R=2.0$ adopted by AASHTO in this case,
302 underestimates the actual energy dissipation capacity of these bridges. The only bridge with multi-
303 column bents in Fig. 3 is G2; for this bridge the estimated force reduction factor is about 4 in the
304 longitudinal direction, which exceeds the value of 3.5 specified by AASHTO, but only 2.4 in the
305 transverse direction. Since this is a rather particular case (a combination of the two types discussed in
306 previous sections) one cannot really draw any definitive conclusions.

307 **Conclusions**

308 A methodology for evaluating the force reduction factors available in concrete bridges was proposed;
309 these available factors are related to the ultimate limit state of the bridge. A key aspect of the
310 approach, which differentiates the way of evaluating the force reduction factors, is the seismic energy
311 dissipation mechanism of the bridge. Another aspect is that the bridge is addressed as a system, and
312 failure modes other than exceedance of available ductility in the piers are also addressed. The
313 methodology was applied for evaluating the available q -factors (for bridges with yielding piers) or q_{eq} -
314 factors (for bridges with bearings and non-yielding piers) of seven actual bridges representative of a
315 broad set of typologies found in Southern Europe.

316 It was found that in all cases the available force reduction factors were higher than those used for
317 design in both the longitudinal and transverse directions. In fact, in many cases the code-specified
318 values (in particular those of AASHTO for single-column bents) seem to significantly underestimate
319 the actual energy dissipation capacity of concrete bridges. Seen from another perspective, this is a
320 clear indication that modern bridges possess adequate margins of safety and are able to withstand
321 seismic actions that are often substantially higher than those used for their design. This high
322 performance is due to their ductility, as well as their overstrength; previous studies that have ignored
323 the latter led to deriving unrealistically low values of q -factors.

324 For bridges with yielding piers of the column type, for which the influence of higher modes is
325 significant in their transverse direction, it is recommended to use the multi-modal pushover curves
326 instead of the standard pushover curves to estimate the 'actual' available q -factor of the bridge.
327 Alternatively, dynamic pushover curves may also be used. On the other hand, when the first mode is
328 dominant (this is typically the case in the longitudinal directions of the bridge) the available q -factor
329 can be calculated using the standard (single-mode based) pushover curves since the difference
330 between the static and dynamic pushover curves is not significant. Importantly, if standard pushover is
331 used for estimating q -factors in the transverse direction, the resulting values are conservative.

332 The influence of the angle of incidence of the seismic action on the pushover curves and the
333 derived q -factors was also studied herein. It was found that although the angle of incidence of the
334 seismic action affects the results of both the available overstrength and ductility factor, the values
335 estimated for the transverse and longitudinal directions seem to bound the estimated values; hence,
336 bearing also in mind all the uncertainties involved, two analyses (longitudinal-transverse) of the bridge
337 are deemed to be sufficient.

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Table Captions

Table 1. Overstrength factor (q_s) and ductility-related factor (q_μ) for bridges with yielding piers of the column type.

Table 2. Characteristic bridge displacements, available ductility ratios, overstrength and ductility factors for Pedini bridge, for all angles of incidence.

Table 3. Available force reduction factor (q) for the selected bridges.

Figure Captions

Fig. 1. Pushover curve of a bridge with inelastically responding piers, (a) without abutment-backfill effect, (b) with abutment- backfill effect.

Fig. 2. Definition of the available q-factor.

Fig. 3. Main characteristics of the bridges selected for analysis.

Fig. 4. Dynamic ‘multi-modal’ pushover curves compared to a standard pushover curve for a bridge where higher modes are significant (T7 Bridge).

Fig. 5. Dynamic ‘multi-modal’ pushover curves compared to a standard pushover curve for a bridge where the 1st mode is dominant (G11 Bridge).

Fig. 6. Pushover curve of a bridge with elastomeric bearings and non-yielding piers.

Fig. 7. Pushover curves of Pedini Bridge for various angles of incidence of the seismic action.

Table 1. Overstrength factor (q_s) and ductility-related factor (q_μ) for bridges with yielding piers of the column type

Bridge name	Longitudinal direction		Transverse direction	
	q_s	q_μ	q_s	q_μ
Pedini	2.1	2.4 (4.0)*	5.8	2.1
T7	2.7	3.3 (5.6)	2.8	3.3
G11	2.9	2.4 (4.0)	1.5	2.5
G2	3.4	1.2 (2.0)	1.6	1.5
Krystallopiği	1.3	7.6 (12.7)	1.2	5.5

* Values in parentheses refer to the case that possible abutment-backfill failure is ignored

Table 2. Characteristic bridge displacements, available ductility ratios, overstrength and ductility factors for Pedini bridge*, for all angles of incidence.

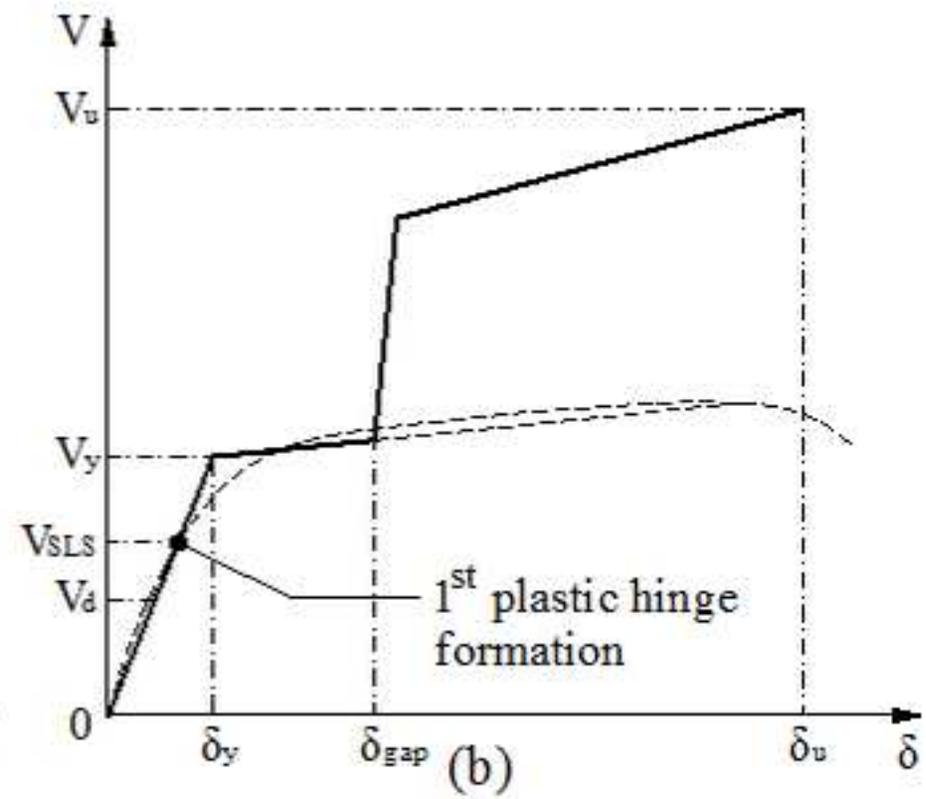
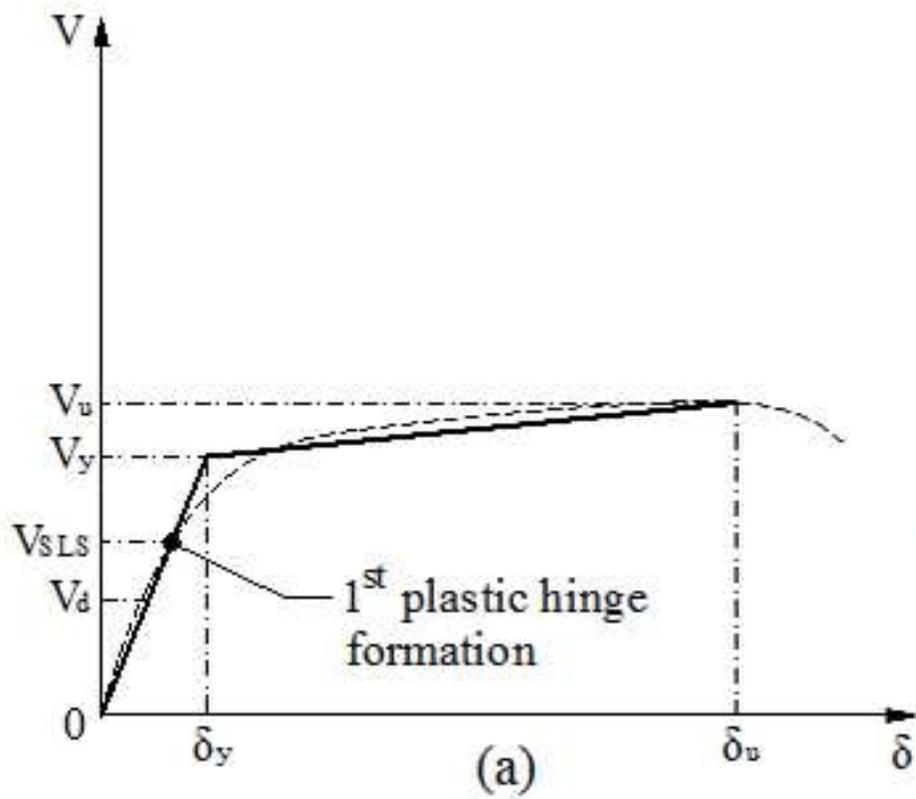
Angle of incidence [°]	δ_y [mm]	δ_u [mm]	q_s	q_μ
0	51.6	270.4	1.8	5.2
15	58.3	288.2	1.9	4.9
30	68.2	335.0	2.1	4.9
45	88.8	408.5	2.4	4.6
60	149.1	530.3	4.1	3.6
75	202.8	580.1	5.4	2.9
90	219.6	582.4	6.0	2.7

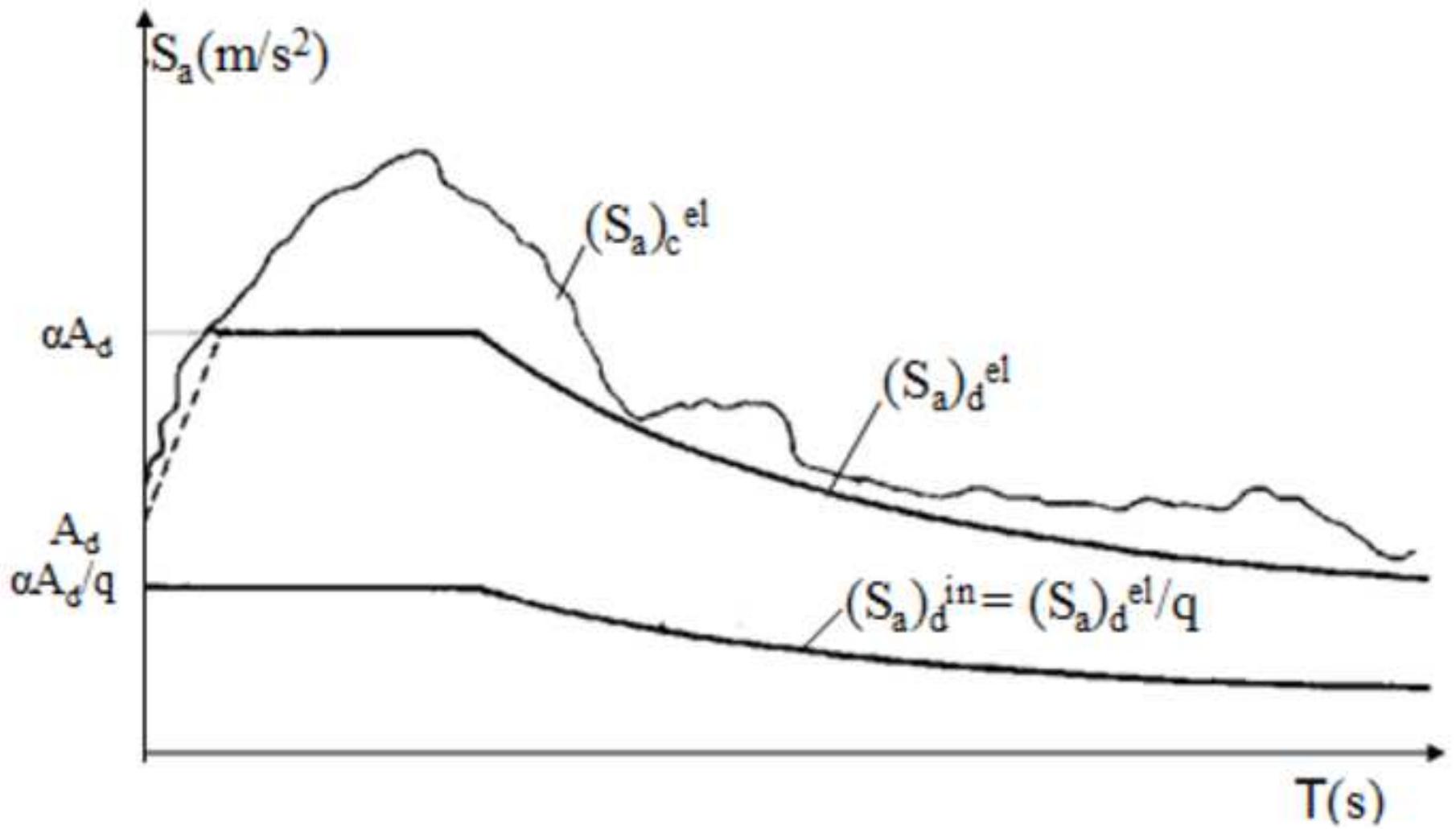
* Using model without foundation compliance

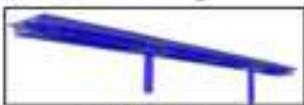
Table 3. Available force reduction factor (q) for the studied bridges.

	Bridge name	Longitudinal direction	Transverse direction
Bridges with yielding piers of the column type (q)	Pedini	5.0 (8.4)*	12.2
	T7	8.9 (15.1)	9.2
	G11	7.0 (11.6)	3.8
	G2	4.1 (6.8)	2.4
	Krystallopigi	9.9. (16.5)	6.6
Bridges with bearings and non-yielding piers (q_{eq})	G2 (approximate evaluation of δ_u')	3.9	-
	Lissos River	6.6	9.3
	Kossynthos River	4.2	4.3

* Values in parentheses refer to the case that possible abutment-backfill failure is ignored.





Bridge name and Structural configuration	No. of spans	Span length	Total length	Pier-to-deck connection	Curvature	Foundation
Pedini bridge 	3	19.0+32.0+ 19.0	70.0	monolithic	in height	pile groups
T7 bridge 	3	27.0+45.0+ 27.0	99.0	monolithic	no	Footings
G11 bridge 	3	64.3+ 118.6+64.3	247.2	monolithic	in plan	Caissons
Krystallopigi bridge 	12	44.17+ 10×54.98+ 44.17	638.19	monolithic/ through bearings	in plan	pile groups
Lissos river bridge 	11	1×29.56+ 3×37.05+ 6×44.35+ 1×26.50	433.31	through bearings	no	pile groups
Kossynthos river bridge 	5	35.0+3×36.0+ 35.0	178.0	through bearings	no	pile groups
G2 bridge 	3	30.7+ 31.7+30.7	93.1	through bearings	no	pile groups

