Accuracy of Nonlinear Static Analysis in Seismic Behavior of Irregular Steel Framed Bridges with Inclined Legs

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SUMMARY:
Nowadays using nonlinear static analysis (NSA) in term of evaluating bridge performances has become prevalent. This kind of analysis acquires acceptable responses in seismic rehabilitation studies of regular framed bridges when sequences of plastic hinge formations and seismic response values are compared with more accurate analyses. While, for irregular framed bridges with inclined legs, analysis accuracy is not predictable. Therefore, necessity of determining the accuracy of NSA in the range of bents inclined angles are the matter of concern in irregular bridges by implementing more reliable analyses. In this study, an existing framed steel bridge under operation is selected as research case study. Four models with diverse bents inclination angles between 45 and 75 degree were selected for nonlinear analyses of steel bridge. First, bridge models were designed based on current codes. Then, nonlinear static analyses (NSA) were carried out as well as incremental dynamic analysis (IDA). Finally by comparing between nonlinear static and dynamic analyses, it was concluded that by decreasing bents inclination angles, NSA accuracy drops significantly from 40% to 75% in both hazard levels of 10% in 50 years and 2% in 50 years.

Keywords: Irregular Framed Bridge, Nonlinear Static Analysis, Incremental Dynamic Analysis, Steel Bridge

1. INTRODUCTION

According to previous researches, the performances of bridges designed by current codes were not acceptable in several earthquakes. Irregularity in configuration has been detected as one of the main reasons among all caused inadequate seismic responses. Most of the modern codes for the seismic design of bridges involve several elastic and inelastic methods for their study. The main supposition in simplified methods is that the ductility demand is uniformly distributed over the entire structure and that the first vibration mode governs the entire response. Therefore, these methods can be used for the analysis of the so-called "regular" structures, in which the domination of the first mode is an acceptable assumption.

Nonlinear static analysis (NSA) has been widely used for evaluating the seismic behavior of bridge structures. It can be used as a method for determining the capacity of bridge structures neglecting the higher mode effects. Many researchers reported the successful use of nonlinear static analysis on building structures especially for low to medium-rise, which is typically dominated by the first mode. However, when the structure becomes more complicated in configurations such as mass and stiffness distribution, the participation of higher modes may increase. These higher mode effects may contribute to the structure's response significantly. Although, the structural behavior of bridges is different from that of other structures (i.e. building structures), most regularly designed bridges have a characteristic mostly similar to their first mode of vibration. So, nonlinear static analysis can obtain appropriate responses for these types of structures. But, as bridge designs become more complex, it is less likely they can be expected to respond in a single degree of freedom fashion. A higher mode response contributes significantly as irregularity of the bridge is increased. As a result, necessity of implementing more accurate studies especially when irregularities determine the fundamental structural behaviors must be considered.

Recent advances in nonlinear dynamic modeling and analysis of structures has made it possible to conduct more reliable prediction of seismic force and displacement demand. A combination of these advances in the modeling strategy with the dispersion assessment framework proposed by Cornell and Vamvatsikos has created a methodology known as Incremental Dynamic Analysis (IDA). In this paper, the median outcome of the IDA curves has been compared with the result of NSA.

2. RIGID-FRAMED BRIDGES

Rigid frame bridges were popularized in the 1920s and 1930s as river crossings and highway grade separations. Also, short and medium span rigid frame bridges are still widely used today for rail/roadway grades. German engineers were the first to pioneer the rigid frame or portal frame bridge, but it gained popularity in the U.S. due to Westchester County, NY engineer Arthur G. The strength of a rigid frame bridge originated from the rigid connection of the vertical or inclined bents with the horizontal deck slab, resulting in a shallow mid-span section. This bridge type had a unique ability to redistribute loads throughout the structure and its considerable strength and rigidity provided an additional safety in the structure.

3. NUMERICAL VALIDATION ANALYSIS

To validate the composite model, specimen CTB1 was selected from six continuous steel-concrete composite beams tested by Ansourian. This specimen was used to demonstrate the ability of the modeling validation with nonlinear behavior composite beams. The continuous beam CTB1 had two unequal spans and was subjected to a central concentrated load in the short span as shown in Fig. 1.1. The dimensions of the cross section are also shown in Fig. 1.2.

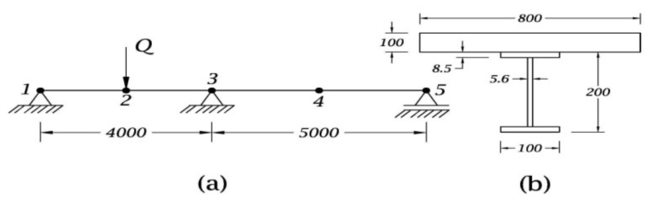

Figure 1. Continuous steel–concrete composite beams tested by Ansourian
a) Continuous beam CTB1, b) Cross-section

The OpenSees model was created with two dimensional assemblages of four nonlinear elements with a fiber section connected at the nodes using the “dispBeamColumn” element command. Opensees “Concrete01” uniaxial material was carried out for unconfined concrete. The bi-linear steel material of “Steel01” was used to model the reinforcements and steel girders.
Variations of the applied load with the vertical deflections of the middle spans are compared with the test results of Ansourian in Fig. 2 for the beam CTB1. As a conclusion, results almost coincide with the test results in both cases.

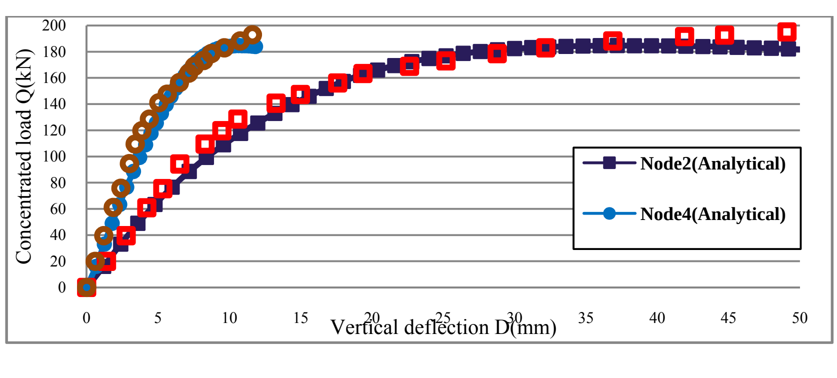

Figure 2. Comparison with test results for continuous steel–concrete composite beam CTB1 of Ansourian.

4. STUDY OF STEEL FRAMED BRIDGES WITH INCLINED BENTS

4.1. Geometric characteristics

The steel bridge studied here is a three-span framed bridge with inclined legs which is 75 m long with a total width of 18 m. The distance between the upper and lower end of each leg is 6.2 m in the vertical direction and 4.35 m in the horizontal direction. The deck is composed of steel girders with a concrete slab. The bents were constructed as six steel I-section piers. The section area increases linearly from the bottom to the top. Bents-foundations connections and Bents-deck connections in the longitudinal direction are pined and fixed, respectively. The bent is connected to abutments through a simple support. Dimensions of the bridge and deck section are presented in Fig. 3 and Fig. 4.

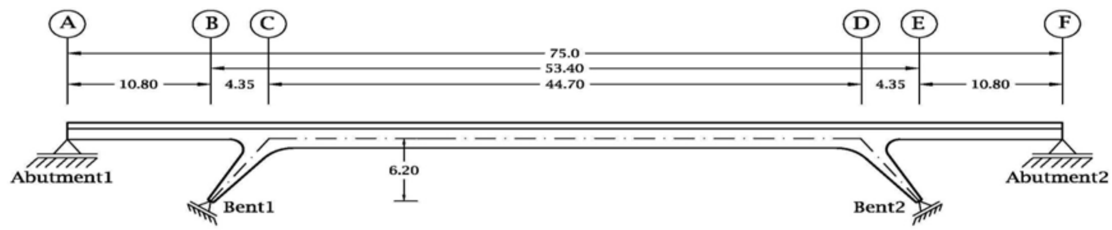

Figure 3. Overview of steel bridge models.

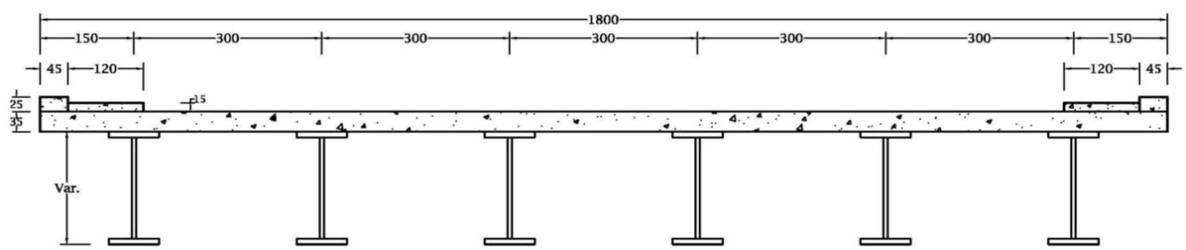

Figure 4. Composite deck section.

Four models with overall characteristics similar to the introduced existing steel bridge were developed for nonlinear analyses. In these models, abutment characteristics are the same as the existing bridge. To develop more models and advance the goals of this study on inclined-bent bridges, the horizontal space between the upper and lower ends of the bents was altered in the foundation level to adjust the inclination angles of the bents in the range of 45° to 75°. The length of the middle and end spans in all the models was kept fixed. Primary data on the models are provided in Table 1.

Table 1. Details of inclined bents in steel bridges.

Bridge Model	Space between the Upper and Lower Ends of Bents		Bent Inclined Angle	Bent Length
	Horizontal Direction	Vertical Direction		
	(m)	(m)		(m)
Bridge1 (α=45)	6.2	6.2	45	8.77
Bridge2 (α=55)	4.35	6.2	55	7.57
Bridge3 (α=65)	2.89	6.2	65	6.84
Bridge4 (α=75)	1.66	6.2	75	6.42

4.2. Designing steel bridges

SAP2000 was used to design the steel bridge models. Tables 2 and 3 present the material characteristics.

Table 2. Concrete material characteristics in the steel bridges.

Materials	Modulus of Elasticity	Weight per Unit Volume	Compressive Strength
	E_c (MPa)	W_c (kN/m^3)	f_c (MPa)
Deck Concrete	2.1×10^4	0	30

Table 3. Steel material characteristics in the steel bridges.

Materials	Modulus of Elasticity	Weight per Unit Volume	Yield Strength	Ultimate Strength
	E_s (MPa)	W_s (kN/m^3)	F_y (MPa)	F_u (MPa)
Deck Reinforcement	2.1×10^5	78.5	400	600
Deck Steel	2.1×10^5	0	360	520
Bent Steel	2.1×10^5	78.5	360	520

Response spectrum dynamic analysis was performed for all four bridge models. Therefore, a design spectrum acceleration diagram appropriate for the Tehran region must be carried out in bridge analyses. To include the goals of the study in the later steps (nonlinear analysis), the spectrum used in the study must be a spectrum with a uniform hazard of 10% over 50 years. To include uncertainties in land movements, probabilistic seismic hazard assessment was used (PSHA). Uniform hazard spectra for three probable spectral accelerations were drawn (Fig. 5) and values corresponding to the first mode periods of the structures were obtained. Moreover, AASHTO [2007] load combinations were used for spectral analysis.

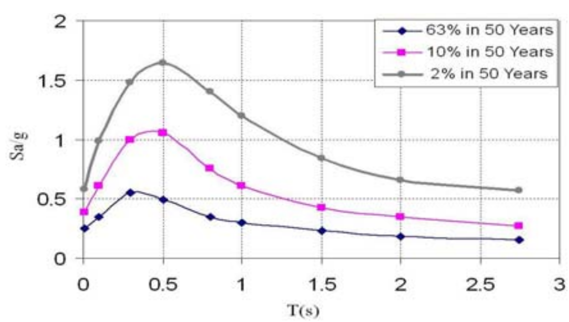

Figure 5. Tehran uniform hazard spectrums (damping 5%).

Design of the four steel bridges was controlled based on AASHTO [2007]. In designing bents and decks, an attempt was made to keep stress ratios within a limited range in order to make more accurate comparison possible. Table 4 shows the detailed designs of the four bridge models and the percentage of axial forces on the bents.

Table 4. Details of the steel models design (SAP2000).

Bridge Model	Bridge Part	Section Part	d (m)	b (m)	t_w (m)	t_f (m)	Pu (kN)	$Pu/(f_y*A_g)$ (%)	AISD ratio
Bridge1 (a=45)	Column	Upper Level	2	1.1	0.05	0.05			0.985 , 0.922
		Lower Level	1.1	1.1	0.05	0.05	3632	6.31	
	Deck	Inner Span	1.6	0.4	0.05	0.05			0.987
		Outer Span	0.9	0.4	0.05	0.05			0.505

Bridge2 (a=55)	Column	Upper Level	2	1.1	0.05	0.05			0.934 , 0.894
		Lower Level	1.1	1.1	0.05	0.05	3220	5.59	
	Deck	Inner Span	1.6	0.4	0.05	0.05			0.86
		Outer Span	0.9	0.4	0.05	0.05			0.512
Bridge3 (a=65)	Column	Upper Level	2	1.1	0.05	0.05			0.911 , 0.869
		Lower Level	1.1	1.1	0.05	0.05	2890	5.02	
	Deck	Inner Span	1.6	0.4	0.05	0.05			0.836
		Outer Span	0.9	0.4	0.05	0.05			0.515
Bridge4 (a=75)	Column	Upper Level	2	1.1	0.05	0.05			0.904 , 0.861
		Lower Level	1.1	1.1	0.05	0.05	2640	4.58	
	Deck	Inner Span	1.6	0.4	0.05	0.05			0.82
		Outer Span	0.9	0.4	0.05	0.05			0.516

4.3. Nonlinear static analysis (NSA) of the steel bridges

OpenSees software was used to analyze the steel bridge models. Since the total structure is comprised of six parallel identical frames and analyses are conducted in two-dimensional conditions, one of these six frames was selected for modeling. In order to prevent any changes in modal characteristics of the structure, the total load applied to this frame was set at one sixth of the total load exerted on the structure. This reduces the time required for analysis without making any changes in the outputs. Fig. 6 shows the deck section for the frame of interest used in modeling and nonlinear analysis.

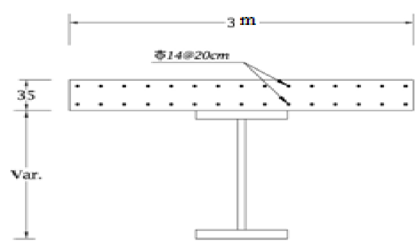

Figure 6. Deck section for the frame of interest used in modeling and nonlinear analysis.

To assign material characteristics, the Modified Kent & Park criterion was carried out to model confined behavior of the concrete. For the unconfined concrete, the strain corresponding to maximum compressive strength and ultimate strength were 0.002 and 0.006, respectively. Steel materials were modeled according to Giuffre-Menegetto-Pinto. In this model, the strain-hardening ratio was set at 0.007. Nonlinear element sections were defined in the form of fiber sections with “Section Fiber” command. Girder section and deck concrete were modeled with 60 fibers and 100 fibers, respectively. Ten elements were used to define the middle span while five elements were used to define each of the end spans. Six elements were employed at each column. The command “dispBeamColumn” was applied to model nonlinear behavior as distributed plasticity over bent elements. Five integrating points were defined in these elements. In bent analysis, co-rotational effects were carried out in order to allow the effects of axial and lateral displacements in structural equilibrium. In analyzing gravity loading, one sixth of the total dead load on the deck (40 kN/m) as well as one sixth of the live load was applied uniformly on the deck. Table 5 shows the resulting periods obtained in the modal analysis of the four steel bridges in the two-dimensional scenario.

Table 5. Modal analysis results of 3-D concrete models (OpenSees).

Mode Number	Period (Sec)			
	Bridge1 (α=45)	Bridge2 (α=55)	Bridge3 (α=65)	Bridge4 (α=75)
1	0.5969	0.6165	0.6162	0.6087

2	0.0598	0.0597	0.0599	0.0609
3	0.029	0.0291	0.0294	0.2099
4	0.0205	0.0198	0.0198	0.0199
5	0.0198	0.0177	0.0163	0.0164

The Capacity-Demand-Diagram method was applied to calculate target displacement. Bilinear approximation was employed to determine yield displacement according to ATC-40. The bilinear capacity spectrum in ADRS format was intersected with inelastic design spectra to obtain target displacements for the four models, as shown in Table 6 and 7. Fig. 7 demonstrates the above mentioned diagram of Bridge model 3 (α=65) as a sample of four models in the two aforementioned return periods.

Table 6. Target displacement of the steel models (return periods of 475 years).

Bridge Model	Yield Displacement	Ductility Factor	Reduction Factor	Target Displacement
	Dy(m)	μ	Ry	Dt(m)
Bridge1 (α=45)	0.0398	1.5100	1.7253	0.0615
Bridge2 (α=55)	0.0608	1.2200	1.3342	0.0759
Bridge3 (α=65)	0.0776	1.0600	1.1194	0.0860
Bridge4 (α=75)	0.0869	1.0000	1.0170	0.0874

Table 7. Target displacement of the steel models (return periods of 2475 years).

Bridge Model	Yield Displacement	Ductility Factor	Reduction Factor	Target Displacement
	Dy(m)	μ	Ry	Dt(m)
Bridge1 (α=45)	0.0398	2.1600	2.5671	0.0871
Bridge2 (α=55)	0.0608	1.7100	1.9881	0.1059
Bridge3 (α=65)	0.0776	1.4800	1.6855	0.1164
Bridge4 (α=75)	0.0869	1.3700	1.5384	0.1197

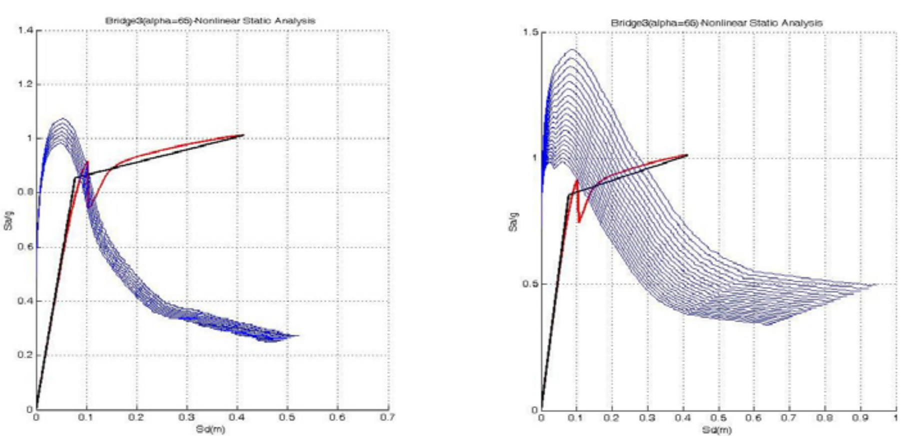

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Figure 7. Intersection of demand diagrams and bilinear capacity diagram for Bridge 3 (α=65): a) Return periods of 475 years, b) Return periods of 2475 years.

4-4 Incremental dynamic analysis (IDA) of the steel bridges

Just like nonlinear static analysis, where the structure is pushed step by step in a static manner, as time-history analysis is incremented, several time-history analyses provide scales of seismic loads. Cornell and

Vamvatsikos first described the concept of this method which was then applied by many researchers. In this step, the same model was used as those developed for NSA through OpenSees. Accelerograms (motions) were selected based on 22 pairs of record (44 components) for far fields as described in FEMA-P695. In this study, spectral acceleration in the first modal period (Sa(T1, 5%)) was assumed to present all intrinsic uncertainties in the distribution of demands in different accelerograms. In addition, given the dominance of the first mode over the bridges behavior, this index is a good bridge representative of intensity measure (IM). In order to compare nonlinear static and dynamic analyses, and since target displacements were calculated in static analysis, damage measure (DM) in incremental dynamic analysis was set at a maximum drift of the bridges.

The first step corresponds to the spectral acceleration of 0.005g which represents the elastic range. Acceleration for the initial step was 0.3g and the increments for the next steps were 0.25g. The Hunt & Fill method was used to scale accelerograms. Structural limit-states considered here are when the slope of diagram reaches 20% of the elastic slope, or drift reaches 10%, and when IM reaches 6g, and finally when dynamic (numerical) instability is reached. To improve convergence, a particular convergence algorithm was implemented. The algorithm was developed to examine different options before convergence criterion is violated. Statistical processing of IDA curves is required in order to achieve a better judgment on the overall analysis and to better compare the two types of nonlinear analysis. Cornell and Vamvatsikos proposed three percentiles at 16%, 50% (median), and 84%. In this approach, the interpolation of demands at different levels is initially used to determine IMs and then the three percentiles are identified. Fig. 8 and Fig. 9 show the IDA curves as well as post-processed curves of sample Bridge 3 (α=65).

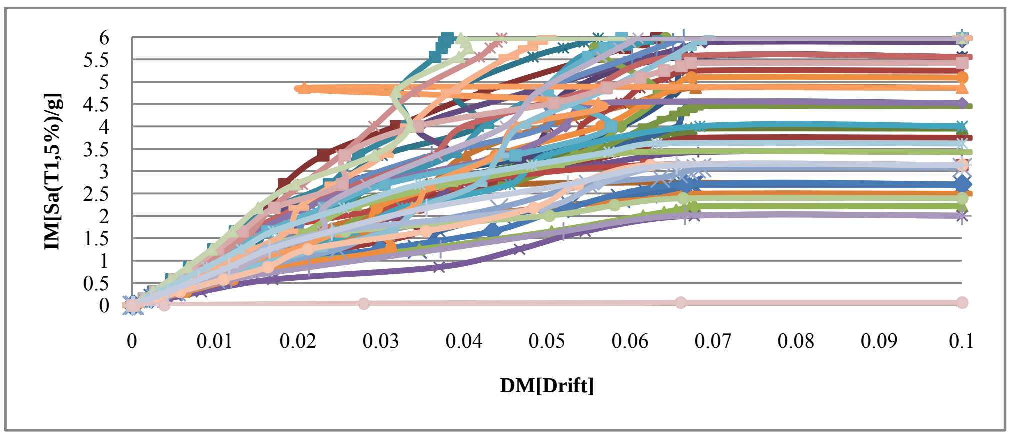

Figure 8. 44-record IDA curves for Bridge 3 (α=65).

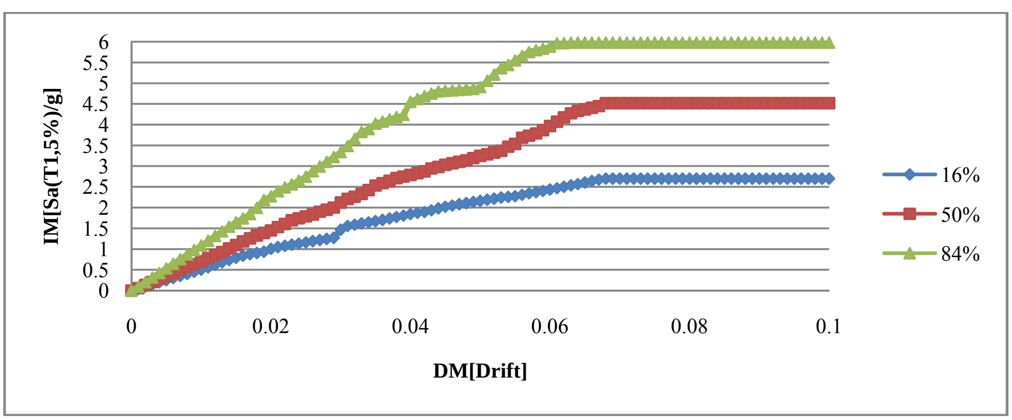

Figure 9. Post-processed IDA curves for Bridge 3 (α=65).

Incremental dynamic analysis was performed to obtain the median (the 50%-percentile) and determine the seismic response of the bridges studied here. To obtain the responses, data on the probability of the occurrence of earthquakes are required in terms of spectral accelerations for structures with different periods. To do so, spectra with a uniform hazard for different probabilities of occurrence should be obtained using the concepts of probability seismic hazard assessment (PSHA). In this way, probability of occurrence can be plotted versus spectral acceleration (Sa-λ). Given a certain probability of occurrence and the main period of structure, one can use this curve to find spectral acceleration. Then, the spectral acceleration may be used on IDA curves to obtain seismic response (drift) based on the return period and probability of occurrence. Fig. 10 shows the calculation procedure. Table 8 and Table 9 provide details on the calculation of drift for the bridge models at the return period of 475 years (probability of occurrence: 10% in 50 years).

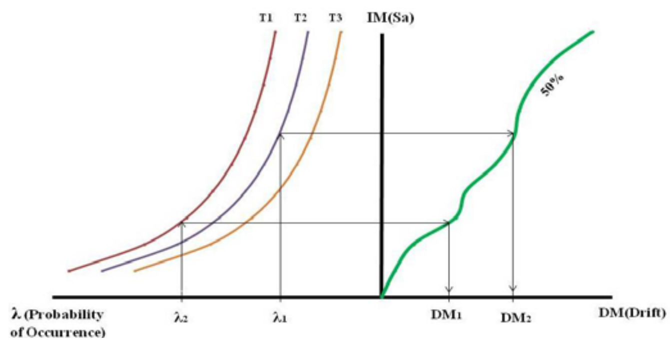

Figure 10. DM Calculation with post-processed IDA curves by considering the probability of occurrence.

Table 8. Displacement of steel models in IDA (return periods of 475 years).

Bridge Model	Probability of Occurrence of 10% in 50 Years			
	T1 (sec)	Sa (T1, 5%)/g	Drift	Displacement (m)
Bridge1 (α=45)	0.5969	0.9772	0.0165	0.1980
Bridge2 (α=55)	0.6165	0.9569	0.0135	0.1620
Bridge3 (α=65)	0.6162	0.9569	0.0135	0.1620
Bridge4 (α=75)	0.6087	0.9670	0.0125	0.1500

Table 9. Displacement of steel models in IDA (return periods of 2475 years).

Bridge Model	Probability of Occurrence of 10% in 50 Years			
	T1 (sec)	Sa (T1, 5%)/g	Drift	Displacement (m)
Bridge1 (α=45)	0.5969	1.5963	0.0275	0.3300
Bridge2 (α=55)	0.6165	1.58055	0.0225	0.2700
Bridge3 (α=65)	0.6162	1.58055	0.0215	0.2580
Bridge4 (α=75)	0.6087	1.5864	0.0195	0.2340

4.5. A comparison of nonlinear analysis for the steel bridges

As seen before, the four steel bridge models were analyzed using both nonlinear static analysis (NSA) and incremental dynamic analysis (IDA). The aim was to determine the computational difference in NSA for inclined-bent bridges by taking into account their degree of inclination. In this study, the difference in displacement (in percent) was calculated as one of the parameters involved in the seismic response of bridges, as shown in Table 10 and Table 11.

Table 10. Difference in percentages of NSA compared to IDA in the steel bridges (return periods of 475 years).

Bridge Model	NSA		IDA			Error (%)
	Yield Displacement (m)	Target Displacement (m)	T1 (sec)	Sa (T1, 5%)/g	Displacement (m)	
Bridge1(α=45)	0.0398	0.0615	0.5969	0.9772	0.1980	+68.9550
Bridge2(α=55)	0.0608	0.0759	0.6165	0.9569	0.1620	+53.1660
Bridge3(α=65)	0.0776	0.0860	0.6162	0.9569	0.1620	+46.8960
Bridge4(α=75)	0.0869	0.0874	0.6087	0.9670	0.1500	+41.7660

Table 11. Difference in percentages of NSA compared to IDA in the steel bridges (return periods of 2475 years).

Bridge Model	NSA		IDA			Error (%)
	Yield Displacement (m)	Target Displacement (m)	T1 (sec)	Sa (T1, 5%)/g	Displacement (m)	
Bridge1(α=45)	0.0398	0.0871	0.5969	1.5963	0.3300	+73.613
Bridge2(α=55)	0.0608	0.1059	0.6165	1.58055	0.2700	+60.776
Bridge3(α=65)	0.0776	0.1164	0.6162	1.58055	0.2580	+54.892
Bridge4(α=75)	0.0869	0.1197	0.6087	1.5864	0.2340	+48.840

5. CONCLUSION

In this research, focus was on the accuracy of Nonlinear Static Analysis (NSA) compared with Incremental Dynamic Analysis (IDA) in irregular steel framed bridges. Using Nonlinear Static Analysis (NSA) in terms of evaluating bridge performances has become prevalent. While, for irregular framed bridges with different inclined legs, analysis accuracy is not predictable. Consequently, in order to show the difference in percentages between nonlinear static analysis (NSA) and incremental dynamic analysis (IDA) in irregular framed bridges, seven models of steel bridges were used. Target displacements as well as drift criteria were the main features obtained from NSA and IDA, respectively. It can be observed that in steel bridges with inclined legs, there is a dramatic decrease in accuracy of NSA when the inclined leg's angle drops from 75˚ to 45˚. Overall, in both return periods of 475 years and 2475 years, the differences are nearly between 40% and 70% for angle changes between 75˚ and 45˚, respectively. Also it has been observed that the accuracy of NSA depends on the hazard level that falls significantly from a hazard level of 10% in 50 years to 2% in 50 years. Therefore, it is concluded that irregularities have substantial effects on bridge structural responses which need to be considered further in order to avoid obtaining absurd calculations and wrong predictions.

REFERENCES

American Association of State Highway and Transportation Officials (2007). AASHTO LRFD Bridge Design Specifications, Customary U.S. Units. 4th edition, Washington D.C.

Ansourian, P. (1981). Experiments on continuous composite beams. Proc., Inst. Civ. Eng., **71**(2): 25–71.

Applied Technology Council (1996). Seismic Evaluation and Retrofit of Concrete Buildings. ATC-40, Vol.1.

Aviram, A., Mackie, K.R., and Stojadinović, B. (2008). Guidelines for Nonlinear Analysis of Bridge Structures in California. Pacific Earthquake Engineering Research Center, Peer2008/03.

California Department of Transportation (Caltrans) (2006). Seismic Design Criteria (SCD). Version 1.4.

Chopra, A.K., and Goel, R.K. (1999). Capacity-Demand-Diagram Methods for Estimating Seismic Deformation of Inelastic Structures SDF Systems. PEER Report 1999/02.

Cornell, C.A., and Vamvatsikos, D. (2002). Seismic Performance, Capacity and Reliability of Structures as Seen Through Incremental Dynamic Analysis. Doctor of Philosophy Dissertation, Department of Civil and Environmental Engineering, Stanford University, California, USA.

Federal Emergency Management Agency (1997). NEHRP Guidelines for the Seismic Rehabilitation of Buildings. FEMA-273.

Federal Emergency Management Agency (2009). Quantification of Building Seismic Performance Factors. FEMA P695.

Isaković, T., and Fischinger, M. (2000). Regularity Indices for Bridge Structures". *In* Proceedings of the 12th World Conference on earthquake engineering, Auckland, New Zealand.

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

MATLAB (R2009b) v7.9.0.529 (2009). The Language of Technical Computing.

Mergel, P., and Almansour, H. (2010). Live load distribution on rigid frame concrete bridge. *In* Proceedings of the 8th International Conference on Short and Medium Span Bridges, Niagara Falls, Ontario, Canada, 3-6 August 2010, pp. 106-1-106-10.

Monteiro, R., Ribeiro, R., Marques, M., Delgado, R., and Costa A. (2008). Pushover Analysis of RC Bridges Using Fiber Models or Plastic Hinges. *In* Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China, 12-17 October 2008.

Muljati, I., and Warnitchai, P. (2007). A Modal Pushover Analysis on Multi-Span Concrete Bridges to Estimate Inelastic Seismic Responses. CIVIL ENGINEERING DIMENSION, **9**(1): 33 – 41.

OpenSees v.2.2.2 (2011). Open System for Earthquake Engineering Simulation. Created at the NSF, sponsored by Pacific Earthquake Engineering (PEER) Center. Available from http://opensees.berkeley.edu/.

Pi, Y.L., Bradford, M.A., and Uy, B. (2006). Second Order Nonlinear Inelastic Analysis of Composite Steel–Concrete Members. II: Applications. Journal of Structural Engineering, **132**(5): 762–771.

SAP2000 Advanced 14.1.0, Static and Dynamic Finite Element Analysis of Structures, University of California Berkeley.

Taghikhani, T., and Mousavi, K. (2011). Seismic evaluation of performance coefficients of designed moment frame structures affected by near-field earthquake based on Iranian Code. M.Sc. Thesis. Department of Civil and Environmental Engineering, Amirkabir University of Technology (Tehran Polytechnic), Tehran, Iran, (in Farsi).