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# Seismic Vulnerability Evaluation of Simply Supported Multi Span RCC Bridge Pier

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**Abstract:** This paper presents the seismic vulnerability evaluation of simply supported multi span RCC bridge pier under different ground motions. To determine the seismic performance Nonlinear analysis has been done. Nonlinear static (Pushover) analysis was used to determine the capacity of the bridge pier and seismic demand of the pier was determined from Nonlinear time history analysis.In the time history analysis seismic inputs are given in the form of earthquake time history data. Four numbers of time history data recorded in peer strong motion database has been used in this study.Damage on the pier was determined by using the output of Nonlinear time history analysis and Nonlinear static(Pushover) analysis.The probability of reaching or exceeding the different defined damage states with respect to the input ground motion was determined and the fragility curves are also developed by using the First Order Second Order method(FOSM). Using this fragility curves, it is concluded that there is no shear and flexural failure occured in the bridge pier rather than shear crackings at PGA of 0.4g which is assumed as design PGA in this study for the seismic hazard level of 10% of probability of exceedence in 50 years(475 years return period).

**Keywords:** Bridge, Damage States, Fragility Curves, Pushover Analysis, Seismic Vulnerability, Time History Analysis

## 1. INTRODUCTION

Nepal is situated in an earthquake prone area. The subduction of the Indian plate under the Tibetan plate is considered as the major source of seismicity in this region. Since, Nepal has experienced large earthquakes like Nepal-Bihar Earthquake (1934), Nepal-Sikkim earthquake (2011), great Gorkha earthquake (2015) etc in the past years, the damage of several infrastructures as well as life loss had occurred. But as per the conventional practice in Nepal we are still practicing conservative seismic design process rather than finite element analysis procedure. The San Fernando Earthquake (1971), Loma Prieta Earthquake (1989), Northridge Earthquake (1994), Hanshin-Awaji Kobe Earthquake (1995) and Tohoku Earthquake (Japan) (2011), are few earthquake name to which caused drastic damage to a considerable number of bridges because of lack of design considerations to seismic counteraction forces, had little or no seismic design consideration. So, after these great earthquakes bridges were damaged due to various reasons some of them are failure of the pier due to inadequate shear strength and lack of the lateral confinement of the transverse reinforcement, transverse reinforcement opened up at the lap splicing, pulled out failure due to the inadequate development length of the longitudinal reinforcement into the foundation, flexural-shear failure due to premature curtailment of the longitudinal reinforcement at the mid height of the splicing [1].

Seismic vulnerability assessment and development of fragility curves for the existing bridges are the main concern for some researchers in these days. Karim [2] adopted an analytical approach to construct fragility curves for highway bridge piers of specific bridges based on the numerical simulation considering both structural parameters and the variation of the input ground motion. But for the component level approach Nielson [3] considered the major components like column, bearings and they were assumed as the main contribution to develop the fragilities. These researchers developed the fragility curves for various bridges and concluded the necessity for the evaluation of seismic vulnerability.

A large number of bridges in Nepal were designed and constructed as per IRC codes without considering seismic forces with no shear design criteria. Likewise in the IRC: 6 2000 the flexibility and dynamic behavior of the bridges were not considered in the calculation of design seismic force. IRC recommended seismic coefficient value for the hard soil lies in the zone V becomes 8% but as per the AASHTO LRFD, 2007 [4] value of seismic coefficient lies in the range of 20-30% of the seismic weight of the structure for the same zone and soil condition. In the present scenario some of the IRC codes are amended which includes the previously discussed lag of the seismic coefficient value and other seismic considerations. Newly constructed bridges followed these improved IRC codes. But for the bridges that were constructed using older codes which are still in use and play an important role in the transportation system which may be susceptible to failure due to their structural deficiencies. In these codes design provisions for the columns and compression members didn't include the shear design even under lateral loading conditions such as during earthquakes. Similarly the possibility of buckling of the longitudinal reinforcement was not considered. Thus the incomplete treatment of the shear and transverse reinforcement may degrade the performance of the bridge pier during strong shaking [5]. Each and every bridge has shown their own different characteristics and performance on the seismic loading. In case of Nepal, there is no any frequent seismic vulnerability evaluation of such bridges which may fail due to hit of great earthquakes. So it is very important to evaluate the seismic vulnerability of the bridges.

## 2. RESEARCH OBJECTIVES

- To develop the analytical seismic fragility curve of reinforced concrete bridge pier.
- To determine the capacity of bridge pier for different damage states.
- To evaluate the seismic responses of RCC Bridge pier excited by different ground motion time histories.

## 3. METHODOLOGY

### 3.1 Bridge Description

A reinforced concrete simply supported multi span T-beam Bridge has been selected for the research purpose. All the necessary dimensions of the bridge components used in this study are listed in the Table 1.Also the typical cross section of bridge superstructure is shown in Fig.1.

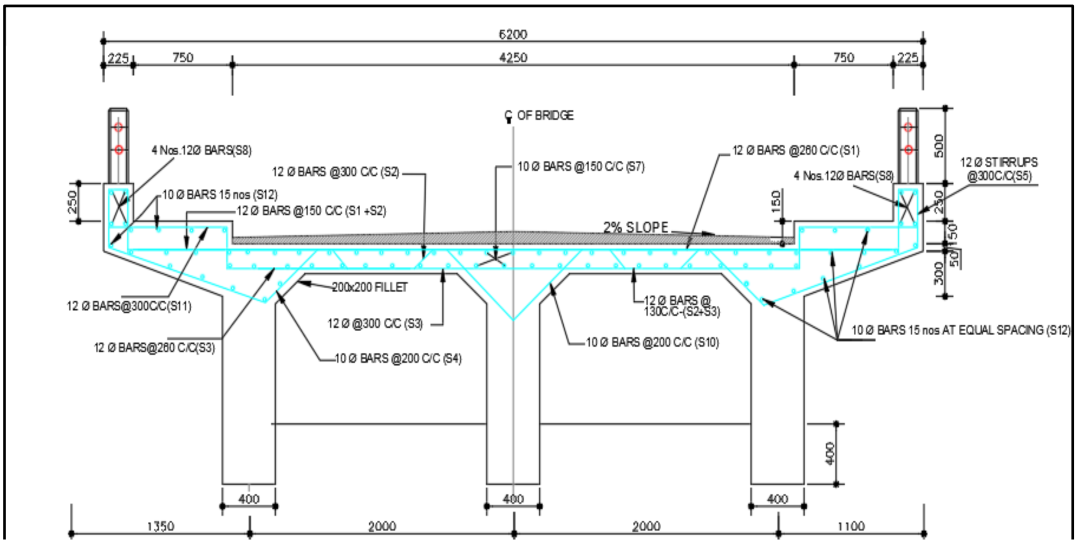


Fig.1 Cross Section of the Bridge Superstructure

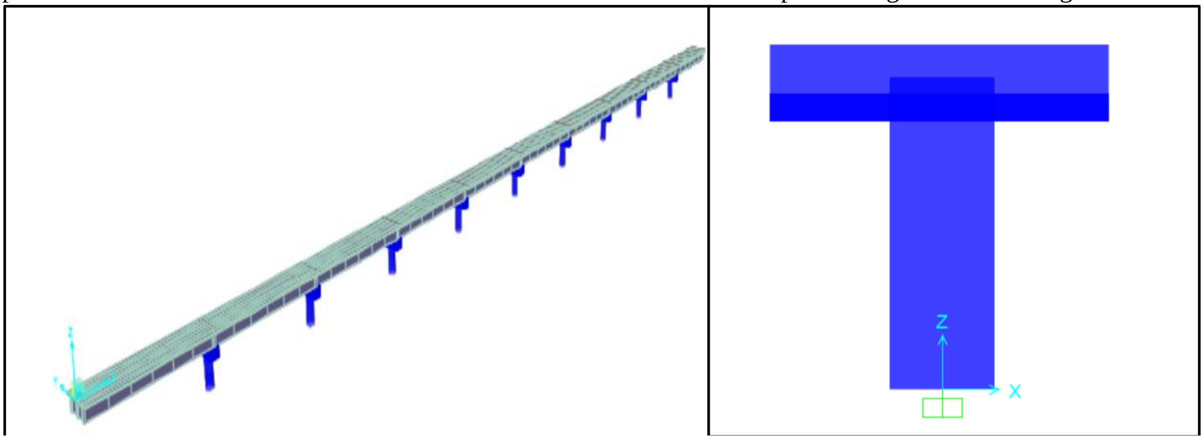
Table 1 Description of the bridge configuration

| S.N | Bridge Components | Data |
|---|---|---|
| 1 | Bridge Name | Lalbakaiya bridge, Rautahat |
| 2 | Overall length of the bridge | 247.55 m |
| 3 | Length of each span | 24.7 m |
| 4 | Effective length of each span | 24 m |
| 5 | Span number | 10 |
| 6 | Diameter of the pier | 1.6 m |
| 7 | Longitudinal reinforcement on pier | 60 no. Ø25mm |
| 8 | Transverse reinforcement | Ø16, @250mm c/c |
| 9 | Overall width of the bridge | 6.2 m |
| 10 | Carriageway width | 4.25 m |
| 11 | Pier cap length | 5.2 m |
| 12 | Pier cap height | 1.25 m |
| 13 | Pier cap width | 1.6 m |

| | | |
|---|---|---|
| 14 | Pier clear height | 4.4 m |
| 15 | Elastomeric bearing size | 0.5×0.4×0.05 m |
| 16 | Depth of the slab | 0.2 m |
| 17 | Depth of the longitudinal girder | 1.8 m |
| 18 | Section of the longitudinal girder | 0.4 m |
| 19 | Section of the cross girder | 0.25 m |
| 20 | Foundation | 6 m diameter well foundation |
| 21 | Bridge live loads | IRC Class A |

### 3.2 Analytical Bridge Modelling

A three dimensional finite element model of the bridge was constructed in the CSI Bridge V20 structural analysis software as shown in the Fig. 2 (a). Bridge was modeled based on the properties of material and geometric dimensions from as built drawings. All the loads and forces carried by the superstructure are transformed through the girder to the pier by means of elastomeric bearing which was modeled as the linear link element. Both longitudinal as well as cross girders, column were modeled as frame element and the bridge deck was modeled as shell element [6]. Cantilever model of the pier considering lumped mass is shown in Fig. 2 (b). In this study, nonlinear analysis of the bridge pier was based upon the lumped mass model. Fiber hinge in the pier section was defined to determine the nonlinear characteristics of the pier during seismic loading.




(a) (b)

Fig.2 Finite Element Modelling of (a) Bridge and (b) Pier from CSI Bridge

### 3.3 Seismic Input

The non-linear response of the structures is very sensitive to the structural modelling and ground motion characteristics. Therefore a set of representative ground motion that accounts for the uncertainties and difference in frequency, severity and the duration characteristics has to be used to predict the possible deformation of the structures for seismic performance evaluation purposes. These ground motions would be actual earthquake records from the region where the structures are located. However actual earthquake records in Nepal are not available remarkably. The ground motions used in this research are shown in the table below. These data were extracted from the PEER Strong motion database. ATC-40 [7] recommends the using of minimum three ground motion to determine the response.

Table 2 Ground Motion Considered for the Seismic Study

| S.N | Earthquake | PGA(g) | Station |
|---|---|---|---|
| 1 | Imperial Valley (1979) | 0.3152 | (Oct.15,1979 USGS Station) |
| 2 | Kobe,1995 | 0.3447 | (Jan. 16,1995 KAKOGAWA(CUE90)) |
| 3 | Loma Prieta (1989) | 0.3674 | (Oct.18,1989 090CDMG Station 47381) |
| 4 | Northridge (1994) | 0.5683 | (Jan. 17,1994 090CDMG Station 24278) |

**3.4 Probabilistic Fragility Function**

The fragility or probability of failure ($P_f$), that the seismic demand (D) exceeds the structural capacity (C) can be described as below. The probability condition on a chosen intensity measure (IM) which represents the level of seismic loading.

$P\{D \geq C/IM\} = P\{C-D \leq 0, IM\}$(1)

This probability is generally modeled as a lognormal probability distribution. In addition, when the structural capacity and demand roughly fit a normal or lognormal distribution, using the central limit theorem, it can be said that the composite performance will be log normally distributed. Thus the fragility curve can be represented by a lognormal cumulative distribution function as follows [8].

$$P_f = \emptyset \left\{ \frac{\ln\left(\frac{S_d}{S_c}\right)}{\sqrt{(\beta_c^2 + \beta_d^2)}} \right\} \quad (2)$$

Where, $S_c$is the median value of the structural capacity defined for the damage state, $\beta c$ is the dispersion or lognormal standard deviation of the structural capacity, $S_d$is the seismic demand in terms of a chosen ground motion intensity parameter, $\beta_d$is the logarithmic standard deviation for the demand and is the standard normal distribution function [1].

The seismic demand is expressed as:

$\ln(S_d) = a \times \ln(X) + b$ (3)

Where a & b are an unknown regression coefficient, X is the ground motion intensity parameter.

**3.5 Damage States**

An important index to express structural performance demand is an acceptable probability of damage. Displacement ductility has been determined from results of nonlinear dynamic analysis based on the force displacement idealization using CSI Bridge. Ultimate ductility is defined as the ratio of maximum displacement to the yield displacement. Further displacement ductility ($\mu_d$) is defined as the ratio of the maximum displacement at the top of the pier obtained by dynamic analysis to the displacement at the yield obtained from the static analysis. The limit states of each damage state in the range slight/minor to the complete collapse state are defined as in the Table 3 below.

Table 3 Bridge Damage States Using Displacement Ductility Ratio [9]

| Damage States | Criteria | Remarks |
|---|---|---|
| Slight/Minor | $\mu_{cy} > \mu_d > \mu_{cy1}$ | $\mu_{cy1}$, First reinforcement yield displacement ductility ratio |
| Moderate | $\mu_{c2} > \mu_d > \mu_{cy}$ | $\mu_{cy}$, Yield displacement ductility ratio |
| Extensive | $\mu_{cmax} > \mu_d > \mu_{c2}$ | $\mu_{c2}$, Displacement ductility ratio with $\varepsilon_c = 0.002\left(\frac{f'_{cc}}{f'_c} - 1\right)$ |
| Collapse | $\mu_d > \mu_{cmax}$ | $\mu_{cmax}$, Maximum displacement ductility ratio |

## 4. ANALYSIS AND RESULTS

**4.1Modal Analysis**

This analysis was used to determine the fundamental time period and natural mode shapes of the structure in the free vibration. Fundamental time period of vibration of the bridge was found to be 0.947 sec.

**4.2 Nonlinear Static (Pushover) Analysis**

Pushover or non-linear static analysis is basically used to determine the capacity of the bridge pier. Simple cantilever model of the bridge pier was formed in which the half of the mass from each span of superstructure was lumped at the top. The non-linear behavior was characterized by fiber PMM hinge. The pier section at the plastic hinge zone was characterized by 61 number of reinforcing bar fiber, 250 concrete and 61 numbers of covered concrete fibers defined in a circular patch.

The analysis was done as per displacement controlled approach. First of all, target displacement was given to 10 mm and then increased up to 225mm at an interval of 10mm. In the pushover analysis primarily all the fibers are in compression under the action of gravity load. After the horizontal displacement of the pier top, the compressive strain of some fibers increases gradually and some have shown the decreasing and finally tensile (Unloading of compressive and loading of tensile). So the strain value of each was monitored.

The analysis gives the pushover curve which was plotted base shear against pier top displacement. The pier capacity at each damage state was also derived. From the pushover result the maximum base shear was found to be 3295.83KN and the maximum pier displacement was found 224.3mm respectively as shown in Fig.3 below.

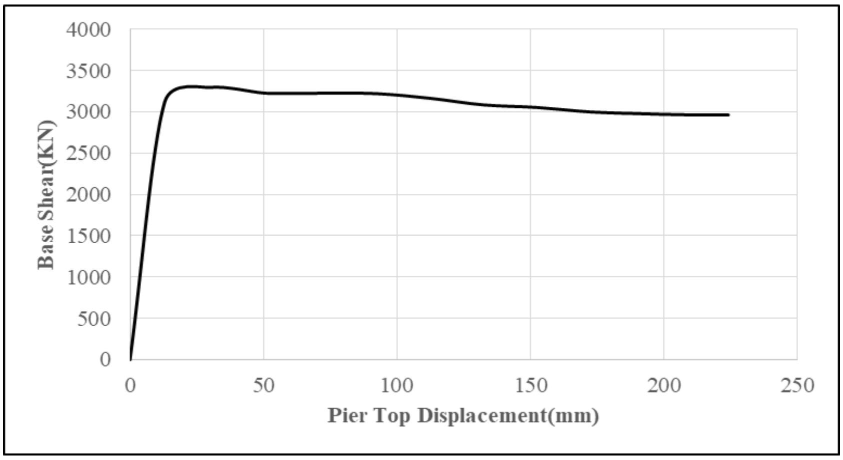



Fig. 3 Pushover Curve of the Bridge Pier

**4.3 Response Analysis**

Nonlinear time history analysis was carried out to determine the response of the bridge pier. The applied seismic input as in the form of the ground motion time histories in the foundation level are as shown in Table 2. The time histories are rescaled from 0.1g to 1.6g in an increment of 0.1g generating altogether 16 numbers of the time history for each single recorded ground motion. Four numbers of recorded ground motion time histories were used in this research work namely Imperial Valley, Kobe, Loma Prieta, and Northbridge. The time histories are applied along the transverse direction of the bridge pier. There are altogether 64 numbers of time history data and the pier top displacement is recorded for each dynamic load case. As the height of the column is less, only the material nonlinearity was considered in this research. The displacement time histories obtained from the input ground motions were used.

**4.4 Regression Analysis**

The maximum displacement of the pier top was obtained from time history analysis. Though the obtained displacement and yield displacement value, displacement ductility for sixteen PGA values of each time history was calculated.

Yield displacement of the section was calculated as [10]

$\Delta_y = Ø_y \times L^{2/3}$(4)

Where, $Ø_y$ is idealized yield curvature defined by an elastic-perfectly plastic representation of the cross section M-Ø curve.L is the distance from point of maximum moment to the point of the contra flexure.

The displacement ductility was plotted against peak ground acceleration in natural logarithmic scale. Regression analysis was then carried out to get the probabilistic seismic model presented in Fig. 4.

$\ln(\mu_d) = 1.183 \times \ln(PGA) + 1.5829$ (5)

The relation obtained as in equation (5) from regression analysis was used to calculate the displacement ductility for arbitrary PGA of input ground motions.

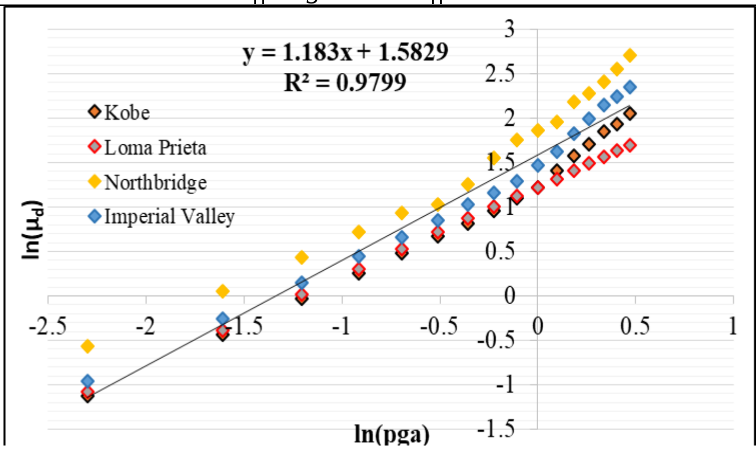



Fig. 4 Seismic Response of the Bridge Pier

**4.5 Development of the Fragility Curves**

First order second moment method was used to find the probability of failure. Response calculated as per the equation (5) was assumed to be mean value of the input peak ground motion acceleration. Capacities calculated from pushover analysis were taken as the mean capacities value for different damage states. Value of composite standard deviation, $\sqrt{(\beta_c^2+\beta_d^2)}$ were taken as 0.55 for slight and moderate damage states and 0.7 for extensive and collapse damage states respectively [11] Which is the combined uncertainty factor representing the sum of square roots of standard deviation of both capacity and demand.

Probability of reaching or exceeding different four damage states for input peak ground motion acceleration was then calculated. For the continuation of the fragility curves, input ground motion was started from 0.05g at an interval of 0.05g up to 2g. Fragility curves have been developed for different damage states presented in Fig. 5. The time history used in this analysis contains seismic hazard level 10% probability of exceedance in 50 years (return period 475 year). For the PGA of 0.4g [12] probability of failure for bridge piers was found to be 51.42%, 6.38%, 3.85% and 0.039% corresponding to a slight, moderate, extensive and complete damage level. The bridge piers had 97.75%, 67.21%, 28.46%, 3.52% and 99.79%, 90.63%, 51.1%, 13.075% probability of slight, moderate, extensive and complete damage for peak input ground motion acceleration of the 1.0g and 1.5g respectively.

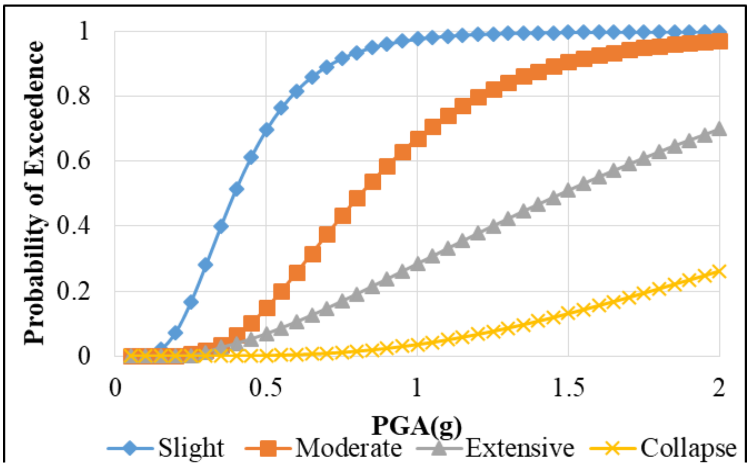



Fig.5Fragility Curve of the Bridge Pier for Different Damage States

## 5. CONCLUSION

From the above results it is observed that

1) The probability of failure of bridge pier for different damage states with different peak ground acceleration are found and tabulated as follows.

Table 4 Probability of failure

| PGA(g) | Probability of Reaching or Exceeding Damage States | | | |
|---|---|---|---|---|
| | Slight | Moderate | Extensive | Collapse |
| 0.4 (For seismic hazard level 10% probability of exceedance for 50 years) | 51.42% | 6.38% | 3.85% | 0.04% |
| 1.0 | 97.75% | 67.21% | 28.46% | 3.52% |
| 1.5 | 99.79% | 90.63% | 51.10% | 13.08% |

2) The acceptable PGA for target 5% probability of failure for bridge pier are found to be 0.18g, 0.37g, 0.45g and 1.1g for slight, moderate, extensive and complete damage respectively.
3) It is concluded that there is no shear and flexural failure occurred in the bridge pier rather than shear cracking at PGA of 0.4g which is assumed as design PGA for the seismic hazard level of 10% probability of exceedance in 50 years (475 years return period) in this study.
4) These curves are very important for making authentic decisions in need of pier retrofitting or replacement, future response planning and loss estimation of the bridge due to strong hit of ground motion.

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