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Pushover analysis for the seismic response prediction of cable-stayed bridges under multi-directional excitation

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A B S T R A C T

Cable-stayed bridges represent nowadays key points in transport networks and their seismic behavior needs to be fully understood, even beyond the elastic range of materials. Both nonlinear dynamic (NL-RHA) and static (pushover) procedures are currently available to face this challenge, each with intrinsic advantages and disadvantages, and their applicability in the study of the nonlinear seismic behavior of cable-stayed bridges is discussed here. The seismic response of a large number of finite element models with different span lengths, tower shapes and class of foundation soil is obtained with different procedures and compared. Several features of the original Modal Pushover Analysis (MPA) are modified in light of cable-stayed bridge characteristics, furthermore, an extension of MPA and a new coupled pushover analysis (CNSP) are suggested to estimate the complex inelastic response of such outstanding structures subjected to multi-axial strong ground motions.

1. Introduction

Cable-stayed bridges represent key points of the transport networks and, consequently, they are conceived to remain nearly elastic under the design seismic action, typically including dampers to control the response when located in seismic-prone areas. However, several important cable-stayed bridges with dampers (e.g. Rion-Antirion in Greece or Stonecutters in China) also allow some structural damage in the towers in order to reduce response uncertainties under unexpectedly large earthquakes. On the other hand, there are many cable-stayed bridges without seismic devices which are exposed to large earthquakes and inelastic excursions. Considering these extreme events, designers need appropriate methodologies to address if the ductility demand along the towers is acceptable, and to verify the elastic response of the deck.

Non-Linear Response History Analysis (NL-RHA) is undoubtedly the most rigorous methodology to deal with inelasticity in dynamic studies, allowing also the consideration of viscous dampers. However, several uncertainties are introduced in the definition of the models and analysis, to the point that there are seismic regulations which preclude this procedure [1]. In this sense, nonlinear static pushover analysis is very appealing.

In recent years, pushover strategies have received a great deal of research, especially since seismic design guidelines [2,3] were published. Their main goal is to estimate the nonlinear seismic response by means of static calculations, pushing the structure up to certain target displacement using load patterns which try to represent the distribution of inertia forces. These methodologies are useful to uncover design weaknesses that could remain hidden in an elastic analysis and yield good estimations of the nonlinear seismic performance under certain conditions, drastically reducing the computational cost [4]. For these reasons many design guidelines recommend the use of pushover analysis to evaluate the inelastic seismic response [2,5,6], whereas the N2 pushover analysis [7] is adopted in Eurocode 8 [8]. However, the mathematical basis of the procedure is far from accurate; it is assumed that the nonlinear response of a multi degree-of-freedom structure can be related to the response of an equivalent single degree-of-freedom model (SDOF), which implies that the response is controlled by a single mode; furthermore it is assumed that this modal shape remains constant through the analysis [4]. Although these assumptions are clearly incorrect, if the structure response is dominated by the first mode of vibration the estimated results have been found to be generally accurate compared with rigorous NL-RHA [4,9,10]. Different proposals have been made to overcome the aforementioned shortcomings, briefly described in the following lines.

Chopra and Goel [11] introduced the Modal Pushover Analysis (MPA) in order to take into account the contributions of several important modes in the nonlinear dynamic response of the structure, neglecting the interaction between modes in nonlinear range and studying their response independently as it is performed in spectral analysis. This procedure, initially proposed for buildings under one-directional ground shaking and included in FEMA-440 [6], has been improved in order to include the effect of higher

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modes through spectral analysis, considering their response completely elastic [12]. The modal contribution is finally combined with standard rules like CQC (Complete Quadratic Combination) or SRSS (Square Root of the Sum of Squares), based on elastic superposition principles unable to retain the sign of the modal force distributions, which may introduce errors [13].

Several adaptive pushover methods have been developed in order to 'update' the load distribution pattern along the structure as long as yielding mechanisms are developed, they can be based on imposed load [14] or displacements patterns [15]. Although the consideration of variable modal properties normally improves the accuracy of the procedure [13,15], its difficulty is inevitably increased and it is somewhat away from the initial objective of a simplified yet accurate method. Moreover, Papanikolau et al. [16] pointed out the misleading results that adaptive pushover strategies could offer, and the numerical difficulties involved in the extraction of vibration modes if large inelastic deformations arise.

Another pitfall of pushover analysis is the difficulty in modeling three-dimensional (3D) and torsional effects, as well as considering multi-directional simultaneous seismic excitation, which in the present work are found to be important in structures with strong mode coupling like cable-stayed bridges [17]. In this direction, Lin and Tsai [18] proposed an extension of MPA, substituting the SDOF by a three degree of freedom system which takes into account the coupling between the two horizontal translations and the vertical rotation, increasing the complexity of the procedure. More practically, Huang and Gould [19] performed a simultaneous bi-directional pushover analysis considering two load patterns along both horizontal directions.

So far, most of the research is currently focused on buildings and few works address the problem of the applicability of pushover analysis to bridges [9,10,20]; the work of Paraskeva et al. [21] proposed key issues to employ MPA to bridges, providing information about the selection of the control point (among other features), and applying the procedure to a strongly curved bridge, where transverse modes present displacements also in longitudinal direction. Nonetheless, no specific studies on this topic about cable-stayed bridges have been found by the authors. On the other hand, bridges are usually more affected by higher modes and, therefore, proposing modal pushover procedures for these structures is even more of a challenge than in the case of buildings.

In this work, several considerations proposed for the applicability of MPA in triaxially excited cable-stayed bridges are first included. Subsequently, two new procedures are presented; the Extended Modal Pushover Analysis (EMPA), which considers the 3D components of the accelerograms, and the Coupled Nonlinear Static Pushover analysis (CNSP), which takes into account the nonlinear coupling between the governing modes. The validation of these pushover methods is performed by comparing their results with the extreme seismic response recorded in NL-RHA, considered as the 'exact' solution.

2. Implementation issues of MPA in cable-stayed bridges

The complex interactions among vibration modes, characteristic of cable-stayed bridges [17], force the designer to consider the full 3D model in pushover analysis. Furthermore, large differences in the stiffness of their constitutive members (towers, deck and cable-system) favor significant contributions of modes higher than the fundamental one, and typically among the first twenty modes (see Section 6.1), which clearly differentiate these structures from buildings. Several special features about the implementation of MPA in three-axially excited cable-stayed bridges have been proposed in this study and are described in the following lines.

MPA has been conceived for structures under one-directional seismic excitation, being its mathematical development included elsewhere [11]; if the bridge is three-directionally excited, in-plane pushover analyses may be conducted separately, deciding first which is the characteristic direction of the nth mode (referred as DR_n) and neglecting its contributions in the other directions.

A previous study about the contributions of each mode below a reasonable upper limit of $f_{\max} = 25$ Hz (higher modes are neglected) should be performed in order to select the governing horizontal modes in longitudinal and transverse directions, i.e. the ones with larger contributions in the corresponding response (see Section 6.1). The inelastic demand is assumed to be governed by the first vibration modes, consequently, it is proposed to include in the nonlinear static analyses all the vibration modes below the limiting frequency f_{gov}, which is established as $f_{\text{gov}} = \max(f_{nX}, f_{nY})$, where f_{nX} and f_{nY} are the frequencies associated with the longitudinal and transverse governing modes respectively (Section 6.1 is devoted to the identification of such values). The modal responses obtained through pushover analysis are combined by means of CQC rule to obtain the inelastic contribution. On the other hand, all the modes between f_{gov} and $f_{\max} = 25$ Hz are considered merely elastic and included by means of response spectrum analysis [12]. This elastic response is combined with the inelastic one obtained previously by employing the SRSS rule. Finally, frequencies above 25 Hz are directly neglected in light of the characteristic dynamic response of cable-stayed bridges. Fig. 1 aims to clarify the distinction of intervals in this proposal.

The nonlinear contribution of the first relevant modes is obtained with pushover analysis, integrating for each one the resulting SDOF differential equation in time-domain to obtain the modal displacement demand (the nonlinear spring cyclic behavior is solved with the algorithm proposed by Simo and Hughes [22]). This procedure is more rigorous than employing inelastic spectra (as it is proposed in Refs. [7,21]), since the contribution of modes in the short-period range has been observed to be relevant in the response of cable-stayed bridges (discussed in Section 6.1), being the estimates of displacement demand employing formulae based on the inelastic spectrum less accurate for these modes [7].

The selection of the roof as the control point in buildings is straightforward because it is generally the level with extreme recorded displacements. However, when dealing with three-directionally excited cable-stayed bridges, this point is not obvious. It is proposed here to establish the control point as the point with maximum modal displacement in the specific studied mode along its dominant direction (defined in Section 6.1). Therefore, optimized control points are considered by this proposal, which may be different from one vibration mode to another.

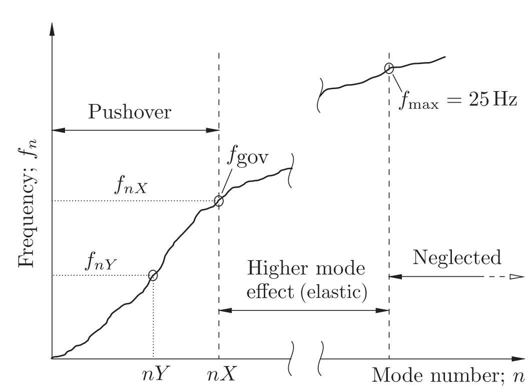

Fig. 1. Scheme of mode selection in MPA and EMPA procedures (in this case $nX > nY$ but it could be reversed).