EVALUATION OF ITERATIVE DISPLACEMENT BASED DESIGN AND EVALUATION METHODOLOGIES FOR BRIDGES USING SIMPLIFIED MODELS

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ABSTRACT :

In this paper, the aims and limitations of current seismic evaluation and design practice and the tendencies of the displacement-based seismic evaluation/design of bridges are discussed. From it, two evaluation/design methods consistent with the performance based seismic design philosophy are proposed. The most effective widely used approaches for the Displacement Based Design of bridges are based on the use of secant stiffness and equivalent viscous damping for the piers, both evaluated at maximum pier displacement, in this paper the stiffness and energy dissipation characteristics, necessary to estimate these parameters, are obtained for reinforced concrete hollow rectangular bridge piers. This work involves the use, first of a continuous non-linear behaviour section model and then a pier model based on the plastic hinge approach and on the results obtained by the section model of the previous step. The first method proposed closely follows current linear displacement based procedures with improvements in the way the equivalent viscous damping and stiffness of the pier sections of a bridge are evaluated. This method takes into consideration the contribution of higher modes of vibration by using for the calculation of performance the complete substitute structure instead of an equivalent single degree of freedom system. The other method, evolution of a previously developed method based on the capacity curve, considers in a direct way the non-linear behaviour of the piers by calculating the nonlinear capacity curve of the structure and deriving from it, the response curve of a reference SDOF from which the overall performance of the structure is determined. It is shown that in both methods proposed the design approach follows in an inverse manner the evaluation approach. To illustrate the application of the DBE/D methods is carried out by applying the evaluation/design approach to four typical reinforced concrete multi-span bridge structures designed in accordance with the Eurocode 8. For comparison purposes, results of non-linear step by step analyses of the chosen examples are also presented.

KEYWORDS: Displacement based design equivalent stiffness, equivalent damping, substitute structure method, reference SDOF system, uniform hazard spectra

1. INTRODUCTION

The occurrence of recent destructive earthquakes all around the world, *e.g.,* Loma Prieta (1989), Northridge (1994), Kobe (1995), Turkey (1999) and Taiwan (1999), has made evident that seismic design methods proposed and used by current bridge codes do not always provide the safety levels and performances expected when bridges are subjected to design demands. Current codes base their recommendations on a design philosophy which accepts that that the seismic design of bridges may be done with design forces derived from design spectra reduced from the real elastic to consider, among other aspects, the over-strength implicit in the design equations and factors which take into account the inelastic behavior of the structure implicitly allowed to develop when different levels of damage are accepted. Unfortunately, with the designs produced with these codes it is in general not possible to guarantee that a structure has a performance that fulfils the expected design objectives. This situation makes evident the need of using alternative procedures of seismic design which

guarantee structures with performances in agreement with those expected when designed.
Within this framework, it is the purpose of this work to carry out a review of a particular class of performance based evaluation/design methods based on displacements and to propose new alternatives which correct some of the deficiencies of existent methods. All methods proposed have as theoretical foundations, concepts of structural dynamics approximated to systems with non-linear behavior, which allow, in a simple and direct way, the calculation of performances in the case of evaluation and of the correct design forces which guarantee an expected seismic performance level.

2. STATE OF THE ART

The procedure for the displacement based design of Single Degree Of Freedom, SDOF, systems or systems which may be reduced to equivalent linear SDOF systems, proposed by Kowalsky *et al*. (1995), Priestley (2000) and Kowalsky (2002), starts from a target design displacement, based on a deformation capacity guaranteed by an appropriate detailing of the structure. Assuming a reasonable value for the yielding displacement, the peak displacement becomes a displacement ductility demand, and starting with this demand and with a set of response displacement spectra, for an equivalent damping ratio which includes the inherent viscous damping characteristics of the structure and that required to consider the energy dissipated by the system through non linear hysteretic behavior, the effective period of an equivalent linear viscoelastic SDOF system corresponding to the peak displacement is determined. The final result of this process is the required yielding strength determined from the peak displacement and the secant stiffness corresponding to the effective period. Calvi and Kingsley (1995) extended this methodology to Multiple Degree Of Freedom (MDOF) structures which may be transformed to an equivalent SDOF system using an assumed deformed configuration of the structure. The final result of this alternative is the required strength that should be given to the structure to attain the objective performance.
An alternative approach to the performance based evaluation/design of structures is based on the use of non-linear static analysis procedures to include, in a simple method, the most important features which influence performance. Regarding the participation to performance of higher modes, most existent methods either neglect this participation or include it in a heuristic and somehow arbitrary way. An improvement of the single mode approximation is to include the contribution of higher modes into the forces used for pushover. Relevant formulations of the single and multi-mode approximation are described in detail in ATC, (2007). Even though the use of modal spectral analysis in the inelastic domain to define the distribution of lateral forces used to determine the capacity curve of the structure is theoretically inconsistent, the reported results from this approach show an acceptable approximation.
Based on the two simplified non-lineal systems considered, this work proposes two methods for the seismic performance evaluation/design of reinforced concrete bridges. In the first method, the original structure is substituted by a reference linear elastic structure with elements with reduced stiffness and energy dissipation characteristics consequent with the obtained/expected performance levels. This method, iterative in nature, involves the definition of a substitute structure from which performance evaluation or the design conditions for the complete structure may be found. The second method, non-linear in nature, has as basic assumption that the performance of the complete structure, generally expressed in terms of a modal index, may be approximately related to that of a reference non-linear SDOF system with a response curve directly derived from the non-linear capacity of the structure.

3. METHOD BASED ON THE SUBSTITUTE STRUCTURE

One of the most divulgated methods used for the displacement based evaluation/design of bridges is one in which the original structure is substituted by its linear viscoelastic equivalent. This substitute structure has the same configuration as the original but equivalent stiffness and damping properties for the elements where damage is assumed to occur under design conditions or where it actually occurs in design evaluation applications.

The idea of introducing viscous damping to represent the energy dissipation characteristics of a SDOF system was first presented by Jacobsen (1960). For the assessment of real structures, Gulkan and Sozen (1974) introduced the concept of substitute structure for a SDOF structure comparing analytical with corresponding experimental results. Later Shibata and Sozen (1976) extended this idea to MDOF systems proposing an approximation to the modal damping ratio of the whole structure as a weighted average of the elements' damping ratios. In this approximation, once the equivalent linear stiffness of the elements and the modal damping ratios of the structure are determined, modal spectral analysis may be used to approximately evaluate its performance.

Considering that the original Jacobsen (1960) approach is strictly applicable to harmonic excitation, several researchers have presented additional empirical equations for the equivalent damping, which reflect the type of assumed hysteretic model and the characteristics of the earthquakes defining the seismic hazard at a particular site. Recent papers, all referred in Ayala *et al.* (200), present a thorough list of different definitions of equivalent viscous damping and, where applicable, effective periods.

4. THE SUBSTITUTE STRUCTURE APPLIED TO THE DISPLACEMENT BASED EVALUATION OF BRIDGES

In this section it is assumed that for evaluation purposes the bridge structure is already designed and that its substitute structure is used to assess its seismic performance when subjected to a seismic demand given by a design spectrum. The steps involved in the evaluation of a bridge are:

Step-1: Determination of the inelastic behavior of the pier sections, as moment vs. curvature, within the potential damaged region when subjected to increasing cyclic curvature. A procedure to find this behaviour is proposed and exemplified in Paulotto *et al.* (2007).

Step-2: Determination of the load-displacement characteristics of the top of the piers. Based on the moment *vs*. curvature curves determined in step 1 and on an assumption for the plastic hinge length, these load-displacement curves for different maximum ductility levels are constructed.

Step-3: Determination of equivalent linear viscoelastic properties of the piers. Based on the non-linear force vs. displacement curves of the piers determined in step 2, the equivalent linear viscoelastic properties of the piers are calculated, *i.e.*, effective secant stiffness and equivalent viscous damping ratio at maximum displacement.

Step-4: Construction of the curves for each pier depicting the variation of the equivalent stiffness and damping ratio in terms of displacement ductility. To consider the transient nature of the earthquake action in the equivalent properties, it is necessary to use a modification factor that takes into account the fact that the maximum displacement attained during an earthquake occurs only a limited number of times, *e.g.,* for narrow band records. Equivalent properties associated to the maximum displacement multiplied by a factor equal to 0.67 have shown to be a good approximation. These curves are given in Paulotto *et al.* (2007).

Step-5: Initiation of the iterative procedure for performance determination. Since the equivalent viscoelastic properties of the piers are functions of the associated maximum displacement, it is required to initially assume a distribution of maximum displacements under design conditions. A simple way to obtain this distribution is to carry out a modal spectral analysis considering for the piers the initial stiffness and the inherent modal viscous damping for this type of structures.

Step-6: Determination of the non-linear bridge performance using an iterative procedure. Once an initial performance is assumed, the viscoelastic properties of the piers are defined using the maximum displacements and the damping ratios defined as the sum of the inherent modal damping and the weighted average of the hysteretic modal damping for all the structural elements, Shibata and Sozen (1976).

Step-7: Comparison of the updated and previous performances. When, during the iteration process, the updated and the previous performances are close enough, *i.e.,* the maximum differences between

the displacement configuration of the bridge do not exceed a given value, the process is stopped, otherwise steps 6 and 7 are repeated, updating the last performance obtained to be the initial.
The steps involved in the above described procedure are illustrated in Figure 1.

5. SUBSTITUTE STRUCTURE APPLIED TO DISPLACEMENT BASED DESIGN OF BRIDGES

A similar procedure to that for evaluation may be used for the DDBD of bridge structures. The design procedure proposed in this paper is derived following similar steps to those presented in the above section and it is different to those presented in Priestley *et al.* (2007) inasmuch as it includes information about a target damaged distribution under design conditions, includes participation of all contributing modes and uses relations between the inelastic deformation at the top of the piers vs. local curvature demands at the hinges at the base of damaged piers as basic design information.
To apply this method it is necessary to have the design curves for different pier geometries and the location of masses in the bridge model. The detailed steps involved in its application are:
Step-1: Perform a conventional force design for permanent plus vehicular plus earthquake loads, choosing an acceptable target performance index, *e.g.,* a global ductility.
Step-2: Considering the results of step 1, check if the obtained damage distribution, defined by comparing the maximum displacements at the top of the piers with their corresponding yield displacements, is acceptable then pass to step 3, otherwise modify the geometry of the piers where no damage is accepted to occur or where the ductility demand is not satisfactory and repeating step 1.
Step-3: Calculate the additional damping ratios and reduced stiffness for the damaged piers using the information presented in Paulotto *et al.* (2007). Using this information, perform the seismic analysis of the corresponding substitute structure.
Step-4: Compare the calculated maximum pier displacement with that considered as target in design. Based on this comparison and the information presented in Paulotto *et al.* (2007), modify, if required, the design of the piers and calculate the new local ductility demands for the damaged piers and go back to step 3, otherwise go to step 5.
Step-5: Perform the seismic analysis of the substitute structure defined in step 4 and compare its performance with the target. Leave the process once the overall performance of the bridge is considered acceptable, otherwise refine the design of the piers and repeat the step. It is important to mention that if the target performance is not reached, this could be due to the choice of an unfeasible damage distribution, in this case an alternative distribution should be considered.
This procedure is schematically described in Figure 2.

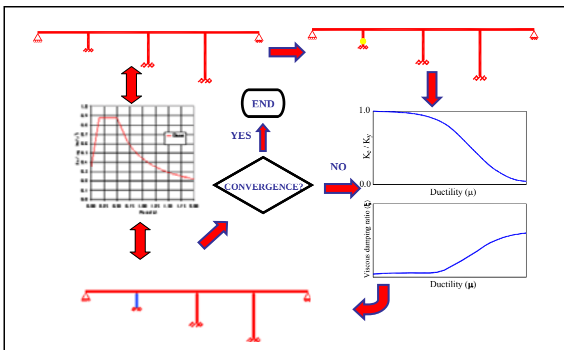


Figure 1. Evaluation procedure based on the substitute structure

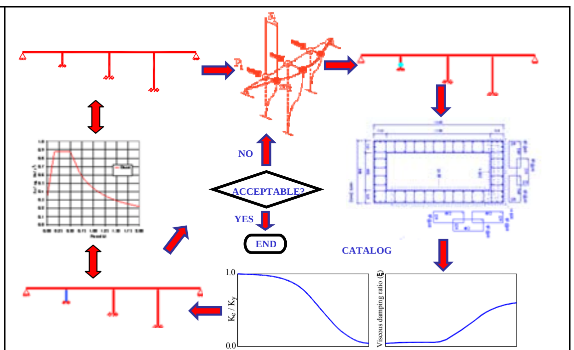


Figure 2. Design procedure based on the substitute structure

6. METHOD BASED ON THE NON-LINEAR CAPACITY OF THE STRUCTURE

From the detailed study of the existing procedures based on the non-linear capacity of structures, it has been found that, in general, all involve the following two tasks:

1. Determination of the deformation capacity of a structure and its corresponding strengths for the sequential occurrence of events associated to predefined limits states (*e.g.,* distribution of plastic hinges, maximum displacements, redistribution of the seismic forces in the structure, etc.).
2. Determination of the seismic performance using displacement/acceleration design spectra; considering SDOF systems (one or several systems, depending on the method) whose non-linear force-displacement relationships are the result of task 1. The use of smooth spectrum produces, for evaluation purposes, the maximum displacement, *i.e.,* the displacement demand for a given design, and for design purposes, the strength demand for a required displacement.

Based on the concepts which support these tasks and the considerations of the method developed by Ayala (2001), a performance evaluation/design method is proposed which explicitly considers the non-linear behavior of the structure on the derivation/postulation of the response curve of a reference SDOF system considering the participation of all modes to determine the performance of the otherwise MDOF structure under evaluation/design. The characteristics of the response curve of the reference system are obtained from the calculated/desired distributions of damage for the considered design objective. In this method, the design seismic demands associated to each of the design objectives are concurrently determined using the characteristics of the calculated/assumed response curve of the reference SDOF. The evaluation version of this method is an evolution of the proposed by Requena and Ayala (2000) where the maximum displacement of the reference system is obtained from one of the variations of the equal displacement rule, *e.g.,* Fajfar (1999) and Ruiz Garcia and Miranda (2004), and directly changed to the maximum displacement of the bridge by *ad hoc* modal spectral analyses. The details of the application of this method are presented in Ayala *et al.* (2007).

A key question in the application of displacement-based evaluation/design methods to MDOF structures is how to transform the global performance into demands of local inelastic deformation in the individual structural members. In this respect, detailed procedures intended to achieve this goal are, for example, those proposed by Ayala *et al*. (2007), however a definite solution to this problem has not been established and it is still the topic of current investigations.

7. NON-LINEAR CAPACITY CONCEPT APPLIED TO THE DISPLACEMENT BASED EVALUATION OF BRIDGES

The application of the proposed method involves the following steps:

Step 1: The seismic demand is defined by a smooth response spectrum corresponding to a chosen seismic design level.

Step 2: The response curve of the reference SDOF system is obtained through a series of Modal Spectral Analyses, MSA, considering as many damage stages as necessary, until its maximum capacity is reached. The contribution of higher modes in the response curve is taken into account using a modal combination rule (*e.g.,* SRSS or CQC).

Step 3: Once the jth MSA is performed, the corresponding scale factor, Sf(j), is calculated at the base of each damaged pier using the equations presented in Ayala *et al*. (2007). The lowest scale factor corresponds to the pier requiring the lowest seismic demand to yield.

Step 4: The scaled pseudo-acceleration, ΔSa, and the scaled spectral displacement, ΔSd, corresponding to the period of the dominant mode of the structure in the jth damage stage, are defined from the scaled spectrum, using the acceleration *vs.* displacement format, ADRS, which is the same format in which the response curve is defined.

Step 5: The capacity of the structure is reached when a local or global instability occurs, indicating that the construction of the response curve is finished and that the methodology for the evaluation of the target spectral displacement may be continued. Otherwise, a new damage stage has to be considered and a new MSA performed for the determination of the next point on the response curve.

Step 6: The inelastic displacement demand, or performance spectral displacement, Sd*, may be calculated using the equal displacement rule (Veletsos and Newmak, 1966), with proper consideration of its short period correction (Fajfar, 1999), (Ruiz-Garcia and Miranda, 2004).
For the majority of large bridges, the initial period of the most relevant mode is larger than the characteristic soil period, *i.e.,* the short period correction is not necessary. However if the fundamental period of the structure is smaller than the characteristic soil period, the spectral displacement must be corrected as specified in Annex B of the EC8 (CEN, 2003).
Step 7: When the available capacity of the structure exceeds the demand, a new scale factor needs to be calculated for the first point of the response curve where the displacement is larger than the target displacement. This is done in accordance with the equations presented in Ayala *et al.* (2007).
The seismic performance of the bridge for the selected performance parameter, in this case the maximum lateral pier displacement, is calculated as the weighted sum of the corresponding parameters for the N modal spectral analyses performed until the target displacement is reached.

8. NON-LINEAR CAPACITY CONCEPT APPLIED TO THE DISPLACEMENT BASED DESIGN OF BRIDGES

The design process for a performance level defined by a design ductility consists of these steps:
Step 1: The response curve of a reference system corresponding to the mode of the structure with the highest contribution to response is built by considering two structures with different properties: one, corresponding to a pre-designed undamaged bridge; the other, the same bridge with modified properties to incorporate a proposed damage distribution expected to occur under design demands.
Step 2: The distribution of the global lateral strength of the bridge is carried out by means of MSAs corresponding to the two performance stages considered, with a design elastic spectrum reduced by factors defined from the strengths of the elastic system.
Step 3: The seismic forces of the last design stage are obtained by combining the element forces of the MSA with the reduced elastic design spectrum. The final element forces are obtained, considering, besides the seismic, those due to gravitational and vehicular loads.

9. APPLICATION EXAMPLES

To illustrate the application and validate the accuracy and potentiality of the methods proposed, four sample reinforced concrete bridge structures are studied. The bridge termed ELSA, is an 80 m long scaled four span single supported concrete bridge tested by Pinto *et al.,* (1996) under a variety of pseudo-static and pseudo-dynamic conditions. The other three bridges designed by Isaković and Fischinger (2006) in accordance with the EC8 (CEN, 2003), are 200 m long. All bridges have the same configuration, with different dimensions and characteristics of the piers and superstructure. The general layout of the considered bridges is illustrated in Figure 3 and their geometric and structural characteristics are given in Ayala *et al.* (2007).

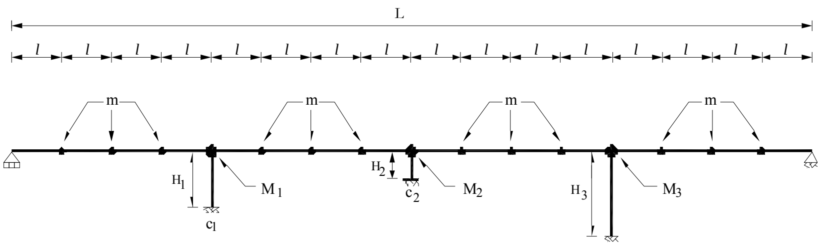
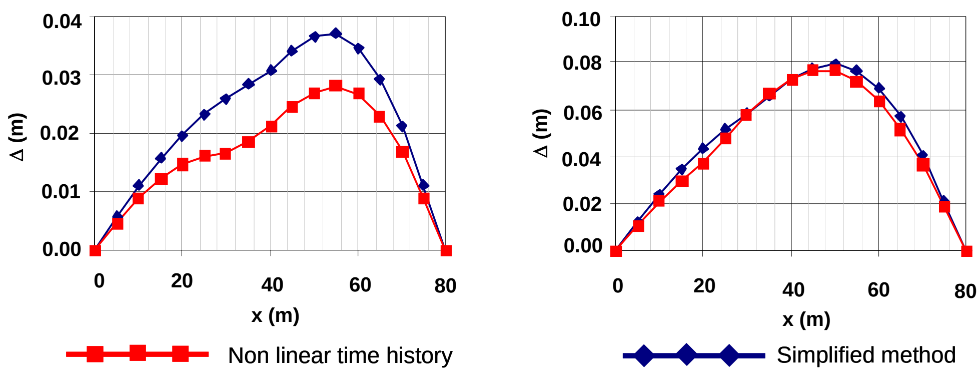


Figure 3. Geometry and location of masses of bridge examples.

For all bridges the seismic design level was defined using different intensities of the EC8 design

spectrum corresponding to soil type B, 5% damping ratio and a 1.2 soil amplification factor. The equivalent properties derived for rectangular reinforced concrete hollow piers described in Paulotto *et al.* (2007) were used to define the substitute structure of the bridges analysed. The seismic demands were artificial records with response spectra matching the EC8 spectrum, with peak accelerations of 0.35g and 0.70g for the ELSA bridge, and ranging from 0.20g to 0.70g for the other bridges.
Figures 4a and 4b show the calculated performances for the ELSA bridge under the two considered scaled earthquake intensities. In this particular example, to show the approximation of the method based on the substitute structure alone, the results presented were obtained using equivalent linear modal time history analyses instead of MSA, in order to avoid the effect of a particular modal combination rule. The results are compared with those obtained from non-linear time history analyses on the same structure subjected to the two scaled synthetic records. The maximum displacements along the bridge axis x depicted in Figure 4a, show that, for the lowest intensity, the approximation of the proposed method is not good enough, whereas the results for the highest intensity record, Figure 4b, are in better agreement with those obtained from non-linear time history analysis.



(a) Intensity 0.35 g (b) Intensity 0.70 g
Figure 4. Maximum displacement distribution for the bridge model tested at ELSA

The results of the evaluation method based on the non-linear capacity concept applied to the remaining three bridges models and all earthquake intensities considered were compared with the mean value of the results of 50 non-linear time history analyses and those obtained when the bridges were subjected to three real earthquake records scaled to match the smooth design spectrum used in the evaluation, *i.e.,* Peega (Petrovac E-W 1979), Tonga (Tolmezzo N-S 1976) and Llnga (Llolleo 1985). These results are not shown in this paper due to space limitations; however the interested reader is referred to Ayala *et al.* (2007) for full details and illustrations. For all models considered, the approximation obtained for the maximum displacement distribution was satisfactory.

10 CONCLUDING REMARKS

This paper presented two methods for the displacement based evaluation/design of bridges which improve previously developed approximations. The results obtained may be directly used to construct a substitute structure or a response curve of a reference SDOF system which lead to the sought performance or to a design for a specified target performance given by a maximum pier displacement.
The work shows that the evaluation and the design options of the proposed methods, may give acceptable results with a limited computational effort. Both methods may be considered enhanced versions of others currently in use or investigation, as they take into consideration the contribution of higher modes of vibration and the displacement reversal nature of earthquake action through evolving modal spectral analyses, rather than from evolving force or displacement based pushover analyses.
Preliminary results show that for bridges with a significant contribution of higher modes and with

large non-linearities, the methods proposed, in particularly the one based on the non-linear capacity of the structure, lead to better results than alternative simplified procedures based on a substitute structure and on an equivalent SDOF system, which do not explicitly consider the contribution of higher modes. For the design versions of the methods proposed, the deformation capacity of the structure is obtained from an assumed damage distribution, explicitly defined in the design process.

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