# Seismic Assessment of Integral Reinforced Concrete Bridges Using Adaptive Multi-Modal Pushover Analysis

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**SUMMARY:**
This paper presents a new adaptive pushover procedure to account for the effect of higher modes in order to estimate the seismic response of bridges more accurate. The suitability and robustness of the proposed method is demonstrated by a parametric study for regular and irregular configuration of integral bridges which includes 9 bridge configurations having 4 spans with the varying length of spans and height of piers. In each step of the nonlinear analysis, the updated displacement capacity of the structure, obtained by the proposed method, is compared with the updated displacement demand, determined from the response spectrum of the scaled earthquake excitation. The procedure is conducted step by step until the performance point of the structure is found. Numerical results indicate that in most cases studied in this research, the proposed method is capable of predicting displacements as well as internal forces with desirable accuracy. The response of the proposed procedure is in good agreement with the response of inelastic time-history analysis and is believed to be quite satisfactory in comparison with the current pushover procedures. This method is therefore recommended to be applied to the seismic performance evaluation of integral bridges for engineering applications.

*Keywords: seismic assessment, adaptive pushover, higher-modes effect, integral bridge*

## 1. INTRODUCTION

In recent years, there has been an increasing attention to Nonlinear Static Procedures (NSPs) for the seismic assessment and evaluation of bridges among other structures. In these methods, a pushover analysis is employed to predict the inelastic behaviour of the structure with emphasis on the inelastic displacements rather than forces within the structural members (Shattarat et. al. 2008).

Damage potential and ultimate failure can usually be directly related to the inelastic displacement capacity of the structural elements (Chandler & Mendis. 2000) and NSPs are generally capable of approximating these capacities with rather acceptable accuracy while offering a compromise between the simplified linear static analysis and the complex nonlinear dynamic analysis.

Nonlinear static approaches for seismic assessment of bridges have been developed over the past decade, generally based on the classic pushover procedures, the Capacity Spectrum Method (CSM), (Freeman et al. 1975) and the inelastic demand spectrum method (Fajfar & Fischinger. 1988) and (Fajfar et al. 1997). The classic CSM has been developed and introduced initially for application to the seismic assessment of buildings. Employing of such a methodology for integral bridges usually requires the consideration of higher modes of vibration, which may significantly influence the seismic response of the bridge (Casarotti & Pinho. 2007). Thus, the development and employment of an adaptive version of the CSM with consideration of higher modes seems to be satisfactory, coupling the simplicity of the classic CSM with the accuracy of an adaptive scheme of the pushover loading with the effect of higher modes to be taken into account.

In this paper, a new adaptive pushover procedure is presented which includes the effect of higher modes of vibration in the assemblage of the pushover load pattern. The proposed procedure can be described as an adaptive capacity spectrum approach which employs the substitute structure methodology to model the inelastic multi degree of freedom (MDOF) structure as an equivalent single degree of freedom (SDOF) system with equivalent elastic properties. The seismic demand is also defined by appropriately over-damped elastic response spectra of the given earthquake record.

To determine the viability of the proposed procedure, a parametric study is conducted on a set of integral bridges subjected to earthquake motion. The procedure mainly consists of determination of capacity and demand curves of the structure. For the capacity curve, the gravity loads are first applied to the bridge and then the proposed static pushover analysis with new features in applying the pushover load pattern is performed to establish the pushover curve of the MDOF structure. The pushover curve is then transformed into the capacity curve of the equivalent SDOF system which represents the MDOF bridge seismic response. On the other hand, the demand curve is derived at each step of the pushover analysis from the over-damped elastic SDOF system, and is updated step by step based on the current stiffness and hysteretic damping of the MDOF structure.

## 2. PROPOSED ADAPTIVE PUSHOVER PROCEDURE

The proposed pushover procedure for the seismic assessment of bridge structures can be described as the following main steps, explained in detail in what follows:

1. Conducting an adaptive multi-mode pushover analysis to obtain capacity curve of the equivalent SDOF system, which represents the load capacity of the MDOF bridge structure,
2. Determination of the updated over-damped elastic response spectrum to be applied to the capacity curve of the SDOF system,
3. Determination of the performance point of the bridge under the applied earthquake action,
4. Determination of displacements and internal forces of the MDOF bridge

The proposed methodology is schematically summarized in Fig. 2.1.

### 2.1. Determination of the Adaptive Capacity Curve

The first step of the seismic assessment is to perform a reliable pushover analysis on a nonlinear model of the MDOF structure. The proposed adaptive pushover analysis is carried out by the open source code OpenSees (McKenna et al. 2000) which performs the following steps at each increment of the pushover loading:

1. Conducting an eigenvalue analysis as $\left| K-\omega^2 M \right|=0$ in order to find the modal shapes. The proposed method gives the opportunity to determine the desired total modal mass ratio (for example, 90%) to be participated into the analysis throughout the whole procedure. Therefore, the number of modes that participate in the assembly of the pushover load pattern may vary during the analysis due to the geometrical nonlinearity and material inelasticity within the structure.

2. Assembling the incremental pushover load pattern, $\{\Delta P_i\}$, based on the combination of multi-modal parameters through Eqn. 2.1 to 2.3:

$$F_{ij} = \Gamma_j S_{a,j} m_i \Phi_{ij} \qquad , \qquad \Gamma_j = \frac{\sum_{i=1}^{n} m_i \Phi_{ij}}{\sum_{i=1}^{n} m_i \Phi_{ij}^2} \tag{2.1}$$

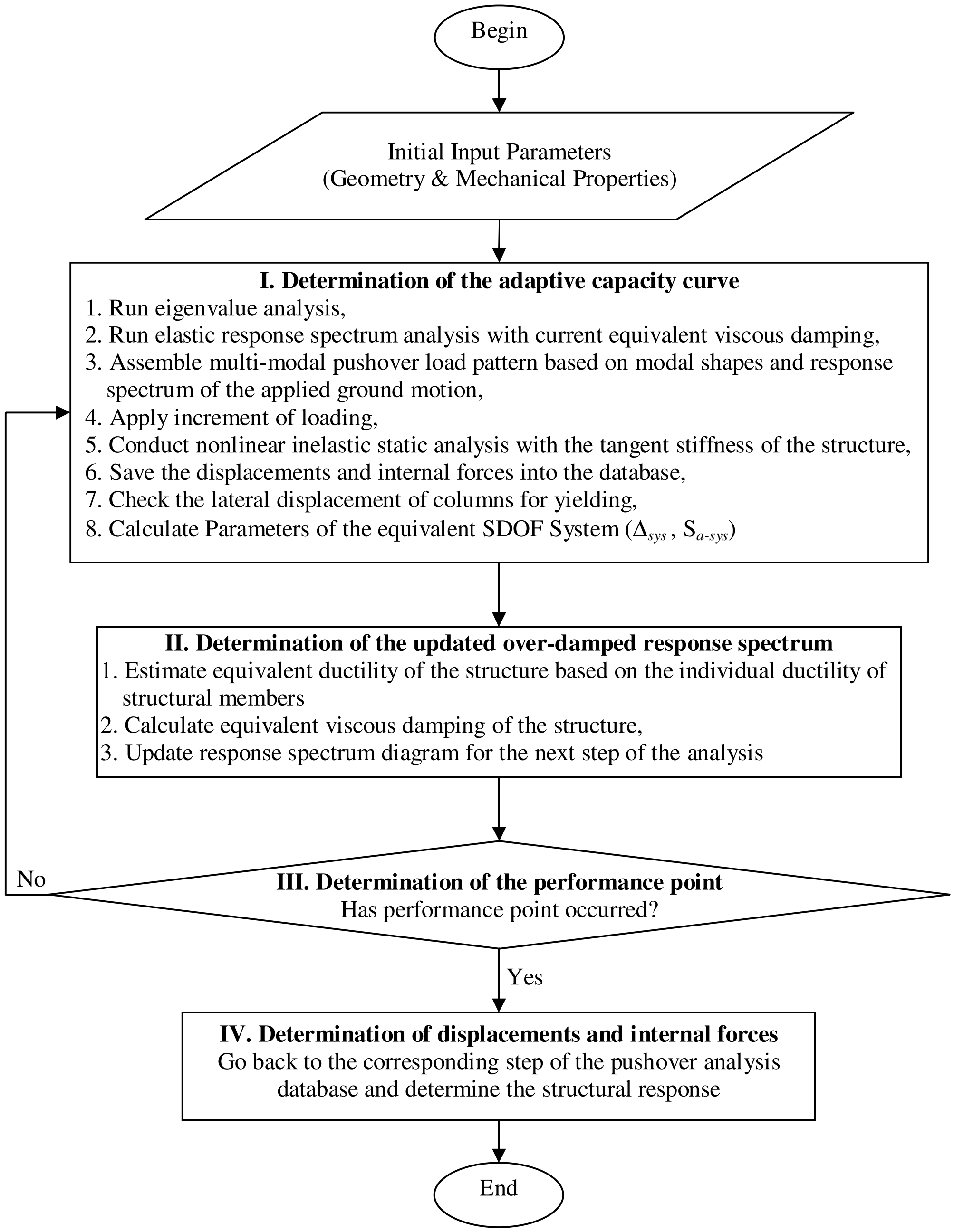


**Figure 2.1.** Flowchart of the proposed seismic assessment methodology

$$F_i = \sqrt{\sum_{j=1}^{m} F_{ij}^2} \quad , \quad \overline{F}_i = \frac{F_i}{\sum_i F_i} \tag{2.2}$$

$$\{\Delta P_i\} = \Delta V_{base} \times \{\overline{F}_i\} \tag{2.3}$$

in which, $m_i$ is the mass of the $i^{th}$ DOF, $\Phi_{ij}$ is the mode shape of DOF $i$ at mode $j$, $\Gamma_j$ is the modal participation factor of mode $j$, and $S_{a,j}$ is the spectral psudo-acceleration of mode $j$ determined from the response spectrum of the earthquake record. $F_{ij}$ is the modal force at the $i^{th}$ DOF at mode $j$, $F_i$ is the combined modal force applied at DOF $i$ based on the SRSS combination rule, $\overline{F_i}$ is the normalized modal force at the $i^{th}$ DOF, and $\Delta V_{base}$ is the incremental base shear defined by the user at the beginning of the analysis.

3. Performing an adaptive pushover analysis by employing the well known Newton-Raphson numerical method:

$$[K_T]\{\Delta D\} = \{\Delta P\} \tag{2.4}$$

where $\{\Delta D\}$ is the incremental displacement vector of the bridge and $[K_T]$ is the tangent stiffness of the structure including geometrical nonlinearity and material inelasticity.

4. Calculating the displacement ductility of piers, as dissipating elements of the hysteretic energy, by comparing the lateral displacement of piers, $\Delta_{col}$ to their yield displacement, $\Delta_y$ as (Priestley. 2003):

$$\mu_{col} = \frac{\Delta_{col}}{\Delta_y} \quad , \quad \Delta_y = \Phi_y \frac{H_e^{\ 2}}{3} = \frac{2.25\varepsilon_y}{D}\frac{(H+2l_{sp})^2}{6} \tag{2.5}$$

in which, $\Phi_y$ is the yield curvature of the column section, $\varepsilon_y$ is the yield strain of the longitudinal bar, $H$ is the height and $D$ is the diameter of the column. $l_{sp}$ is the strain penetration length and is given by (Priestley et al. 1996):

$$l_{sp} = 0.022 f_y d_{bl} \tag{2.6}$$

where $f_y$ and $d_{bl}$ are the yield stress and diameter of the longitudinal bar, respectively. It should be noted that Eqn. 2.5 is given for the case of circular column with monolithic connections to both the superstructure and the base.

5. Deriving the equivalent SDOF adaptive capacity curve by calculating the equivalent system displacement $\Delta_{sys,k}$ and system acceleration $S_{a\text{-}sys,k}$ based on the deformed shape of the structure at each analysis step $k$, according to Eqns. 2.7 and 2.8 (Casarotti & Pinho. 2007):

$$\Delta_{sys,k} = \frac{\sum_i m_i \Delta_{i,k}^2}{\sum_i m_i \Delta_{i,k}} \tag{2.7}$$

$$S_{a-sys,k} = \frac{\sum_i m_i \Delta_{i,k}^2}{(\sum_i m_i \Delta_{i,k})^2} \frac{V_{base,k}}{g} \tag{2.8}$$

where $V_{base,k}$ is the total base shear of the structure at step $k$. It should be noted that similar to the method proposed in (Casarotti & Pinho. 2007), $\Delta_{sys,k}$ and $S_{a\text{-}sys,k}$ are calculated step by step based on the current deformed shape of the structure and thus vary at each step, unlike the invariant elastic or inelastic modal shapes used in most pushover procedures.

### 2.2. Determination of the Updated Over-damped Response Spectrum

At each step of the proposed pushover analysis, the response spectrum of the ground motion is updated to match the current state of the structure. In order to determine the updated over-damped response spectrum, it is necessary to determine the equivalent damping of the structure at each load step. The equivalent damping of the structure can then be evaluated by the damping-ductility relationship for the Takeda degrading stiffness hysteretic response (Takeda et. al. 1970) and (Kowalsky et al. 1995) as:

$$\xi_{eq} = 0.05 + \frac{1}{\pi}\left[1 - \frac{1-r}{\sqrt{\mu_{eq}}} - r\sqrt{\mu_{eq}}\right] \tag{2.9}$$

where $\mu_{eq}$ is the equivalent ductility of the structure and $r$ represents the post-yield stiffness ratio and is assumed to be 0.05 in this paper. Also a viscous damping ratio of 5% is assigned to the bridge and is added to the equivalent damping obtained from the hysteretic energy dissipation.

To assess the equivalent ductility of the structure, it is assumed that $\mu_{eq}$ is the average ductility of all members of the substructure, weighted by the shear force of each member according to Eqn. 2.10, which explicitly considers the contribution of the elastic deck, abutments and piers into the total lateral resistance of the structure:

$$\mu_{eq} = \frac{(\sum_{j=1}^{2} V_{j,abt}) \times 1 + \sum_{k=1}^{n_{pier}} V_{k,pier}\mu_{k,pier}}{V_{base}} \tag{2.10}$$

In which, $V_{j,abt}$ and $V_{k,pier}$ are the shear force of abutment $j$ and pier $k$ at the current step of the analysis, respectively. In Eqn. 2.10, the ductility of the abutments is conservatively assumed to be 1, neglecting the rather poor soil-abutment interaction in the transverse direction.

### 2.3. Determination of the Performance Point

The developed adaptive capacity curve is intersected with the demand spectrum, providing an estimate of the inelastic acceleration and displacement demand of the input motion on the structure, as shown in Fig. 2.2. It should be noted that if the demand spectrum is described by an earthquake response spectrum rather than a smoothed design spectrum, more than one intersection with the capacity curve may be found. It has been verified by (Casarotti & Pinho. 2007) that generally the intersection corresponding to the largest displacement is the correct performance point because after such a point, the capacity of the structure is well below the demand of the employed ground motion.

### 2.4. Determination of Displacements and Internal Forces

Once a performance point on the SDOF capacity curve is established, it is sufficient to go back to the

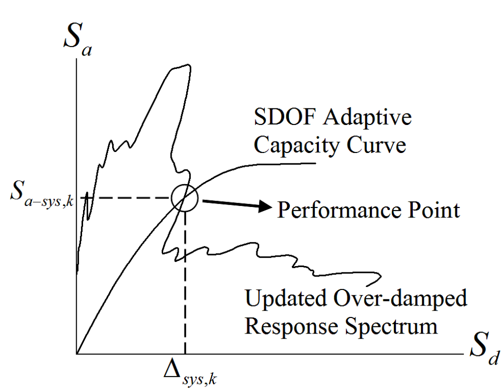


**Figure 2.2.** Determination of the performance point

corresponding step of the pushover analysis database and determine the structural response in terms of displacements and forces, as well as modal properties of the bridge under the applied ground motion.

## 3. PARAMETRIC STUDY

The accuracy and robustness of the proposed pushover procedure is verified by assessing the seismic response of nine, 180 meters long, bridge configurations. The parametric study consists of four-span integral reinforced concrete box girder bridges with two-column integral bents supported on pile foundations (Fig. 3.1). A fiber based distributed plasticity finite element program is implemented in OpenSees by the authors for the seismic assessment of bridge structures. For the bridge models used in this study, the potential of inelasticity is restricted to the columns, while the elements representing the superstructure are assumed to remain elastic. However, geometrical nonlinearity is taken into account for all elements of the structure.

The material used for all members of the bridge is reinforced concrete. The concrete has a Young modulus of 28 GPa and a shear modulus of 12 GPa, while a 200 GPa modulus of elasticity is assumed for all the steel reinforcing bars. The constitutive law of the concrete is described by Mander's confined concrete model (Mander et. al. 1988) with the compressive strength of 35 MPa. Also Menegotto-Pinto (Menegotto & Pinto 1973) model was utilized for the constitutive behaviour of the steel reinforcing bars which have yield stress of 450 MPa.

A typical finite element model of bridge is shown in Fig. 3.2. This model consists of three-dimensional (3D) frame elements which pass through the geometrical center of members. The mass of each element is equally distributed at its end nodes. The seismic weight of the bridge includes the weight of superstructure, cap beams, and half of piers.

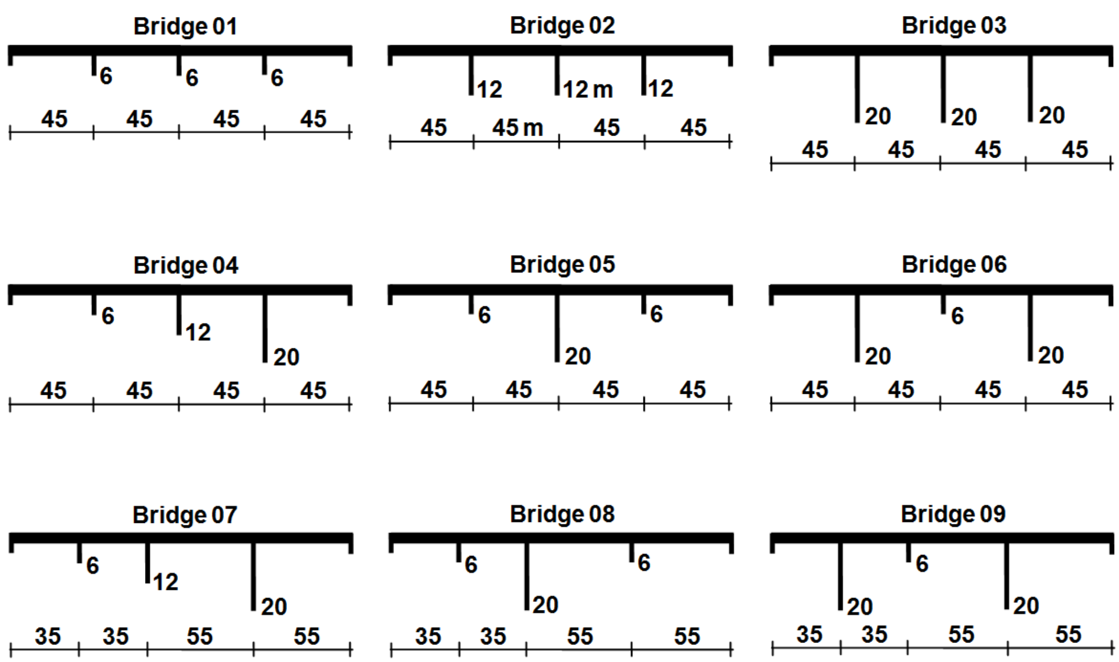


**Figure 3.1.** Bridge configurations of the parametric study

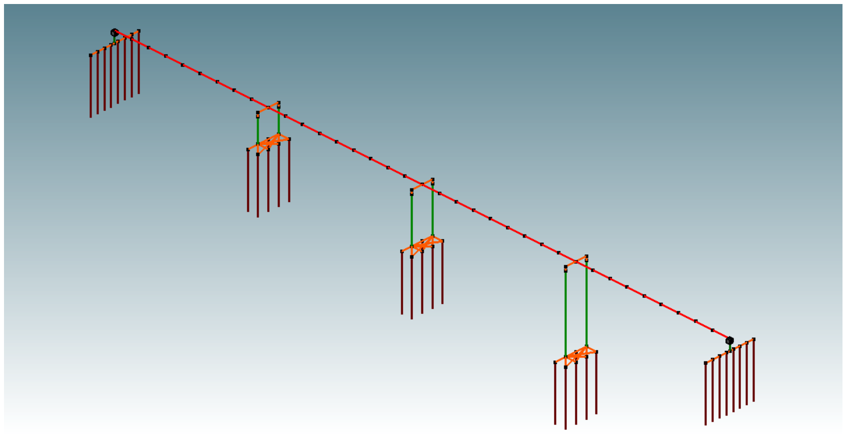

**Figure 3.2.** 3D structural model of the bridge in OpenSees

The bridge deck is a multi-cell box girder with total width of 15m and depth of 2.3m. The deck model consists of 3D elastic nonlinear beam-column elements with elastic sections, fully characterized by the cracked sectional properties. The superstructure is assumed to be connected to the abutments and piers monolithically and thus full continuity is achieved at the connections of superstructure to substructure.

The circular columns of piers are modeled by 3D inelastic force-based beam-column elements (Mazzoni et al. 2005). The transverse volumetric steel ratio of columns is 1% and the concrete cover to the main reinforcement is 50 mm. A rigid end zone of 1.0 m is located at the top of columns to account for the offset between the lower soffit of the box girder and the geometric center of superstructure.

In order to realistically model the soil-structure interaction, nonlinear springs are utilized to simulate the horizontal reaction of the abutments' backfill as well as the soil around the piles and pile caps. The p-y spring model is employed for soil-pile interaction (API. 2000), while the hyperbolic relationship proposed by (Shamsabadi et al. 2007) is employed for the interaction of pile cap and soil. The soil-abutment interaction in the transverse direction is represented by bilinear spring behaviour, determined by the shear capacity of the wing walls (Goel & Chopra. 1997). Also equivalent linear springs are employed to simulate the vertical reaction of the soil surrounding the bridge supports with characteristics determined by (Das. 2010).

The employed seismic excitation is defined by Tabas earthquake record, applied to the bridge in the transverse direction. The characteristics of the input ground motion are shown in Fig. 3.3, and are summarized in Table 3.1.

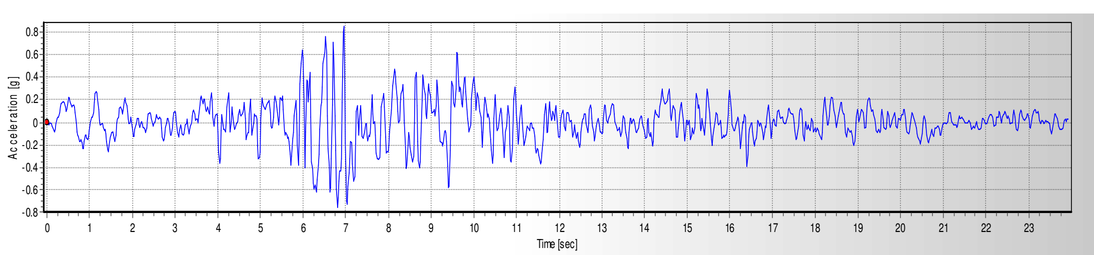


**Figure 3.3.** Input ground motion (Tabas, 1978)

**Table 3.1.** Characteristics of the Input Ground Motion

| Earthquake | Date | Station | Magnitude (M) | Component | PGA(g) |
|---|---|---|---|---|---|
| Tabas, Iran | 19/09/1978 | 9101 Tabas | 7.4 | E–W | 0.836 |

The accuracy of the proposed method is evaluated by means of nonlinear time-history analysis (THA), which represents the most accurate tool to estimate the dynamic response of structures. In the time-history analysis, a 2% Raleigh damping was assigned to the structure, proportional to the first two natural modes of vibration in the direction of the applied ground motion.

## 4. NUMERICAL RESULTS

The typical numerical results of the parametric study are presented in Figs. 4.1 and 4.2, as well as Tables. 4.1 to 4.3.

Fig. 4.1 shows the inelastic displacement pattern of the bridge deck for the models bridge04 and Bridge05 as representatives of asymmetric and symmetric configurations, respectively (see Fig. 3.1). It is observed that the deformed shape of the deck is very well captured by the proposed method with quite good accuracy. Similar results can be seen in Fig. 4.2 for the hysteretic response of columns. Figs. 4.1 and 4.2 clearly indicate the capability of the proposed method in estimation of the displacement response of bridges.

Force responses considered in this study are in terms of axial and shear force of columns as well as base moment at the pile caps. These quantities are summarized in Tables. 4.1 to 4.3, for the critical column in each of the nine bridge configurations. Herein, the critical pier (two-column pier) is

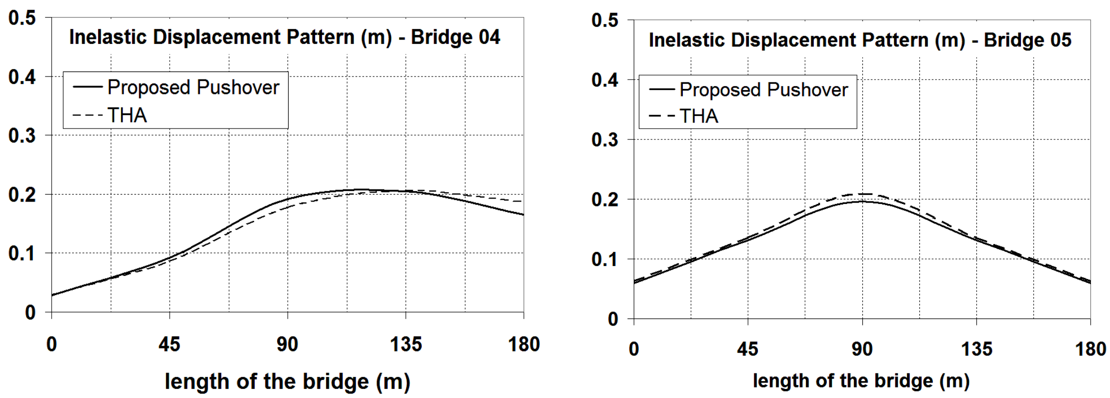


**Figure 4.1.** Inelastic displacement pattern of the deck for the models: Bridge04 and Bridge05

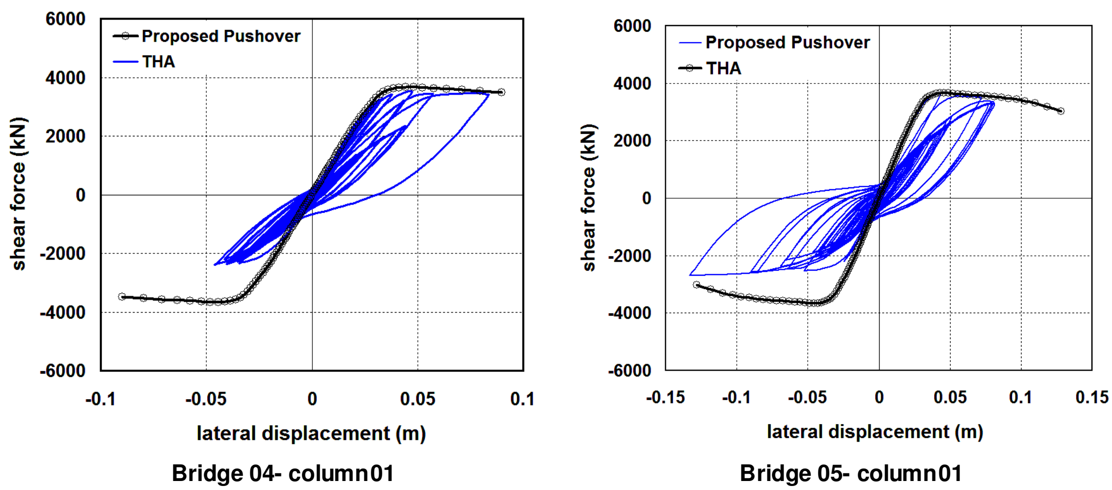


**Figure 4.2.** Hysteretic lateral response and estimated pushover curve of column01 in Bridge04 and Bridge05

**Table 4.1.** Verification of the proposed pushover method by the THA, for column axial force response (kN)

| | Bridge 01 | Bridge 02 | Bridge 03 | Bridge 04 | Bridge 05 | Bridge 06 | Bridge 07 | Bridge 08 | Bridge 09 |
|---|---|---|---|---|---|---|---|---|---|
| critical pier | Pier 1,3 | Pier 1,3 | Pier 1,3 | Pier 1 | Pier 1,3 | Pier 2 | Pier 3 | Pier 3 | Pier 2 |
| Proposed Pushover | 9559 | 10581 | 13109 | 10908 | 10897 | 9800 | 12447 | no P.P. | no P.P. |
| THA | 9879 | 10719 | 12558 | 11313 | 11972 | 11554 | 12414 | failed | failed |
| error (%) | 3.2 | 1.3 | 4.4 | 3.6 | 9.0 | 15.2 | 0.3 | — | — |

**Table 4.2.** Verification of the proposed pushover method by the THA, for column shear force response (kN)

| | Bridge 01 | Bridge 02 | Bridge 03 | Bridge 04 | Bridge 05 | Bridge 06 | Bridge 07 | Bridge 08 | Bridge 09 |
|---|---|---|---|---|---|---|---|---|---|
| critical pier | Pier 1,3 | Pier 1,3 | Pier 1,3 | Pier 1 | Pier 1,3 | Pier 2 | Pier 3 | Pier 3 | Pier 2 |
| Proposed Pushover | 2320 | 1833 | 2033 | 3667 | 3661 | 3546 | 1093 | no P.P. | no P.P. |
| THA | 2341 | 1693 | 1850 | 3528 | 3643 | 3599 | 1085 | failed | failed |
| error (%) | 0.9 | 8.3 | 9.9 | 3.9 | 0.5 | 1.5 | 0.7 | — | — |

**Table 4.3.** Verification of the proposed pushover method by the THA, for base moment response (kN.m)

| | Bridge 01 | Bridge 02 | Bridge 03 | Bridge 04 | Bridge 05 | Bridge 06 | Bridge 07 | Bridge 08 | Bridge 09 |
|---|---|---|---|---|---|---|---|---|---|
| critical pier | Pier 1,3 | Pier 1,3 | Pier 1,3 | Pier 1 | Pier 1,3 | Pier 2 | Pier 3 | Pier 3 | Pier 2 |
| Proposed Pushover | 7769 | 12059 | 22645 | 12104 | 12093 | 11713 | 12800 | no P.P. | no P.P. |
| THA | 7845 | 11051 | 20154 | 11667 | 12005 | 11914 | 12499 | failed | failed |
| error (%) | 1.0 | 9.1 | 12.4 | 3.7 | 0.7 | 1.7 | 2.4 | — | — |

defined as the pier with maximum damage among all piers. It is observed that for all the three internal forces, the values predicted by the proposed method are in excellent agreement with those predicted by the inelastic time-history analysis. This is a very good result considering the general shortcoming of most current pushover procedures in estimation of internal forces rather than displacements.

## 5. CONCLUSION

An adaptive pushover procedure with new features is presented in this paper for the seismic assessment of integral reinforced concrete bridges. The proposed method can be categorized as a multi-mode adaptive capacity spectrum method, with improvements in modal combination during the process of assembling the pushover load pattern. The authors' computer program gives users the opportunity to determine a minimum desired total modal mass ratio (for example, the common value of 90% or any other desired value) to be participated in every step of the pushover analysis. Therefore, the number of modes participate in the assembly of the pushover load pattern may vary during the analysis due to the geometrical nonlinearity and material inelasticity. This can generally improve the accuracy of the proposed pushover procedure in comparison to the current adaptive pushover approaches, which take into account only the first mode or a predefined number of modes for the analysis. Numerical results of the parametric study conducted in this paper, indicate the excellent accuracy and robustness of the proposed pushover procedure. This method is therefore recommended to be applied to the seismic performance evaluation of integral bridges for engineering applications.

## REFERENCES


American Petroleum Institute (API). (2000), Recommended practice for planning, designing and constructing fixed offshore platforms—working stress design. 21th ed., Washington (DC): API RP 2A-WSD.

Casarotti, C. and Pinho, R. (2007). An adaptive capacity spectrum method for assessment of bridges subjected to earthquake action. *Bulletin of Earthquake Engineering*. **5**, 377–390.

Chandler, A.M. and Mendis, P.A. (2000). Performance of reinforced concrete frames using force and displacement based seismic assessment methods. *Engineering Structures* **22**,352–363.

Das, B. M. (2010). Principles of Foundation Engineering, 7th Edition, Cengage Learning.

Fajfar, P. and Fischinger, M. (1988). N2–A method of non–linear seismic analysis of regular buildings. *9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan*. 111–116.

Freeman, S.A., Nicoletti, J.P. and Tyrell, J.V. (1975). Evaluation of existing buildings for seismic risk–A case study of Puget Sound Naval Shipyard, Bremerton, Washington. *U.S. National Conference on Earthquake Engineering, Berkley, California.* 113–122.

Goel, R.K., Chopra, A.K. (1997). Evaluation of bridge abutment capacity. *Earthquake Spectra* **13:1**, 225–254.

Kowalsky, M. J., Priestley, M. J. N. and MacRae, G. A. (1995). Displacement-based design of RC bridge columns in seismic regions. *Earthquake Enginerring. and Structural Dynamics.* **24:12**, 1623–1642.

Mander, J.B., Priestley, M.J.N. and Park, R. (1988). Theoretical stress-strain model for confined concrete. *Journal of the Structural Engineering, ASCE* **114:8**,1804–1826.

Mazzoni, S., McKenna, F. and Fenves, G.L. (2005). OpenSees command language manual, *Pacific Earthquake Engineering Research (PEER) Center.*

McKenna, F., Fenves, G. and Scott, M. (2000). Open System for Earthquake Engineering Simulation. *University of California, Available from: http://opensees.berkeley.edu.*

Menegotto, M. and Pinto, P.E. (1973). Method of analysis for cyclically loaded R.C. plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending. *Symposium on the resistance and ultimate deformability of structures acted on by well defined repeated loads, International Association for Bridge and Structural Engineering, Zurich, Switzerland.* 15–22.

Priestley, M.J.N. (2003), Myths and Fallacies in Earthquake Engineering, Revisited, The Ninth Mallet Milne Lecture, IUSS Press, Pavia, Italy.

Priestley, M.J.N., Seible, F. and Calvi, G.M. (1996), Seismic Design and Retrofit of Bridges, John Wiley and Sons, New York.

Shamsabadi, A., Rollins, K.M. and Kapuskar, M. (2007). Nonlinear Soil-abutment-bridge Structure Interaction for Seismic Performance-based Design. *J. Geotech. & Geoenviron. Eng., ASCE* **133:6**, 707–720.

Shattarat, N.K., Symans, M.D., McLean, D.I. and Cofer, W.F. (2008). Evaluation of nonlinear static analysis methods and software tools for seismic analysis of highway bridges. *Engineering Structures* **30**, 1335–1345.

Takeda, T., Sozen, M.A. and Nielsen, N.N. (1970). Reinforced concrete response to simulated earthquakes. *Journal of the Structural Division, ASCE* **96:12**, 2557–2573.