SEISMIC FRAGILITY ANALYSIS OF DAMAGED CONCRETE BRIDGE- A PERFORMANCE BASED APPROACH

D. M. Patil[1], Amit Melani[2], Rakesh K. Khare[3]
[1]Department of Civil Engineering and Applied Mechanics,
Shri Govindram Seksaria Institute of Technology and Science, Indore (M. P.), India

Abstract
Recent earthquakes and the resulting losses have highlighted the structural design inadequacy of important structures like concrete bridges to carry seismic loads. Hence there is a necessity of performance based procedures in seismic design code for assessment of concrete bridges in terms of seismic resistance. In view of this, best suited methodology and guidelines are very much necessary for seismic evaluation of existing bridges and also for design of new bridges. Therefore, in present study, seismic fragility analysis of concrete bridge damaged during 1995 Kobe Earthquake has been carried out using SAP2000 for pushover analysis and IDARC-2D for incremental dynamic analysis with two sets of time histories, to insist on performance based earthquake engineering procedure.
Keywords: concrete bridge, evaluation, fragility, seismic.

I. INTRODUCTION

Current codes and modern engineering practice address the issues of collapse prevention and life safety by conservatively predicting nominal demands and strengths of structural members, but provide little indication of the actual state of a structure after an earthquake. In post earthquake condition, a bridge may still be standing but structural and nonstructural members may be damaged, resulting in costly repairs. The economic losses due to downtime may even be larger. In contrast to current codes, performance-based earthquake engineering (PBEE) attempts to explicitly predict damage states and assess the probability of reaching multiple levels of damage. PBEE has the potential to improve structural engineering practice by providing engineers the capability of designing structures to achieve a variety of performance levels. The impact of implementing PBEE goes beyond improving engineering practice and extends to a wide range of decision making. The potential impact of PBEE is summarized in Pacific Earthquake Engineering Research Center's report [1].

In this paper, existing results of Hanshin Expressway damaged during Hyogo-ken Nanbu Earthquake of 17 January 1995 [2] are used to match the results of fragility analysis. The drift-ratio approach provides a simple means of estimating damage displacements (which is essential to engineering practice), but it has significant limitations. This approach neglects the effects of cycling on damage and it is difficult to implement for columns with biaxial bending, variable axial loads or variable shear spans.

The objective of this paper is to evaluate column-modeling strategies that are capable of accurately modeling column behavior under seismic loading, including global and local deformations, as well as progression of damage. The focus of this research will be on ductile reinforced bridge columns, for which shear failure is not a consideration.

Seismic fragility analysis of damaged concrete bridge has been carried out using SAP2000 and IDARC-2D to establish importance of PBEE. SAP2000 is used for pushover analysis and IDARC-2D [3] is used for incremental dynamic analysis using two sets of time histories.

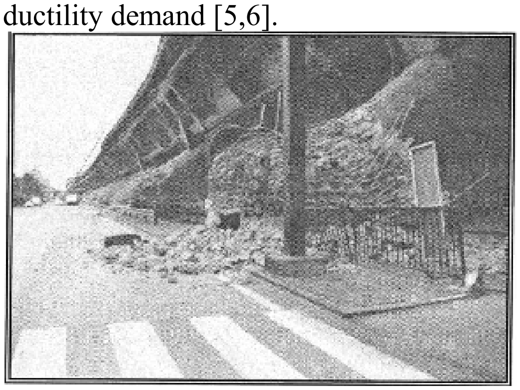
II. CASE STUDY OF HANSHIN EXPRESSWAY

The Hanshin Expressway is a network of major toll highways and interlinking feeder routes combining to form one of the primary arterial roadways within the Hanshin (Osaka-Kobe) region. The Kobe Route 3 section of the Hanshin Expressway is a major toll highway extending from Osaka, alongside Osaka Bay through to the western side of Kobe and is almost entirely elevated for approximately 40 kilometers. The Kobe Route sustained heavy damage from the earthquake, resulting in a total of 47 collapsed piers, 82 severely damaged piers and 471 partially damaged piers.

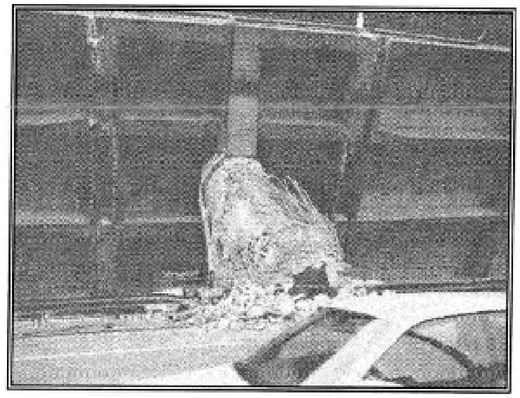
A. Description of Damaged Concrete Bridge

Collapse of typical concrete column was one of the major causes of the extensive damage during the 1995, Kobe earthquake [4]. Collapse of this section of the elevated highway was initiated by formation of a local hinge mechanism near the base of the columns, at the height of cut-off of the inner layer of longitudinal reinforcement. This mechanism led to the entire superstructure overturning until the deck edge rested on the ground. With excessive rotation in the column due to overturning, the longitudinal reinforcement effectively split the column, spalling large sections of concrete away and further weakening the column. Stripped sections of spalled concrete running the length of the columns are evident in Fig. 1.

Other contributing factors reported for the collapse were, failure of the pressure gas welds on reinforcing bars and high shear stresses coincident with the high flexural ductility demand [5,6].



(a) Hinge at base reinforcement cut-off



(b) Column separation due to tension generated in longitudinal reinforcement on overturning

Figure 1. Hanshin Expressway, Kobe Route 3

B. General Description and Structural Characteristics

The material strength used in design is 26.5 MPa for the compressive strength of the concrete while the steel yield and ultimate stress values are 300 and 450 MPa, respectively

The 20.25-meter-wide reinforced concrete deck over the column is cast in-situ to form a monolithic deck/column unit with effective deck span of 35 meters between columns.

The columns are circular reinforced concrete of 3,100mm diameter and 14,967mm in length. 2,300mm of which forms the upper part of the pile cap. A further 2,800 mm at the top of the column is cast monolithically with a reinforced concrete girder deck of the substructure, leaving 9,867mm of free column.

The longitudinal column reinforcement consists of three concentric rings of 60 bars of 35mm diameter (D35) up to a height of 2500mm above the top of the pile cap. At this point the inner ring is cut-off while the two outer layers continue full length of the column. The column is confined by 16mm diameter bars (D16) at 200mm centers on the outer transverse reinforcement and 400mm centers on the inner transverse reinforcement up to the 2500mm cut-off. The bars are then spaced at 300mm centers on the remaining outer layers of transverse reinforcement for 4200mm at which spacing reduces to 200mm centers for the remainder of the column. Cover to the reinforcement is 100mm with 100mm between layers. The distance between ground level and the top of the pile cap varied with location between 500 and 1500mm.

C. Analytical Modeling of Structure Using SAP 2000

The expressway is modeled in three dimensions, assuming inelastic behavior to be restricted to the column and pile sections while the deck remains elastic. The structural models are assembled by considering the structure to be a sequence of identical units connected at the deck. The overall model representing piers 126 through 142 is shown in Fig.2.
This is an assembly of units linked with the appropriate connection boundary conditions. An average ground level elevation of 1 meter above the centroid of the pile-cap is assumed for each pier.

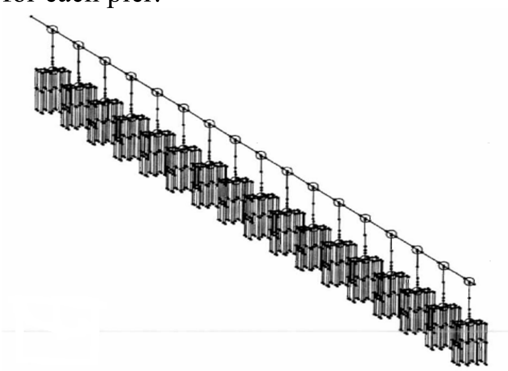


Figure 2. Analytical model of expressway

The deck is assumed to remain elastic throughout the analysis. Three elements are utilized to replicate the properties of the deck in terms of vertical and transverse stiffness and distributed mass contribution. The assumption of the deck remaining elastic facilitated the use of a three-dimensional elastic element. The deck mass is distributed on identical axis as structural elements, using a cubic three-dimensional mass element. The connection between deck and pier is represented by a rigid link element. The length of the rigid link is determined so as to allow connection between the centroidal node of the deck and the top of the column. A three-dimensional non-structural lumped mass element is applied at the top of the link to account for the increased weight of the reinforced concrete girder configuration at the deck-to-column connection.
The piers are split into three separate elements of 5600mm, 4200mm and 2500mm to reflect the changing reinforcement and confinement conditions along the length. Each of the sections is modeled as a circular reinforced concrete section. The element is selected as it employs a cubic shape function and allows the spread of plasticity both along the individual members and across the cross-section to be monitored.
Confining pressure results from the transverse steel in the reinforced concrete, which is passively resisting the lateral expansion of the concrete under axial compressive load. This confining action increases the actual compressive strength and ductility of the section. The concrete model used accounts for the increase in core compressive strength due to the confinement by evaluating the instantaneous confined strength from the state of axial stress for each monitoring area within the section. A bilinear elasto-plastic model with kinematic hardening is adopted for the reinforcing steel. A strain hardening parameter of 0.01 is utilized. For confined and unconfined concrete Mander model is used.

D. Analytical Modeling of Structure Using IDARC-2D

For incremental dynamic analysis of the Hanshin Expressway concrete bridge, a two-dimensional single column bent analytical model is considered, loaded axially by 10221kN force as shown in Fig.3.

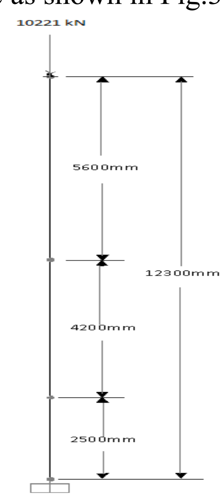


Figure 3. Analytical model of single column bent of Hanshin Expressway

The column is split into three separate elements of 5600mm, 4200mm and 2500mm to reflect the changing reinforcement and confinement conditions along the length. Each of the sections is modeled as a circular reinforced concrete section. Column is considered as fix at the bottom with plastic hinge modeling. Moderate degrading conditions are used for hysteresis modeling.

E. Nonlinear Analysis Programs

For investigating response of the Hanshin Expressway concrete bridge to the strong ground motion the finite element programs SAP2000 and IDARC-2D are utilized.

Two different sets of ten ground motions each are used for incremental dynamic analysis to consider all ground motion parameters. First set is selected randomly from 20 ground motion records used by Maniyar and Khare [7] and second set is selected randomly from 20 ground motion records used by Vamvatsikos and Cornell [8].

F. Pushover Analysis Results

In order to assess the ductility supply of the piers and to allow the accurate determination of failure criteria for use in interpreting the output from the dynamic analysis, a simple cantilever non-linear static analysis is carried out. The model is essentially identical to the column configuration used in the dynamic analysis, but is restrained for both rotation and displacement in all three planes at the base. The objective is to determine the capacity and displacement ductility supply of various sections of the pier to ascertain the realism of the adopted geometry and material characterization. To account for the axial dead load acting on the column, a vertical load of 10,221 kN is applied.

From the pushover analysis, base shear and displacement capacities are obtained at performance point for the pier to identify the mode of failure. Fig. 4 shows base shear and displacement capacities for single column bent of Hanshin Expressway.

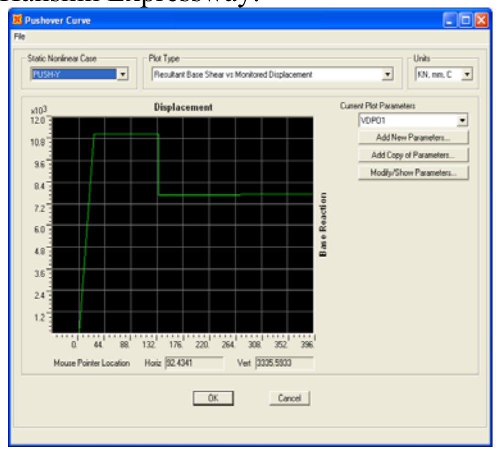


Figure 4. Pushover curve for transverse direction.

The results of pushover analysis are tabulated in Table I from SAP2000. Values of vulnerable PGA (g) are shown in Table II which are calculated from the obtained base shear capacities using equation $V = A_h*W$, as per Indian standards, Where $A_h = Z*I/R*Sa/g$ (Here Z/2 is considered as Z as per AASHTO).

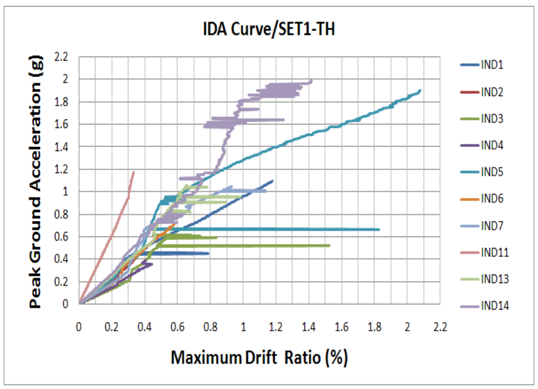
Table I. Base shear capacities at different performance level from pushover analysis

Push Step	Displacement mm	Base Shear kN	Damage State
1	24.898	11053.544	Immediate occupancy
2	64.366	11053.981	Life safety
3	103.834	11054.418	Collapse prevention
4	134.349	11054.755	Damage

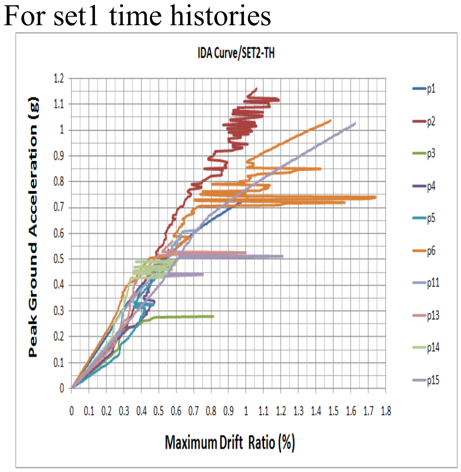
Table II. Vulnerable PGA (g) calculated from base shear capacity using pushover analysis

Plastic Hinge Stage For Flexure Failure	Base Shear Capacity (kN)	PGA(g) From Base Shear Capacity	Displacement (mm)
@ Life safety	11053.981	0.720	64.366
@ Collapse Prevention	11054.418	0.721	103.834
@ Damage	11054.755	0.721	134.349

G. Incremental Dynamic Analysis (IDA) Results



(a) For set1 time histories



(b) For set2 time histories

Figure 5. IDA results

The data collected through IDA can provide more realistic behavior of structures for dynamic response. IDA curves of single column bent of Hanshin Expressway for first set of ten time histories are shown in Fig. 5 (a) for moderate degradation of section with results summarized in Table III. Similarly, IDA curves of single column bent of Hanshin Expressway for second set of ten time histories are shown in Fig. 5 (b) for moderate degradation of section with results tabulated in Table IV.

Table III. IDA Result Summary for Set1 Time History

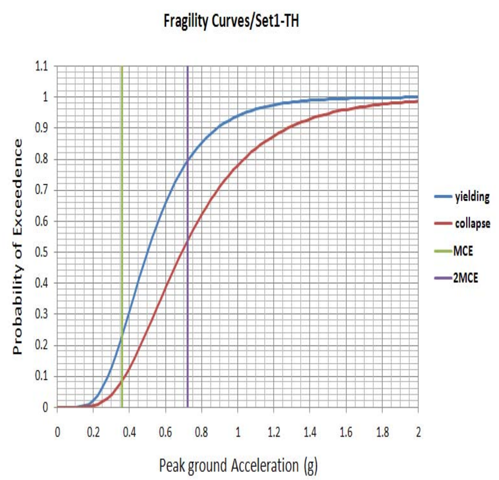
Damage States	Single-column bent (SCB) of Hanshin Expressway	
	Range of PGA(g)	Range of % Drift Ratio
At-Yield	0.295 to 0.97	0.255 to 0.677
At-Collapse	0.39 to 1.64	0.335 to 1.807

Table IV. IDA Result Summary for Set2 Time History

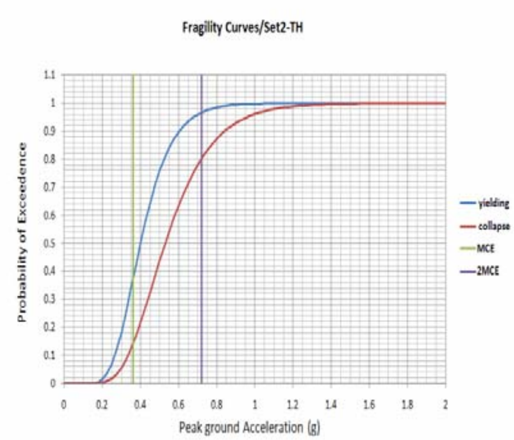
Damage States	Single-column bent of Hanshin Expressway	
	Range of PGA(g)	Range of % Drift Ratio
At-Yield	0.23 to 0.525	0.306 to 0.754
At-Collapse	0.28 to 0.945	0.528 to 1.554

H. Development of Analytical Fragility Curves

Fragility curves can be expressed in the form of two parameters (median and log-standard deviation) lognormal distribution functions. Fragility curves (FC) are constructed with respect to PGA (g) [9]. The damage indices of the bridge piers are obtained from a non-linear dynamic response analysis. Then using the damage indices and the ground motion indices, the fragility curves for the single column bent of Hanshin Expressway bridge are constructed. Fragility curves of single column bent of Hanshin Expressway for first set of ten time histories are shown in Fig. 6 (a) for moderate degradation of section with results summarized in Table V. Similarly, Fragility curves of single column bent of Hanshin Expressway for second set of ten time histories are shown in Fig. 6 (b) for moderate degradation of section with results tabulated in Table VI.



(a) For set1 time histories.



(b) For set2 time histories.

Figure 6. Fragility results

Table V. Fragility Results for Set1 Time History

Type of Earthquake	PGA(g) of earthquake	Probability of Damage (%) for Single-column bent (SCB) of Hanshin Expressway	
		At Yield	At Collapse
MCE*	0.36	23	8
2MCE	0.72	80	54

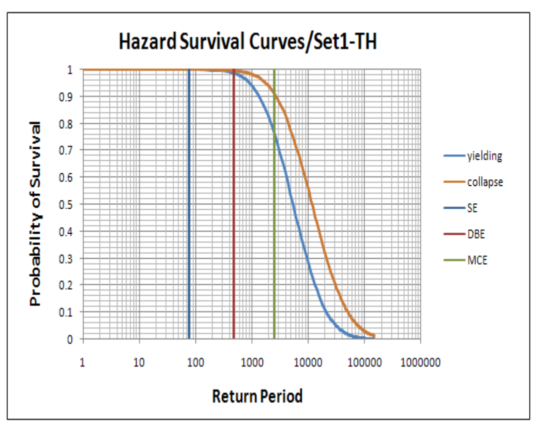
*MCE: Maximum Considered Earthquake as per Indian Standard.

Table VI. Fragility Results for Set 2 Time History

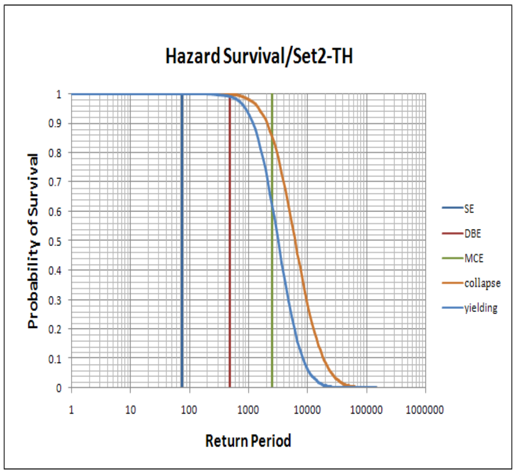
Type of Earthquake	PGA(g) of earthquake	Probability of Damage (%) for Single-column bent (SCB) of Hanshin Expressway	
		At Yield	At Collapse
MCE	0.36	36	14
2MCE	0.72	96	80

I. Development of Hazard Survival Curves

Hazard survival curve (HS) is a plot of probability of survival versus return period. Probability of survival is equal to one minus probability of yielding or collapse at respective damage state [10]. Return period is calculated using the equation defining relation between PGA of earthquakes and their annual probability of occurrence. Hazard survival curves of single column bent of Hanshin Expressway for first set of ten time histories are shown in Fig. 7 (a) for moderate degradation of section with results summarized in Table VII. Similarly hazard survival curves of single column bent of Hanshin Expressway for second set of ten time histories are shown in Fig. 7 (b) for moderate degradation of section with results tabulated in Table VIII.



(a) For set1 time histories



(b) For set2 time histories

Figure 7. Hazard Survival results

Table VII. Hazard Survival Results for Set 1 Time History

Type of Earthquake	Return period in Years	Probability of survival (%) for Single-column bent (SCB) of Hanshin Expressway	
		At Yield	At Collapse
SE*	75	100	100
DBE	475	98	100
MCE	2475	76	92

*SE: Serviceability Earthquake; DBE: Design Base Earthquake

Table VIII. Hazard Survival Results for Set 2 Time History

Type of Earthquake	Return period in Years	Probability of survival (%) for Single-column bent (SCB) of Hanshin Expressway	
		At Yield	At Collapse
SE	75	100	100
DBE	475	99	100
MCE	2475	62	86

III. CONCLUSIONS AND RECOMMENDATIONS

In this paper, seismic fragility analysis of concrete bridge damaged during Hyogo-ken Nanbu Earthquake of 17 January 1995 has been carried out using SAP2000 and IDARC-2D to establish analytical modeling and analysis procedure.

From incremental dynamic analysis results it is clearly observed that the range of vulnerable PGA(g) for single-column bent of Hanshin Expressway found to be 0.28 to 1.64, for set of time histories other than Kobe earthquake. A pushover analysis result shows vulnerable PGA(g) as 0.721. Flexural failure is observed in both the analysis with proper formation of fiber plastic hinge. For single-column bent of Hanshin Expressway, results and failure patterns obtained in this research study are resembling with actual designed value of PGA(g) i.e. 0.72, flexural failure and recorded PGA(g) i.e. 0.84 at the site. Thus, importance of PBEE is established in this paper. Present study highlights the need of pushover and incremental dynamic analysis to get more realistic seismic behavior of structure. Hence following recommendations are necessary:

- For seismic design of concrete bridges, performance based earthquake engineering must be incorporated in a codal provision, that is, at least pushover analysis should be included based on capacity design concept as per AASHTO guidelines of 2009.
- Use of advanced, well tested and validated computer programs must be insisted to overcome model and computational complexities.
- Two suits of Ground motions, one given by Maniyar and other given by Vamvatsikos are quite sufficient to use for incremental dynamic analysis which covers most of the ground motion parameters.
- This study can be used as guidelines for fragility analysis, damage state definitions and evaluation of damage states or performance levels.
- Seismic assessment and evaluation of existing and upcoming concrete bridges is necessary.

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