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

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

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

The foregoing methodology is then used to answer the very legitimate (and relevant to practicing engineers) question ‘what are the actual q-factors of modern bridges?’ More specifically, the available $q$-factors (or $q_{eq}$-factors) are estimated for seven actual bridges, typical of those used in European motorways, in particular in Southern Europe, which is a high seismicity region. They include typologies of both the first (inelastically responding piers) and the second category (bearings on elastic piers), as well as a ‘mixed’ type of structure, combining features of both categories. The available force reduction factors calculated for these bridges are then compared with the values specified in the European (Eurocode 8) and North American (AASHTO) codes for seismic design of bridges.

## Methodology

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

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

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

### *Bridges with inelastically responding piers*

In bridges with yielding piers of the column type, pushover curves, i.e. plots of base shear vs. displacement of the ‘monitoring’ point on the deck (taken as the one above the critical pier or abutment) are derived by performing a standard (fundamental mode based) pushover analysis. Some of the bridges have also been analyzed using a modal pushover analysis for each mode independently (Paraskeva et al. 2006). When the modal pushover method is used, a "multi-modal" curve can be constructed by an appropriate combination of the values from individual curves (Kappos and Paraskeva 2008, Kappos et al. 2012). Alternatively, for bridges where the higher modes are significant (for the transverse response of the bridge) non-linear response history analysis may also be applied to derive dynamic pushover curves. The derived (through any of these procedures) pushover curve is then idealized as a bilinear one in order to define a conventional yield displacement, $\delta_y$ and ultimate displacement $\delta_u=\mu_u \cdot \delta_y$ , both referring to the entire bridge, not to a single pier ($\delta_u$ is taken here to correspond to a 20% drop in the base shear capacity, see Figure 1).

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

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

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

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

$$q_s = V_y / V_d \tag{2}$$

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

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

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

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

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

Obviously, when the pushover curve is idealized as elastic-perfectly-plastic, the two definitions of equations (2) and (4) coincide. A minimum value of the overstrength factor can be defined as the ratio of the strength of the structure at the time where the first plastic hinge takes place to the design base shear

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

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

The ductility factor, $q_\mu$, is derived as a function of the available ductility of the bridge, which is defined as the ratio of the ultimate limit state displacement ($\delta_u$) to the yield displacement ($\delta_y$), depending on the prevailing period. Veletsos and Newmark (1960) related $q_\mu$ to the kinematic ductility demand $\mu$ by the following expressions:

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

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

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

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

where $\delta_u$ is the ultimate displacement of the bridge without the abutment- backfill effect. The value for *a* was found to be about 0.6 for the analyzed overpass (Kappos & Sextos 2009); this approximate value of the $\delta_u'$ was used for bridges where the "full-range" analysis is not performed.

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

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

In the case of bridges with elastomeric bearings (with or without seismic links) and non-yielding piers of the wall type, pushover curves are derived by performing a standard pushover analysis given that the first (fundamental) mode of the bridge is similar to the first (fundamental) mode of the deck since the wall-type piers are much stiffer than the bearings, and as a consequence this mode has a very high participating mass ratio. In the longitudinal direction the first mode of the deck is a rigid-body displacement, while in the transverse direction it has a sinusoidal shape or it consists of a quasi-rigid-body displacement and rotation, depending on whether the transverse displacement of the deck at the abutments is restrained or free. In addition, the derived pushover curve has a bilinear shape because of the corresponding bilinear behavior of the bearings (Figures 6(a) and 6(b)). Note that in the usual case that common (low-damping ratio, $\zeta \approx 5\%$) bearings are used, the pushover curve is essentially a straight line, whose slope is defined by the effective shear stiffness that does not change substantially (the hysteresis loop of these bearings is very thin). The choice of this linear approximation is advisable for both the economy of the analysis procedure and the more accurate assessment of the target displacement, since the definition of the first branch of the bilinear diagram of the bearings is subject to substantial uncertainty. Whenever seismic links (stoppers) are present, the pushover curve has a similar shape but an apparent hardening/softening is noticed, due to the successive activation and failure, respectively, of seismic links (Fig. 6(b)).

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

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

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

## Available behavior factors for concrete bridges

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

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

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

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

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

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

*Some comparisons with code-specified values*

The estimated available force reduction factors for the typical bridges studied here can be compared with values prescribed by modern seismic codes. Eurocode 8 – Part 2 (CEN 2005) qualifies for the most direct comparison, since the studied bridges were designed according to provisions that are similar, albeit not identical, to those of this code. For concrete bridges with piers expected to yield under the design earthquake the Eurocode specifies a behavior factor equal to $3.5\lambda(\alpha_s)$ for ductile 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 span of the pier columns and *h* the depth of their cross-section in the direction of flexure of the plastic hinge), which implies that its response is predominantly flexural, whereas for $3 > \alpha_s > 1$, $\lambda(\alpha_s) = \sqrt{\alpha_s / 3}$. For the studied bridges in this category a value of 3.5 would be appropriate ($\alpha_s \geq 3$ for most columns); this does not necessarily mean that this was indeed the q-factor used in their design, since minor discrepancies exist between Eurocode 8 and the previous Greek Code (for instance, q=3.5 applied for $\alpha_s \geq 3.5$, in lieu of 3). Notwithstanding the aforementioned minor discrepancies, the fact that the estimated q-factors (Table 3) vary from 4.2 to 10.1 in the longitudinal direction and from 3.7 to 11.6 in the transverse direction, is a clear indication that the code-prescribed value is not only feasible but in several cases is actually an underestimation of the actual energy dissipation capacity of the bridge, which is the result primarily of its ductility, but also of its overstrength.

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

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

## Conclusions

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

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

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

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

**Acknowledgements**

Part of the work presented herein has been performed within the framework of the research project 'ASProGe: Seismic Protection of Bridges', funded by the General Secretariat of Research and Technology (GGET) of Greece.

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**Table Captions**

Table 1. Overstrength factor ($\boldsymbol{q_s}$) and ductility-related factor ($\boldsymbol{q_\mu}$) 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 1$^{st}$ 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 (***$q_\mu$***) for bridges with yielding piers of the column type

| Bridge name | Longitudinal direction | | Transverse direction | |
|---|---|---|---|---|
| | $q_s$ | $q_\mu$ | $q_s$ | $q_\mu$ |
| 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.

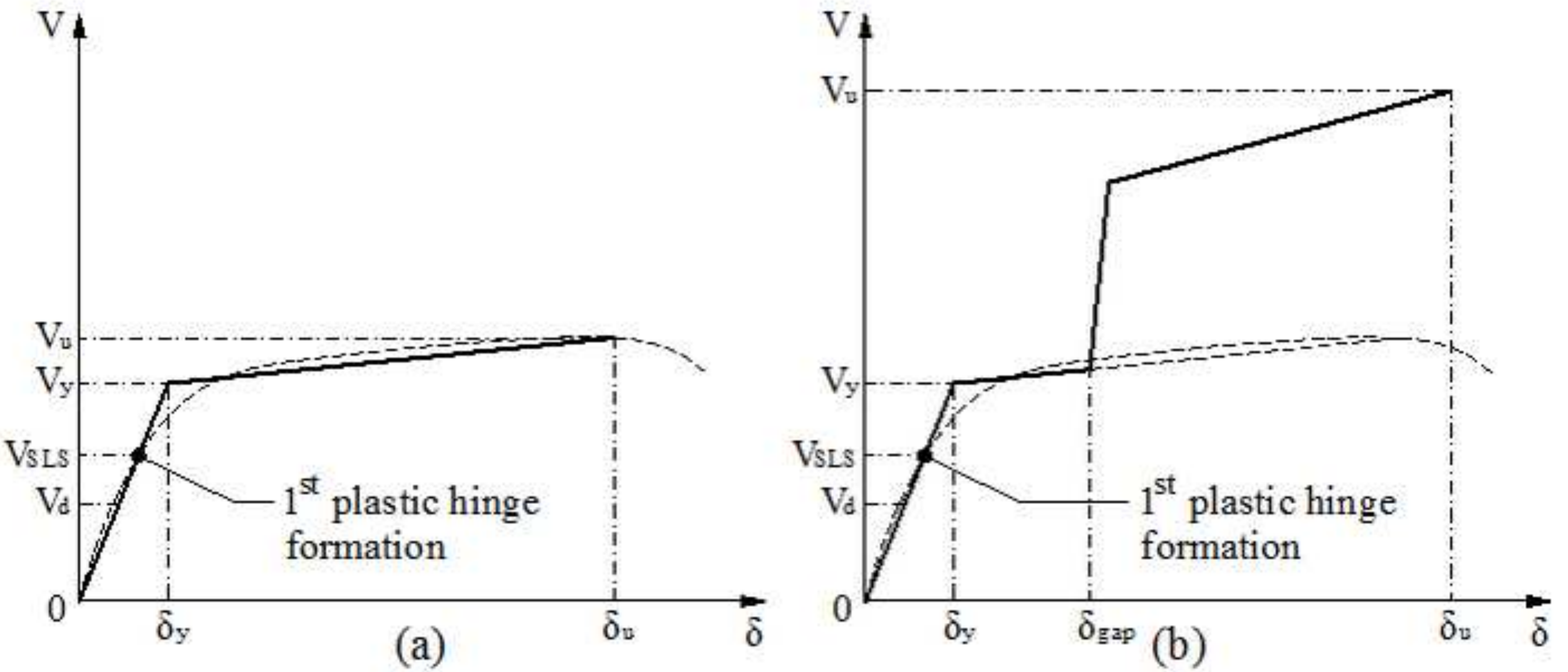
| Angle of incidence [$^o$] | $\delta_y$ [mm] | $\delta_u$ [mm] | $q_s$ | $q_\mu$ |
|---|---|---|---|---|
| 0 | 51.6 | 270.4 | 1.8 | 5.2 |
| 15 | 58.3 | 288.2 | 1.9 | 4.9 |
| 30 | 68.2 | 335.0 | 2.1 | 4.9 |
| 45 | 88.8 | 408.5 | 2.4 | 4.6 |
| 60 | 149.1 | 530.3 | 4.1 | 3.6 |
| 75 | 202.8 | 580.1 | 5.4 | 2.9 |
| 90 | 219.6 | 582.4 | 6.0 | 2.7 |

* Using model without foundation compliance

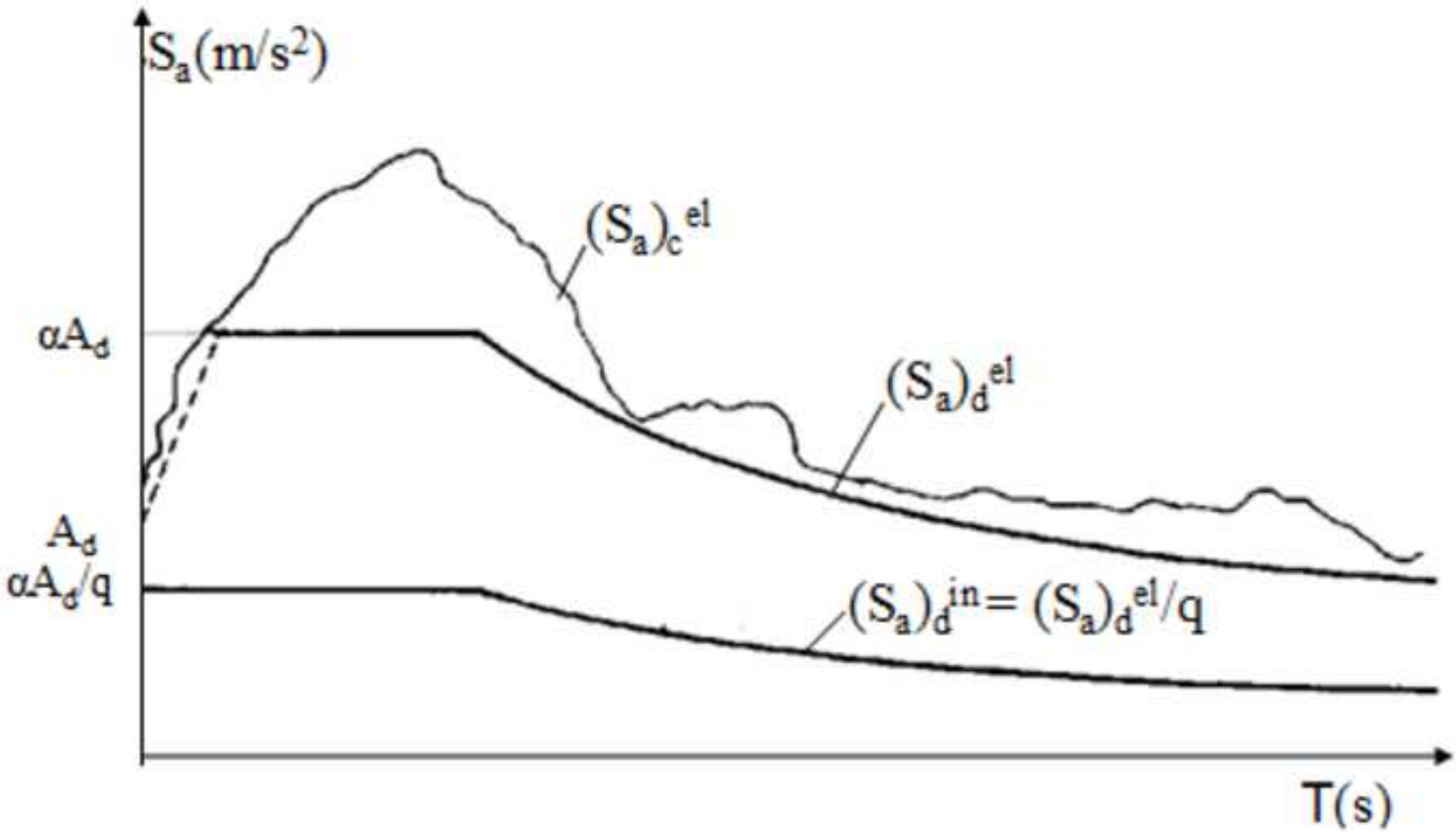
Table 3. Available force reduction factor (*q*) for the studied bridges.

| | 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 $\delta_u'$) | 3.9 | - |
| | Lissos River | 6.6 | 9.3 |
| | Kossynthos River | 4.2 | 4.3 |

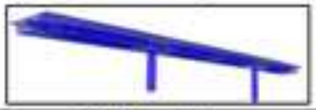
* Values in parentheses refer to the case that possible abutment-backfill failure is ignored.

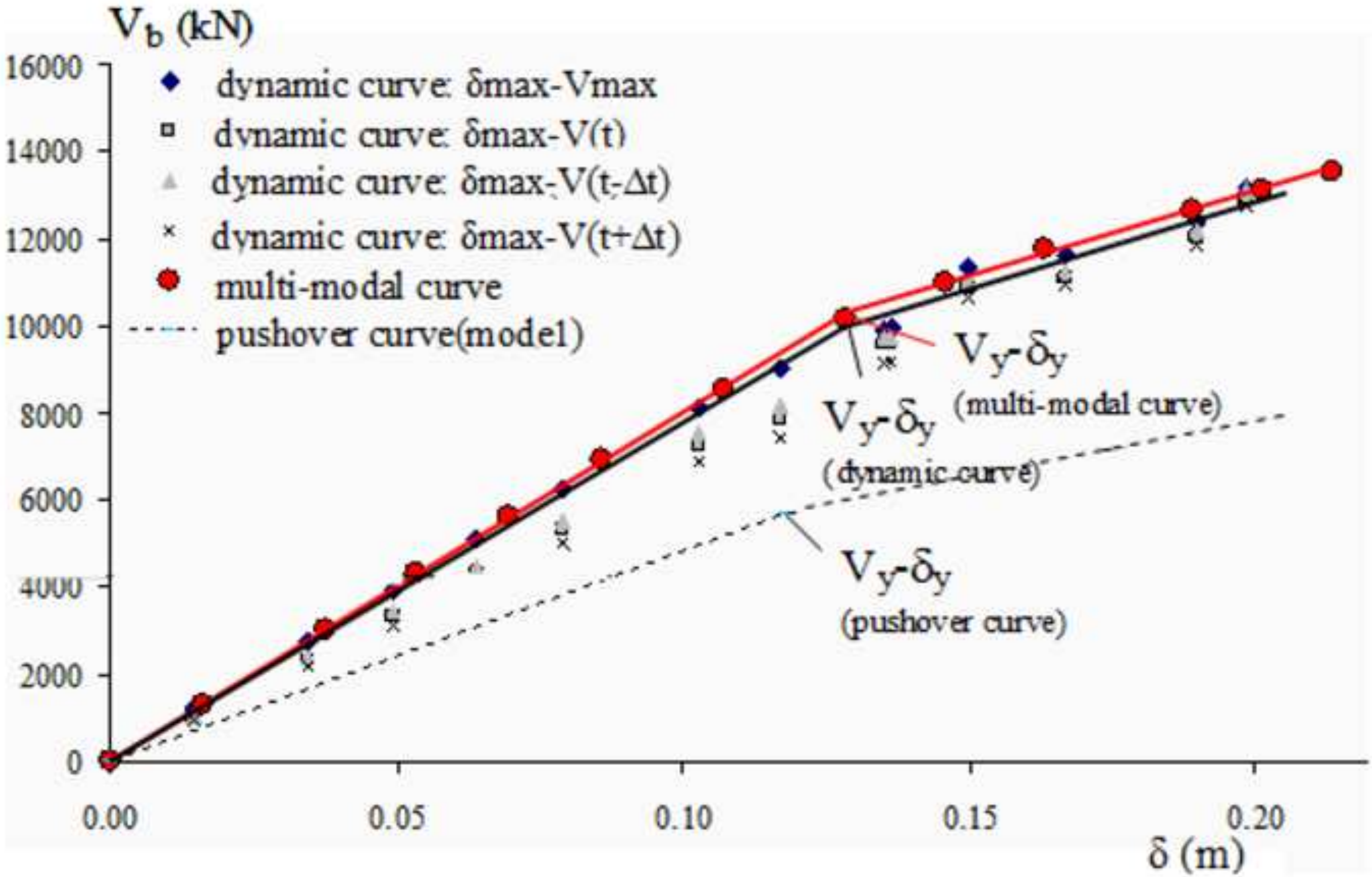


V
Vu
Vy
VSLS
Vd
0
δy
δu
δ
1st plastic hinge formation
(a)
V
Vu
Vy
VSLS
Vd
0
δy
δgap
δu
δ
1st plastic hinge formation
(b)

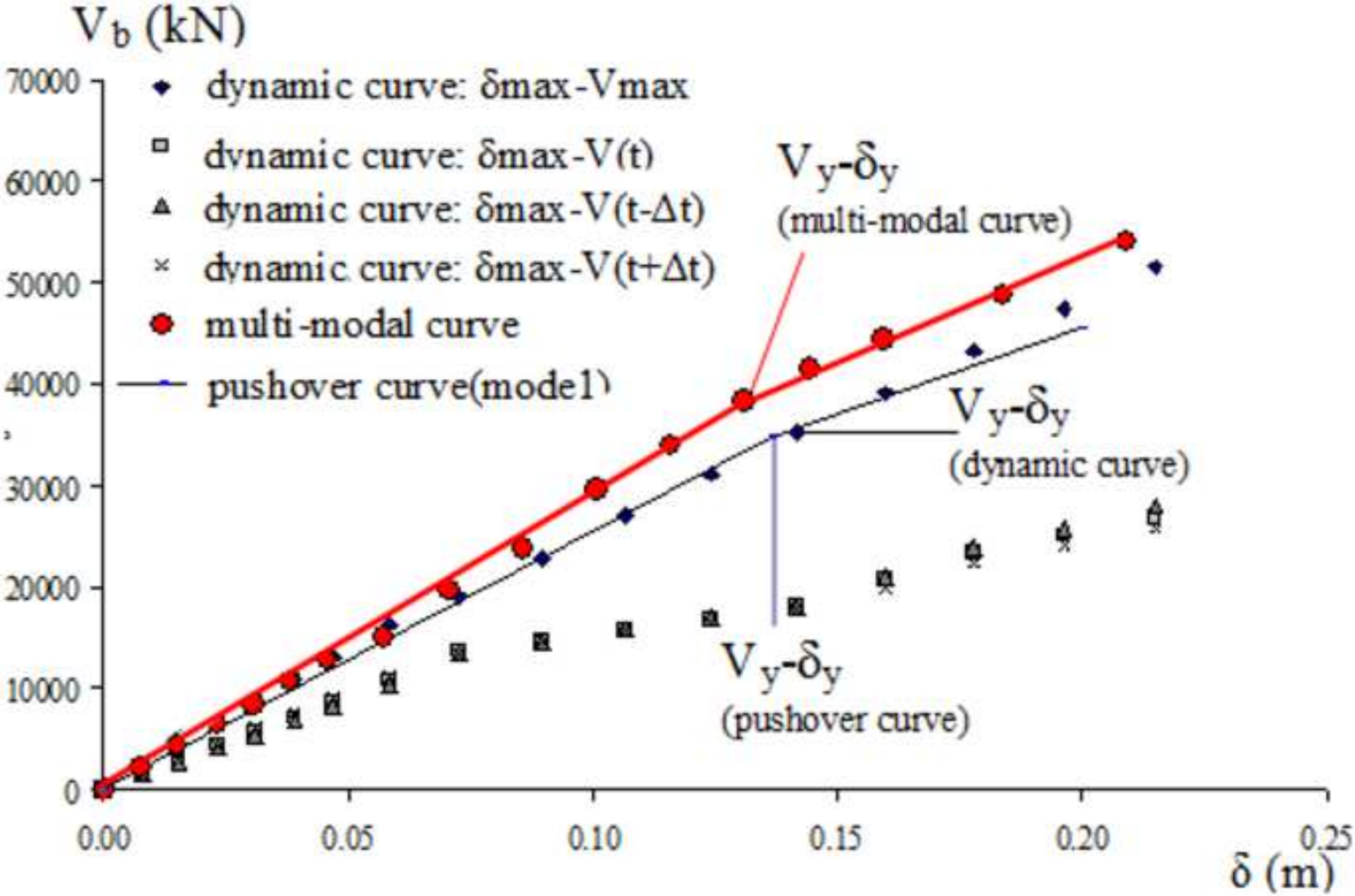


$S_a(m/s^2)$
$(S_a)_c^{el}$
$\alpha A_d$
$(S_a)_d^{el}$
$A_d$
$\alpha A_d/q$
$(S_a)_d^{in} = (S_a)_d^{el}/q$
T(s)

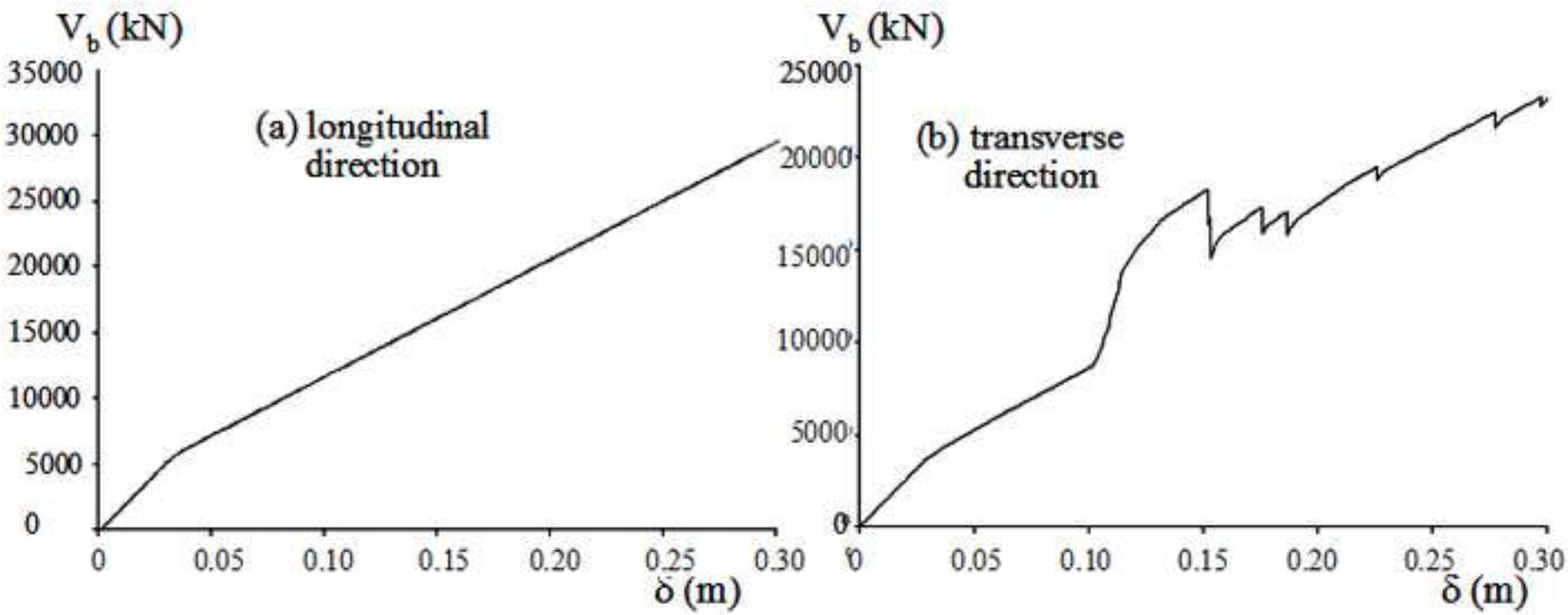
| Bridge name and Structural configuration | No. of spans | Span length | Total length | Pier-to-deck connection | Curvature | Foundation |
|---|---|---|---|---|---|---|
| Pedini bridge  | 3 | 19.0+32.0+19.0 | 70.0 | monolithic | in height | pile groups |
| T7 bridge  | 3 | 27.0+45.0+27.0 | 99.0 | monolithic | 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 |



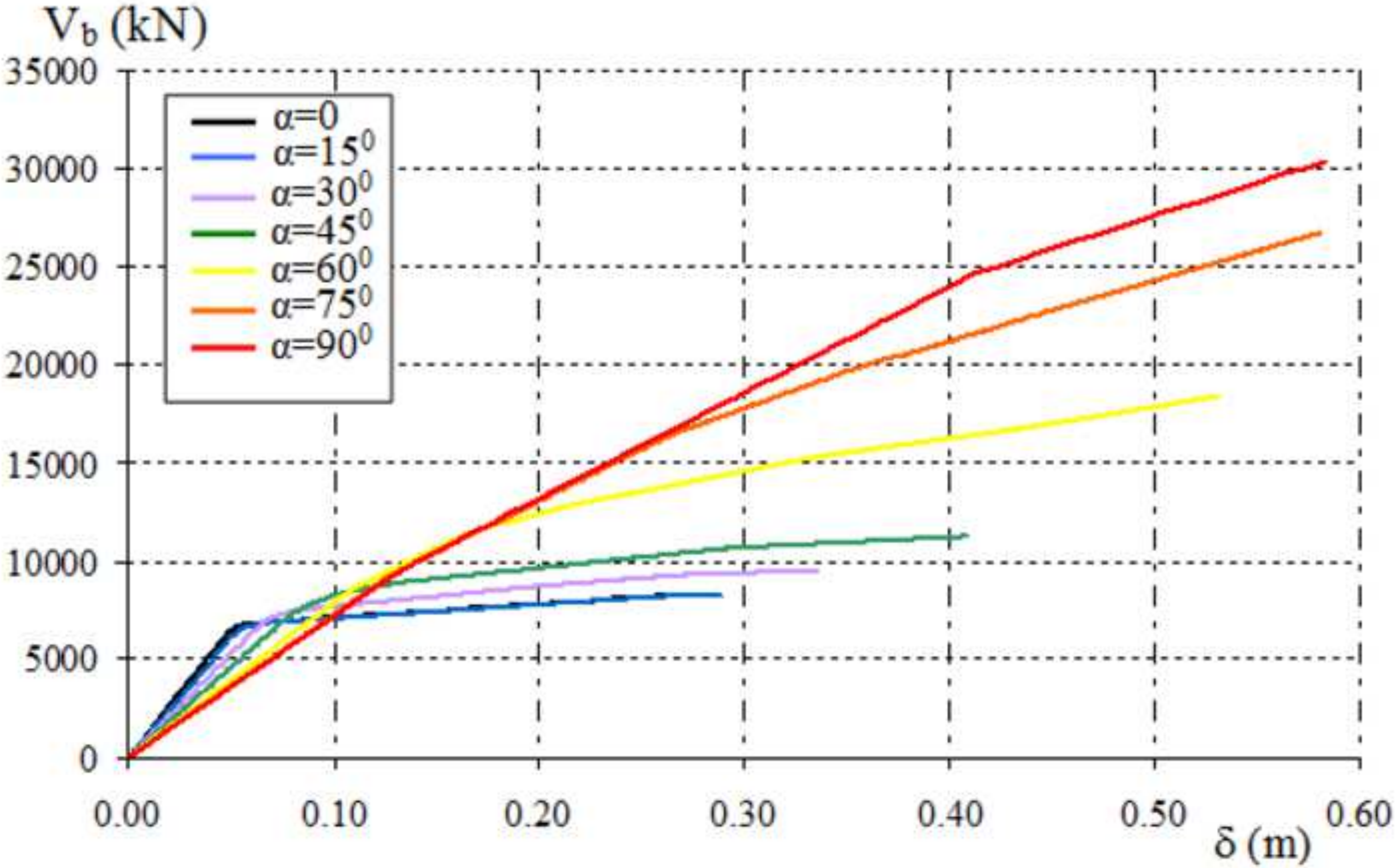
V_b (kN)
16000
14000
12000
10000
8000
6000
4000
2000
0
dynamic curve: δmax-Vmax
dynamic curve: δmax-V(t)
dynamic curve: δmax-V(t-Δt)
dynamic curve: δmax-V(t+Δt)
multi-modal curve
pushover curve(mode1)
Vy-δy
(multi-modal curve)
Vy-δy
(dynamic curve)
Vy-δy
(pushover curve)
0.00
0.05
0.10
0.15
0.20
δ (m)



V_b (kN)
70000
60000
50000
40000
30000
20000
10000
0
dynamic curve: δmax-Vmax
dynamic curve: δmax-V(t)
dynamic curve: δmax-V(t-Δt)
dynamic curve: δmax-V(t+Δt)
multi-modal curve
pushover curve(mode1)
Vy-δy
(multi-modal curve)
Vy-δy
(dynamic curve)
Vy-δy
(pushover curve)
0.00
0.05
0.10
0.15
0.20
0.25
δ (m)



V_b (kN)
35000
30000
25000
20000
15000
10000
5000
0
(a) longitudinal direction
0
0.05
0.10
0.15
0.20
0.25
0.30
δ (m)
V_b (kN)
25000
20000
15000
10000
5000
0
(b) transverse direction
0
0.05
0.10
0.15
0.20
0.25
0.30
δ (m)



V_b (kN)
35000
30000
25000
20000
15000
10000
5000
0
α=0
α=15⁰
α=30⁰
α=45⁰
α=60⁰
α=75⁰
α=90⁰
0.00
0.10
0.20
0.30
0.40
0.50
0.60
δ (m)