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ORIGINAL RESEARCH PAPER

An adaptive capacity spectrum method for assessment of bridges subjected to earthquake action

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Abstract Estimating seismic demands on structures, to predict their performance level with confidence, requires explicit consideration of the structural inelastic behaviour: to this end, the use of nonlinear static procedures is inevitably going to be favoured over complex nonlinear time-history methods.

The currently available assessment procedures have been tested predominantly against building frames. A newly derived assessment procedure is proposed within the scope of bridge applications, based on an innovative displacement-based adaptive pushover technique. The procedure, which can be incorporated into a performance-based engineering philosophy, is applicable to MDOF continuous span bridges with flexible or rigid superstructures, and for varying degrees of abutment restraint.

As a first application to determine the viability of the proposed procedure, a parametric study is conducted on a ensemble of bridges subjected to earthquake motion. It is shown that, compared to the seismic demand estimated by means of the more accurate nonlinear dynamic analysis tool, the novel static assessment method can lead to the attainment of satisfactory predictions, both in terms of displacement as well as moment demand on members.

Keywords ACSM · Adaptive · Capacity spectrum method · Displacement-based pushover · Seismic assessment · Bridges

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1 Introduction

Design/assessment for seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from 'strength' to 'performance'. One of the major developments over the past 10 years has been increased attention on limit states, i.e. Performance Based Seismic Engineering (PBSE). The definition of the philosophy of PBSE is to design a structural system able to sustain a predefined level of damage under a predefined level of earthquake intensity, or, in assessment terms, to identify the damage level of a structure under a predefined earthquake intensity level. It is generally agreed that deformations are more critical parameters for defining performance, and as a result it is argued that seismic design and assessment methods should largely be based on them.

Nonlinear static assessment approaches based on pushover analysis developed over the past two decades, such as the N2 method (Fajfar et al. 1997) or the Capacity Spectrum Method (CSM, Freeman et al. 1975) among others, constitute the expression of the well-established tendency towards PBSE. They basically consist in identifying the structural performance applying a response spectrum approach to a bilinear representation of an equivalent SDOF model derived from a pushover analysis of a MDOF model of the structure under a force vector compatible with an assumed displacement profile (it is recognised that nonlinear static assessment methods that do not make use of single-run pushover analysis have also been, however they do not constitute the scope of the current paper).

In the N2 method, an estimate of the seismic displacement demand is associated with the 'equal displacement rule', in the medium- and long-period ranges, employing inelastic spectra related to displacement-ductility demand. From the limit to displacement imposed by the calculated displacement demand, the structural performance quantities are extracted from the pushover analysis, and hence, local and global damage indices are determined.

More recently, several researchers have built upon the Capacity Spectrum Method (CSM), initially proposed by Freeman et al. (1975): by means of a graphical procedure, the structural capacity and the seismic demand on the structure are plotted on the same graph. The capacity of the structure is represented converting the force–displacement curve, obtained by pushover analysis, to the so-called 'capacity spectrum', i.e. a plot of the accelerations and displacements of an equivalent SDOF system (other researchers, such as Chopra and Goel (1999), have suggested the adoption of the term "capacity diagram", a conceptually more correct terminology given the fact that a force–displacement curve is certainly not a spectrum). The demand of the earthquake ground motion is defined by over-damped acceleration–displacement response spectra, with the appropriate level of equivalent elastic damping interpreted from the ductility demand. Iterations are required in order to match the ductilities at the performance point obtained on the demand spectrum and the capacity diagram within a given tolerance. The main differences with respect to the N2 method appear to be that damage indices are not specifically referenced in the CSM, and that seismic demand is expressed in terms of an equivalent highly damped elastic spectrum rather than by an inelastic spectrum.

Fajfar (1999), arguing that inelastic spectra are expected to be more accurate than elastic spectra with equivalent damping especially in the short-period range and in the case of high ductilities, combined the two approaches, employing inelastic spectra, related to displacement-ductility demand, rather than equivalent viscous damping.

Recently, a modal pushover analysis (MPA) procedure for building structures has been developed in order to include in the structural analysis the contributions of several modes of vibration (Chopra and Goel 2002), still retaining the conceptual simplicity of procedures with invariant force distribution. The MPA procedure is organized in two phases: first, the pushover curves obtained from forcing vectors representing the various modes of vibration (typically, two or three modes are deemed to be enough) are transformed into bilinear curves of equivalent SDOF systems, in order to calculate the target deformation and the corresponding response parameters for each mode separately. The total demand, for a predetermined earthquake level, is then determined by combining the peak modal demands using the SRSS rule. Owing to its basis on structural dynamics, MPA is deemed to provide fair estimates of the response peak values for inelastic structures.

The main drawback associated to all of the above-described methods is that these have been calibrated and/or verified for application to the seismic assessment of buildings, rather than bridges. In addition, whilst the application of pushover methods in the assessment of building frames has been extensively verified in the recent past, nonlinear static analysis of bridge structures has been the subject of only limited scrutiny. Due to the marked differences of the structural typology, observations and conclusions withdrawn from studies on buildings cannot really be extrapolated to the case of bridges, as shown in Fishinger et al. (2004), who highlighted the doubtful validity of the application of standard pushover procedures to bridge structures.

To somehow address such relative shortage of pushover studies on bridge structures, Casarotti et al. (2005) conducted an extensive parametric study, applying different adaptive and conventional pushover procedures within the scope of bridge application, demonstrating that the recently proposed adaptive pushover methods (e.g. Reinhorn 1997, Elnashai 2001) lead to the attainment of improved predictions, which match very closely results from dynamic nonlinear analysis, specially in the case of the innovative displacement-based variant of the method (Antoniou and Pinho, 2004).

The current work therefore focuses on the development and preliminary verification of an integrated seismic assessment method that makes use of such innovative and seemingly more accurate displacement-based adaptive pushover procedure (DAP), noting that the latter has been implemented in an Internet-downloadable Finite Element program, therefore readily available for general use to the practising and research communities. The procedure is applied in the assessment of continuous concrete bridges with flexible superstructure and varying degrees of abutment restraint, and is shown to lead to improved seismic response predictions.

2 Proposed adaptive capacity spectrum method

The proposed performance-based approach combines elements from the Direct Displacement-Based Design method (e.g. Priestley and Calvi 2003) and the Capacity Spectrum Method (Freeman 1998, ATC 1996), elaborated and revised within an 'adaptive' perspective, for which reason it can also be viewed as an Adaptive Capacity Spectrum Method (ACSM). The procedure is defined as a response spectrum-based approach which employs the substitute structure methodology to model an inelastic system with equivalent elastic properties. The seismic demand is defined by appropriately over-damped elastic response spectra of a given earthquake.

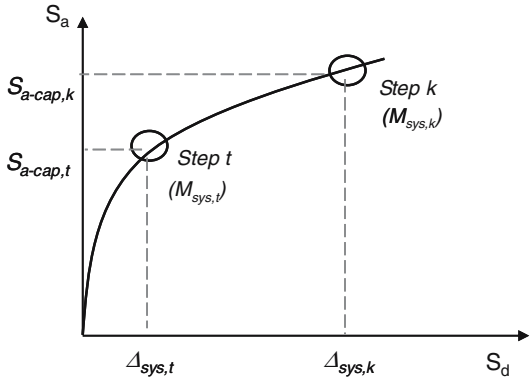

Fig. 1 Equivalent SDOF adaptive capacity curve

The proposed assessment method for the verification of MDOF bridge structures can be reduced to the following basic steps, explained in detail in what follows: (i) determination of the 'equivalent SDOF adaptive capacity curve', (ii) application of the demand spectrum to the 'equivalent SDOF adaptive capacity curve', (iii) determination of the inelastic displacement pattern and of the base shear distribution, (iv) check of acceptability criteria (pier required strength and displacement).

2.1 Description of the assessment algorithm

(i) Step 1 — Determination of the 'equivalent SDOF adaptive capacity curve'
The first step is to perform a reliable pushover analysis on a nonlinear model of the MDOF structure. The 'equivalent SDOF adaptive capacity curve' (Fig. 1) is then step-by-step derived by calculating the equivalent system displacement $\Delta_{\mathrm{sys},k}$ and acceleration $S_{\mathrm{a-cap},k}$ based on the actual deformed shape at each analysis step k, according to Eqs. 1 and 2, where $V_{b,k}$ is the total base shear of the system and $M_{\mathrm{sys},k}$ is the effective system mass, as defined in Eq. 3:

$$\Delta_{\mathrm{sys},\,k} = \frac{\sum_i m_i \Delta_{i,\,k}^2}{\sum_i m_i \Delta_{i,\,k}} \tag{1}$$

$$S_{\mathrm{a-cap},\,k} = \frac{V_{b,\,k}}{M_{sys,\,k} g} \tag{2}$$

$$M_{\mathrm{sys},\,k} = \frac{\sum_i m_i \Delta_{i,k}}{\Delta_{sys,\,k}} \tag{3}$$

It is noted that $\Delta_{\mathrm{sys},k}$ and $\mathrm{M}_{\mathrm{sys},k}$ are defined as the inverse of the modal participation factor and modal mass for a modal displacement shape, with the important difference that they are calculated step-by-step based on the current deformed pattern, rather than on invariant elastic or inelastic modal shape (thus implying that also the $\mathrm{M}_{\mathrm{sys},k}$ varies at each step, for which reason the curve is termed 'adaptive').

(ii) Step 2 — Application of the demand spectrum to the 'equivalent SDOF adaptive capacity curve'

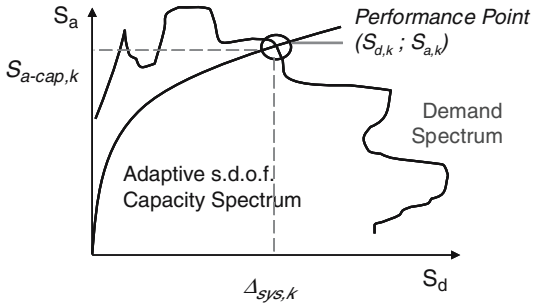

Fig. 2 Individuation of the performance point

The developed adaptive capacity curve is intersected with the demand spectrum, providing an estimate of the inelastic acceleration and displacement demand (i.e. performance point) on the structure, as shown in Fig. 2. A swift iterative procedure is required at this stage in order to use the appropriate value of equivalent viscous damping to be applied to the demand spectrum: with the performance point obtained with the 10% damped spectrum (any initial value will work, in order to have a starting point), the actual system damping is calculated with the SDOF damping model based, for instance, on the Takeda degrading–stiffness–hysteretic response (Takeda et al. 1970):

$$\zeta_{\text{sys,eff}} = 0.05 + \frac{1 - (1 - r)/\sqrt{\mu_{\text{sys}}} - r\sqrt{\mu_{\text{sys}}}}{\pi} \tag{4}$$

In Eq. 4, r and μ_{sys} are the post-yielding ratio and the ductility of the SDOF system at the performance point, as calculated by bi-linearising the capacity curve at the performance point, according to the equivalence of areas (i.e. work) between the actual and the bi-linear curve. The procedure is repeated with the spectrum damped with the updated amount of damping and iterated up to the convergence of the damping value: the procedure can be easily implemented in a simple worksheet, and usually converges within two or three iterations.

It is noted that if the demand is described by a real earthquake spectrum rather than a smoothed design spectrum, more than one intersection with the capacity curve may be found. In the present work it has been verified that often just one of those intersections provides the convergence with the damping value, and it is generally the intersection corresponding to the largest displacement value. In case more than one intersection converges with the damping, it was found again that the one corresponding to the largest displacement gives results closer to the inelastic time history analyses results. The highest deformation as performance point would be in any case a reasonably conservative choice, because generally the different intersections have comparable base shear, due to their occurrence in the post-elastic range, but different displacement demand.

(iii) Step 3 — Determination of the inelastic displacement pattern and of the base shear distribution

Once a performance point on the SDOF capacity curve is established, it suffices to go back to the corresponding level of the pushover database and pick up the actual displacement pattern, base moment and shear values, as shown in Fig. 3.

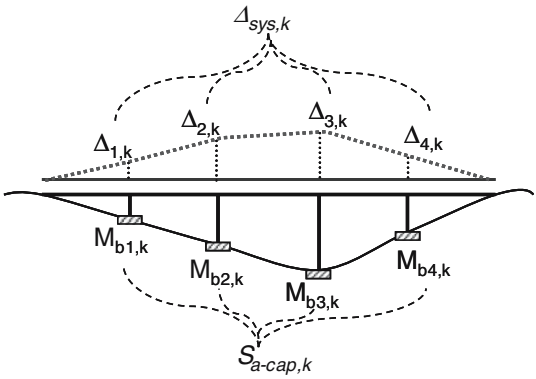

Fig. 3 Displacement and moment demand on members from pushover results

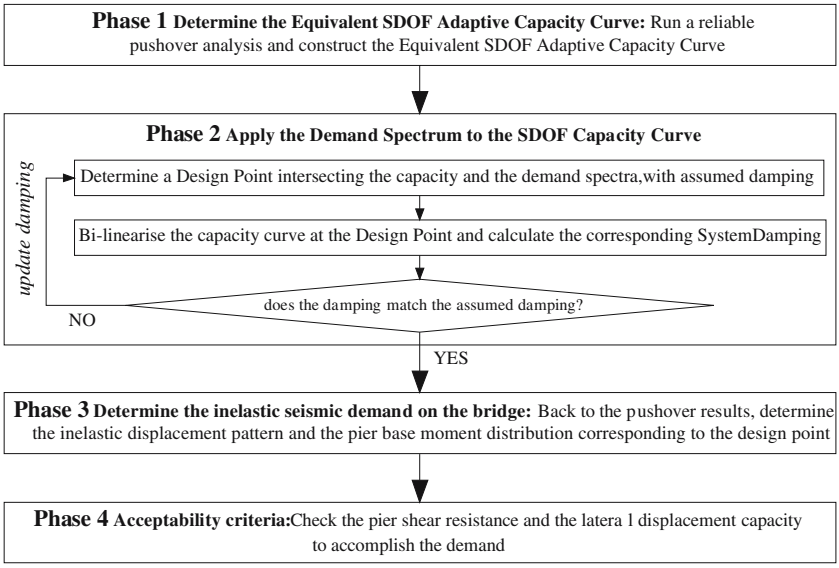

Fig. 4 ACSM flowchart

(iv) Step 4 — Check of acceptability criteria (column required strength and displacement)
Having obtained, for each element the force and displacement demand, members are checked to accomplish the shear and the displacement demand. The proposed methodology is schematically summarised in Fig. 4.

2.2 Discussion of the method

In the definition of the capacity curve, there is general agreement about the force to be plotted, i.e. the total base shear of piers and abutments, whilst the choice of the representative displacement is somehow discussed. For the longitudinal response, where all the deck locations are likely to displace of the same quantity, the displacement at

deck can be employed as reference, but for the transverse direction the selection of the characteristic displacement is not as straightforward. In the majority of current assessment methods, the displacement of a physical location (structural node) is sort of 'manipulated' to convert the curve in a SDOF equivalent by means of a modal transformation, or multiple pushover curves corresponding to multiple monitored locations are used, in order to check the state of solicitation of different members contemporarily. In viaducts where the station of maximum displacement depends on the load intensity, Isakovic and Fischinger (2006) propose to monitor the maximum displacement in the post-yield region, at the current position. In Fajfar et al. (1997) the characteristic displacement is referred to as the point of deck where the maximum displacement is expected. In this case, the resulting capacity curve may be described by a location where a local collapse mechanism has taken place, but not corresponding to a structural collapse, especially in case of irregular bridges, where the maximum deck displacement does not correspond to the critical member, likely the shortest column. As an alternative to monitor a physical location, the proposed ACSM method defines the representative displacement by an equivalent structural displacement built on the current deformed pattern, avoiding any reference to a specific structural node or a particular modal shape. In this manner, the displacements at each location contribute to the equivalent system displacement at that particular step.

As observed by Freeman (1998), another controversial issue of currently developed assessment methods based on nonlinear static analyses is the relationship between inelastic response spectra and equivalent linear elastic response spectra. In the initial version of the CSM, highly damped elastic spectra were employed for the determination of seismic demand. As an alternative to ATC-40, Freeman presented effective damping ratios based on Newmark and Hall studies (1982). Other studies (Chopra and Goel 1999, 2000) report results for SDOF systems, in which deficiencies highlighted in the ATC40 procedures are deemed to be bypassed by the use of constant-ductility demand spectra. Due to the fact that for SDOF bilinear systems relationships between the hysteretic energy dissipation of the maximum excursion and the equivalent viscous damping have been derived (e.g. Eq. 4), and because such relationships explicitly account for the ductility achieved by the system, the authors suggest the use of highly damped elastic spectra in a first application of the method because results proved to be efficient. A future study is envisaged in order to establish the possibility and the effectiveness of employing constant ductility inelastic, as opposed to highly damped elastic, spectra.

As a recognition of the similarities between the original CSM and the proposed ACSM, an explicit description of their differences is provided herein. The CSM is an iterative procedure essentially applied to buildings, which uses: (i) code-mandated elastic damped acceleration and displacement spectra S_{a-damp}/S_{d-damp} or inelastic spectra S_{a-inel}/S_{d-inel} (Freeman 1998, Fajfar 1999), (ii) force-based conventional pushover curve, either first- or multi-modal, (iii) equivalent SDOF curve explicitly related to the elastic first mode or to an assumed deformed shape, and based on a modification of the capacity curve built on the displacement of a reference node $\Delta_{\text{reference node}}$, according to the first column of Eq. 5, where Γ_1 and M_1^* are respectively the modal participation factor and the modal mass of the first mode, $V_{b-pushover}$ is the total base shear obtained by the pushover analysis, and T the structural period.

The proposed method, which has been developed with bridge application in mind (though there is no reason for it not to be applied to buildings as well), makes use of: (i) elastic over-damped spectra, either code-defined or site-specific, (ii) more reliable

displacement-based adaptive pushover curves, (iii) equivalent SDOF curve without reference either to any given elastic or inelastic mode shapes, but calculated step by step based on the actual deformed pattern, and not built on a modification of the capacity curve referred to the displacement of a specific physical location. As a consequence, all the 'equivalent SDOF quantities' (i.e. system displacement $1/\Gamma_{\text{sys}}$ and mass M^*_{sys}), even though of same 'format' of the corresponding modal quantities, are also calculated step-by-step based on the actual deformed pattern, according to the second column of Eq. 5:

$$\begin{array}{ll} \textbf{Capacity spectrum} & \textbf{Proposed procedure} \\ S_{\text{a/d-demand}} = S_{\text{a/d-inel/damp}} & S_{\text{a/d-demand}} = S_{\text{a/d-damp}} \\ S_{\text{a-capacity}} = \dfrac{V_{\text{b-pushover}}}{M^*_1 g} & S_{\text{a-capacity}} = \dfrac{V_{\text{b-pushover}}}{M^*_{\text{sys}} g} \\ S_{\text{d-capacity}} = \dfrac{\Delta_{\text{reference node}}}{\Gamma_1 \phi_{1,\text{reference node}}} & S_{\text{d-capacity}} = \dfrac{1}{\Gamma_{\text{sys}}} \end{array} \tag{5}$$

In other words, the proposed method features two types of "adaptiveness". To start with, the pushover analysis algorithm is fully adaptive, due to the impossibility of a fixed force pattern, characteristic of conventional pushover force-based methods, to accomplish the collapse mode characteristic of a bridge. Indeed, Casarotti et al. (2005) showed that the employment of the DAP algorithm leads to better estimates of the inelastic deformed pattern, as well as of the distribution of base forces at a given inelasticity level, independently of structural regularity. The second element of adaptiveness of the method resides in the way the capacity diagram is computed: as stated above, the proposed procedure can be viewed as an adaptive variant of the CSM approach, because all the 'equivalent SDOF quantities' vary at each step depending on the current deformed shape, which is not the case in traditional CSM.

3 Parametric study: case-studies, modelling assumptions, obtained results

As a preliminary verification, the proposed method has been applied to the assessment of a series of concrete bridge configurations with flexible superstructure and varying degrees of abutment restraint. In order to evaluate the reliability of the assessment procedure, it was compared to the results of dynamic time-history analyses. The parametric study has considered two bridge lengths (four and eight 50-m spans), with regular, irregular and semi-regular layout of the pier heights and with two types of abutments. The total number of bridges is therefore 12, as shown in Fig. 5, where the label numbers 1, 2, 3 characterise the pier heights of 7, 14 and 21 m, respectively.

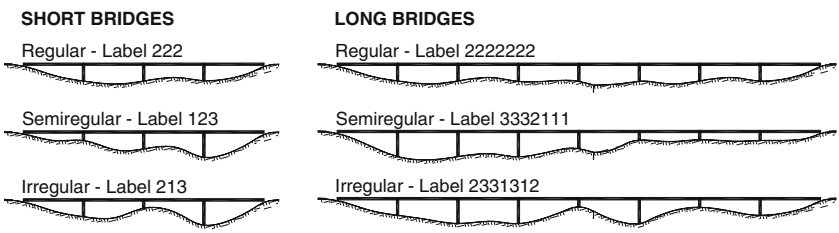

Fig. 5 Analysed bridge configurations

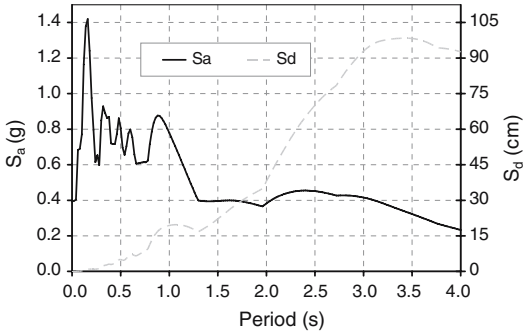

Fig. 6 Elastic 5% damped pseudo-acceleration and displacement spectra of the employed seismic excitation (Imperial Valley, 1979)

The employed seismic excitation is defined by a real earthquake of magnitude 6.5 (Imperial Valley, 1979), recorded on firm ground at a distance of 4.1 km and scaled to match the 475 years return period uniform hazard spectrum of Los Angeles (SAC Joint Venture 1997). The elastic 5% damped pseudo-acceleration and displacement response spectra of the record are presented in Fig. 6, and the characteristics of the record are the peak ground acceleration of 0.39 g, the peak response acceleration of 1.43 g and the significant duration of 8.52 s (out of 40 s total duration), defined as the interval between the build up of 5% and 95% of the total Arias intensity (Bommer and Martinez-Pereira 1999).

Finite element analyses were carried out with SeismoStruct (Seismosoft 2005), a fibre element based program for seismic analysis of framed structures, which can be freely downloaded from the Internet. The program is capable of predicting the large displacement behaviour and the collapse load of any framed structural configuration under static or dynamic loading, accounting for geometric nonlinearities and material inelasticity. Its accuracy in predicting the seismic response of bridge structures has been demonstrated through comparisons with experimental results derived from pseudo-dynamic tests carried out on large-scale models (see Casarotti et al. 2005).

The piers are modelled through a 3D inelastic beam-column element, with a rectangular hollow section of 2.0 m ×4.0 m, a wall thickness of 0.4 m, and a longitudinal steel ratio of 0.76%; the constitutive laws of the reinforcing steel and of the concrete are respectively described by the Menegotto and Pinto (1973) and Mander et al. (1988) models, with strength of 500 and 42 MPa, respectively. The deck is a 3D elastic nonlinear beam-column element, fully characterised by the sectional properties values, based on Young and shear modules of 25,000 and 10,000 MPa; a 2% Raleigh damping was assigned to the deck, proportional to the two first transversal modes of the structure. Piers and Deck cross-sections are shown in Fig. 7 further details and discussion are found in Casarotti et al. (2005).

Equivalent linear springs are used to simulate the abutment restraints, which should reflect the dynamic behaviour of the backfill, the structural component of the abutment and their interaction with the soil. The two types of abutments are modelled as (i) continuous deck-abutment connections supported on piles, with a bilinear behaviour, and (ii) deck extremities supported on linear pot bearings. Employed stiffness values for the bilinear and linear models were found respectively in Goel and Chopra (1997) and from an actual bridge with similar dimensions and loads.

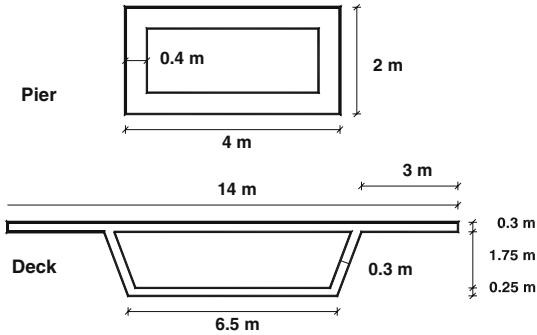

Fig. 7 Piers and deck cross sections

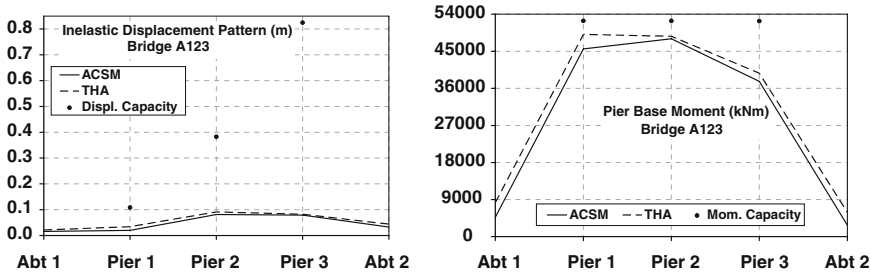

Fig. 8 Bridges A123 — Inelastic displacement pattern and pier base moments: ACSM and time-history assessment

The reliability of the proposed ACSM procedure is verified by assessing the structural response to the specific earthquake of the ensemble of the 12 bridge configurations.

The seismic demand on the bridge models is evaluated by means of (i) nonlinear time-history analyses (THA), which represent the most accurate tool to estimate the 'true' dynamic response of the structure, and (ii) the proposed ACSM procedure. Results are presented in terms of the estimated displacement pattern of the bridge deck and of the moment demand at the pier bases: the response quantities corresponding to the performance intersection are compared with the results obtained from the inelastic time-history analysis of the same bridge subjected to the employed seismic excitation.

In Figs. 8–11, the left-hand-side plots show the inelastic displacement patterns whilst their right-hand-side counterparts display the moment demand at pier bases; in both cases the dashed line represent the THA results, the solid line corresponds to the ACSM predictions at the performance point, and the dots represent the displacement/moment pier capacities, as calculated for a 'damage control' performance limit state, corresponding to repairable damage; it is assumed that this limit state can be characterized with respect to concrete compression and steel tension strains of 0.018 and 0.06, respectively (Kowalsky 2000). The drift limits of each pier are calculated based on the defined strain limits following the approach of Priestley et al. (1996). As shown in Figs. 8–11, the matching of results, between ACSM and dynamic analyses, is very good for the majority of cases considered.

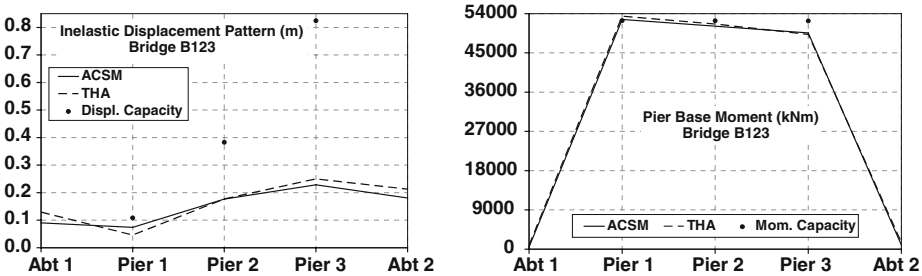

Fig. 9 Bridge B123 — Inelastic displacement pattern and pier base moments: ACSM and time-history assessment

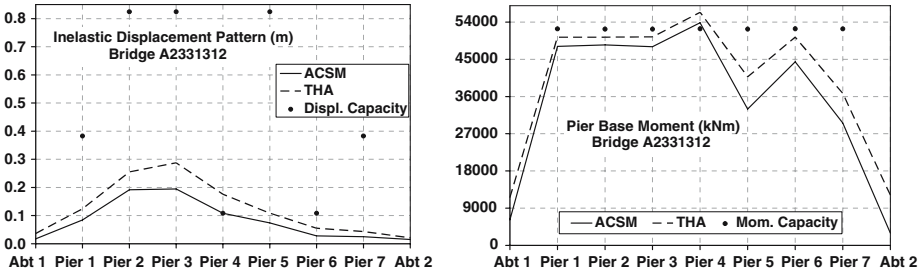

Fig. 10 Bridge A2331312 — Inelastic displacement pattern and pier base moments: ACSM and time-history assessment

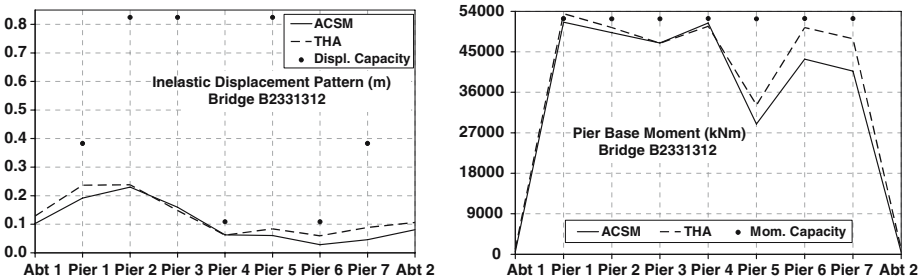

Fig. 11 Bridge B2331312 — Inelastic displacement pattern and pier base moments: ACSM and time-history assessment

Tables 1 and 2 show numerical results as ratios of the ACSM prevision to time-history outcomes, in terms of deck displacement and pier base moments for the short and long bridge configurations respectively. It is possible to see that the prediction of base moments is generally good (the same has been observed for the shear profiles, not included here for the sake of succinctness). For what concern the deformed pattern, even if the shape is well captured, as shown in figures, drift values sometimes are good and other times still underestimating, especially for long-span bridges.

Table 1 Short bridges: ratio of ACSM prevision to time-history results

	Pier 1	Pier 2	Pier 3
Deck displacement			
A123	58%	89%	96%
A213	91%	102%	86%
A232	86%	91%	86%
B123	161%	100%	91%
B213	253%	237%	133%
B232	75%	69%	75%
Pier base moment			
A123	93%	99%	95%
A213	97%	100%	91%
A232	99%	95%	99%
B123	99%	99%	101%
B213	110%	107%	107%
B232	95%	95%	95%

Table 2 Long bridges: ratio of ACSM prevision to time-history results

	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
Deck displacement							
A2222222	82%	87%	77%	69%	77%	87%	83%
A2331312	68%	75%	68%	62%	68%	50%	57%
A3332111	87%	93%	94%	90%	64%	57%	53%
B3332111	93%	96%	85%	80%	61%	54%	32%
B2331312	81%	97%	108%	101%	72%	48%	52%
B2222222	55%	59%	71%	71%	71%	59%	56%
Pier base moment							
A2222222	84%	99%	92%	97%	92%	99%	84%
A2331312	96%	96%	95%	96%	81%	88%	81%
A3332111	94%	98%	95%	98%	95%	71%	68%
B3332111	99%	97%	98%	97%	70%	62%	57%
B2331312	96%	98%	100%	101%	87%	86%	85%
B2222222	94%	96%	98%	93%	98%	96%	94%

4 Concluding remarks and future developments

To the author's knowledge, while a number of simplified nonlinear static methods has been developed, or currently in development, mainly for building structures, the feasibility of employing simplified assessment procedures for the performance-based seismic evaluation of bridges has been only recently under investigation by a number of researchers. Such deficiency of well-established knowledge consisted an important gap in the field of bridge engineering, given that current performance-based trends require, as a matter of necessity, the availability of simple, yet accurate methods for estimating seismic demand, without resorting to more complex nonlinear time-history analyses.

Therefore, and with a view to 'imitate' the developments on the buildings front, an assessment procedure (ACSM), which can be view as an adaptive re-interpretation of the well-known CSM, has been proposed within the scope of bridge applications,

whereby innovative adaptive pushover methods are employed to analyse the structural performance of bridges subjected to earthquake motion.

As a first application of the method, the study examines the seismic performances of a suite of bridge configurations predicted by using different analytical approaches: one is the time-history analysis tool and the other makes use of the newly proposed adaptive capacity spectrum method. The comparison of inelastic displacement patterns and base moment demands by the nonlinear static ACSM with those by the dynamic analysis indicates a very good agreement in the majority of cases. A similarly good agreement has been observed in the prediction of shear forces.

It is recognised by the authors that nonlinear static procedures such as the CSM method are conceptually valid because they are applied to single mode response. On the contrary, in the proposed adaptive variant of such method, an important theoretical approximation is carried out; the results of a "combined" multi-modal pushover response curve are employed to construct an equivalent single-mode capacity curve, and as such the effects of the various modes contributing to the response can no longer be de-aggregated. The authors are happy to note, however, that such apparent theoretical inconsistency did not prevent the proposed method from producing very good response predictions of the bridge case-studies considered in this work, under varying levels of ground motion intensity (inducing both elastic as well as highly inelastic response of the bridges).

In other words, the results obtained by the proposed static procedure seem to grant at least some validity in employing pushover analysis in the context of performance-based seismic assessment of bridges. Recognising the preliminary nature of the study, however, future work is clearly needed in order to compare the effectiveness of the proposed procedure to the other currently employed nonlinear static procedures for the seismic assessment of structures.

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