Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1638

EVALUATION OF AVAILABLE FORCE REDUCTION FACTORS FOR CONCRETE BRIDGES

A. J. Kappos[1], Th. S. Paraskeva[2] and J. F. Moschonas[3]

ABSTRACT

The paper presents a methodology for evaluating available force reduction factors (R) for seismic design of concrete bridges. The usual procedure for analytically estimating this 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 with bearings and strong, elastically responding, piers. The methodology was applied for evaluating the available R-factors (for bridges of the first category) or R_{eq}-factors (for bridges of the second category) of seven actual bridges, forming part of Egnatia Highway, in Greece. It was found that in all cases the available force reduction factors were higher than those used for design.

Introduction

A critical issue of seismic assessment of bridges is the relationship between the design seismic action and that for which failure of the bridge occurs. This study focuses on the estimation of ductility and overstrength factors (the two components of the available force reduction factor, see Kappos 1999) for actual concrete bridges. Evaluating the 'actual' value of R 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. The procedure for analytically estimating this factor is based on nonlinear static (pushover) analysis of the entire bridge, wherein pushover curves are derived for the bridge in (at least) its longitudinal and transverse direction. In the present study pushover curves are also derived for an arbitrary angle of incidence of the seismic action using a procedure recently developed by Moschonas and Kappos (2009). 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

[1]Professor, Dept. of Civil Engineering, Aristotle University of Thessaloniki, Greece. email:ajkap@civil.auth.gr
[2] Civil Engineer (MSc), PhD Candidate, Department of Civil Engineering, Aristotle University of Thessaloniki, Greece email: thparask@civil.auth.gr
[2] Civil Engineer (MSc), PhD Candidate, Department of Civil Engineering, Aristotle University of Thessaloniki, Greece email: jfmoschon@kat.forthnet.gr

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, with a view to defining a (conventional) yield point and an ultimate point. The available R-factor (in each direction) is then estimated as the product of two components, a ductility-based one, and an overstrength-based one ($R=R_{\mu}\cdot R_s$). The overstrength factor, R_s, is defined as the ratio of (conventional) yield strength to the design base shear (in the pertinent direction), while the ductility factor, R_{μ}, is derived as a function of the available ductility of the bridge ($\mu=\delta_u/\delta_y$) depending on the prevailing period (equal energy or equal displacement approximation, the latter being commonly the case in bridges). For bridges of the second category, wherein the deck is resting on elastically responding piers through bearings, a different procedure is proposed herein, since no meaningful bilinear pushover curves can be derived in this case. Hence the concept of equivalent R-factor is introduced, which is defined as the ratio of the spectral acceleration (corresponding to the pertinent prevailing period of the bridge) for which failure occurs, to the design spectral acceleration.

The foregoing methodology was applied for evaluating the available R-factors (or R_{eq}-factors) of seven actual bridges, forming part of the Egnatia Highway (in Nothern Greece). Four of them belong to the first category (inelastically responding piers), two to the second one (bearings on elastic piers) and one is an interesting ‘mixed’ type of structure, combining features of both categories. It was found that for all bridges and in both directions the available force reduction factors were higher than those used for design. It was further noted that while in some of the bridges one of the principal directions (longitudinal or transverse) was the most critical one (lowest R), this was not the case with other bridges wherein ‘intermediate’ directions were more critical, mainly due to different available ductilities in each direction.

Procedure for evaluating the available R- factors for concrete bridges

The procedure for analytically estimating the available force reduction factors is based on nonlinear static (pushover) analysis of the entire bridge. A critical issue that differentiates the way of evaluating the aforementioned factors is the seismic energy dissipation mechanism of the bridges; accordingly, bridges are classified into two main categories:

1. 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 one of the solutions used in modern ravine bridges in Europe. The inelastic behaviour is developed due to the formation of plastic hinges at the pier base, and also the top, if the pier- to – deck connection allows the development of substantial bending moment.

2. Bridges with bearings (with or without seismic links, like stoppers) and non- yielding piers of the wall type. The inelastic behaviour develops due to the inelastic ressponse of bearings and seismic links. In most cases the deck is supported by massive wall-type piers which remain in the elastic range even for high levels of seismic action.

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, whose slope is defined by the effective stiffness of the bearings. Reinforced concrete members are modelled using the point hinge model of SAP2000 (CSI, 2005) with multilinear moment – rotation law for each hinge, accounting for residual strength after exceeding the available rotational capacity; relevant details are given in Kappos et al. (2007).

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 analysed 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 (Paraskeva and Kappos, 2009). For bridges where higher modes are significant (for response in the transverse direction of the bridge) non-linear Response History Analysis may also be applied to derive the more rigorous 'dynamic pushover' curves. However, to retain uniformity among all typologies studied, such 'multi-modal' or 'dynamic' curves are not further addressed herein. The derived pushover curve is then idealized as a bilinear one (ATC, 1996, ASCE, 2000) in order to define a conventional yield displacement, δ_y and ultimate displacement $\delta_u=\mu_u{\cdot}\delta_y$, both referring to the entire bridge, not to a single pier; δ_u corresponds to more than 20% drop in the base shear capacity (Figure 1).

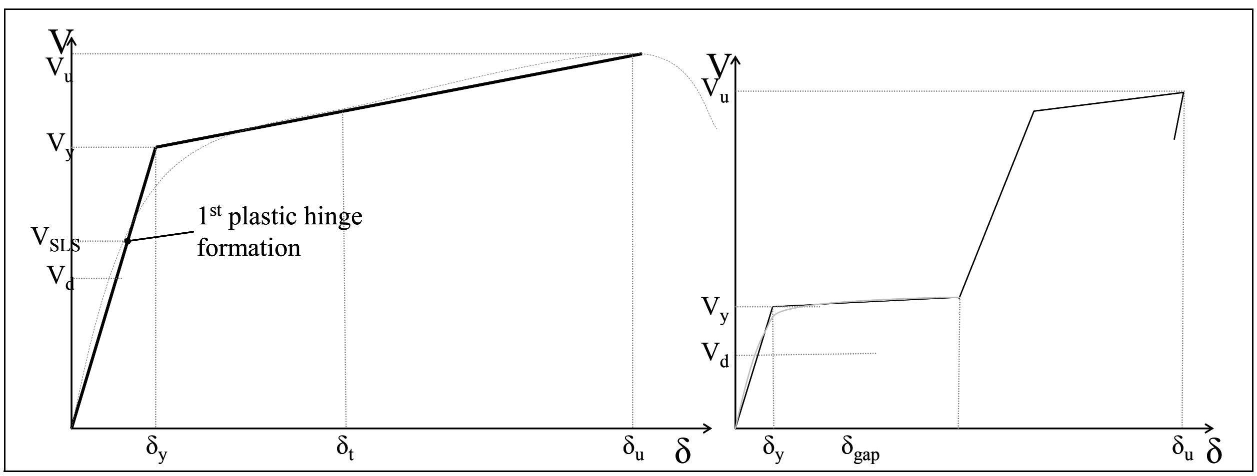


Figure 1. Pushover curve of a bridge with inelastically responding piers: without abutment-backfill effect (left); with abutment-backfill effect (right).

Overstrength-dependent component (R_s)

The overstrength-dependent component of R, is commonly defined as the ratio of the yield strength to the design base shear of the structure (eq. 1)

$$R_s = (V_y/V_d) \quad (1)$$

where V_y is the (conventional, i.e. derived from a bilinearization of the actual curve) yield strength and V_d is the design base shear of the structure. The overstrength factor (upper limit) can also be defined as the ratio of the ultimate strength to the design base shear of the structure

$$R_{s(max)} = (V_u/V_d) \qquad (2)$$

where V_u is the (conventional) ultimate strength of the bridge. Similarly, 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 forms to the design base shear

$$R_{s(min)} = (V_{SLS}/V_d) \qquad (3)$$

where V_{SLS} is the strength of the bridge when the first plastic hinge formation occurs (it is noted that, for a deterministic assessment, mean values of material strength must be applied). In the longitudinal direction, the activation of the abutment-backfill system due to closure of the gap between the deck and the abutments strongly affect the damage mechanism and an increases in the total strength of the bridge is noted (see fig. 1-right). In this case, the overstrength factor is estimated from eq. 1 and is not affected by the new seismic energy dissipation mechanism.

Ductility-dependent component (R_μ)

The ductility-dependent component, R_μ, is derived as a function of the available ductility of the bridge, which is defined as the ratio of the ultimate limit state displacement (δ_u) to the yield displacement (δ_y), depending on the prevailing period. Veletsos and Newmark (1960) related R_μ to the kinematic ductility demand μ by the now familiar expressions:

$$R_\mu = \begin{cases} \sqrt{2\mu - 1}, \ T < 0.5s \\ \mu, \ T \geq 0.5s \end{cases} \qquad (4)$$

which are based on the equal energy and the equal displacement approximations; note that usually T>0.5s for concrete bridges.

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 modelling of the abutment-backfill system (in both the longitudinal and the transverse direction), including soil flexibility (nonlinear behaviour and consideration of both stiff and soft soils) and pile non-linearity (in flexure and in shear), was made in the case of Pedini bridge (Kappos et al. 2007). 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-right) and the additional parameter that has to be defined is the displacement at failure of the abutment-backfill system, δ_u'. In the more common in design practice case that the analysis of the bridge is performed ignoring the abutment-backfill effect, the following approximate ultimate displacement of the bridge is suggested

$$\delta_u' = a\, \delta_u \tag{5}$$

where δ_u' is the ultimate displacement of the bridge with the abutment-backfill effect and δ_u is the ultimate displacement of the bridge without the abutment-backfill effect. The value for a was 0.63 for the Pedini bridge (Kappos et al. 2007). The approximate value of δ_u' is applied to bridges whenever a "full-range" analysis is not performed. Due to the approximate nature of eq. (5) additional parameters must be defined in order to take into account the displacement at gap closure, δ_{gap} in the longitudinal direction of the structure, i.e.

$$\delta_u' = \max\begin{Bmatrix} \delta_u, \text{for } \delta_u < 1.1_{DS3} \\ \max(\alpha \cdot \delta_u, 1.1_{DS3}) \end{Bmatrix} \tag{6a}$$

$$\delta_{DS3} = \begin{cases} 3.0\delta_y \\ \delta_y + \frac{2}{3}(\delta_u - \delta_y) \\ 1.2\delta_{gap} \end{cases} \tag{6b}$$

wherein δ_y is the yield displacement δ_{gap} is the displacement at gap closure and δ_{DS3} is the displacement at the threshold of 'damage state 3 - major damage' (see Moschonas et al. 2009).

R- factor (R)

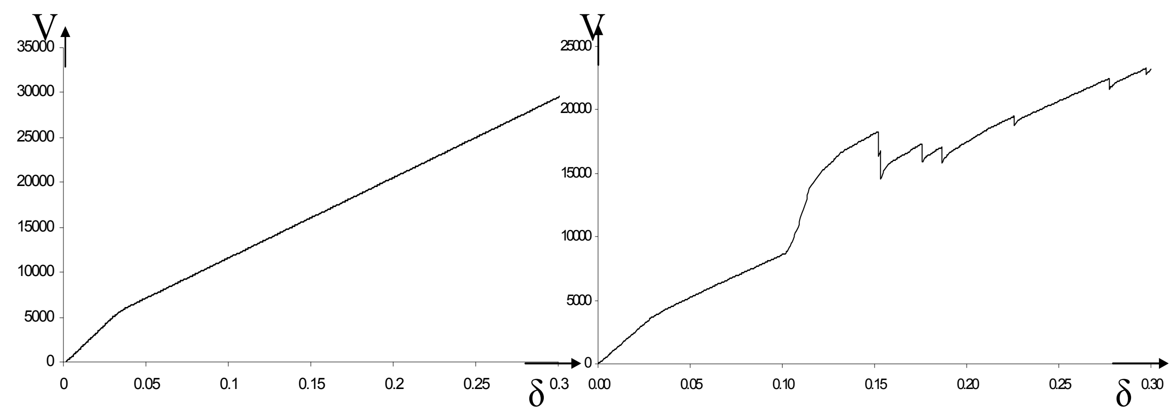
By definition, the value of the force reduction factor (R) for a structure is

$$R=(S_a)_d^{el}/(S_a)_d^{in}=F_{el}/F_d=(F_{el}/F_y)\cdot(F_y/F_d)=R_s\cdot R_\mu \tag{7}$$

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, 1999).

Bridges with bearings and elastically responding piers

In the case of bearing-supported bridge decks (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 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 rigid-body displacement and rotation, depending on whether the transverse displacement of the deck at the abutments is restrained or not. In addition, the derived pushover curve has already a bilinear shape because of the corresponding bilinear behaviour of the bearings (Figures 2(a) and 2(b)). 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. 2(b)). Furthermore, in the case of common elastomeric bearings the bilinear behaviour may be represented by a simple quasi-elastic (linear) behaviour given that the hysteresis loop of these bearings is very thin (low equivalent damping ratio, $\zeta \approx 5\%$). This choice is advisable for both the economy of the analysis 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 uncertainty.



(a) longitudinal direction (b) transverse direction

Figure 2. Pushover curve of a bridge resting through bearings on non-yielding piers.

For bridges wherein the deck rests through bearings on elastic piers, a different procedure for evaluating the force reduction factor is proposed herein, since no meaningful bilinear pushover curves can be derived in this case. Hence the concept of equivalent R-factor is introduced, which is defined as the ratio of the (elastic) 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 Kappos 1991)

$$R_{eq} = S_{au}(T)/S_{ad}(T) \tag{8}$$

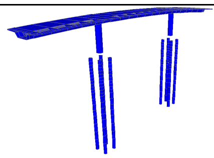
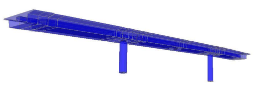
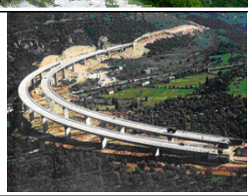
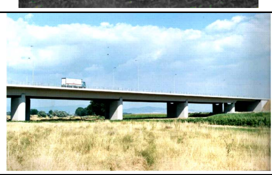
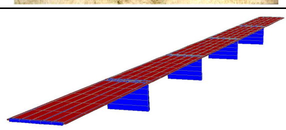
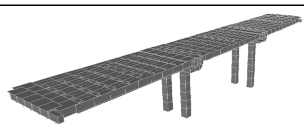
.

Available force reduction factors for concrete bridges

Case studies

Seven common types of bridges along the Egnatia Highway were selected. Four of them belong to the first category (inelastic piers), two to the second one (bearings on elastic piers) and one is a ‘mixed’ type of structure, combining features of both categories. The main characteristics of the selected bridges are given in Table 1.

Table 1. Main characteristics of the bridges selected for analysis.

Structural configuration	**Bridge name and class***	**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 (Section 14.1.2) bridge	3	27.0+ 45.0+ 27.0	99.0	monolithic	no	footings
	G11 bridge (right branch)	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 (Section 1.1.6) Bridge	3	30.7+ 31.7+ 30.7	93.1	through bearings	no	pile groups

Overstrength and ductility factor

The selected bridges were assessed using standard (fundamental mode based) pushover analysis for the longitudinal as well as the transverse direction. The corresponding pushover curves were constructed and then idealized as a bilinear curve in order to define a conventional yield displacement, δ_y and ultimate displacement δ_u. The overstrength-dependent components of R for bridges with yielding piers are given in Table 2, whereas the ductility-dependent components for the same bridges are given in Table 3.

Table 2. Overstrength factor R_s for bridges with yielding piers of the column type.

Bridge name	Longitudinal direction	Transverse direction
Pedini bridge	2.05	5.83
T7 (section 14.1.2) bridge	1.71	1.31
G11 bridge (right branch)	2.88	1.49
G2 bridge (section 1.1.6)	3.41	1.59
Krystallopigi bridge	1.32	1.22

Table 3. Ductility factor R_μ for bridges with yielding piers of the column type.

Bridge name	Longitudinal direction	Transverse direction
Pedini bridge	2.36	2.07
T7 (section 14.1.2) bridge	5.25	7.10
G11 bridge (right branch)	2.44	2.48
G2 bridge (section 1.1.6)	1.23	1.48
Krystallopigi bridge	7.62	5.53

Static pushover curves for Pedini Bridge 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 (2009), to investigate the influence of the shape of the pushover curve on the estimation of both ductility and overstrength-dependent components of R. Pushover curves were plotted on the same diagram (Figure 3), from which a rather smooth and gradual transition from the pushover curve in the longitudinal direction to that in the transverse direction is observed, as expected for a symmetric bridge such as the Pedini overpass. The conventional yield displacement, δ_y, ultimate displacement, δ_u, the ductility factor and the overstrength, as well as the resulting R-Factor for all angles of incidence are reported in Table 4. The R_μ-factor was calculated using eq. 4, ignoring the displacement at gap closure. It is noted that the angle of incidence of the seismic action affect the results of both the available overstrength and ductility factor; the resulting R-factor also varies, and the two main directions do not correspond to the minimum and maximum R values (differences are small, though).

Table 4. Characteristic bridge displacements, available ductility ratios, overstrength and ductility factors for Pedini bridge, for different angles of incidence

Angle of incidence [°]	δ_y [mm]	δ_u [mm]	R_s	R_μ	R
0	51.6	270.4	2.1	5.2	10.7
15	58.3	288.2	2.1	4.9	10.3
30	68.2	335.0	2.1	4.9	10.5
45	88.8	408.5	2.2	4.6	10.2
60	149.1	530.3	4.3	3.6	15.5
75	202.8	580.1	5.1	2.9	14.6
90	219.6	582.4	5.8	2.6	15.4

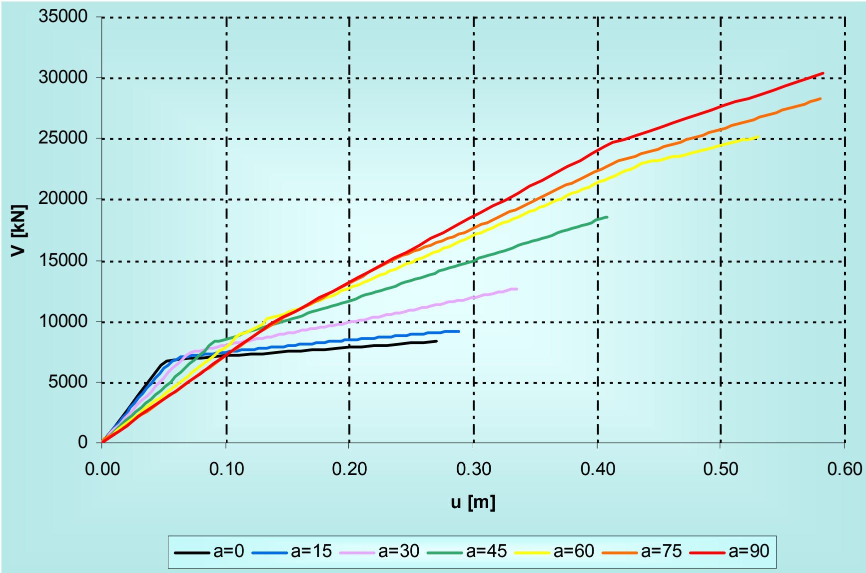


Figure 3. Pushover curves for Pedini Bridge, for various angles of incidence of the seismic action.

Available force reduction factor

For bridges of the first category, the available R-factor (in each direction) is estimated as the product of two components, a ductility based one, and an overstrength-based one ($R=R_{\mu}\cdot R_s$), whereas for bridges of the second category the concept of equivalent R-factor is introduced, which is defined as the ratio of the spectral acceleration (corresponding to the pertinent prevailing period of the bridge) for which failure occurs, to the design spectral acceleration.

Table 5. Available force reduction factor for the selected bridges.

	Bridge name	**Longitudinal direction**	**Transverse direction**
Bridges with yielding piers of the column type (***R***)	Pedini bridge	4.8	11.6
	T7 (section 14.1.2) bridge	8.9	9.3
	G11 bridge (right branch)	7.0	3.7
	G2 bridge (section 1.1.6)	4.2	2.4
	Krystallopigi bridge	10.1	6.8
Bridges with bearings and non-yielding piers (***R_{eq}***)	G2 bridge (approximate evaluation of δ_u')	3.9	-
	Lissos River Bridge	6.6	9.3
	Kossynthos River Bridge	4.2	4.3

Conclusions

A methodology for evaluating available force reduction factors, R, for seismic design of concrete bridges was proposed. A key aspect of the approach is that it differentiates the way of evaluating the R-factors depending on the seismic energy dissipation mechanism of the bridge. The methodology was applied for evaluating the available R-factors (for bridges with yielding piers) or R_{eq}-factors (for bridges with bearings and non-yielding piers) of seven actual bridges, part of Egnatia Higway, in Greece. It was found that in all cases the available force reduction factors (in each direction) were higher than those used for design. It was further noted that while in some of the bridges one of the principal directions (longitudinal or transverse) was the most critical one (lowest R), this was not the case with other bridges wherein other directions were more critical, mainly due to different available displacement ductilities in each direction.

References

ASCE (American Society of Civil Engineers), 2000, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356: Washington DC

ATC, 1996 "*Seismic Evaluation and Retrofit of Concrete Buildings*", Rep. No. SSC 96-01: ATC-40,1, Redwood City, CA

Computers and Structures Inc, 2005. SAP2000: Linear and Non linear Static and Dynamic Analysis and Design of Three-Dimensional Structures. Berkeley, California

Kappos, A.J., 1991. Analytical Prediction of the Collapse Earthquake for R/C Buildings: Suggested Methodology. *Earthquake Engineering & Structural Dynamics*, V. 20, No 2, 167-176

Kappos, A.J., 1999. Evaluation of behaviour factors on the basis of ductility and overstrength studies. *Engineering Structures*, V. 2, No 9, 823-835.

Kappos, A.J., Potikas, P. and Sextos, A.G., 2007. Seismic assessment of an overpass bridge accounting for non-linear material and soil response and varying boundary conditions. Proceed. of the *Conference on Computational Methods in Structural Dynamics and Earthquake Engineering* (COMPDYN), Rethymno, Greece, paper no. 1580

Moschonas J.F and A. J. Kappos, 2009. Generalized fragility curves for bridges, for arbitrary angle of incidence, *ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering(COMPDYN)*, Rhodes, Greece, CD186

Veletsos, A., Newmark, N.,1960. Effect of inelastic behaviour on the response of simple systems to earthquake motions. Proceedings, *Second World Conference on Earthquake Engineering*, 895-912

Paraskeva, T., Kappos, A., Sextos, A., 2006. Extension of modal pushover analysis to seismic assessment of bridges. *Earthquake Engineering & Structural Dynamics,* V. 35, No 10 1269-1293

Paraskeva, Th., Kappos, A.J. 2009. Further development of a multimodal pushover analysis procedure for seismic assessment of bridges. *Earthquake Engineering & Structural Dynamics* (in press); *published on-line* 4-8-09, DOI: 10.1002/eqe.947.