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Seismic Assessment of Structures Using Approximate Methods Based on the Capacity Curve

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Abstract

This paper investigates the reliability of approximate seismic assessment methods by critically examining their fundamental assumptions and limitations. Particular attention is given to the influence of lateral load application techniques on the construction of the capacity curve, including force-based, displacement-based, and adaptive spectral modal procedures. Within force-based approaches, the effect of the lateral load vector shape is analyzed, since the estimation of the seismic performance point is governed by the characteristics of the capacity curve. A seismic assessment of three reinforced concrete structures is performed: two mid-rise frame buildings and one continuous viaduct-type bridge. The results obtained from (1) the original Capacity Spectrum Method, (2) a recently developed approximate procedure (2025), proposed by the authors, based on the Dynamic Capacity Curve, constructed through evolutionary spectral modal analyses in which each point represents a performance point, and (3) the benchmark procedure, Incremental Dynamic Analysis (IDA), are compared. The results show that both the lateral load application technique and the load vector shape significantly affect the estimated seismic performance. Approximate methods can provide reliable results when the equal displacement rule is valid and the Multi-Degree-of-Freedom (MDOF) system can be represented by an equivalent Single-Degree-of-Freedom (SDOF) oscillator.

Keywords: Capacity Curve; Pushover Analysis; Adaptive Pushover Analysis; Modal Spectral Analysis; Seismic Assessment; Incremental Dynamic Analysis (IDA).

1. Introduction

Over the past four decades, several intense and destructive earthquakes—such as those in Mexico (1985), Northridge earthquake (1994), Kobe earthquake (1995), Chile earthquake (2010), Tōhoku earthquake and tsunami (Japan, 2011), and Mexico City earthquake (2017), among others—have exposed persistent vulnerabilities in many existing structures, even where seismic-resistant design principles had been implemented. Consequently, numerous experimental and analytical studies have focused on the development of approximate seismic assessment and design methodologies, with particular emphasis on the principles of performance-based seismic design. The primary objective of this approach is to ensure satisfactory structural behavior under one or more seismic scenarios. In addition to meeting design objectives, most of these methodologies seek to provide procedures that are transparent and straightforward to implement, thereby facilitating their incorporation into design codes and professional structural engineering practice.

Despite the increasing acceptance of approximate methods in contemporary engineering practice, only a limited number of seismic design codes—such as Eurocode 8, part 1 [1]—have incorporated some of these procedures (e.g., the

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N2 Method proposed by Fajfar & Gašperšič [2]). The majority of current design codes continue to rely on force-based procedures which, according to numerous studies, do not always produce results consistent with design objectives or with those obtained from numerically rigorous procedures such as nonlinear time-history analysis. This discrepancy arises primarily because the assumptions underlying approximate methods are not always valid for Multi-Degree-of-Freedom (MDOF) systems [3].

Among the most widely accepted and commonly applied approximate seismic assessment procedures in engineering practice are the Capacity Spectrum Method, the Coefficient Method, and the N2 Method. The Capacity Spectrum Method, originally proposed by Freeman et al. [4], determines the target displacement through graphical procedures in which the capacity curve—typically idealized as an equivalent bilinear representation—and the seismic demand are plotted on the same graph. In this method, a response spectrum reduced by equivalent damping is employed, and the intersection between the capacity curve (assumed to represent the fundamental mode response) and the demand spectrum defines the seismic performance point (see Figure 1).

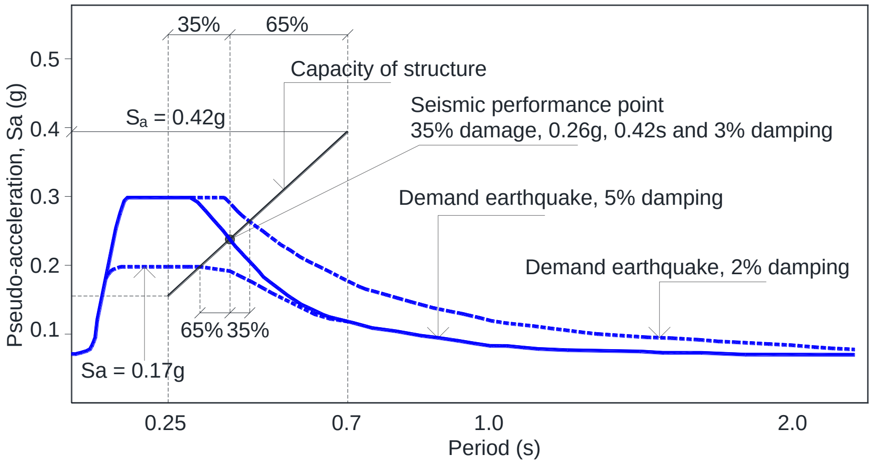


Figure 1. Estimation of the seismic performance point using simplified graphical methods (adopted from [4])

The Coefficient Method, first introduced in FEMA 273 [5], is based on the validity of the equal displacement rule, which assumes that inelastic displacement is approximately equal to elastic displacement. To determine the performance point, correction factors (C1, C2, and C3) are applied to account for parameters such as hysteretic energy dissipation and second-order effects.

The N2 Method [2] is conceptually similar to the Capacity Spectrum Method [4], differing mainly in the procedure used to reduce the demand spectrum. The N2 Method employs a ductility-based reduction factor, whereas the Capacity Spectrum Method uses spectral reduction factors directly related to the equivalent viscous damping associated with maximum displacement.

In recent years, a considerable number of studies have sought to improve the approximations provided by both the Capacity Spectrum Method and the Coefficient Method. FEMA 440 [6] and American Society of Civil Engineers ASCE 41-23 [7] present the most recent modifications to these methodologies. The revised Capacity Spectrum Method introduces the use of inelastic design spectra with constant ductility, while the Coefficient Method incorporates updated formulations for calculating coefficients C1, C2, and C3.

For more than two decades, research has focused on improving the accuracy of and overcoming the limitations associated with, the N2 Method, the Capacity Spectrum Method, and the Coefficient Method. Gencturk & Elnashai [8] proposed an extension to the Capacity Spectrum Method that incorporates nonlinear time-history analysis, thereby eliminating errors associated with equivalent linear approximations. Jing et al. [9] introduced a variant in which each point on the capacity curve is directly computed as a function of ductility, using a dynamic pseudo-acceleration factor relating period and damping for an elastic system. Kreslin & Fajfar [10] further extended the N2 Method to account for higher-mode effects in both plan and elevation, under the assumption that the structure remains elastic in higher modes. This modification substantially improved the prediction of seismic demand in the upper stories of buildings.

Ruiz-García & González [11] proposed an adaptation of the Coefficient Method for the seismic assessment of existing buildings founded on very soft soils. Their study demonstrated that seismic response is strongly influenced by the ratio between the fundamental period of the structure and the predominant period of ground motion. A displacement coefficient approach was employed to estimate maximum inelastic roof displacements. Worku & Hsiao [12] developed a simplified procedure for assessing the seismic performance of steel buildings using a single-step pushover analysis with a novel load pattern up to a target displacement. Their results demonstrated improved accuracy in estimating peak and inter-story responses compared with other existing methods.

Jalilkhani et al. [13] proposed a multi-stage modal pushover analysis in which the lateral force distribution is updated each time a new plastic hinge forms. Final structural responses are obtained by combining the seismic demands from each stage using an appropriate modal combination rule. Other researchers— Soliman et al. [14], Sharifi & Toopchi-Nezhad [15], Ferraoli & Lavino [16], Asikoglu et al. [17], Kuria & Kegyes-Brassai [18], Raheem et al. [19], Suliman & Lu [20], Fan et al. [21], Tsai & Hsu [22], and Shendkar et al. [23]—have validated the use of pushover analysis for the seismic assessment of masonry, reinforced concrete, and steel structures. Their findings suggest that approximate methods may produce satisfactory results under specific conditions, particularly where higher-mode participation is significant and structural regularity is preserved.

Despite the proliferation of approximate seismic assessment procedures in recent years, most remain founded on the same fundamental assumptions: the validity of the equal displacement rule and the representation of MDOF behavior by an equivalent Single-Degree-of-Freedom (SDOF) system. Most procedures generate a capacity curve to estimate structural response under lateral loading; however, insufficient attention has been paid to the procedure used to construct this curve, and even less to the definition of the lateral load vector used to represent seismic demand. Such oversights may result in inaccurate approximations of the capacity curve and, consequently, unreliable performance assessments.

Numerous simplified procedures have been proposed for constructing the capacity curve; however, most rely on similar assumptions and employ either force-controlled or displacement-controlled pushover techniques. This paper reviews the most representative approaches reported in the literature for constructing the capacity curve. It provides a discussion of the capacity curve, its potential as an intrinsic structural property, and its role in performance-based seismic engineering. These procedures are classified according to the evolutionary characteristics of the lateral load pattern (constant or adaptive), their influence on seismic performance, and the type of pushover analysis employed—force-based, displacement-based, or spectral modal—to characterize seismic demand of increasing intensity.

Finally, the seismic performance of two mid-rise structures and one continuous viaduct-type bridge is assessed using: (1) traditional pushover analysis; (2) a recently developed methodology, proposed by the authors, that generates the dynamic capacity curve through a sequence of evolutionary spectral modal analyses; and (3) Incremental Dynamic Analysis (IDA), as proposed by Vamvatsikos & Allin Cornell [24]. Based on these results, the advantages and limitations of both traditional procedures (e.g., the Capacity Spectrum Method) and more advanced approximate methodologies are discussed, using IDA as the benchmark reference procedure.

The paper is organized into six main sections. Following the introduction, the theoretical basis of force-based pushover analysis is presented, highlighting its fundamental assumptions and limitations in the seismic assessment of Multi-Degree-of-Freedom (MDOF) structures. The subsequent sections examine the most representative lateral load patterns and adaptive pushover procedures, with particular emphasis on force-controlled and spectral modal approaches for constructing the capacity curve. Thereafter, the principles of Incremental Dynamic Analysis (IDA) are introduced as the benchmark nonlinear dynamic procedure for validation purposes. The paper then presents the numerical application of the proposed methodology to reinforced concrete frame buildings and a continuous viaduct-type bridge, analyzing the influence of lateral load patterns and pushover techniques on structural response and seismic performance indices. Finally, the principal findings are discussed and the main conclusions are drawn, highlighting the capabilities and limitations of approximate seismic assessment procedures in performance-based earthquake engineering.

2. Force-Based Pushover Analysis

Traditional pushover analysis has gained considerable acceptance in recent decades, primarily owing to its simplicity and ease of implementation. This procedure consists of statically pushing a structure by applying a monotonically increasing lateral load vector until either a predefined target displacement is reached or a collapse mechanism develops within the structural system. At each analysis step, the capacity of the structural elements is compared with the seismic demand induced by a constant lateral load pattern representing progressively increasing seismic intensity.

The principal objective of pushover analysis is to estimate the nonlinear behavior of a structure subjected to external actions (typically seismic loading) through the construction of a curve commonly referred to as the capacity curve. This curve provides information regarding structural performance that conventional force-based procedures, as adopted by most current design codes, are unable to capture directly.

The analysis of a monotonically increasing lateral load, and particularly the information contained within the capacity curve, allows the approximate identification of cracking and yielding mechanisms as the structure progresses towards collapse under increasing seismic intensity. More importantly, it provides performance indices associated with specific seismic demand levels (see Figure 2).

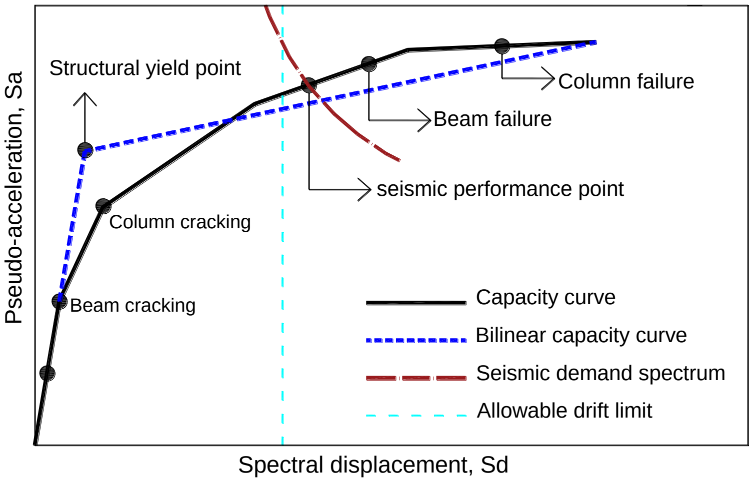


Figure 2. Anatomy of the capacity

Force-based pushover analysis lacks a fully rigorous theoretical basis, since it assumes that the response of a Multi-Degree-of-Freedom (MDOF) structure can be represented by an equivalent Single-Degree-of-Freedom (SDOF) system associated with the fundamental mode of vibration (see Figure 3). This assumption implies that the structural response is governed by the fundamental modal shape, which is assumed to remain invariant throughout the inelastic range. However, this assumption is not always valid, particularly in structures exhibiting modal irregularity, as demonstrated by Ayala & Escamilla [3]. Nevertheless, several studies have shown that, for structures whose response is predominantly governed by the first mode and which exhibit regular seismic behavior under increasing seismic demand, this procedure may provide a reasonable approximation for defining the capacity curve.

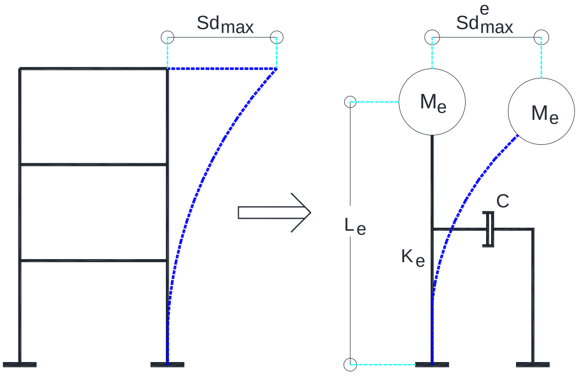


Figure 3. Equivalent single-degree-of-freedom system

To overcome the limitations of traditional pushover analysis, several methodologies have been developed to incorporate higher-mode participation into the definition of the lateral load vector used to push the structure. This is typically achieved through eigenvalue analysis combined with an appropriate modal combination rule. Paret et al. [25] and Requena & Ayala [26] proposed pushover procedures employing multiple lateral load patterns, each proportional to its corresponding modal shape.

One of the most widely accepted force-based procedures in engineering practice that accounts for higher-mode participation is Modal Pushover Analysis (MPA), proposed by Chopra & Goel [27]. This procedure estimates structural performance through a series of pushover analyses employing invariant lateral load vectors throughout the inelastic response of the structure.

The distribution of these lateral load vectors is defined by the modal shapes, participation factors, and the mass matrix, as expressed in Equation 1. In this procedure, a capacity curve is generated for each mode making a significant contribution to the structural response (see Figure 4), and a target displacement is determined for each mode. The overall seismic performance point is subsequently estimated by applying a modal combination rule to integrate the individual modal responses.

$$S^n = \Gamma^n m^n \Phi^n \tag{1}$$

where S: lateral load vectors, Γ: the participation factor, m: mass matrix, Φ: modal shapes and superscript n indicates the mode considered in the analysis.

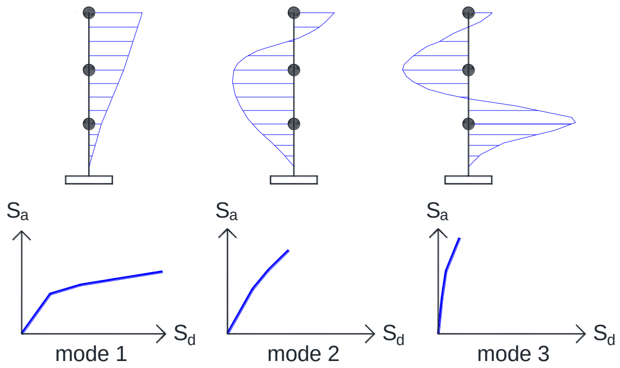


Figure 4. Modal capacity curves associated with their lateral force vector (adopted from [27])

2.1. Lateral Force Patterns Used in Pushover Analysis

Lateral load vectors are intended to represent the distribution of seismic forces induced by an earthquake. However, accurately defining this distribution is as complex as estimating the structural performance itself. At present, no seismic design code provides a universally accepted definition of the lateral force distribution required for pushover analysis. In reality, seismic force distribution varies over time and depends on both the characteristics and intensity of the earthquake. This temporal variability complicates the representation of seismic demand by means of a monotonically increasing load pattern. Nevertheless, under certain conditions, such load vectors may provide sufficiently accurate approximations of seismic demand.

The capacity curve generated through pushover analysis is highly sensitive to the lateral load pattern employed to characterize seismic demand. Consequently, the shape of the capacity curve may vary significantly depending on the selected lateral load vector. For this reason, seismic design guidelines permitting nonlinear static procedures recommend the application of multiple lateral load patterns when assessing structural response. The most widely used patterns include the Uniform Load Vector (ULF), the Equivalent Lateral Force vector (ELF), the Fundamental Mode vector (MF), and the Multimodal vector (SRSS) (see Figure 5).

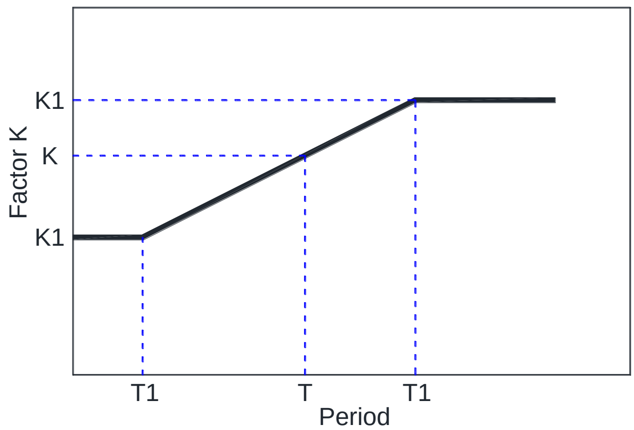


Figure 5. Factor k. FEMA-273 (adopted from [5])

Uniform Load Vector (ULF): This load vector assumes a uniform distribution of lateral forces across all stories of the structure. Although such an assumption rarely reflects the actual distribution of seismic forces during an earthquake, it highlights the reserve capacity of lower stories and emphasises the influence of shear forces in upper stories relative to overturning moments. The uniform load vector is calculated using Equation 2.

$$F_i = \frac{W}{\sum_{i=1}^{n} W_i} \tag{2}$$

where F: lateral load pattern, W: weight of story and subscript i indicates the story

Equivalent Lateral Force (ELF): This vector estimates the distribution of lateral forces as a function of structural flexibility. Its definition requires a coefficient k, which remains constant for extreme values of the fundamental period and varies for intermediate periods. For short-period structures (T_1 < 0.5 s), the distribution adopts a parabolic shape. For long-period structures (T_1 > 2.5 s), the distribution approaches a parabolic tympanum shape. For intermediate periods, the load pattern resembles an inverted triangular distribution.

$$F_i = \frac{Wh^k}{\sum_{i=1}^{n} W_i h_i^k} \tag{3}$$

where h_i: story height, subscript i indicates the story

Fundamental Mode Vector (MF): This lateral load distribution is similar to that employed by several seismic design codes for calculating equivalent seismic forces. The distribution shape is defined by the fundamental mode shape of the structure. The MF vector is calculated using Equation 4.

$$F_i = \frac{W_i \Phi_i^1}{\sum_{i=1}^{n} W_i \Phi_i^1} \tag{4}$$

where superscript 1 indicates fundamental mode, subscript i indicates the story and Φ modal shape.

Multimodal Vector (SRSS): This load pattern incorporates higher-mode participation and defines the force distribution by means of eigenvalue analysis combined with a modal combination rule, specifically the Square Root of the Sum of the Squares (SRSS) method. The magnitude of the load pattern is determined using spectral acceleration values obtained from the elastic response spectrum. The SRSS vector is calculated using Equation 5.

$$F_i = \Gamma_i m \Phi_i s_{ai} \tag{5}$$

where: s_a: acceleration and subscript i indicates the mode.

Figure 6 shows different forms of loading patterns used in pushover analysis proposed in some technical documents such as FEMA 273 [5], ATC-40 [28], ASCE 41-23 [7], etc.

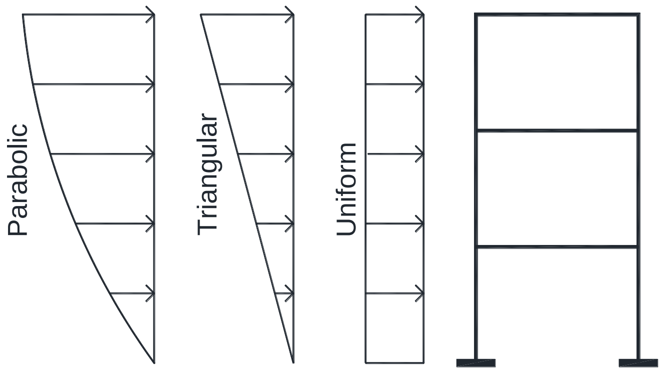


Figure 6. Different forms of loading patterns used in pushover analysis

2.2. Adaptive Force-Based Procedures

In general, adaptive pushover analysis procedures are founded on the same fundamental assumptions as traditional pushover analysis; consequently, the differences between them are relatively minor. Nevertheless, adaptive procedures have been developed as an alternative means of incorporating higher-mode effects into the seismic assessment process. The first adaptive force-based pushover analysis procedure was proposed by Bracci et al. [29], in which the structure is pushed using an evolving lateral load pattern. In this approach, the distribution of lateral forces is updated at each analysis step based on the inter-story shear forces obtained from the previous step.

Requena & Ayala [26] proposed a simplified seismic assessment procedure for reinforced concrete buildings based on the assumption that the structural response remains governed by the fundamental mode throughout the inelastic range. Consequently, the performance point may be estimated using an equivalent SDOF reference system. This procedure generates a capacity curve by applying an adaptive monotonically increasing lateral load pattern until the target displacement is reached.

In this procedure, both the lateral load vector and the dynamic properties of the structure are updated at each analysis step. Two alternative lateral load vectors are proposed. Equation 6 considers only the contribution of the fundamental mode, whereas Equation 7 incorporates higher-mode effects through eigenvalue analysis and modal superposition.

$$F^i = \frac{m\Phi_1^i}{\sum_i m\Phi^i} \tag{6}$$

$$F^i = \sum_{j=1}^{n} \frac{\sum_{j=1}^{n} m\Phi_j^k}{m\Phi_1^i} \tag{7}$$

where: subscript 1 indicates fundamental mode, superscript i indicates the story, subscript j indicates the mode.

Antoniou & Pinho [30, 31] proposed a methodology for constructing the capacity curve by applying a sequence of adaptive lateral load vectors until the structure reaches either a predefined roof displacement or develops a collapse mechanism. The lateral load vector is calculated at the beginning of each analysis step and defines the load increment shape through eigenvalue analysis combined with modal superposition. At each step, the load vector is updated by adding the increment required to drive a critical point of the structure to its maximum resistance.

$$F_i = F_{t-1} + \Delta\lambda_t F_t P_O \tag{8}$$

where F_{t-1}: load vector used in a previous step, $\Delta\lambda_t$: load increase factor, P_O: nominal load vector, F_i: normalized updated load vector

Adaptive force-based pushover analysis procedures have addressed some of the limitations inherent in traditional methods. Nevertheless, the nonlinear response of a structure subjected to seismic excitation can be captured more accurately using displacement-based procedures. Accordingly, more recent adaptive pushover procedures have evolved within this framework. Figure 7 schematically illustrates the updating process of the lateral load vector throughout the analysis.

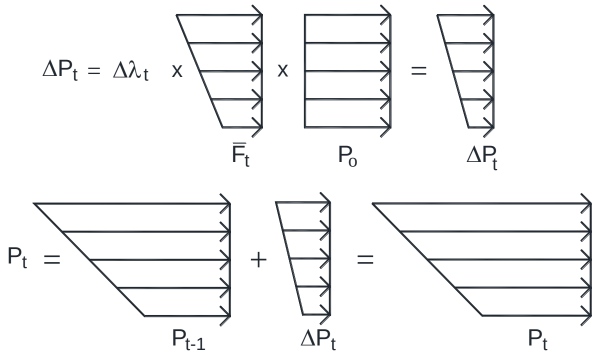


Figure 7. Updating the lateral force vector at each analysis step (adopted from [30])

3. Adaptive Spectral Modal Analysis

Although the application of spectral modal analysis to structures that have entered the nonlinear range is theoretically inconsistent, several studies have demonstrated that, for structures exhibiting regular seismic behavior governed predominantly by the fundamental mode, such analyses may provide sufficiently accurate results.

Adaptive spectral modal procedures were developed to overcome the limitations of force-based methodologies, particularly for structures undergoing deformations beyond their maximum resistance point, a condition that conventional force-based procedures are unable to represent adequately. These procedures not only characterize structural behavior through a capacity curve relating base shear to displacement, but also provide the evolution of inter-story drifts, which constitutes a more informative performance index for structural designers. This is because maximum seismic demand does not necessarily correspond to maximum structural damage, as demonstrated by the application of adaptive spectral modal procedures (see Figure 8).

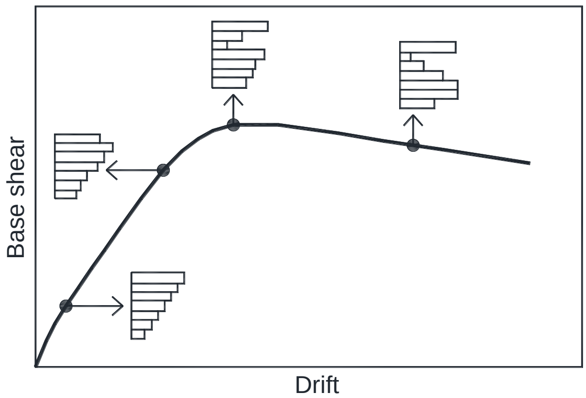


Figure 8. Story drift history during an adaptive spectral modal analysis

Aydınoğlu [32] proposed a simplified multimodal seismic assessment procedure known as Incremental Response Spectrum Analysis (IRSA). This methodology estimates the nonlinear response of a structure through a sequence of linear spectral modal analyses, each corresponding to a segment of the structural behavior curve associated with a specific level of seismic demand. The procedure generates independent modal behavior curves using an adaptive spectral modal approach (see Figure 9). To estimate the response of each mode, the equal displacement rule originally proposed by Veletsos & Newmark [33] is employed.

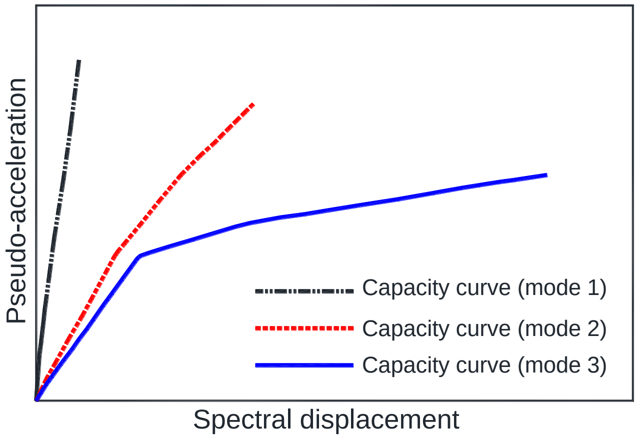


Figure 9. Modal capacity curves (adopted from [32])

The IRSA procedure idealizes structural damage through the formation of plastic hinges, and at each analysis step the stiffness matrix is updated to account for both accumulated damage and stiffness reductions arising from second-order (P–Δ) effects in previous steps. However, this procedure does not incorporate the short-period correction to the equal displacement rule, thereby limiting its applicability to structures whose fundamental period is equal to or greater than the predominant period of the supporting soil.

Alba et al. [34] proposed a simplified seismic assessment procedure derived from the force-based pushover methodology developed by Requena & Ayala [26]. This procedure employs adaptive modal analysis to characterize the nonlinear behavior of a structure subjected to a defined seismic demand, generating a capacity curve in spectral coordinates (behavior curve) associated with the fundamental mode of vibration. This is achieved through a sequence of linear spectral modal analyses, each corresponding to a seismic demand increment and a specific level of accumulated structural damage.

The behavior curve is constructed incrementally, such that a complete capacity curve requires as many spectral modal analyses as there are damage events in the structure. For practical purposes, however, the methodology allows multiple damage states (plastic hinges) to be considered within a single analysis step. The procedure employs a scale factor (Sf) to define the seismic demand increments required to induce each damage state and, consequently, to estimate each point of the capacity curve. The scale factor represents the ratio between the resistant moments of the structural elements, considering accumulated damage, and the acting moments associated with 100% of the seismic demand. Equation (9) is used to calculate the scale factor for the first damage event, while Equation (10) is used for subsequent damage states. This procedure is applicable to structures with short fundamental periods and estimates the performance point using the equal displacement rule together with a short-period correction.

Bañuelos et al. [35] proposed an extension of the methodologies developed by Requena & Ayala [26], Alba et al. [34], and Mendoza Pérez & Ayala Milián [36]. In this updated procedure, stiffness degradation and hysteretic energy dissipation associated with increasing seismic demand are incorporated into the evolutionary spectral modal analyses employed for the approximate construction of the Dynamic Capacity Curve (DCC). As in previous methodologies, this procedure assumes that the seismic performance of an MDOF structure may be approximated by the response of an equivalent bilinear SDOF oscillator used as a reference system. It further assumes that the capacity curve constitutes an intrinsic structural property for a given type of seismic demand.

The approximate DCC is constructed through a sequence of evolutionary spectral modal analyses, each associated with an increasing level of seismic demand and a corresponding damage state. For each branch of the curve, a correction is applied to account for stiffness degradation and hysteretic energy dissipation by solving a bilinear oscillator whose properties are defined by a reference system associated with the fundamental mode of vibration (see Figure 2). In principle, one spectral modal analysis is required for each damage state until the target displacement is reached. For practical implementation, however, several damage states may be grouped within a single seismic intensity increment.

The calculation of the approximate capacity curve requires obtaining the response of an SDOF oscillator whose properties are associated with a reference modal system, including modal mass, fundamental period, and both elastic and post-yield stiffness.

The proposed procedure and its application steps are described as follows:

- ***Define a seismic demand*:** Seismic action may be characterized by a design spectrum, a response spectrum, or a set of ground-motion records.
- ***Calculate the elastic branch of the capacity curve:*** To estimate this stage of the curve, a load increment (S_f) is applied, representing the increment required to generate the first damage in the structure. This calculation is performed for each end of the structural elements, considering the smallest value obtained in each event (Equation 9). The capacity of each element is associated with the yield moments (M_y), determined by constructing moment–curvature diagrams or interaction diagrams. The load increment is calculated using Equation 9.

$$S_{f_{i=1}} = \frac{M_{y_k} - M_{G_k}}{M_{us_k}} \tag{9}$$

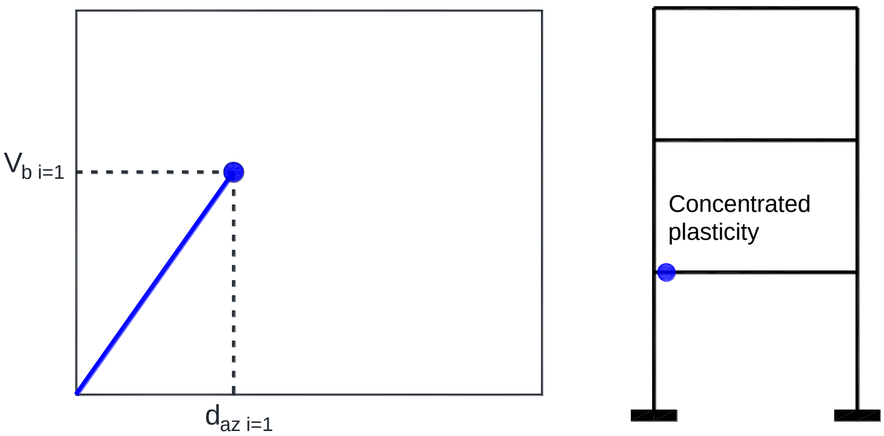
where M_y: yield bending moment, M_G: bending moment associated with the gravity load, M_{us}: bending moment associated with the application of 100% of the seismic demand. The subscripts i and k indicate the intensity increment and the structural element, respectively.

The base shear (V_{bi}) and the roof displacement (d_{azi}) can be obtained directly from structural analysis or by multiplying the scale factor by the total base shear and total roof displacement, respectively (Equations 10 and 11).

$$V_{b_{i=1}} = S_{f_{i=1}} V_b \tag{10}$$

$$d_{az_{i=1}} = S_{f_{i=1}} d_{az} \tag{11}$$

Structural damage was characterized using a concentrated plasticity model, in which inelastic deformations are localized at the ends of structural elements and represented by zero-length plastic hinges (see Figure 10). Residual stiffness was modelled using inelastic springs, assuming 15% of the original bending stiffness.



(a) (b)

Figure 10. Elastic branch of capacity curve, a) first point of dynamic capacity curve, b) first damage of structure

- ***Calculation of the second branch of the capacity curve (first inelastic branch)*:** To define this branch, a new load increment is applied, determined by the scale factor required to generate a second group of damage events (Equation 12). In this stage, the structural model incorporates the damage accumulated from the first load increment (see Figure 11). The corresponding roof displacement and base shear are then calculated using Equations 13 and 14.

$$S_{f_{i=2}} = \frac{M_{y_k} - M_{ac_k}}{M_{us_k}} \tag{12}$$

$$V_{b_{i=2}} = V_{b_{i=1}} + S_{f_{i=2}} V_{b_t} \tag{13}$$

$$d_{az_{i=2}} = d_{az_{i=1}} + S_{f_{i=1}} d_{az_t} \tag{14}$$

where $S_{f_{i=2}}$: scale factor for construction of the second branch of the capacity curve, M_{ac}: accumulated bending moment of the structural elements.

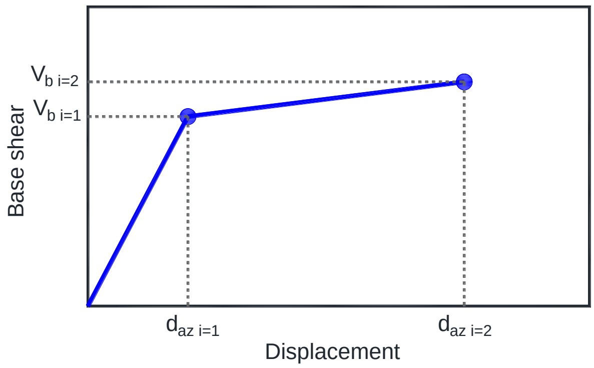


Figure 11. Capacity curve associated with a second intensity increment (adopted from [35])

- ***Calculation of the performance curve*** $(S_a - S_d)$***:*** This curve is derived using a reference system, generally associated with the fundamental mode of the structure. The basic structural dynamics equations from ATC 40 [28] are applied. The performance point of the capacity curve, defined by the first two load increments, is corrected by calculating the maximum response of a bilinear single-degree-of-freedom (SDOF) oscillator. This calculation accounts for the characteristics of the performance curve obtained in the previous step and the seismic demand intensity, defined as the sum of the scale factors. Figure 12 schematically illustrates a performance point of the capacity curve corrected for two load increments.

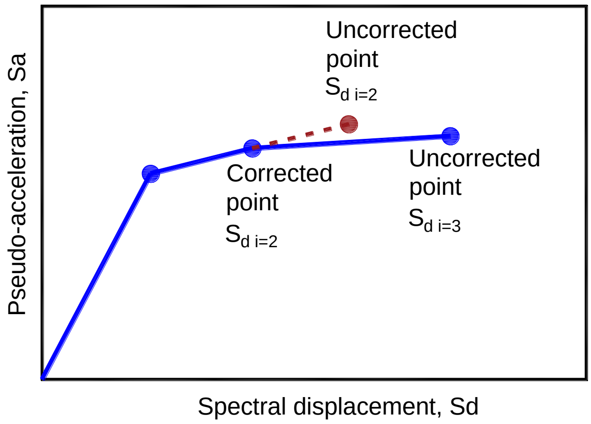


Figure 12. Behavior curve, associated with a second intensity increment, considering the energy correction by hysteresis (adopted from [35])

- ***Calculation of the subsequent branches of the capacity curve*****:** The procedure is repeated from Step 4. For each increment, a hysteresis correction is applied using a bilinear oscillator associated with a reference system that reflects the characteristics of the first vibration mode (see Figure 13).

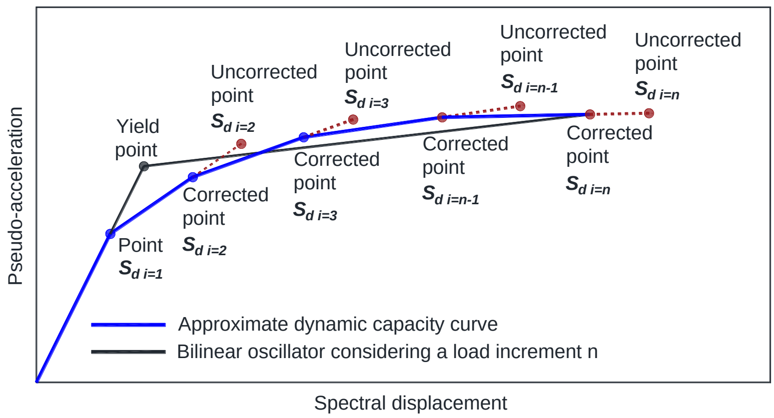


Figure 13. Bilinear approximations of a capacity curve for different damage states (adopted from [35])

Figure 14 schematically illustrates the approximate seismic assessment procedure proposed by the authors of this study.

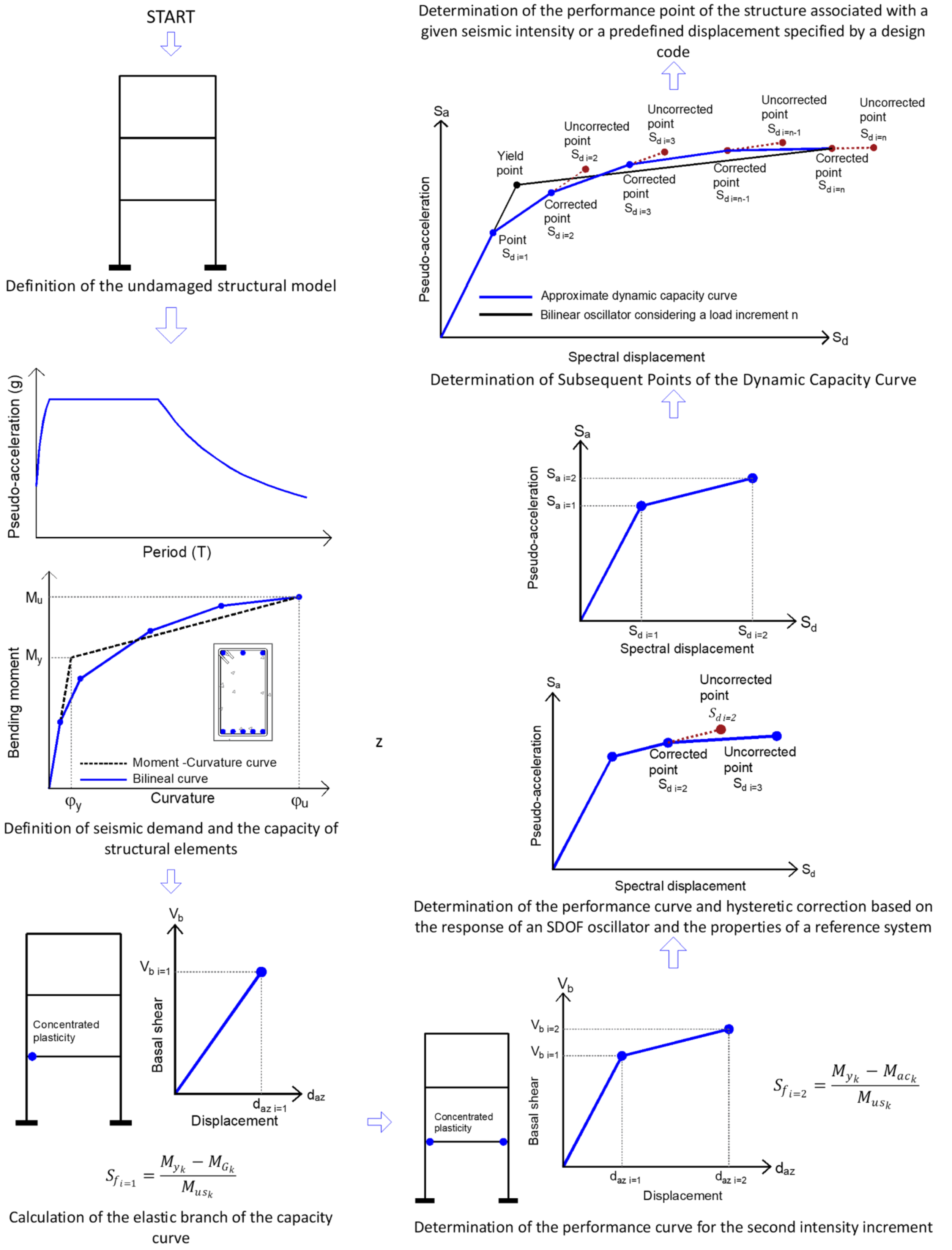


Figure 14. Proposed Method (adopted from [35])

4. Dynamic Incremental Analysis

Vamvatsikos and Cornell [24] proposed a parametric methodology for estimating the structural response under a suite of ground-motion records or a specific seismic event, using a numerically rigorous framework known as Incremental Dynamic Analysis (IDA). This procedure consists of performing a sequence of nonlinear time-history analyses using one or more ground-motion records, each scaled to different intensity levels in order to represent increasing seismic demand. The resulting analyses provide structural performance indices that may be regarded as benchmark solutions, which are represented through Incremental Dynamic Analysis (IDA) curves (see Figure 15). These curves enable the visualization of various performance parameters as functions of seismic intensity (e.g., inter-story drift versus spectral acceleration), as well as dynamic capacity curves that characterize structural response in terms of base shear versus inter-story drift or roof displacement [37].

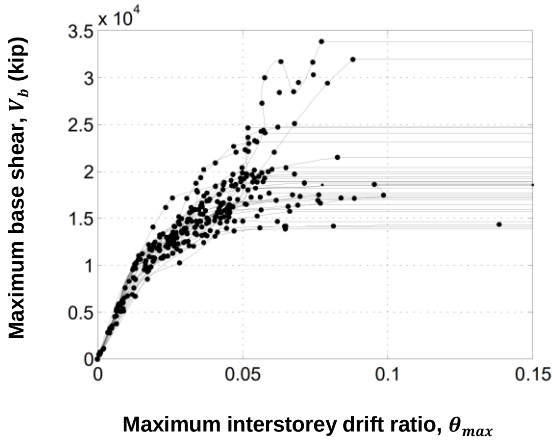


Figure 15. Dynamic capacity curve [37]

5. Results and Discussion

To validate the proposed procedure and illustrate its practical application, three reinforced concrete structures were analyzed: a continuous viaduct-type bridge (V123P) and two frame structures comprising 8 and 17 stories (Models M1 and M2, respectively). The analyses were carried out using different lateral load patterns and damage models in order to evaluate the influence of the modeling assumptions on the estimated seismic response. Based on the obtained results, recommendations are provided regarding the appropriate application of pushover analysis procedures.

The results obtained using the methodology proposed by the authors are compared with those derived from the Capacity Spectrum Method, a procedure widely accepted within the earthquake engineering community. In addition, the results are compared with those obtained from a numerically rigorous benchmark procedure, namely Incremental Dynamic Analysis (IDA).

5.1. Description of the Structures Studied

For the structural design of both building models, Class I concrete was used, with a specified compressive strength (f'c) of 25 MPa, an elastic modulus (Ec) of 21,700 MPa, and a unit weight (γc) of 23.5 kN/m³. Deformed reinforcing steel with a yield strength (fy) of 412 MPa and an elastic modulus (Es) of 200,000 MPa was considered. Tables 1 and 2 present the dimensions and reinforcement details of the structural elements comprising Models M1 and M2.

Table 1. Dimensions and reinforced steel of the structural elements for the 17-story structure, M1

Story	Element	A_s cm^2	A'_s cm^2	width (b) m	Depth (h) m
1	Beam	25.35	40.56	0.35	0.9
2	Beam	25.35	40.56	0.35	0.9
3-8	Beam	35.49	50.70	0.35	0.9
9-10	Beam	25.35	40.56	0.35	0.9
11-12	Beam	20.28	35.49	0.35	0.9
13-14	Beam	15.21	15.21	0.35	0.9
15-16	Beam	15.21	15.21	0.35	0.9

Story	Element	$A_s$cm^2	width (b) m	Depth (h) m
1-4	Column	253.35	1.10	1.10
5-7	Column	126.68	1.10	1.10
8-11	Column	126.68	0.9	0.9
12-14	Column	63.33	0.75	0.75
15-17	Column	63.33	0.60	0.60

Table 2. Dimensions and reinforced steel of the structural elements for the 8-story structure, M2

Story	Element	A_s cm^2	A'_s cm^2	width (b) m	Depth (h) m
1-4	Beam	15.84	23.13	0.30	0.70
5-6	Beam	10.14	15.21	0.30	0.70
7	Beam	10.14	15.21	0.30	0.70
8	Beam	10.14	10.14	0.30	0.70

Element	width (b) m	Depth (h) m	$A_s$cm^2
Column	0.7	0.7	103.84
Column	0.7	0.7	72.2
Column	0.7	0.7	72.2
Column	0.7	0.7	72.2

Structures M1 and M2 (Figure 16) form part of a reinforced concrete building system comprising moment-resisting frames in both orthogonal directions and a 150 mm thick solid concrete slab floor system. The structural system was designed in accordance with the Complementary Technical Standards for Seismic Design, NTC-DS, GCDMX (2023). These models belong to a broader set of reinforced concrete buildings developed by the authors for the purpose of investigating the seismic performance of existing structures in the Valley of Mexico under both current and previous seismic design regulations. In the present study, only the planar frame response was considered, since the primary objective is to validate the proposed methodology under controlled structural conditions. This simplification avoids the influence of bidirectional response and torsional effects, which may introduce additional uncertainties not directly related to the formulation of the proposed procedure. For the same reason, a single seismic record or response spectrum was adopted as the seismic demand, in order to isolate and evaluate the accuracy of the proposed methodology without introducing record-to-record variability.

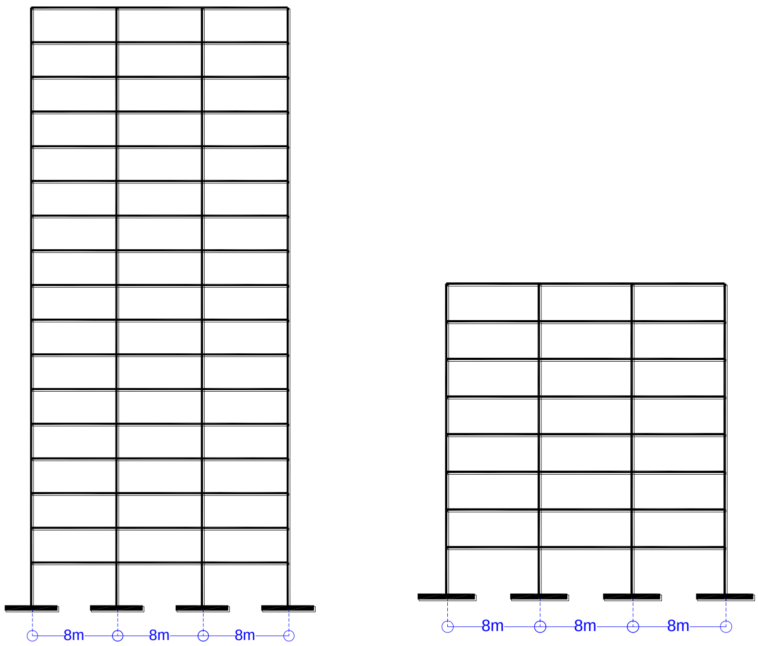


Figure 16. Geometry of the frame structures, a) model M1 and b) model M2

Model M1 is a symmetrical reinforced concrete frame comprising 17 stories, with three bays of 8 m each and a typical story height of 3.20 m, except for the ground floor, which has a height of 4.00 m (see Figure 16-a). Model M2 consists of an 8-story reinforced concrete frame with three bays and a uniform story height of 3.30 m (see Figure 16-b). Both structures incorporate a 150-mm-thick solid slab and were designed in accordance with the Complementary Technical Standards for Seismic Design (NTC-2023) [38], corresponding to an office building complex located in Mexico City.

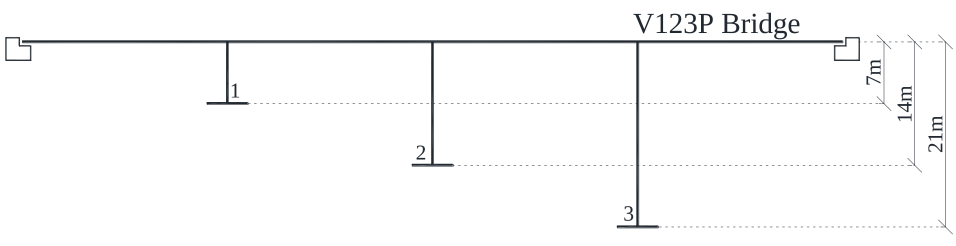


Figure 17. Geometry of the long-span continuous bridge studied

To assess the accuracy of the most representative pushover analysis procedures, as well as the influence of lateral load patterns on the shape of the capacity curve, lateral displacements, and inter-story drifts, two reinforced concrete frame structures were analyzed. According to the classification established by the Complementary Technical Standards for Seismic Design (NTC-2023) [38], Model M1 is classified as a tall building, whereas Model M2 corresponds to a medium-rise structure.

The bridge evaluated in this study is a long-span continuous viaduct-type bridge with a total length of 200 m, divided into four equal spans. According to the nomenclature adopted in the literature [39, 40], the letter **"V"** denotes the bridge typology (viaduct), while the letter **"P"** indicates the support condition at the abutments (pinned supports). The intermediate numerical sequence in the designation V123P represents the ratio of the pier heights with respect to a reference height of 7 m (see Figure 17).

For the modal analysis of the bridge, the mass of the entire structural system was assumed to be uniformly distributed along the deck, with a distributed weight of 196 kN/m. Tables 3 and 4 summarize the mechanical properties and yield moments of the analyzed bridge structure.

Table 3. Mechanical characteristics of the studied bridges

Element	F_c MPa	E_c (MPa)	Γc (kN/m^3)	Moment of inertia (m^4)
Deck	35	26716	24	50
Piers	35	26716	24	1.92

Table 4. Geometrical characteristics and yield moments of the evaluated bridges

Bridge	Diameter piers (m)	Yield moment (kN-m)		
		Pier 1	Pier 2	Pier 3
V123P	2.5	47240	43102	66236

5.2. Seismic Demand Definition

To characterize the seismic response of Models M1 and M2, a seismic demand spectrum and record was adopted (see Figure 18), derived from a ground-motion record obtained at the Secretariat of Communications and Transportation station during the 1985 Mexico City earthquake, 19 September 1985.

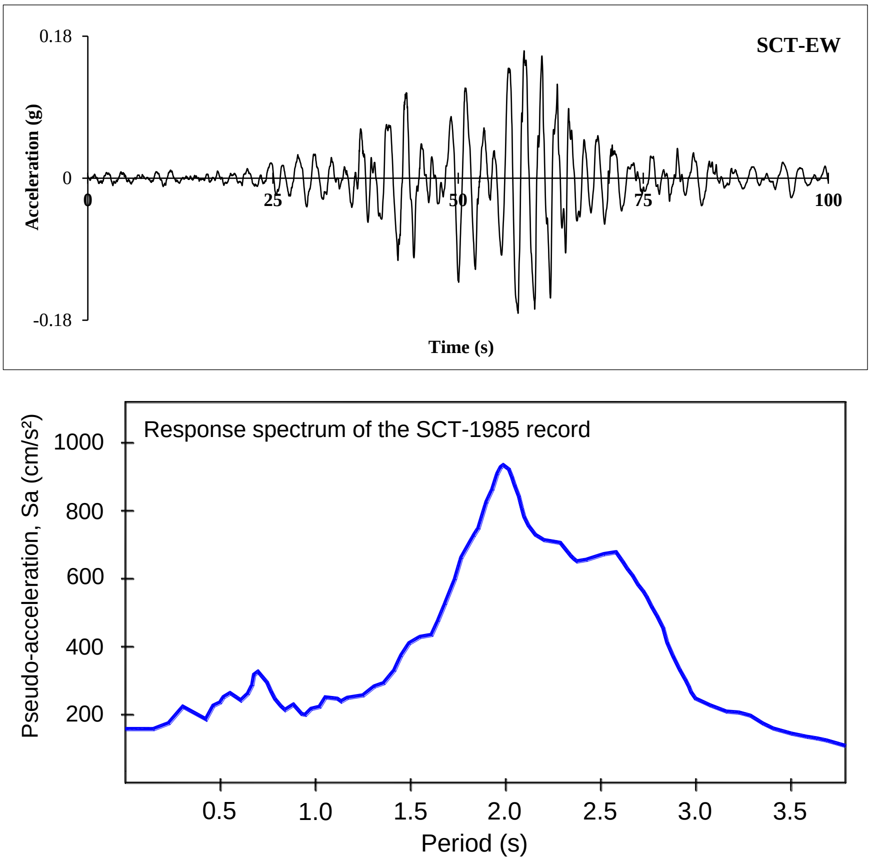


Figure 18. Seismic demand used in numerical examples, M1 and M2

For the adaptive spectral modal analyses, the commercial structural analysis software SAP2000 [41] was employed, whereas the nonlinear dynamic analyses were performed using SeismoStruct (SeismoSoft [42]).

The viaduct-type bridge examined in the present study was designed in accordance with the provisions of Eurocode 8, part 2 [43]. According to the requirements established in current bridge design standards, including the American

Association of State Highway and Transportation Officials (AASHTO) Design Specifications [44] and Eurocode 8, part 2 [43], the V123P bridge is classified as structurally irregular due to its stiffness distribution.

Nevertheless, several studies by Ayala & Escamilla [3], Isakovic & Fischinger [39], and Maalek et al. [45] have proposed a more robust framework for assessing the degree of bridge regularity by considering the structural response within the nonlinear range. These studies introduce a regularity index that enables engineers to make informed decisions regarding the most appropriate analysis procedure for seismic assessment. When a structure is identified as irregular according to this regularity index, its seismic assessment should be carried out using nonlinear dynamic analyses, since approximate procedures may not provide results consistent with its actual structural response.

To validate the seismic performance estimated by the proposed procedure (SNAP) through Nonlinear Time-History Analysis (NLTHA), two ground-motion records were adopted as seismic demand for the bridge assessment: the east–west (EW) acceleration record obtained at the Takatori Station during the 17 January 1995 Kobe earthquake (see Figure 19-a), and the east–west acceleration record obtained at the OTE Building Station during the 13 September 1986 Kalamata earthquake in Greece (see Figure 19-b).

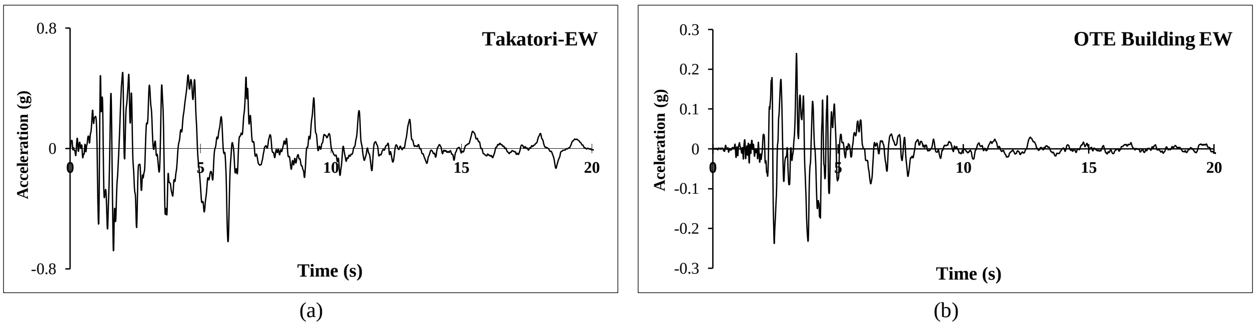


Figure 19. Seismic record used in the seismic assessment of the example bridge

5.3. Influence of Lateral Load Pattern on Capacity Curve

To evaluate the influence of lateral load pattern shape on the capacity curve, the nonlinear behavior of Model M1 was analyzed. In this study, the structural response was assessed using the lateral load patterns proposed in FEMA 273 [5]. Figure 20 illustrates the distribution of these load patterns, showing that the lateral force vectors exhibit markedly different shapes, except for the Fundamental Mode (MF) and the Multimodal (SRSS) vectors, which are nearly identical. This similarity is attributed to the dominance of the fundamental mode throughout the inelastic response, whose participation factor accounts for nearly 100% of the total modal contribution.

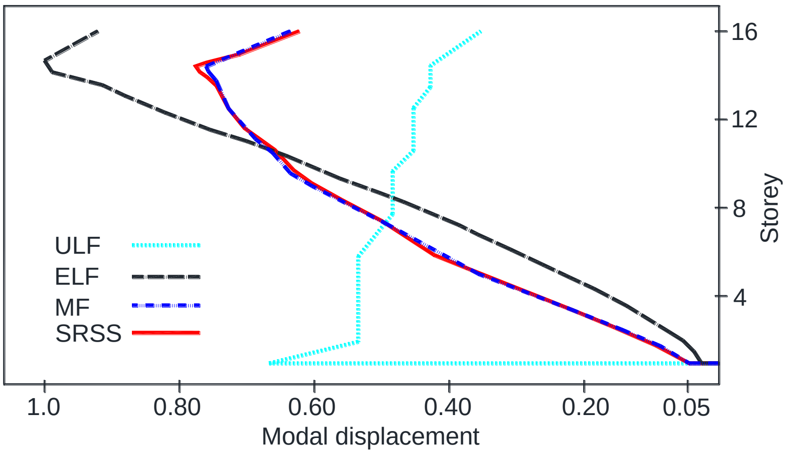


Figure 20. Lateral load pattern shape, FEMA 273 [5]

To generate the capacity curves associated with the load patterns defined in FEMA 273 [5], a force-based pushover analysis was carried out. As shown in Figure 21, the capacity curve obtained using the fundamental mode load pattern (MF) closely matches that obtained through the multimodal combination rule (SRSS). This similarity is associated with the regular dynamic behavior of the structure. Among the evaluated load patterns, the MF and SRSS curves provide the closest approximation to the dynamic capacity curve obtained through Incremental Dynamic Analysis (IDA). However, this comparison applies only to the shape of the capacity curves, since force-based pushover analyses do not provide performance indices associated with specific seismic demand intensities.

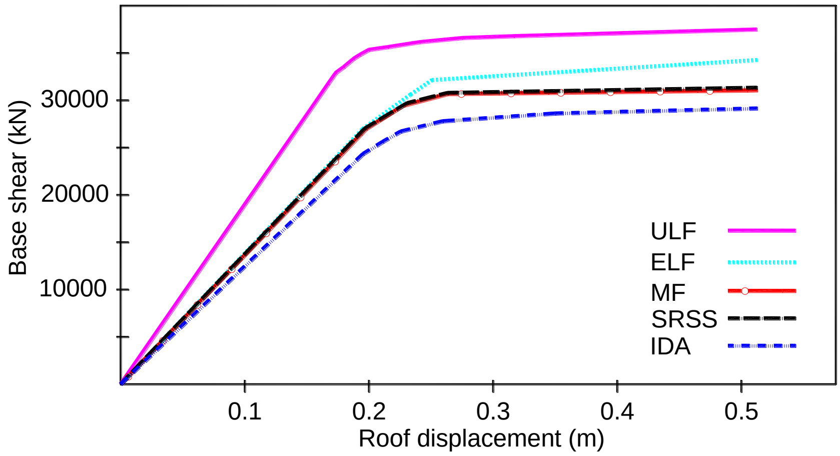


Figure 21. Capacity curves obtained using different lateral load patterns

By contrast, the capacity curves obtained using the Uniform Load Vector (ULF) and the Equivalent Lateral Force (ELF) load patterns differ significantly from the IDA curve in terms of their shape. Nevertheless, the yield point of the structure remains comparable for all load patterns, including the IDA curve. Therefore, if the objective is to estimate the seismic performance point, the selection of the lateral load pattern is not a critical factor. Conversely, if the objective is to characterize structural behavior within the nonlinear range, current guidelines recommend the use of more than one lateral load pattern.

5.4. Influence of the Lateral Pushover Technique in the Construction of the Capacity Curve

Figure 22 compares the dynamic capacity curve obtained from Incremental Dynamic Analysis (IDA) with that derived from the adaptive spectral modal procedure proposed by the authors. The results show that both curves exhibit similar behavior in the linear range, while in the nonlinear range the correspondence remains reasonably close. The main advantage of constructing the capacity curve by means of an adaptive spectral modal analysis, proposed by the authors, is that the estimated seismic performance is directly associated with a defined seismic demand and provides results comparable to those obtained from incremental dynamic analyses. In contrast, conventional force-based pushover procedures generate capacity curves that only represent the structural response under an imposed lateral load pattern, without establishing a direct correspondence with a specific seismic performance level.

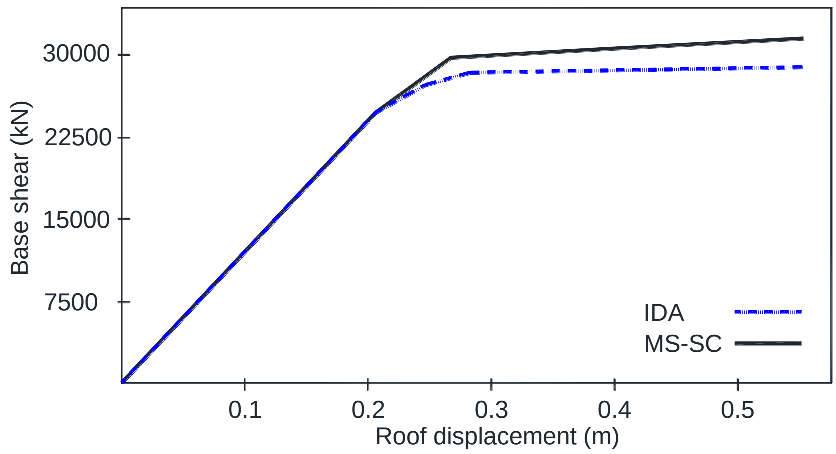


Figure 22. Comparison between dynamic capacity curves obtained from pushover analysis and nonlinear dynamic analysis (IDA)

5.5. Seismic Assessment Results of the Analyzed Structures

Nonlinear static procedures constitute simplified seismic assessment methodologies that provide insights beyond those obtainable through conventional equivalent static analysis. However, the most representative procedures within this category—namely, traditional pushover analysis and adaptive modal analysis—estimate the seismic response of a structure by generating a capacity curve that is comparable to that obtained through a numerically rigorous benchmark procedure only in terms of its overall shape. This limitation arises because these pushover-based procedures do not explicitly account for critical phenomena that significantly influence structural response, such as hysteretic energy dissipation, which is inherently captured in Incremental Dynamic Analysis (IDA).

5.6. Seismic Assessment Results of the Frame Structures

To evaluate the performance of these pushover-based procedures, the nonlinear behavior of Model M2 was analyzed. Key seismic performance indices—such as lateral displacements, inter-story drifts, and deformation profiles—associated with a defined seismic demand were assessed. Figure 23 compares the capacity curve obtained through an evolutionary spectral modal analysis without hysteresis correction (MS), proposed by Mendoza and Ayala [36], with the dynamic capacity curve obtained through Incremental Dynamic Analysis (IDA), adopted herein as the benchmark reference procedure.

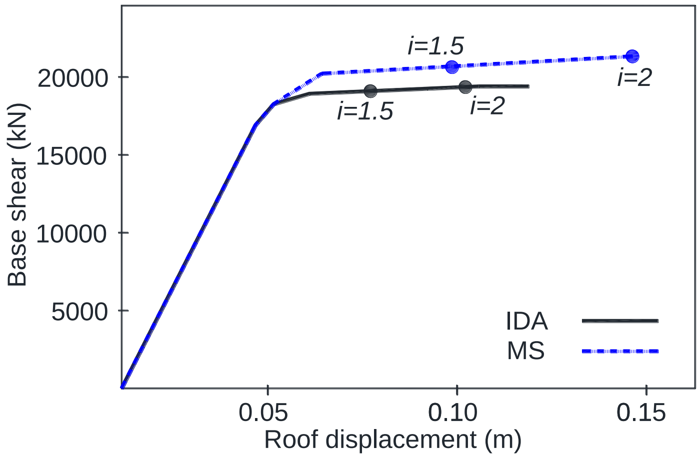


Figure 23. Comparison between dynamic capacity curves obtained from IDA and modal spectral analysis (MS)

Both capacity curves exhibit similar performance points within the elastic range; however, once the structure enters the nonlinear range, the predicted performance points diverge significantly. This observation indicates that, although the proposed procedure provides a reasonable approximation of the capacity curve, the accuracy of the estimated performance point depends on the type of correction applied and on the validity of the equal displacement rule. Figure 23 further shows that, when the equal displacement rule is not applicable, the seismic performance predicted by an approximate procedure may differ substantially from that obtained through a numerically rigorous nonlinear dynamic analysis.

Figure 24 presents a comparison between the dynamic capacity curve obtained through Incremental Dynamic Analysis (IDA), the capacity curve derived from the hysteresis-corrected adaptive spectral modal procedure proposed by Bañuelos-García et al. [35], and the Capacity Spectrum Method (CSM) [4], one of the most widely used approximate seismic assessment procedures in engineering practice. The results show that the capacity curve obtained using the proposed method exhibits behavior closely consistent with that obtained from IDA, whereas the curve derived from the traditional pushover-based CSM differs significantly from the benchmark nonlinear dynamic response. It can also be observed that the seismic performance points estimated by the proposed method and IDA are in close agreement, while the performance point predicted by the Capacity Spectrum Method shows a significant deviation. Although the Capacity Spectrum Method may provide acceptable approximations under specific conditions, its reliance on a traditional force-based pushover analysis may lead to inconsistencies in the estimation of structural response. Overall, the proposed methodology provides a more reliable approximation of structural behavior in both the elastic and nonlinear ranges.

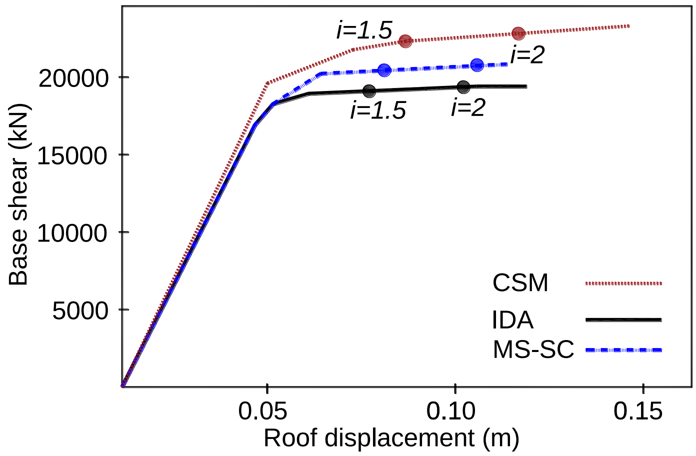


Figure 24. Dynamic capacity curves vs. adaptive modal analysis, considering energy dissipation by hysteresis

Figure 25 presents a comparison of the inter-story drifts and lateral deformation profiles obtained through Incremental Dynamic Analysis (IDA), the hysteresis-corrected adaptive spectral modal procedure proposed by Bañuelos-García et al. [35], and the Capacity Spectrum Method (CSM) [4]. The results show that the inter-story drift distribution and deformation profile estimated by the proposed method are in close agreement with those obtained from the benchmark nonlinear dynamic analysis, providing a reliable representation of the structural performance associated with a defined seismic demand. By contrast, the drift profile estimated using the Capacity Spectrum Method exhibits significant differences, particularly in the stories where nonlinear demand is concentrated. These results confirm that the proposed methodology provides a more accurate estimation of seismic performance indices, particularly inter-story drifts, than conventional pushover-based procedures.

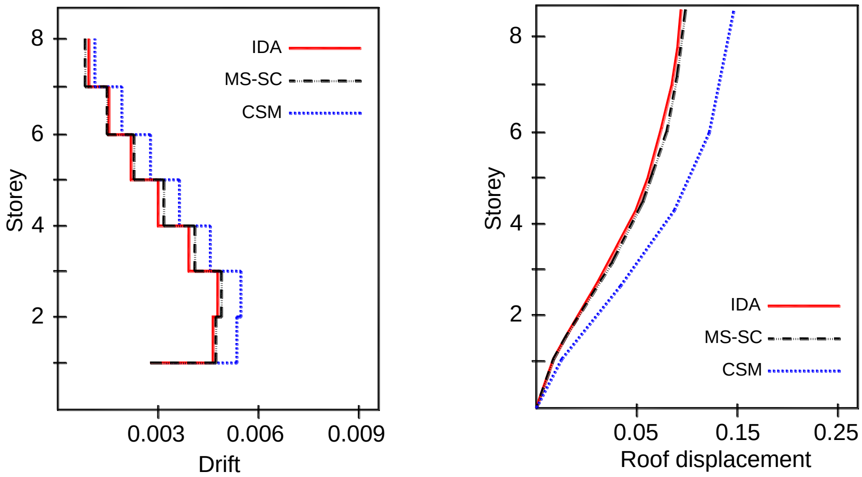
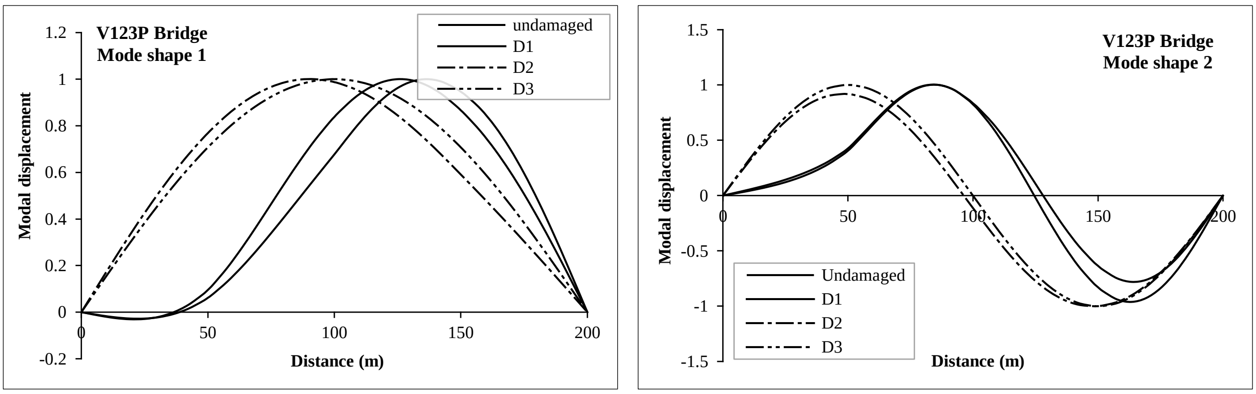


Figure 25. IDA performance indices vs. pushover analysis

5.7. Seismic Assessment Results of the Bridge Structure

As discussed in the preceding paragraphs, several studies have established that the degree of structural irregularity does not depend solely on the stiffness distribution but is also significantly influenced by the structural response within the nonlinear range. In a previous study, the authors of the present work [22] proposed a regularity index based on the evolution of modal shapes throughout the inelastic response. Figures 26-a and 26-b illustrate the evolution of the modal shapes within the elastic and inelastic ranges. A significant variation between the elastic and inelastic modal shapes can be observed, indicating that the bridge exhibits structural irregularity. Consequently, characterising the inelastic response of a Multiple-Degree-of-Freedom (MDOF) system by means of the response of an equivalent Single-Degree-of-Freedom (SDOF) oscillator is not always feasible, since one of the fundamental underlying assumptions, namely the equal displacement rule, is not satisfied, as reported by Escamilla [46].



(a) (b)

Figure 26. Mode shapes of the irregular bridges studied for different damage states

Figure 27 presents a comparative assessment of the seismic performance of the V123P bridge, considering a seismic demand characterized by the previously described ground motion records, where the first record (1995 Kobe earthquake Takatori) exhibits a peak ground acceleration of 0.68 g and the second record (1986 Kalamata earthquake Kalamata) exhibits a peak ground acceleration of 0.24 g. Figures 27-a and 27-c show the deformed displacement profile and the capacity curve. Figures 27-a and 27-b show that, for the same displacement magnitude, there is a clear correspondence with the intensity of the seismic demand; that is, the performance estimated using the proposed approximate method, SNAP, is consistent with the performance obtained from Incremental Dynamic Analysis (IDA).

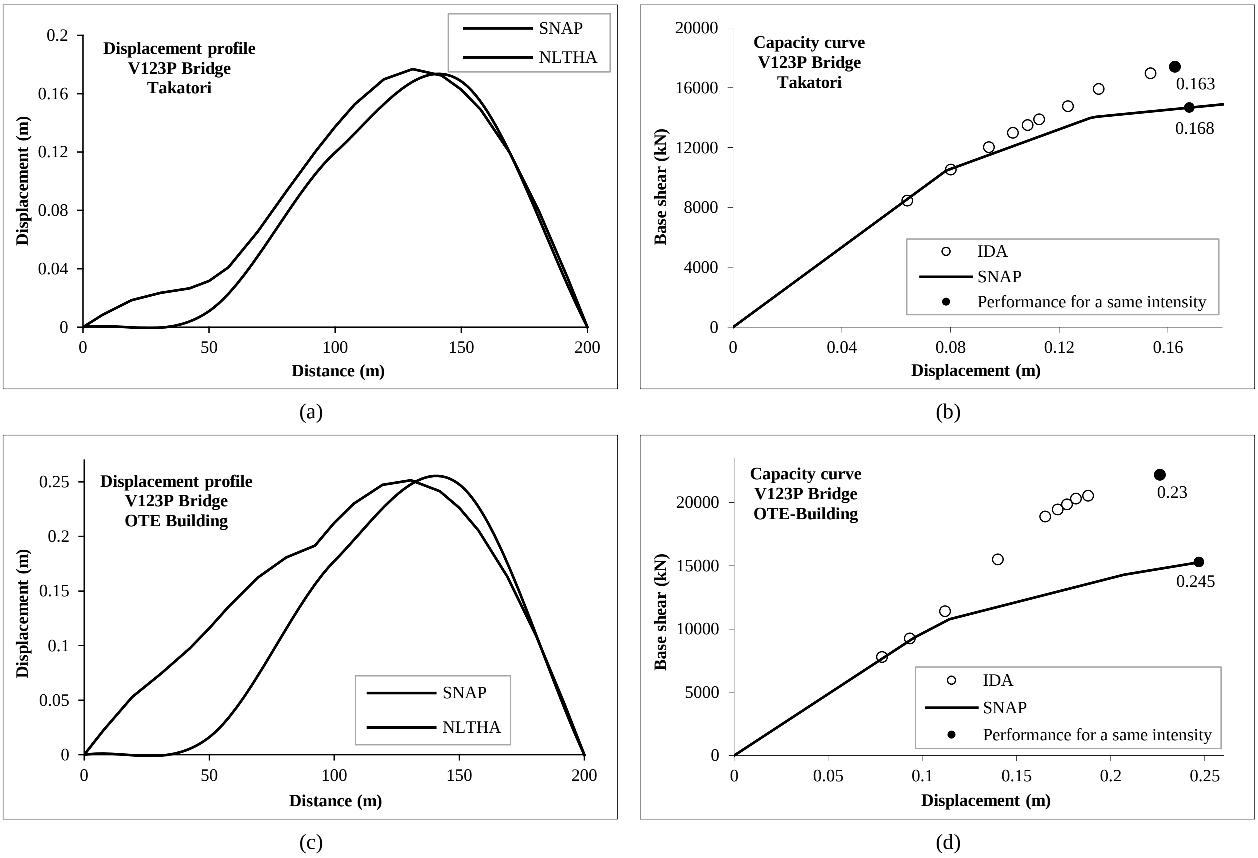


Figure 27. Comparison of the transverse displacements and of the dynamic capacity curve calculated using the proposed method and IDA for the modal irregular bridge V123P under the Takatori and OTE Building seismic records

By contrast, Figures 27c and 27-d show that, for the same displacement magnitude, there is no clear correspondence with the intensity of the seismic demand; in other words, the performance estimated using the proposed approximate method, SNAP, is not fully consistent with that obtained from IDA. It can be observed that the inelastic displacement predicted by the proposed method is similar to that obtained from IDA; however, the base shear differs significantly. Furthermore, the displacement profiles obtained from both methods are not consistent, particularly in the region where the piers contribute the majority of the structural stiffness.

6. Conclusion

This paper presents the findings of an ongoing investigation into the capabilities and limitations of the capacity curve, and its influence on the estimation of seismic performance indices. The most representative pushover-based procedures—both force-based and displacement-based—were analyzed in order to assess their influence on structural response under a defined seismic demand. The conclusions presented herein are based on both a conceptual study and a numerical evaluation employing the lateral load patterns proposed in FEMA 273 [5], together with the most representative pushover analysis procedures.

Traditional pushover analysis remains widely accepted in engineering practice, as it provides information regarding structural performance that cannot be obtained through conventional equivalent static analysis. However, it also presents important limitations. When applied to structures whose inelastic response is not governed by the fundamental mode, the procedure assumes that modal shapes remain invariant throughout the nonlinear range—an assumption valid only for idealized structures rarely encountered in professional practice (e.g., planar frame systems). Despite these limitations, traditional pushover analysis may still provide valuable insight into structural behavior under external actions, particularly when adaptive pushover procedures are employed to better represent both seismic demand and the evolving dynamic characteristics of the structure.

The selection of the most appropriate pushover analysis procedure for assessing structural response under seismic loading depends primarily on two factors: the accuracy of the results and the practical applicability of the methodology. The former is influenced by the assumptions and simplifications inherent in the formulation of the procedure, whereas the latter depends on practical considerations such as computational cost, transparency, and ease of implementation for practicing engineers.

The results of this investigation demonstrate that structural behavior may be represented more accurately through displacement-controlled procedures than through force-controlled procedures. Consequently, displacement-based methodologies are considered the most appropriate procedures for estimating structural response and seismic performance indices. However, many currently available displacement-based procedures focus solely on generating a capacity curve that approximates the benchmark dynamic capacity curve, without addressing the limitation that the associated performance indices do not correspond to the same seismic intensity levels as those obtained through Incremental Dynamic Analysis (IDA).

The use of proposed procedure in this paper for structures produces approximations of seismic performances close to those obtained with procedures considered as "exact". However, in structures where several mode shapes significantly contribute to its seismic performance in the non-linear range and one or more mode shapes in the non-linear range do not correspond to any of the corresponding elastic mode shapes, *e.g*. mode shapes in the non-linear range influenced by two or more elastic modal shapes, the correction for modal regularity proposed in this paper does not lead to correct results

The hysteresis-corrected adaptive spectral modal methodology, based on the developments of Mendoza & Ayala [36] and Bañuelos-García et al. [35], addresses this limitation by incorporating equivalent damping and stiffness degradation at each analysis step

7. Declarations

7.1. Author Contributions

Conceptualization, M.A.E.; methodology, M.A.E. and G.A.; software, M.A.E., F.B.G., and H.G.; validation, M.A.E. and F.B.G.; formal analysis, M.A.E. and H.G.; investigation, M.A.E. and G.A.; writing—original draft preparation, M.A.E.; writing—review and editing, F.B.G. and H.G.; supervision, M.A.E and G.A. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

7.3. Funding and Acknowledgments

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7.4. Conflicts of Interest

The authors declare no conflict of interest.

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