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STUDY OF THE EFFECT OF THE NUMBERS OF MIDDLE PIERS (BASES) IN THE NUMBERS OF EFFECTIVE MODES IN SEISMIC EVALUATION OF BRIDGE

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ABSTRACT

The aim of this study is testing of nonlinear static analysis method for seismic evaluation of bridges. So, the application of the mentioned method was evaluated about three bridges with different openings and irregular piers (bases). In addition, the effect of the number of middle piers (bases) in the numbers of effective modes in seismic evaluation of bridges was assessed. According to the results, the effect of higher modes in modal pushover analysis increases by increasing of the bridge opening length and the numbers of middle piers (bases).

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1. Introduction

Introduction

The word of "pushover analysis" is expressive of a type of advanced alterations of classic disintegration analysis. In other words, it refers to a kind of analytical procedure which contains Newton-Raphson method (Newton`s iteration) - incremental method for statics equilibrium equations. In fact, it is applied for obtaining of the response of dependant structures of lateral load pattern with equable augmentation (Kunnath , 2004). Although the application of this method has been extremely miscellaneous in investigation of construction frames, nonlinear static analysis of bridge structure needs more details. Since the bridges are different structures in comparison with constructions and buildings, the observations and conclusions that are relevant to the constructions, cannot be attributed to the bridges. According to the main differences between structural behavior of bridges and constructions and studying of all pushover methods which have been considered in recent years , it has been tried- in this collection- to examines the modal pushover method as an appropriate way for seismic evaluation of bridges with different openings completely.

The Parameters of Nonlinear Static Analysis for Evaluation of Bridges

1. Idealization of Pushover Curve

The procedure of pushover curve idealization has been shown in figure 1. Here, it is performed by using of complete pushover curve- i.e. the analysis until the failure time of structure that is expressed by decreasing of strength until the maximum of 20%- and the attraction law of equal energy – i.e. the equal areas under the main and two-line curves-.

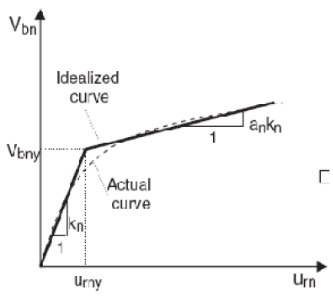

Figure 1 : Idealization of pushover curve

2. Target Displacement

There are some methods for determination of target displacement according to each delineated pushover curve. We can refer to capacity spectrum method (CSM) and displacement coefficient method, among the usual method. Capacity spectrum method

(CSM) needs numerous repetition but displacement coefficient method doesn't need any reiteration (Kunnath , 2004).

3. Formation of Capacity Curve

In this stage, the idealized pushover curve changes to capacity diagram. As it has been shown in figure 2, shear force of pier (base) and the resultant transformation, in each pushover curve, will be changed to accelerative spectrum S_a and displacement coefficient S_d.

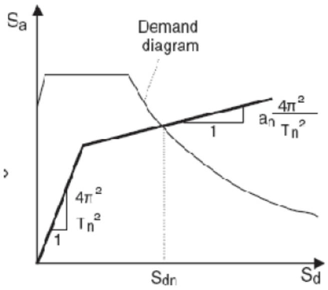

Figure 2: Changing of pushover curve to capacity curve

These operations can be performed by using of relation 1 and 2:

$$S_a = \frac{V_{bn}}{M_n^*} \ (1)$$

$$S_d = \frac{U_m}{\Gamma_n \phi_{rn}} \ (2)$$

In which ϕ_{rn} is the amount of ϕ_n in control M_n^* point, is the effective modal mass, and $.\Gamma_n$is the generalized mass.

4. Determination of Target Point

The natural choice of control points in bridge is in gravity centre of bridge deck (Kunnath , 2004) or above the nearest point to its pier , if displacements of these two points are equal ; For example, for articular or integrated joints of pile to bridge deck but not for slippery or transformable ones. The other suggestion (Kunnath , 2004; Kappos et al. , 2004) is a point of deck whose displacement is maximum. In this state, a primary analysis of structure is necessary for each mode in elastic mode that defines the most critical point and it can be applied for delineation of pushover curve.

5. Seismic Load Pattern

The first pattern is integrated and it acts based on the lateral forces that fit with the total mass of each knot for bridges. It is in the form of relation 3:

$$F_i = m_i g \ (3)$$

F_i is lateral force in knot, n is the numbers of knots, m_i is mass of knot and g is earth acceleration. The second pattern is the modal pattern that can be described in the form of relation 4:

$$F_i = (m_i\varphi_i / \sum_{i=1}^{N} m_i\varphi_i)V \ (4)$$

φ_i is the main mode domain in knot and V is base shear . This pattern is used when more than 75% of total mass participate in main mode of the relevant direction. The third load pattern, which is called spectral pattern, is used when the effect s of high mode are important. This pattern is based on the combinationally modal force and it can be described by using of the methods of "the root of sum of squares" or "complete quadratic combination" (CQC).

$$F_i = (m_i\delta_i / \sum_{i=1}^{N} m_i\delta_i)V \ (5)$$

δ_i isplacement of the knot i of. In addition, it is resultant of incremental response spectrum analysis (IRSA).

Modal Pushover Analysis Method:

1. Modal pushover methods with positive load pattern (Moghadam 2000; Chopra & Goel 2002 ; 2004 ; Chopra & et al. 2004 ; Goel &Chopra 2005; Shakeri & et al 2007 ; Kunnath 2004)
2. Modal pushover methods with adaptive load pattern (Gupta & Kunnath , 2000; Albani & et al. 2002 ; Aydin oğlu 2003 ; 2004 ; Antoniou & Pinho 2004)

Modal pushover method will be represented in the consequent stages (briefly) in the following way (Pinho & et al , 2006) :

- Accomplishment of special analysis for determination of natural periods of vibration and modal shapes of structure
- Accomplishment of pushover analysis for each mode with load distribution pattern and determination of base shear curve of the relevant displacement
- The idealization of pushover curve in the form of two-line curve (figure 3)
- The calculation of maximum transfiguration in n^{th} inelastic mode
- The maximum estimation of control point displacement, relevant to n^{th} inelastic mode

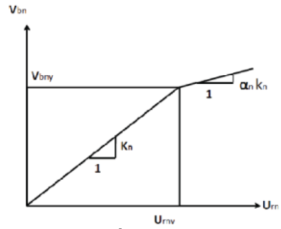

Figure 3 : The idealization of pushover curve in the form of the two- line curve

The Materials and the Methods

Modeling

In this study, three examples of concrete bridges with different structural system have been used for assessment and inspection of the effect of the numbers of openings in the height of bridge piers (bases) on effective mode numbers in nonlinear static analysis. The analysis was performed by using of structural analysis software of "SAP2000 ADVANCED 12.0.1" . The deck bridge was considered as integrated. In addition, the latitudinal displacement of piers (bases) in latitudinal direction was avoided. In nonlinear static analysis, we need capacity curve of structure for finding the characteristic of single-degree- of- freedom system , for each vibration mode . Therefore, it is necessary to perform nonlinear static analysis until displacement of assumed target and the structure is pushed till that target displacement by using of the forces which is proper to the shape of each mode. Then , the program pushes the structure stage to stage automatically and in each stage of shear , the pier (base) is found by displacement of target point . Finally, the capacity curve of structure is formed. Also, the software of Nonlin has been used for obtaining of target displacement of single- degree-freedom system. Control point in bridges has been considered as a point in deck whose displacement is maximum. The method of Square root of sum of squares has been used for obtaining of a proper combination of modal responses .

The Structural Characteristics of Bridge

The bridge no. 1 contains three openings whose middle opening is 60 meter and its lateral opening is 35 meter. Therefore, the total length of bridge is 130 meter and its total width is 6.5 meter. In addition, expansion joint has been considered in its two ends. The numbers of openings for bridges no. 2 and no. 3 are 4 and 5 respectively. Also, in regarding to bridge no. 2, we can say that the length of middle openings are 75 meter and the length of lateral opening is 45 meter. Also, in concerning to bridge no. 3, we can say that the length of middle openings are 75 meter and the length of lateral opening lateral opening are 40 meter. The vertical profile of these three bridges have been represented in the figures 4 ,5 and 6.

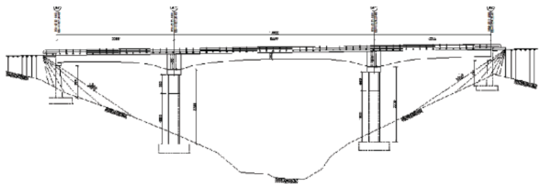

Figure 4. Vertical profile of bridge no. 1

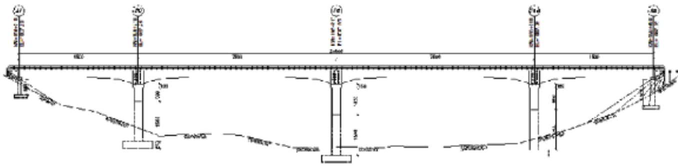

Figure 5. Vertical profile of bridge no. 2

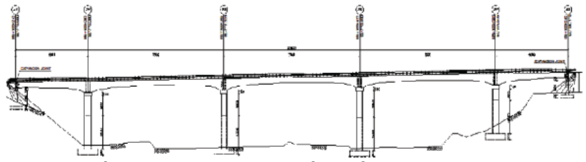

figure 6 . Vertical profile of bridge no. 3

Middle Piers (Bases)
Middle piers (bases) have been considered in the form of hollow box with dimensions of 3.6 m *3.8 m and the walls thickness of 40 cm for these bridges. The height of middle piers (bases) have been chosen 22 m for bridge no. 1 and 24.5 m , 26.5 m and 21.5 m for bridge no. 3 and 22 m, 28.5 m, 26.5 m and 25.5 m for bridge no. 3.

Bridge Abutments
Open bridge abutments have been used here. The height of these abutments is 9.5 m and 7.5 m for bridge no. 1 and 5.5 m and 9.5 m for bridge no. 2 and 6.5 m and 7 m for bridge no. 3. The mentioned bridge abutment have been represented in figure 7.

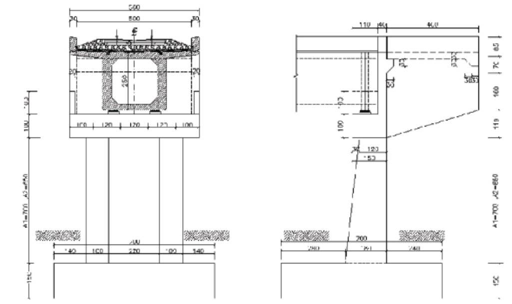

Figure 7 : The face of the abutments of the studied bridge

Bridge Deck
The discussed bridge decks was formed of reinforced concrete boxes, with external dimensions of 2.8 m*2.5 m. the thickness of concrete slab over deck is 25 cm. the concrete slab was performed in integrated way for removing of expansion joint which had been placed over middle piers(bases) and its width was considered 5.6 m in vertical section. The horizontal section of the studied deck bridges has been shown in figure 8.

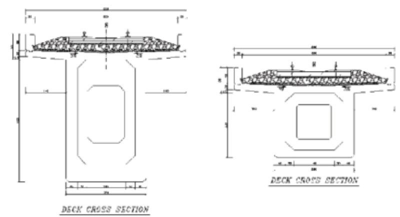

Figure 8 : Horizontal section of the studied deck bridges

Introduction of the Applied Earthquakes
Three accelerographs (seismometers) of Kobe , Lomaprieta and San Fernando have been used as the time history of sever motion of ground for performing of nonlinear dynamic analysis of bridges , nonlinear dynamic analysis of single- degree-freedom system and finding of target point displacement in modal pushover analysis. These accelerographs (seismometers) have become commensurate and coequal with the spectrum of bylaw project of 2800 for type II soil. Also, the scaled PGA , based on g, which belongs to the discussed accelerographs (seismometers)have been mentioned.

Table 1: The commensurate accelerographs (seismometers) with the spectrum of bylaw project of 2800 of Iran for type II soil

code	accelerographs (seismometers)	the real PGA , based on g	the scaled PGA , based on g
1	Kobe	0.821	0.433
2	Lomaprieta	0.450	0.851
3	San Fernando	0.366	1.218

Foresight of Deck Displacement in the Place of Middle Bearings for Bridge No. 1
Seismic evaluation was performed on bridge no. 1. The modal characteristics as well as the maximum of deck displacement in the place of middle piers (bases) have been brought according to single-mode pushover analysis and dynamic analysis of time history in tables 2, 3 and figure 9.

Table 2: Modal characteristics, based on the first two latitudinal modes for bridge no. 1

Pier (base)		First pier	Second pier	Third pier	Fourth pier
The intensive mass in each knot	ton	247	1039	1039	247
Modal shape of first mode	$\phi 1$	0	1	1	0
Modal shape of second mode	$\phi 2$	0	1	-1	0
Modal force of first mode	S* 1	0	1039	1039	0
Modal force of second mode	S* 2	0	1	-1	0

Table 3: The maximum of latitudinal deck displacemnt in the place of the piers (bases) , according to the accelerographs (seismometers) in bridge no. 1

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier
	Single-mode method	Kobe	0	17.9	17.9	0
		Loma prieta	0	8.4	8.4	0
		San Fernando	0	7	7	0
	Dynamic method of time history	Kobe	0	17.46	17.46	0
		Loma prieta	0	9.02	9.02	0
		San Fernando	0	6.905	6.905	0

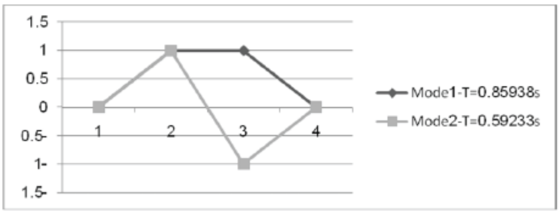

Figure 9: The first two latitudinal modes and alternation time of each mode for bridge no. 1

Foresight of Deck Displacement in the Place of Middle Bearings for Bridge No. 2

The modal characteristics and the maximum of deck displacement in the place of middle piers (bases) - based on single-mode , two-modes and three-modes pushover analysis- as well as dynamic analysis of time history - based on different accelerographs (seismometers) and the comparison of the modal pushover analysis results with dynamic analysis of time history have been brought for this bridge, in tables 4 , 5,6,7 and figure 10.

Table 4 : The modal characteristics , based on the first three latitudinal modes for bridge no. 2

Pier (base)		First pier	Second pier	Third pier	Fourth pier	Fifth pier
The intensive mass in each knot	ton	335	1423	1669	1423	333
Modal shape of first mode	$\phi 1$	0	0.2	1	0.4092	0
Modal shape of second mode	$\phi 2$	0	-0.2061	-0.2988	1	0
Modal shape of third mode	$\phi 3$	0	1	-0.2030	0.1285	0
Modal force of first mode	S^*_1	0	285.3058	1669	582.31	0
Modal force of second mode	S^*_2	0	-293.351	-498.764	1423	0
Modal force of third mode	S^*_3	0	1423	-338.907	182.884	0

Table 5 : The comparison of the maximum responses , according to Kobe accelerographs (seismometers) for bridge no. 2

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier
	Modal method	Two-modes	0	3.7	25.3	9	0
		Three-modes	0	3.92.17	25.350	11.5256	0
	Dynamic method of time history	Kobe	0	16.4736	25.3702	11.553	0
			0	14.8	18.4	16.8	0
		Single-mode	0	75	37.5	46.4285	0
	Errors in percentage	Two-modes	0	73.5018	37.7746	31.3951	0
		Three-modes		11.3082	37.8818	31.23	0

Table 6 : The comparison of the maximum responses , according to Lomaprieta accelerographs (seismometers) for bridge no. 2

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier
	Modal method	Two-modes	0	2	11.2	3.7	0
		Three-modes	0	2.1189	11.2361	4.827	0
	Dynamic method of time history	Lomaprieta	0	7.7935	11.28	4.877	0
			0	8.4	10	7.1	0
		Single-mode	0	76.19	12	47.8873	0
	Errors in percentage	Two-modes	0	74.7742	12.3610	32.013	0
		Three-modes		7.2192	12.8051	21.3028	0

Table 7 : The comparison of the maximum responses , according to San Fernando accelerographs (seismometers) for bridge no. 2

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier
	Modal method	Two-modes	0	1.7	9.4	3.3	0
		Three-modes	0	1.8027	9.4429	3.2638	0
	Dynamic method of time history	San Fernando	0	6.2649	9.4957	4.3058	0
			0	6.8	7	6.8	0
		Single-mode	0	75	34.2857	51.47	0
	Errors in percentage	Two-modes	0	73.4885	34.8998	37.297	0
		Three-modes		7.8679	35.6541	36.6792	0

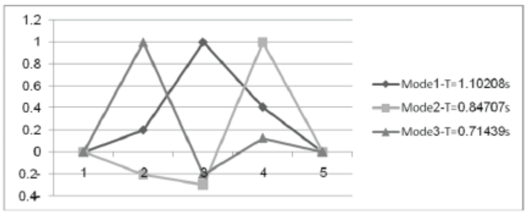

Figure 10 : The first three latitudinal modes and alternation time of each mode for bridge no. 2

Foresight of Deck Displacement in the Place of Middle Bearings for Bridge No. 3

The modal characteristics and the maximum of deck displacement in the place of middle piers (bases) - based on single-mode , two-modes , three-modes and four-modes pushover analysis- plus dynamic analysis of time history and the comparison of the modal pushover analysis results with dynamic analysis of time history have been brought for this bridge, in tables 8 , 9,10 and figure 11.

Table 8 : The modal characteristics, according to four latitudinal modes for bridge no. 3

Pier (base)		First pier	Second pier	Third pier	Fourth pier	Fifth pier	Sixth pier
The intensive mass in each knot	ton	331	14.26	1642	1644	1372	277
Modal shape of first mode	$\phi 1$	0	0.1861	0.6378	1	0.1740	0
Modal shape of second mode	$\phi 2$	0	0.7661	1	-0.7217	-0.20136	0
Modal shape of third mode	$\phi 3$	0	1	-0.49809	0.1502	0.10904	0
Modal shape of fourth mode	$\phi 4$		-0.0368	0.0585	-0.1648	1	0
Modal force of first mode	S^*_1	0	265.497	1047.335	1644	238.8391	0
Modal force of second mode	S^*_2	0	1092.574	1642	-1186.47	-276.266	0
Modal force of third mode	S^*_3	0	1426	-817.86	247	149.6125	0
Modal force of fourth mode	S^*_4	0	-52.5624	96.1144	-270.997	1372	0

Table 9 : The comparison of the maximum responses , according to Kobe accelerographs (seismometers) for bridge no. 3

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier	sixth pier
	Modal method	Two- modes	0	2.8	17.9	24.6	4.2	0
		Three-modes	0	5.5569	19.3693	24.7072	4.2579	0
		Four-modes	0	11.5277	19.4273	24.7145	4.2871	0
	Dynamic method of time history	Kobe	0	11.5282	19.4283	24.7487	14.2597	0
			0	17.2	15.9	20.8	14.6	0
		Single-mode	0	83.72	12.5786	18.2692	71.2328	0
	Errors in percentage	Two- modes	0	67.6919	21.8195	18.7846	70.836	0
		Three-modes	0	32.9779	22.1842	78.8197	70.6356	0
		Four-modes	0	32.9745	22.19	18.9841	2.33	0

Table 10 : The comparison of the maximum responses , according to Lomaprieta accelerographs (seismometers) for bridge no. 3

Latitudinal Displacement (cm)	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier	sixth pier
	Modal method	Two- modes	0	2.4	11.4	11.9	2.3	0
		Three-modes	0	3.3941	11.7512	12.032	2.3769	0
		Four-modes	0	5.2459	11.8175	12.03647	2.3958	0
	Dynamic method of time history	Lomaprieta	0	5.2469	11.8189	12.04046	7.8733	0
			0	6.9	9.8	9.9	8.7	0
		Single-mode	0	65.2173	16.3265	20.202	73.56322	0
	Errors in percentage	Two- modes	0	50.8099	19.91	21.5353	72.6784	0
		Three-modes	0	23.9717	20.5867	21.5797	72.4617	0
		Four-modes	0	19.05886	20.601	21.6202	9.5014	0

Table 11 : The comparison of the maximum responses , according to San Fernando accelerographs (seismometers) for bridge no. 3

	Piers (bases)		First pier	second pier	Third pier	fourth pier	Fifth pier	sixth pier
Latitudinal Displacement (cm)	Modal method	Two- modes	0	1.1	5	9.4	1.3	0
		Three-modes	0	2.3706	5.6824	9.59	1.3928	0
		Four-modes	0	4.4788	5.877	9.6031	1.4247	
	Dynamic method of time history	San Fernando	0	4.4799	5.8804	9.6114	6.1668	0
			0	6.9	6.6	8.6	6.8	0
		Single-mode	0	84.0579	24.2424	9.3023	80.8823	0
	Errors in percentage	Two- modes	0	65.6427	13.9025	11.5116	79.51708	0
		Three-modes	0	35.8929	10.9534	11.6639	79.04734	0
		Four-modes	0	35.07311	10.9018	11.7604	9.31107	0

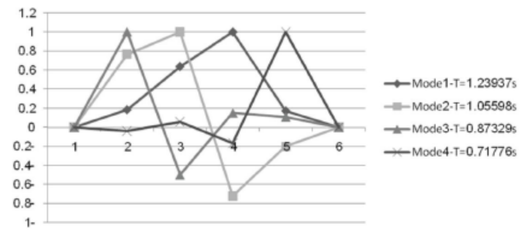

Figure 11 : The first four latitudinal modes and alternation time of each mode for bridge no. 3

Result and Discussion

The accuracy of this method was considered for studying of modal pushover analysis application in regarding to the bridges with different openings. So, the following results were obtained based on the domain of assumptions and the limits of the accomplished studies of this paper:

- If the collaboration of the higher modes is insignificant and trivial, single-mode pushover analysis (regular bridges, with few numbers of openings) will be too accurate , In other case, this method cannot present proper estimation at all.
- In high bridges with long opening, the effect of considering of higher modes is very perceptible by increasing of the collaboration coefficient of other modes, except the first mode.
- The effect of modal pushover analysis is very effective in improving of the accuracy of the estimations of deck displacement in base (pier) place in earthquake of the project . In addition, the response values are in favorable level.
- The number of the mentioned modes in modal pushover analysis must increase based on the escalating of the numbers of bridge openings. The purpose is reaching to proper accuracy.
- It is suggested to consider the minimum numbers of the selected modes in proportion to the size of the numbers of middle piers (bases).

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