13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1351

DEVELOPMENT OF FRAGILITY AND RELIABILITY CURVES FOR SEISMIC EVALUATION OF A MAJOR PRESTRESSED CONCRETE BRIDGE

Fariborz NATEGHI-A. [1] and Vahid L. SHAHSAVAR[2]

SUMMARY

Bridges in recent earthquakes have proven to possess the most threat to transportation system during and after earthquakes. In addition, well being of bridges plays a major role in the post earthquake emergency structures for earthquakes. To address the physical aspects of the seismic performance of bridges, fragility functions or damage probability matrices are developed and used for evaluation purposes. These fragility curves represent the probability of structural damage due to various ground shakings. And more so they describe a relationship between ground motion and level of damage. In this paper, fragility and reliability curves are developed. The seismic vulnerability of a prestressed concrete railway bridge then is assessed based on developed relations. These relations are derived from dynamic nonlinear finite element analysis. A software package in MATLAB is also generated for easy study of the results. Important aspects of this study are; modeling of bridge using 3D nonlinear models and modeling of abutments, bearings, effect of falling of girder on its bearings and nonlinear interaction of soil-structure action. Reliability curves developed in this study are unique in its case. This paper presents the method as well as the results in the form of vulnerability and structural reliability relations based on two damage functions.

INTRODUCTION

Bridges are potentially one of the most seismically vulnerable structures in the highway system during earthquake. It is known that the seismic performance of transportation systems plays key role for the post earthquake emergency management [1, 2]. Hence, it is necessary to be evaluated both physical and functional aspects of bridge structures. The physical aspects of the seismic performance of bridges are evaluated with the seismic fragility functions or damage probability matrices of transportation facilities. A fragility curve, represent the probability of structural damage due to various ground shakings [3, 4]. And more so they describe a relationship between ground motion and level of damage. To exactly define such a ratio, the correct choosing of the ground motion in the target area, is of a great importance. The index defining the intensity of the ground seismic motion properly for the fragility analysis, are as follows; PGA,

[1] Professor, IIEES, Structures, Tehran, Iran. Email: Nateghi@dena.iiees.ac.ir

[2] Technical office manager, Atorpat co & IIEES, Tehran, Iran. Email: Vahidlal@hotmail.com

PGV, PGD, Sa, Sv and Sd. These curves can be obtained by the simulated and/or real analysis of the logic regression of the damage data [5, 6]. About 120 time history of the real ground motion with 30 artificial simulated time history, have been used in this study. Such fragility curves can be used for evaluating the total risk of infrastructures [7-9]. These curves indicate the probable level of damage for a specific class, and by identifying the most vulnerable bridges we can retrofit them. These curves are also used in governmental management institutes and insurance companies to assess the damage rate after earthquake (post-earthquake damage rate) [8-10].

The analytical steps of bridge fragility analysis used in this paper, are as follows:

1. Crating the nonlinear finite element analytical model (fiber element) from the bridge structure (3D);
2. Choosing the proper hysteretic nonlinear models for doing the analysis;
3. Performing a pushover analysis to determine the capacity of members;
4. Choosing the real ground motions and creating the artificial ground motions;
5. Normalization of the records under various levels of PGA and PGV;
6. Performing a nonlinear dynamic finite element analysis using the obtained records;
7. Obtaining the output results, ductility factors and the maximum response of the structure;
8. Calibration of the seismic damage functions dependent on the input motion levels;
9. Calibration of the damage levels and number of their occurrence in the different levels of the input motions;
10. Construction of the fragility curves using the obtained mean and standard deviation with respect to the ground motion indicates for each damage rank assuming a lognormal distribution.

STRUCTURAL SPECEFICATIOS

In this study, studying have been done on a railway bridge with prestress girder, called "Taleh Zang" located in a 600-meter distance from a station with the same name in 587 km of Tehran-Ahvaz railway, and about 70 km away from Andimeshk city to the north, on the river. The mentioned bridge is a concrete bridge with three spans with the length 215 meters, having prestress and continues box girder, which the girder of the bridge is 6.6 meter in width. Figure 1 shows the mentioned bridge (Taleh Zang).

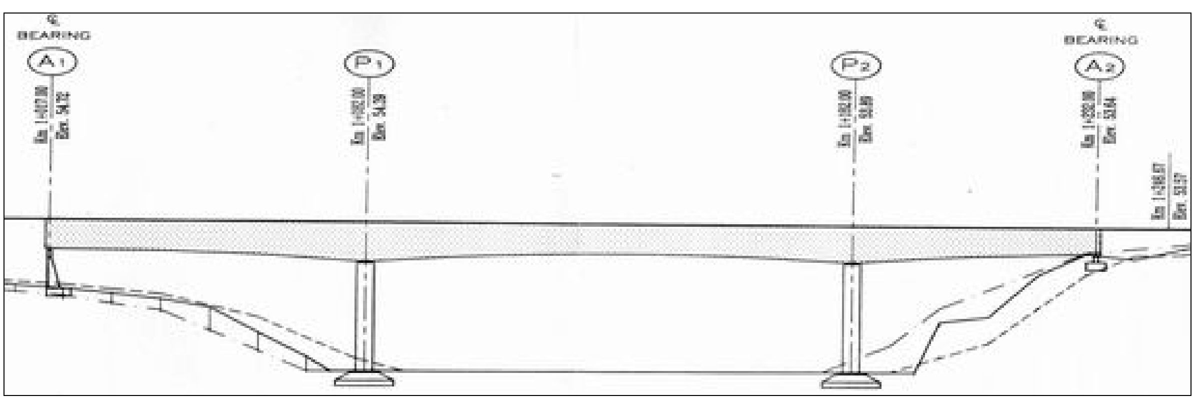

Figure 1. Mentioned bridge (Taleh Zang)

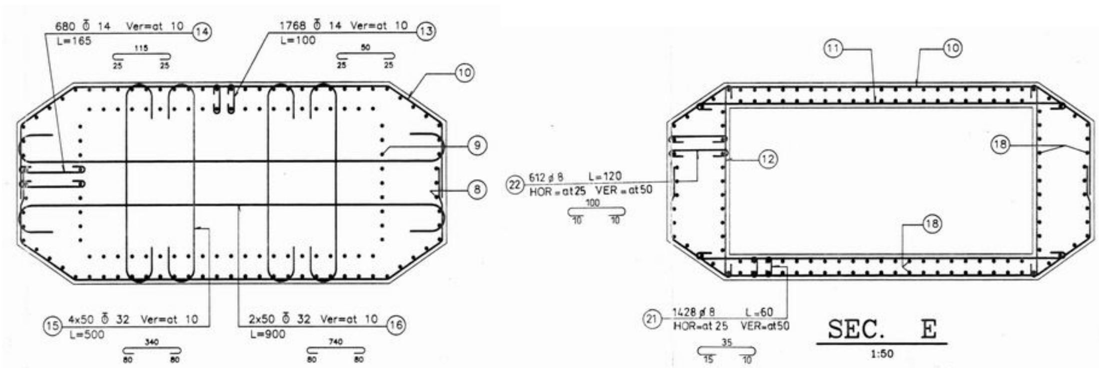

Figure 2. Cross section of piers at top and middle elevations

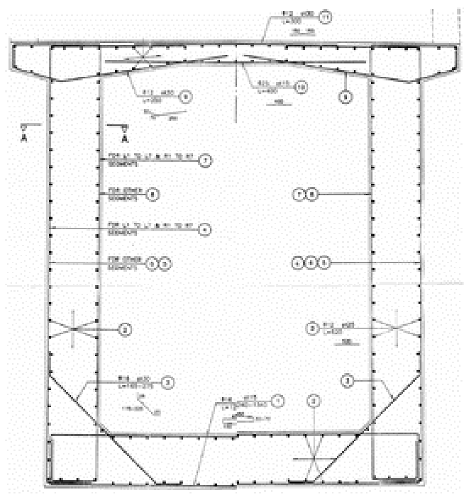
Figure 3. General Cross section of girder

MODELING THE BRIDGE WITH OPENSEES SOFTWARE

"OpenSees" software was obtained by the scientists of Berkeley university [12], and is a finite element program with unique facilities and outstanding flexibility. One can get this software in www.opensees.berkeley.edu. Introduction the pier sections and girder to OpenSess is done in the form of fiber elements with "Xtract" software, which is cross section analysis software [11]. The columns are introduced like nonlinear elements of beam-column in complete 3D, to OpenSess software.

The nonlinear geometry impacts and P-Delta are considered in the related options. For modeling the girder with the variable height, 10 separated cross sections are used in OpenSees, which ultimately the 215-meter length of the girder is divided and modeled to 44 sections. For considering the nonlinear effects of girder in this study, the girder is modeled by the nonlinear element of the column related to the displacement and distributed plasticity, which is one of the advantages of this study in comparison with the others. It should be considered that in most of the research activities, the girder of bridge is supposed liner and modeled with this supposition. However, the studyings done in the reference 11 shows that, the prestressed girders are so sensitive to the vertical seismic motions and in high accelerations penetrate the nonlinear zones.

In this research, to study the nonlinear behavior of the abutments, introducing the abutment is done by Gap elements and nonlinear spring with modeling the neoprenes with the form of nonlinear springs. The nonlinear neoprene springs are completed by Elastoplastic 3D elements, is another advantage of this study. Cause of the nonlinear behavior of the soil during the earthquake, in this study the soil-structure interaction is considered by nonlinear springs with 3D modeled components in soil.

About 120 real time history away from the close zone (R>15 km) were received from Berkley university website and with 30 artificial time history from the university website, scaling 0.1g to 1.2g, 1800 time history were created. The main plate of the fault in the studyings related to the simulated motions was 140x33 km and the average share wave velocity is supposed 760 m/s [11].

For scaling the records and studying them, a graphical software was needed, so a 2000-lined program was written in MATLAB and providing the software package [11], editing the records and studying the fragility and related graphs, was ignored [L. Shahsavar 2003]. After introducing the bridge structure and the analytical options to the software, the nonlinear pushover analysis on the bridge in longitudinal and transverse directions was done. The bridge capacity curve in longitudinal and transverse direction is shown in figure 4. Modeling the falling of girder from the its bearings is another advantage of this study. The girder falling from the bearings took place in the 97-cm displacement and caused the complete failure of the bridge. The dynamic analysis was done using the Newmark integration hierarchy [13] existing in OpenSees considering, controlling the increased energy options related to the system with 1e-10 accuracy.

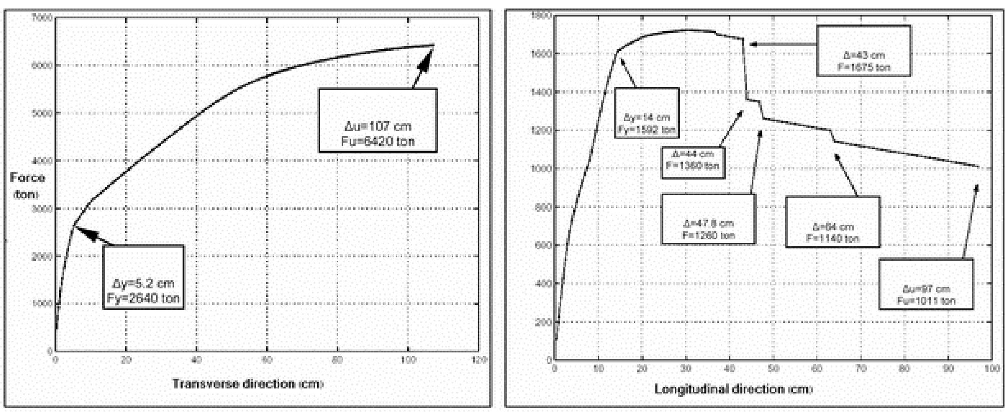

Figure 4. The bridge capacity curve in longitudinal and transverse direction

DEFINING THE SESIMIC DAMAGE STATES

Inelastic nonlinear finite element analysis

Further more, different kinds of functions and parameters being studied in this study will be considered. Other than the indicated parameters in this section, other parameters like, the acceleration time history and the 3-component velocity on girder during the recorded dynamic analysis as the structure response parameters, were studied.

The inelastic displacement ductility ratio was defined by Powel & Allah Abadi (1998) and Kosenza (1993) as follows [14, 15]:

$$IDDR = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} = \frac{\mu_m - 1}{\mu_u - 1} \tag{1}$$

In the above relation, IDDR is the inelastic displacement ductility ratio, δ_m is the maximum displacement from the dynamic analysis of the structure, δ_y is the yield displacement in the resulted member from the Pushover analysis, δ_u is the final displacement of the failure state in the resulted member from the Pushover analysis, $\mu_m = \delta_m / \delta_y$ is the displacement ductility demand by the earthquake and $\mu_u = \delta_u / \delta_y$ is the maximum displacement ductility demand by Pushover analysis.

The damage function related to the Hysteretic energy which is a combination of the maximum displacement and the absorbed hysteretic energy by the element during the alternative loading, is indicated by Park & Ang (1988) and Gobera (1997) as follows [16-18]:

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta_0 \cdot E_H}{\delta_u \cdot F_y} \tag{2}$$

$$\beta_o = 0.36 \times \left[v + (W_l - 0.2)^2 \right] \times 0.9^{W_t} \tag{3}$$

in the above relation, $DI_{P\&A}$ is the Park-Ang damage function, E_H is the absorbed hysteretic energy, gathered by the member during the cyclic loads, equaling the total area under the force-deformation curve of the studied member, F_y is the yield strength of the studied member resulted from the pushover analysis and β_o is the constant parameter of the model which differs for the different structures having a limit from -0.03 to 0.15. The states of similar seismic damage and the operational areas are in the table 1.

Table 1. The states of similar seismic damage and the operational areas

Description	Damage	Damage state	Damage Index (DI)
No damage	N (No)	1	0.00< DI <= 0.14
Slight damage	S (Slight)	2	0.14< DI <= 0.40
Moderate damage	M (Moderate)	3	0.40 < DI <= 0.60
Extensive damage	E (Extensive)	4	0.60 < DI < 1.00
Complete damage	C (Complete)	5	1.0 <=DI

STUDYING THE FRAGILITY OF THE BRIDGE

If the structural capacity and the seismic demand are the two variables following the Normal or Lognormal distribution, according to the mid-term theorem, the efficiency of the resulted complex, is distributed in the form of Lognormal. So the probable distribution mentioned in the fragility curves form, is indicated by the probability density function. For determining such a curve, only two parameters are needed, the average standard deviation (standard deviation in the 50th percentage) and the normalized logarithmic standard deviation [3-6]. According to the lognormal theory, a fragility curve (F(a)) has the following analytical form:

$$F(a_g) = \Phi \left[\frac{\ln a_g - \sigma}{\mu} \right] \tag{4}$$

in the above relation, $F(a_g)$ is the seismic fragility function, a_g indicates the amount of the ground motion (PGA or PGV), μ is the natural logarithmic standard deviation of the input motion levels, σ is the average natural logarithm of the input motion levels and $\Phi[.]$ is the standard normalized distribution function.

The extraction method of the two lognormal distribution parameters, is distributing the parameters on the lognormal probability paper and extracting the average and the standard deviation from the superposition line [7, 19].

CREATING THE FRAGILITY MATRIX

To create the fragility matrix related to the states of the seismic damage, the results of the damage analysis were inspected and collected and the out come is indicated according to the related figures. The fragility curves were obtained in the different levels of acceleration and basic input velocity, based on the two famous and reliable damage functions, as follows: 1- Inelastic displacement ductility ration, 2- Park-Ang function, and the results were compared.

First, the accruing times of each damage state in different levels of acceleration and the peak ground velocity are counted. The accruing times of the damage states related to the IDDR & $DI_{P\&A}$ functions in different levels of the acceleration and peak ground velocity is shown in figure 5.

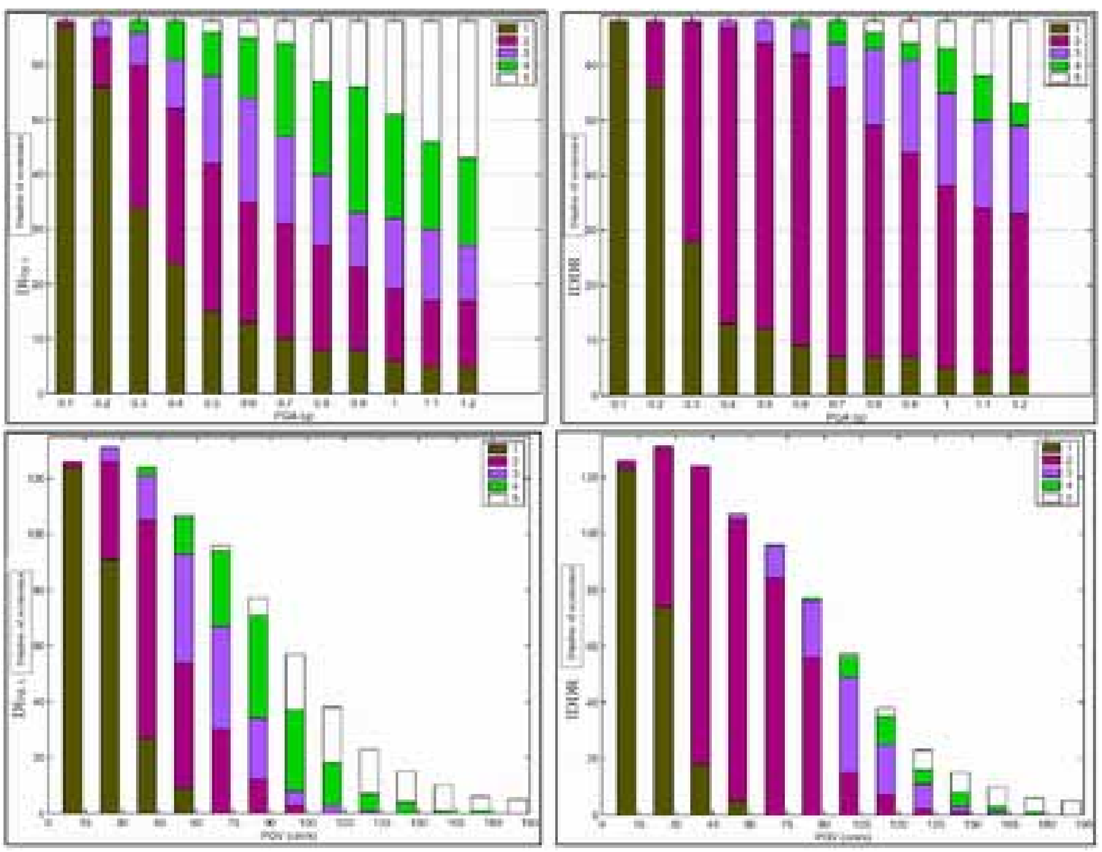

Figure 5. accruing times of the damage states related to the IDDR & $DI_{P\&A}$ functions

Then for determining of the average and standard deviations of the bridge fragility parameter, the number of the accruing damage states related to the IDDR & $DI_{P\&A}$ functions on figure 6, were drawn on lognormal probability paper. After that, by a nonlinear regression analysis among the data, the purpose equations with the mean and standard deviation of the data were extracted.

Accordingly, the seismic fragility curves were obtained from the results and relation 4, and are shown in figure 7. In the above curves, the zero occurring probability equals non-occurring the damage or the zero probability of the damage, so the probability of occurrence 1 equals the complete occurrence of the damage or the maximum probability of the damage.

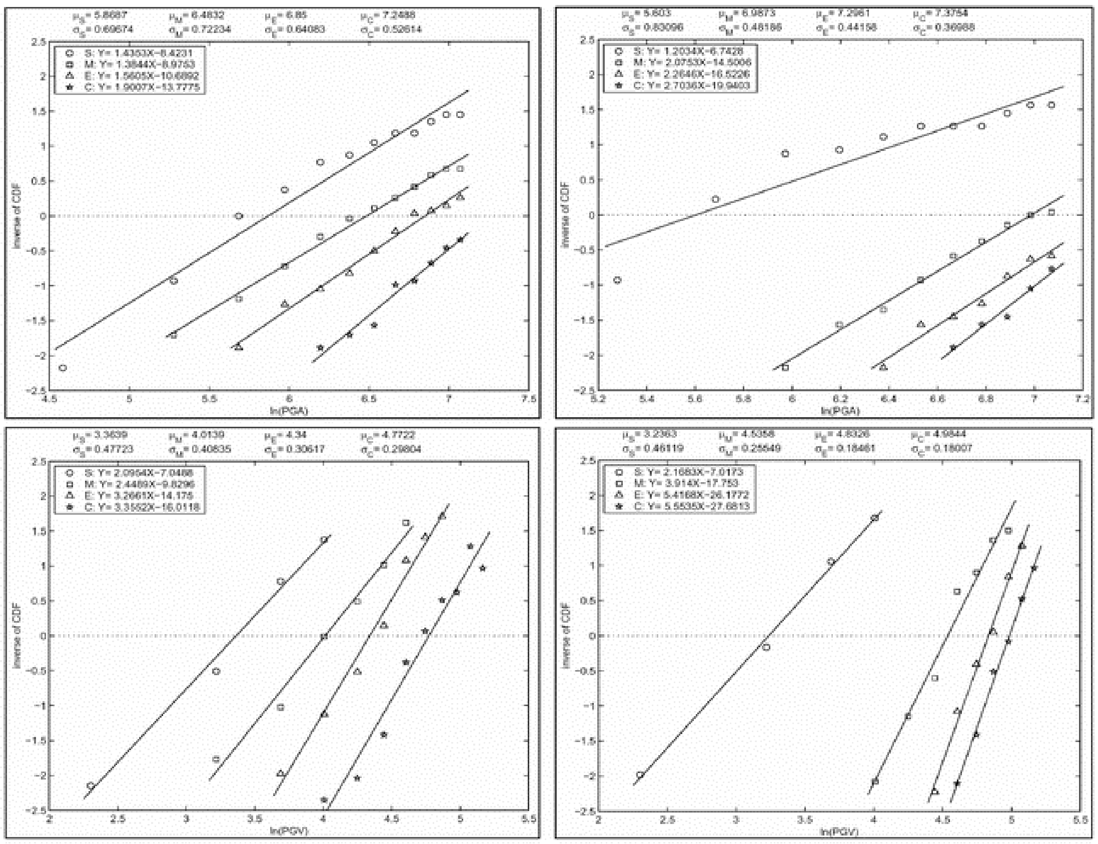

Figure 6. Number of the accruing damage states related to the IDDR & $DI_{P\&A}$ functions on lognormal probability paper

For using the results of these curves and determining the seismic operation of the bridge, the different levels of the operation should be defined. For proper operating of the bridge, the probability of the occurrence of the defined damage states (table 1) in each operational level should be below 50%, otherwise, the function of the bridge in that operational level wouldn't be evaluated properly. For instance, in figure 7 the occurrence probability below 50% in the peak ground acceleration level of g, or the level of the peak ground velocity of 100 cm/s, is defined as a reasonable level of seismic operation in that damage state.

From the resulted fragility curves it is understood that even under the peak ground acceleration of 1.2 g, the probability of the minor damage state doesn't reach 1, however, under the peak ground velocity of 60 cm/s, this probability reach 1. These curves are another reason for the reliability of the results depended on the peak velocity of the ground.

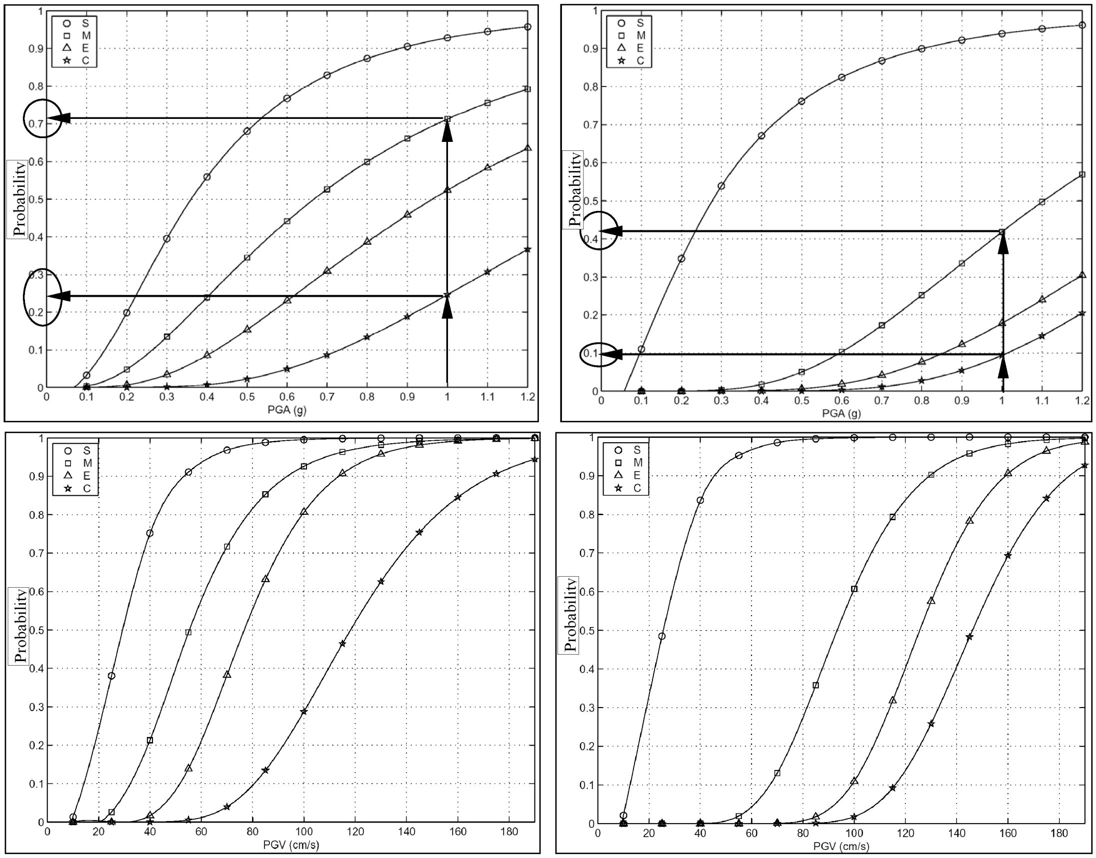

Figure 7. Fragility curves related to the IDDR & $DI_{P\&A}$ functions

THE RELIABILITY INDEX OF THE STRUCTURE (β)

In previous sections it was discussed that, for determining of the seismic operation of the bridge from the fragility curves, the probability of the damage occurrence and defining the probability line in 50%, should be checked. Because of the low slope of the curves in the upper and lower levels of the ground motions, reading the occurrence probability from such curves is not simply done without errors. More over, the probabilities more & less than 50% are not individually meaningful, and just indicate the probability of the damage occurrence. For instance, the occurrence probability of 0.6, means the extra occurrence probability of 0.1 from the 50% line, however this is not enough for determining the needs for retrofitting. To compensate such faults, the seismic reliability curves were expanded by the authors [L. Shahsavar 2003]. The reliability of the structure is defined as follows:

$$\beta = \Phi^{-1}(1 - P_f) \tag{5}$$

in the above relation, β is the reliability of the structure and P_f is the occurrence probability of the definite damage under the definite acceleration or the velocity. In the fragility analysis, the definite amount of β is varied between -5 to 6. -5 equals the failure of the structure and non-reliability of the structure, and 6 shows the occurrence probability around zero and indicates that the structure is 100% reliable. This coefficient shows that on the zero line, the structure has a probability of 50% failure. So the area under the zero line is defined as the critical zone for the structure [11], (figure 8).

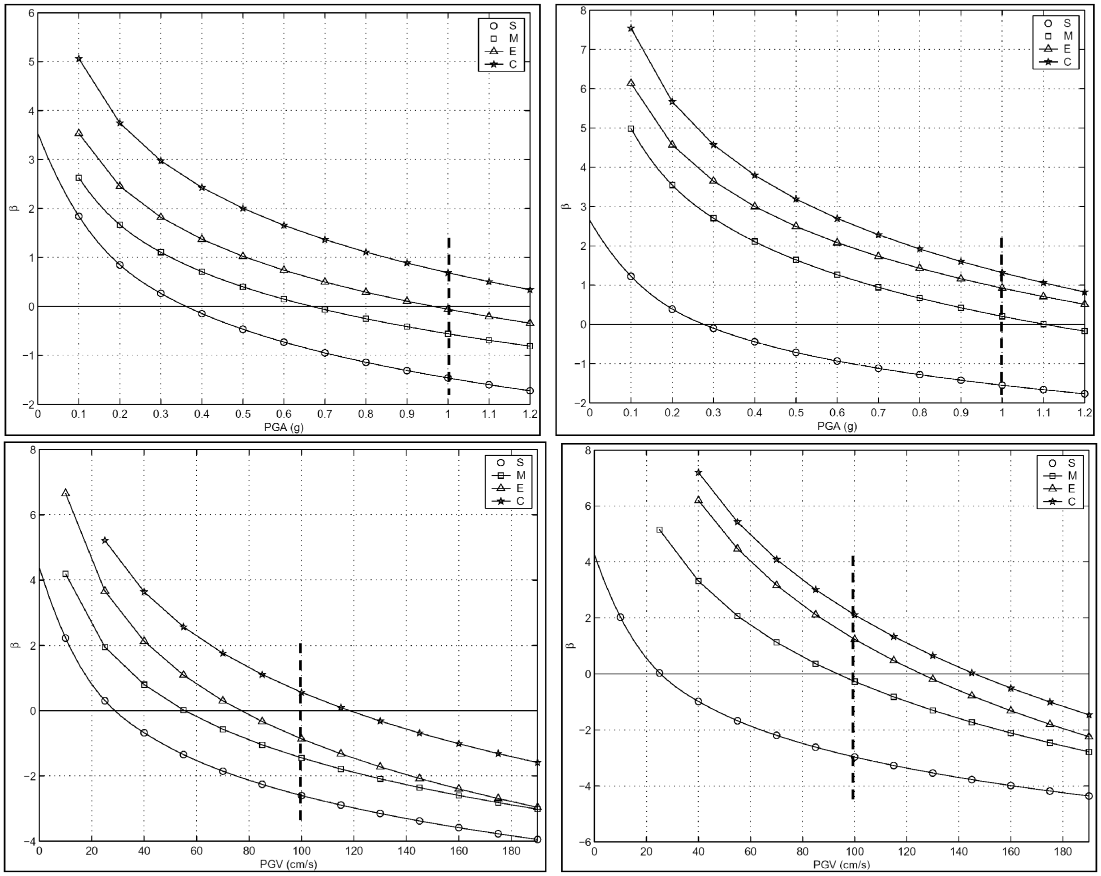

Figure 8. Reliability curves related to the IDDR & $DI_{P\&A}$ functions

RESULTS AND DISCUSSION

The identification of the earthquakes in the nonlinear finite element analysis can influence the results, as in comparing the damage, dependent on the input acceleration, some records were found in which the amount of the g acceleration still cause minor damage. For structures with long period (like the studied bridge), the response of the damage analysis under the peak ground acceleration and velocity was compared and was cleared that reliability ratio of the results under the maximum input acceleration is low. However, this ratio for the results under the peak ground velocity is very high. This proves that the scientists would better use the PGV in their researches on long period structures instead of the PGV.

From the fragility curves, it is got that even under the PGA, 1.2g, the occurrence probability of the minor damage state doesn't reach 1, but under PGV, 60 cm/s, this probability reaches 1. These curves are another reason for reliability of the results depended on the PGV, (figure 7).

The above fragility curves are used for this point whether the capacity of the current bridge, considering the supposed earthquake facts, is enough or not? It was determined that the bridge had a good strength and stability (the bridge in the major damage limit state doesn't penetrate non-reliability zone).

The suggested amount of the μ by the researches of Berkley university in the theoretical section of the fragility curves was 0.6 which was a maximum amount for the researches under PGA [6,20]. In this study the above amount, under the studyings on PGA was suggested as 0.646 and under the studyings on PGV was suggested as 0.37. These amounts are recommended for the similar activities in IRAN, figure 6.

From the seismic reliability curves in figure 8, we can read and recite all the data in the entire input motion zone witch the minimum error ignoring the details. The zero line (occurrence probability of 50%) is another advantage of these curves which makes the inspecting of the definite operation level, possible. The areas over the zero line (positive numbers) are the safety level of the structure and the areas below the zero line (negative numbers) are the unsafe level of the structure. For instance, in figure 8 (left-top) the bridge structure in moderate and slight damage levels (table 1) has penetrated the unsafe area. In figure 8 (left-bottom) which shows the reliability curves deriving from the damage function $DI_{P\&A}$ dependent on the PGV, the levels of the heavy damage, moderate and slight, have penetrated the unsafe area.

The difference between the two indeces, the damage of the inelastic displacement ductility and Park-Ang, is very much [11, 15, 18]. In this study, the fragility curves were created based on the both damage indexes. The comparison between the created fragility curves from the real and simulated records and the curves created based on the two above damage indexes, has been done graphically in the related section.

At the end, it should be mentioned that the fragility curves method or the reliability, is the most perfect way to determine the probabilistic damage of the bridges, so no any suggestion is given on doing damage analysis related to other methods. Working on the structural reliability indexes is recommended as one of the unlimited research horizons for the researchers.

REFERENCES

1. MCEER, “Seismic Retrofitting Manuals for Highway Systems, Volume II: Seismic Evaluation and Retrofitting Manual for Highway Bridges", Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York, 1999(b).
2. Werner, S.D., Taylor, C.E., Moore, J.E. and Walton, J.S., “Seismic Retrofitting Manuals for Highway Systems, Volume I: Seismic Risk Analysis of Highway Systems”, and “Technical Report for Volume I”, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York, 1999.
3. Loh C.-H. and Huang Y.-J., “Seismic fragility analysis of Highway bridges”, Monte Carlo Simulation, Balkema, Rotterdam, ISBN 90 5809 188 0, 505-511, 2001.
4. Mander John B., “Fragility Curve Development for Assessing the Seismic Vulnerability of Highway Bridges”, MCEER Highway Project/FHWA National Institute of Building Sciences (NIBS), 1-10, 1999.
5. Saxena Vinita, Deodatis George, Shinozuka Masanobu and Q. Feng Maria, “Development of Fragility Curves for Multi-Span Reinforced Concrete Bridges”, Dept. of Civil and Environmental Engineering, Princeton University, Princeton, 1-7, 2001.
6. Shinozuka Masanobu, Grigoriu Mircea, R. Ingraffea Anthony, L. Billington Sarah, Feenstra Peter, T. Soong Tsu, M. Reinhorn Andrei and Maragakis Emmanuel, “Development of Fragility Information for Structures and Nonstructural Components”, National Science Foundation,Earthquake Engineering Research Centers Program, Federal Highway Administration, 17-27, 2000.
7. Saxena Vinita, “Spatial variation of earthquake ground motion and development of bridge fragility curves”, PhD Dissertation, princeton university, 1-25, 2000.
8. Basz, N. and Kiremidjian, A.S., “Evaluation of Bridge Damage Data from the Loma Prieta and Northridge”, CA Earthquakes, Technical Report MCEER-98-0004, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York., 1-35, 1998.
9. Kiremidjian Anne A. and Basöz Nesrin, “Evaluation of Bridge Damage Data from Recent Earthquakes”, NCEER Bulletin, Vol. 11, No. 2, April 1997.

10. Wilshire H.G. and Nakata J.K., “Damage to Transportation Facilities due to The 1989 Loma Prieta Earthquake”, U.S. Geological Survey, 1989.
11. L. Shahsavar V., "Seismic vulnerability analysis of concrete bridges using fragility curves", Msc Dissertation, Azad university of science & research, advisor: Nateghi. E. F., 2003.
12. OpenSees, http://opensees.berkeley.edu, 2003.
13. Clough, Ray W., “Dynamics of Structures”, Saadatpour M. M., Isfahan University of Technology, 609-721, 1998.
14. Powell, G.H. and R. Allahabadi, “Seismic damage prediction by deterministic methods Concepts and procedures,” Earthquake Engineering and Structural Dynamics, Vol. 16, pp.719-734, 1988.
15. Cosenza, E., G. Manfredi, and R. Ramasco, “The use of damage functionals in earthquake engineering: A comparison between different methods,” Earthquake Engineering and Structural Dynamics, Vol. 22, pp.855-868, 1993.
16. Park, Y.J., and A.H-S. Ang, “Mechanistic seismic damage model for reinforced concrete,” ASCE Journal of Structural Engineering, Vol. 111, No.4, pp.722-739, 1985.
17. Park, Y.J., A.H-S. Ang, and Y.K. Wen, “Damage-Limiting aseismic design of buildings,” Earthquake Spectra, Vol. 3, No.1, pp.1-26, 1987.
18. Ang, A.H-S., and D. de Leon, “Reliability and response control of R/C buildings,” Proceedings, ASCE Structures Congress XII, Vol. 2, pp. 1593-1599, 1994.
19. Karim Kazi R, Yamazaki F., “Effect of earthquake ground motions on fragility curves of highway bridge piers based on numerical simulation”, Earthquake Engineering and Structural Dynamics, 30(12):1839-1856, 2001.
20. Shinozuka M., Uzawa T. and Sheng L-H., “Estimation and testing of fragility parameters”, Monte Carlo Simulation, Balkema, Rotterdam, ISBN 90 5809 188 0, 491-498, 2001.