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

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        $$q_s = V_y / V_d \qquad (2)$$

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

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

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
$$q_{s(\mathbf{min})} = V_{SLS} / V_d \tag{5}$$

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
$$q_\mu = \begin{bmatrix} \sqrt{(2\mu\text{-}1)}, T < 0.5\text{s} \\ \mu \qquad , T > 0.5\text{s} \end{bmatrix} \tag{6}$$

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    $$\delta_u' = \alpha \cdot \delta_u \tag{7}$$

171   where δ_u is the ultimate displacement of the bridge without the abutment- backfill effect. The value for

172   a 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    $$q_{eq} = (S_{au}(T)/S_{ad}(T)) \cdot q_d \Rightarrow q_{eq} = S_{au}(T))/S_{ad}(T) \tag{8}$$

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).

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.

338 **Acknowledgements**

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387

Table Captions

Table 1. Overstrength factor ($\mathbf{q}_s$) and ductility-related factor ($\mathbf{q}_\mu$) 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
Krystallopigi	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.

(a)

(b)
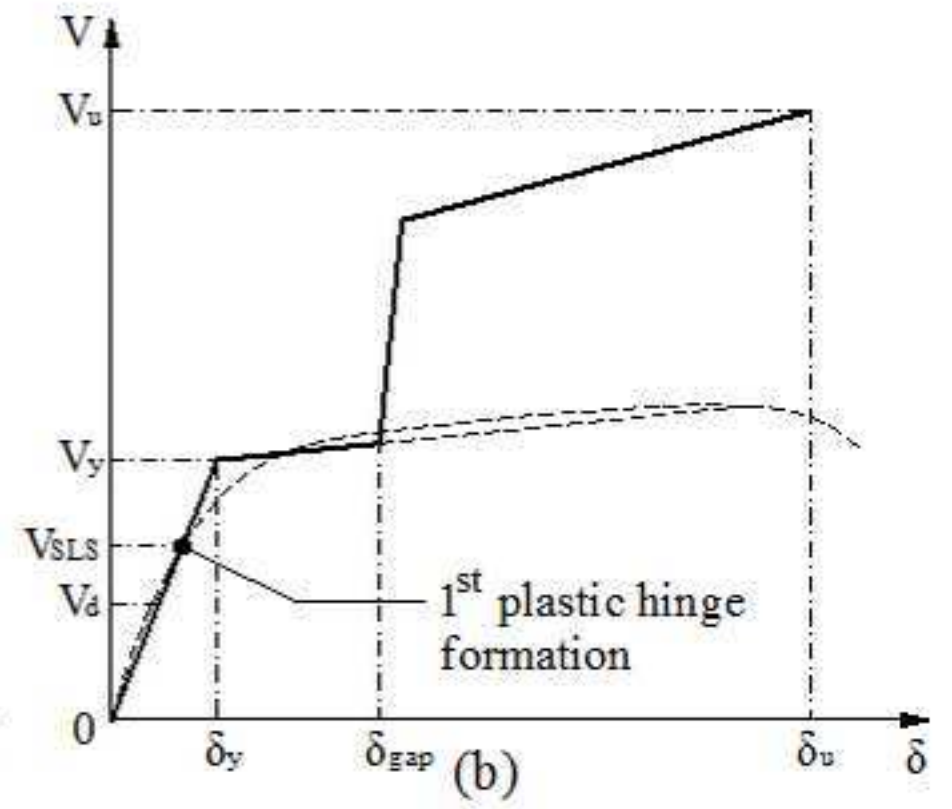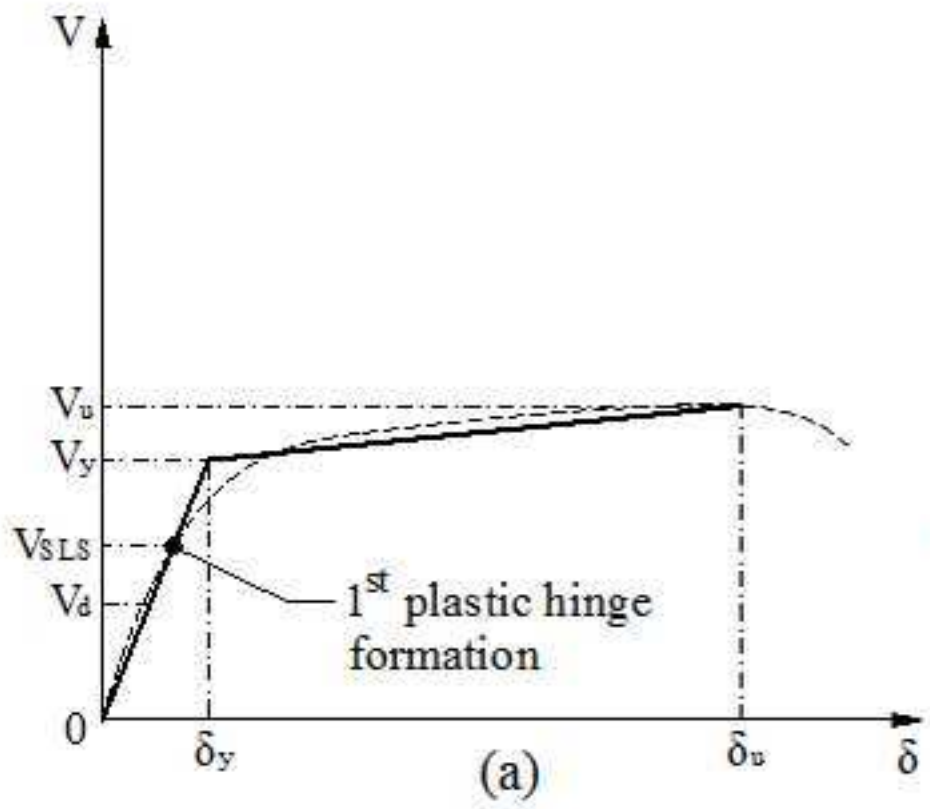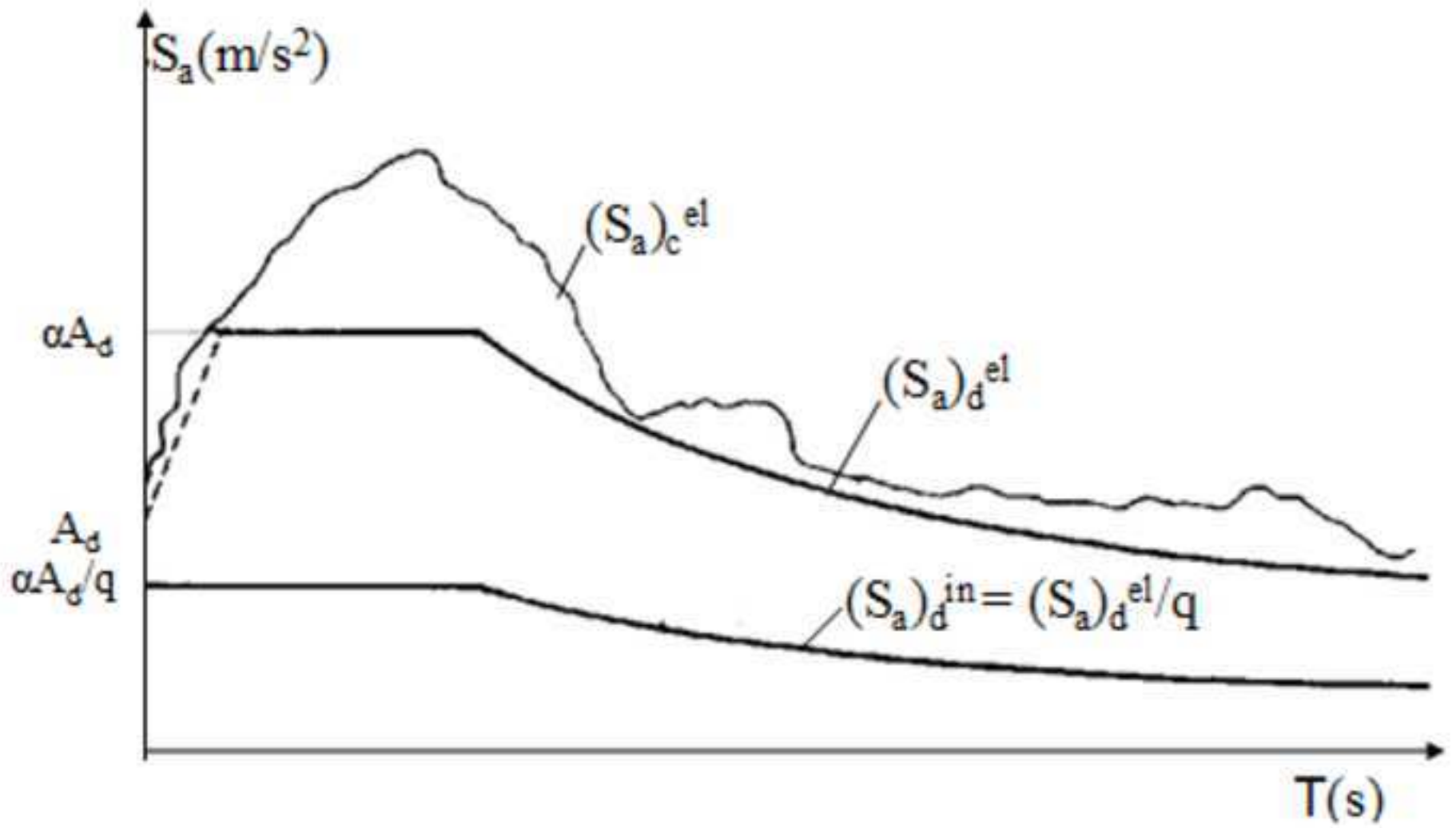
Bridge name and Structural configuration	No. of spans	Span length	Total length	Pier-to-deck connection	Curvature	Foundation
Pedini bridge	3	19.0+32.0+ 19.0	70.0	monolithic	in height	pile groups
T7 bridge	3	27.0+45.0+ 27.0	99.0	monolithic	no	Footings
G11 bridge	3	64.3+ 118.6+64.3	247.2	monolithic	in plan	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

(a) longitudinal direction

(b) transverse direction