# Development of Fragility Curves for Seismic Performance Comparison of Hammerhead and Multicolumn Bridge Pier

**Bhabuk Raj Aryal[1], Nabin Chandra Sharma[2]**

[1]Department of Civil Engineering, Institute of Engineering, Tribhuvan University, Lalitpur, Nepal

[2]Department of Civil Engineering, Institute of Engineering, Tribhuvan University, Lalitpur, Nepal

**Abstract:** ***In this research, it is primarily intended to assess seismic vulnerability of bridge piers that are commonly used in context of Nepal. Seismic vulnerability are compared for a double span bridge with a typical span of 25m. These two pier sections are analysed in finite element based software SAP 2000. Displacement capacity is determined by non-linear static (Pushover) analysis. Capacity analysis of the pier section shows that the top level displacement is more in case of hammerhead pier than multicolumn pier section. Response analysis is done using time history data of Gorkha earthquake to calculate the demand, after this analysis the top level displacement corresponding to the subjected ground motion is more in case of hammerhead pier than multicolumn pier section. The seismic vulnerability of piers are determined by developing analytical fragility curves. Using the fragility curves, it is concluded that for 0.45g PGA the probability of failure corresponding to slight, moderate, extensive and complete damage are 71.2%, 12.7%, 4.6% & 3.4 % for hammerhead bridge piers and 56.3%, 9.1% 3.5 % & 2.1 % for multicolumn bridge piers.***

**Keywords:** fragility curve; multicolumn pier; hammerhead pier; capacity analysis; Seismic vulnerability

## 1. Introduction

Nepal is seismically vulnerable country because it lies in subduction zone of India-Australia and Eurasian plate. After the Gorkha Earthquake, April 29th 2015, it is clear that Nepal has the danger of huge earthquake in coming days too.

For any land transportation system bridges are the most susceptive components, which was illustrated in some previous earthquake like 1971 San Fernando earthquake, the 1994 Northridge earthquake. Following an earthquake, the highway transportation systems might not be fully functional for a long period, and the regional economy might suffer significantly (Werner, Jernigan et al. 1995; Basoz and Kiremidjian 1998; Shinozuka, Moore et al. 1998)

Moreover, in the context of Nepal, we do not have a specific code of practice for design and construction of highway bridges, most of them are mainly borrowed from India. Indian design codes , Indian Road Congress (IRC) standard specification and code of practice of road bridges treat reinforced concrete bridge piers as gravity load carrying compression member, and no provisions are available for their shear design (GOSWAMI and Murty 2003) except the detailing procedures. The motivation behind this research stems from the recognition of this fact.

For seismic assessment Fragility curve, which is a conditional probability statement of a bridge's vulnerability as a function of ground motion intensity, is used in this research. Many works has been done in the field of fragility curve generation by researchers like (Karim and Yamazaki 2001; Liao and Loh 2004; Kurian, Deb et al. 2006; Nielson and DesRoches 2007; Kibboua, Naili et al. 2011) . Fragility curve methodology using analytical approaches is gaining popularity in the field of seismic analysis because this method can be used to different bridge types and places with insufficient seismic records.

Pushover analysis determines the capacity of the bridge pier. Gorkha earthquake time history data is taken as seismic input and is normalized to different excitation levels. Nonlinear dynamic response analysis is performed using the scaled time histories. Displacement ductility factors are calculated. Regression analysis is done, and the fragility curves are developed using the ground motion and damage parameters.

The purpose of this paper is to develop analytical fragility curves for a double span bridge with a single span of 25m. The standard bridge drawings are published by Local Roads Bridge program (LRBP), Local Roads Bridge Support Unit (LRBSU).

## 2. Methodology

The global finite element model of the bridge pier structures prepared using finite element based software SAP 2000. The response of the pier is to be same as the response of overall bridge and suggested that a bridge can be represented by single column with tributary mass from the adjacent half spans of the superstructure (Priestley, Seible et al. 1996). In the current research only the pier section is modeled.

### 2.1 Capacity Analysis

For capacity analysis, the nonlinear static (Pushover) analysis is performed to determine the capacity of the bridge pier. The analysis is done in displacement controlled i.e. target displacement of pier top is given depending upon the geometry of pier. The displacement is increased at the interval of 1mm up to 500mm for hammerhead pier and 0.7mm up to 350 mm for multicolumn. The nonlinear behavior is represented by fiber hinge. The fibers are arranged in a circular patch for both piers. The stress-strain relationship is assigned for each fiber and strain value of extreme reinforcement, cover concrete and core concrete fibers is monitored. For Pushover analysis monotonically increasing lateral force is applied to each pier section, this force will be similar to the inertial force experienced by the structure when subjected to ground shaking. The pushover analysis is refined to get exact capacities. The damage states are defined in four categories namely slight, moderate, extensive and complete damage. A quantitative definition of damage states is set considering the mechanical properties of reinforcing steel and concrete as presented in (FEMA 2003). The displacement ductility is calculated as per the eq. **Error! Reference source not found.** (Caltrans 2004)

$$Displacement\ Ductility(\mu_d) = \frac{\Delta_T}{\Delta_Y} \quad (1)$$

Where $\Delta_T$ is the displacement of pier top, $\Delta_Y$ is the displacement corresponding to first yield of reinforcing steel.

### 2.2 Response Analysis

Seismic input for structural analysis is provided either in time domain or frequency domain or in both time and frequency domain. The most common way to describe a ground motion is with a time history record (Datta 2010).For Response analysis, Time history analysis is carried, the seismic input is in the form of ground motion time history at the surface. Gorkha earthquake, 2015 is used as time history data for this analysis, peak ground acceleration 0.177g and duration of 55 sec. The time history is rescaled from 0.1g to 2.0g in an increment of 0.1g, generating twenty number of time histories with varying peak ground acceleration. (Bommer, Acevedo et al. 2003) Linear scaling of the amplitude of records is acceptable, in particular for those records of earthquakes with a similar magnitude to that of the earthquake scenario, since the shape of response spectrum is not highly sensitive to distance. The bridge pier model using fiber hinge is considered for the modeling nonlinear behavior of the pier. The analysis is carried out for each of twenty time histories and pier top responses (Displacements) are recorded for earthquake data.

### 2.3 Fragility Analysis

Fragility theory is a generalized branch of structural reliability which assesses the vulnerability of a structure conditioned upon some other input parameter. Fragility curves demonstrate the probability of a bridge reaching or surpassing a ground motion for a precise damage state. Moreover, this curve facilitates the process of overall seismic risk assessment of a transportation network. The Fragility analysis generally includes three major parts:

a) The simulation of the ground motion,
b) The simulation of the bridge,
c) The generation of fragility curves from the seismic response data.

The seismic response data can be obtained from time history analysis, elastic spectral analysis, or nonlinear static analysis. For present research, time history analysis is performed. For seismic loading, the fragility simply looks at the probability that the seismic demand placed on the structure (D) is greater than the capacity of the structure (C). This probability statement is conditioned on a chosen intensity measure (IM) which represents the level of seismic loading. The generic representation of this conditional probability is given as (Nielson and DesRoches 2007)

$$Fragility = P[D \geq C / IM] = P[C - D \leq 0.0 / IM] \quad (2)$$

Evaluation of this equation is most easily accomplished by developing a probability distribution for the demand conditioned on the IM, also known as a probabilistic seismic demand model (PSDM), and convolving it with a distribution for the capacity. The demand on the structure is quantified using the parameter-displacement ductility. (Cornell, Jalayer et al. 2002) suggested that the estimate for the median demand ($S_d$) can be represented by a power model as

$$S_d = aIM^b \quad (3)$$

Where, IM is the seismic intensity measure of choice and, both a & b are regression coefficients.

The actual regression used to estimate the parameters a and b from Eq. (**Error! Reference source not found.**) is more easily facilitated in a transformed natural logarithmic space.

$$\ln(s_d) = a\ln(IM) + b \quad (4)$$

The structural reliability is calculated using first order second moment method. In this method random variables are characterized by their first and second moments. In evaluation the first and second moments of the failure function say mean and standard deviation the first order Taylor's approximation is used. That is why these methods are called first order second moment methods. In order to calculate the reliability limit state equation is defined as

$$M = S_c - S_d \quad (5)$$

In which C is capacity, D is response of bridge pier. M is margin of safety.

$$P_f = P\left[\frac{S_d}{S_c} \geq 1\right] \quad (6)$$

Fragility is often modeled by lognormal cumulative distribution function where structural response and capacity are assumed to be log normally or normally distributed. Thus, closed form solution for the fragility may be presented by the equation (Ranganathan 1990)

$$P_f = \Phi\left(\frac{\ln\left(\frac{S_d}{S_c}\right)}{\sqrt{\beta_d^2 + \beta_c^2}}\right) \quad (7)$$

Where $S_c$ is the median value of structural capacity defined for the damage state and presented in Table 4, $S_d$ is the seismic demand in terms of chosen ground motion intensity parameter and $\sqrt{\beta_d^2 + \beta_c^2}$ is known as the dispersion which incorporates logarithmic aspect of uncertainty and randomness for both capacity and demand. Dispersion of damage states for the pier response (Hazus 1997) is 0.5, 0.55, 0.7,0.7 which correspond to slight, moderate, extensive and complete damage respectively.

### 2.4 Damage states

Quantitative definition of damage states is set considering the mechanical properties of reinforcing steel and concrete. Definitions of qualitative damage states are available in (FEMA 2003); same are used for disaggregation of the vulnerability of bridge piers. No damage and slight damage are placed in the same category. The quantitative assignment is made considering the qualitative definitions in (FEMA 2003). First yielding of extreme fiber reinforcing steel in cross section of bridge piers is defined as the slight damage state. The compressive strain in the pier section as per respective damage state is calculated as per equations defined in (Mander, Priestley et al. 1988).

Table 1 provides different damage states with their quantitative definitions.

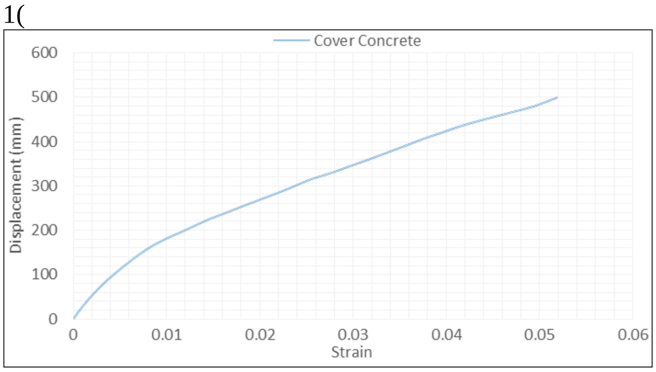
**Table 1:** Damage State Definition of Bridge Pier

| S.N | Damage State | Qualitative Definition in HAZUS 2003 | Quantitative Definition | Strain in concrete | |
|---|---|---|---|---|---|
| | | | | Hammer head Pier | Multi column Pier |
| 1 | Slight | Minor Spalling in Column | First yielding of extreme reinforcement bar | 0.002 | 0.002 |
| 2 | Moderate | Spalling in Column | Maximum compressive strain at cover concrete | 0.0020 | 0.002 |
| 3 | Extensive | Column Degrading without Collapse | Maximum compressive strain at core concrete | 0.00207 | 0.00207 |
| 4 | Complete | Column Collapsing | Ultimate Compressive strain at core concrete | 0.00428 | 0.0042 |

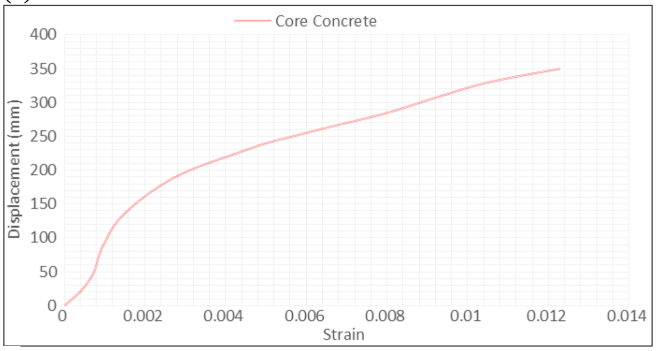
## 3. Result and Discussion

The peak response and displacement ductility for each peak response quantity is calculated and presented in Table 4. The displacement ductility are plotted against peak ground acceleration in natural logarithmic scale to show pier response in power model. Regression analysis is carried out to get the probabilistic seismic response model. One key output from the current research is seismic response model shown in
Table 2

Pier top displacement, for damage states and displacement ductility is present in Table 4. Displacement versus strain for the extreme reinforcement, cover concrete and core concrete fibers are plotted and shown in Figure 1(




(a)-




(f)). The plots are used for estimating the capacity of pier corresponding to strain value mentioned in
Table 1 The capacity analysis of the pier section shows that the top level displacement corresponding to the slight, moderate, extensive and complete damage is more in case of hammerhead pier compared with multicolumn pier section. Using this response model, displacement ductility can be calculated for arbitrary PGA of input ground motion using (Caltrans 2004) guidelines and Eqs. (**Error! Reference source not found.**)-(**Error! Reference source not found.**) . The response analysis of the pier section shows that the top level displacement corresponding to the subjected ground motion is more in case of hammerhead pier compared with multicolumn pier section.

For fragility analysis, response calculated using
Table 2 is taken as the seismic demand for the corresponding input PGA and capacities presented in
Table 1**Error! Reference source not found.** is the considered as median value of structural capacity for different damage states. Combined uncertainty factor representing square root of sum of squares of standard deviation of capacity and response is adopted from (Hazus 1997). To generate a continuous curve, calculation is done starting from 0.05g and increasing to 2.0g at an interval of 0.05g.

Fragility curves for different damage states in case of hammerhead and multicolumn pier are presented in Figs.1-**Error! Reference source not found.**Figure 1: Fragility Curve for Hammerhead Bridge Pier. From the seismic hazard analysis map of Nepal, its shows that PGA for 10% probability of exceedance in 50 years (return period 475 years) is expected to be 0.45g in Kathmandu Valley. Information on the PGA for another return period is not available, so two arbitrary peak ground acceleration 1.0g and 1.5g are taken for stating probability of failures.
For 0.45g PGA, the probability of failure corresponding to slight, moderate, extensive & complete damage are 71.2%,12.7% , 4.6% & 3.4 % in case of hammerhead bridge piers and 56.3%, 9.1% 3.5 % & 2.1 % for multicolumn respectively. It is demonstrated that seismic vulnerability of the bridges can be quantified with the help of fragility curves. The curves, when read along with seismic hazard map of bridge location, will provide the probability of failures for different damage states namely slight, moderate, extensive and complete. Large bridge stock demanding retrofit and replacement can be prioritized determining the probability of failure corresponding to desired damage states. Therefore, the curves are useful for making rational

International Journal of Science and Research (IJSR)
ISSN: 2319-7064
ResearchGate Impact Factor (2018): 0.28 | SJIF (2018): 7.426

decisions on the necessity of strengthening or replacement of existing bridges. The curves are equally useful for pre-disaster planning and loss estimation of bridge stock due to potential earthquake disaster.

**Table 2:** Regression Equation for Gorkha Earthquake

| Pier Type | Regression Equation |
|---|---|
| Hammerhead | $\ln(\mu_d) = 1.004 \ln(PGA) + 1.2567$ |
| Multicolumn | $\ln(\mu_d) = 0.996 \ln(PGA) + 1.18$ |

**Table 3:** Summary of Peak Response and displacement ductility of bridge pier

| PGA | Hammerhead | | | Multicolumn | | |
|---|---|---|---|---|---|---|
| | Max Displacement (mm) | $\mu_d$ | ln ($\mu_d$) | Max Displacement (mm) | $\mu_d$ | ln($\mu_d$) |
| 0.1 | 14.05 | 0.35 | -1.05 | 12.24 | 0.33 | -1.11 |
| 0.2 | 28.11 | 0.7 | -0.35 | 24.5 | 0.66 | -0.41 |
| 0.3 | 42.11 | 1.05 | 0.05 | 36.72 | 0.99 | -0.01 |
| 0.4 | 56.21 | 1.41 | 0.34 | 48.98 | 1.32 | 0.28 |
| 0.5 | 70.26 | 1.76 | 0.56 | 61.21 | 1.65 | 0.5 |
| 0.6 | 84.32 | 2.11 | 0.75 | 73.74 | 1.99 | 0.69 |
| 0.7 | 98.37 | 2.46 | 0.9 | 85.71 | 2.31 | 0.84 |
| 0.8 | 112.4 | 2.81 | 1.03 | 97.96 | 2.64 | 0.97 |
| 0.9 | 126.5 | 3.16 | 1.15 | 110.2 | 2.97 | 1.09 |
| 1 | 140.5 | 3.51 | 1.26 | 112.4 | 3.03 | 1.11 |
| 1.1 | 154.6 | 3.87 | 1.35 | 134.7 | 3.63 | 1.29 |
| 1.2 | 168.6 | 4.22 | 1.44 | 146.9 | 3.96 | 1.38 |
| 1.3 | 182.7 | 4.57 | 1.52 | 152.2 | 4.1 | 1.41 |
| 1.4 | 196.7 | 4.92 | 1.59 | 171.4 | 4.62 | 1.53 |
| 1.5 | 210.8 | 5.27 | 1.66 | 183.7 | 4.95 | 1.6 |
| 1.6 | 224.8 | 5.62 | 1.73 | 195.9 | 5.28 | 1.66 |
| 1.7 | 238.9 | 5.97 | 1.79 | 195.9 | 5.28 | 1.66 |
| 1.8 | 253 | 6.33 | 1.84 | 220.4 | 5.94 | 1.78 |
| 1.9 | 267.4 | 6.69 | 1.9 | 232.6 | 6.27 | 1.84 |
| 2 | 281.8 | 7.05 | 1.95 | 244.9 | 6.6 | 1.89 |

**Table 4:** Capacity Calculation of Bridge

| Damage State | Definition | Pier Type | Displacement (mm) | Displacement Ductility | Median Ductility |
|---|---|---|---|---|---|
| Slight/ Minor Damage | First yielding of extreme reinforcement bar | Hammerhead | 40.00 | 1 | 1.1625 |
| | | Multicolumn | 37.10 | 1 | 1.349057 |
| Moderate | Maximum compressive strain at cover concrete | Hammerhead | 53.00 | 1.325 | 2.9625 |
| | | Multicolumn | 63.00 | 1.698113 | 3.066038 |
| Extensive | Maximum compressive strain at core concrete | Hammerhead | 184.00 | 4.6 | 5.1375 |
| | | Multicolumn | 164.50 | 4.433962 | 5.254717 |
| Complete | Ultimate Compressive strain at core concrete | Hammerhead | 255.00 | 5.675 | 5.675 |
| | | Multicolumn | 225.40 | 6.075472 | 6.075472 |

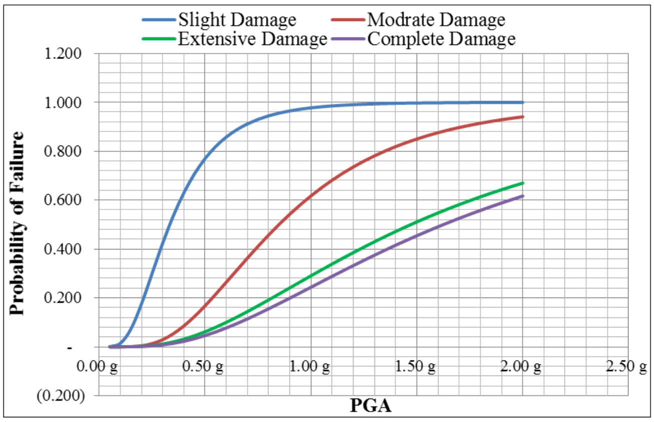



**Figure 1:** Fragility Curve for Hammerhead Bridge Pier

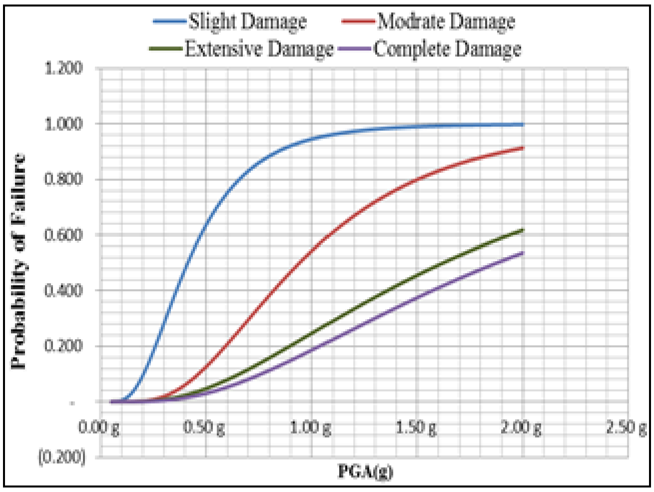
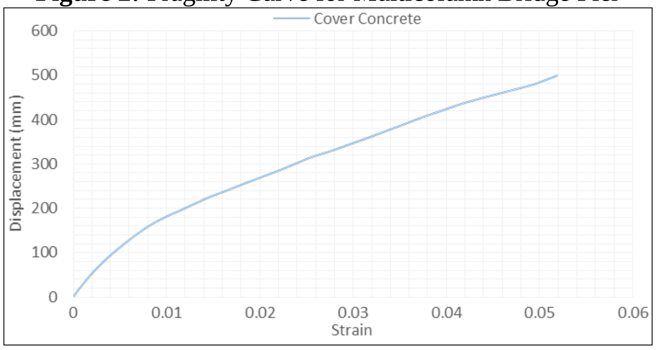
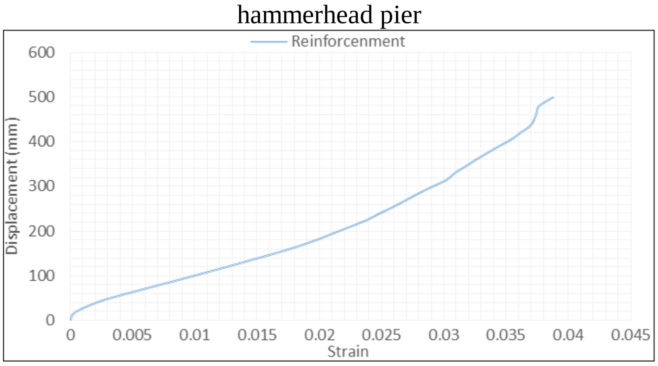


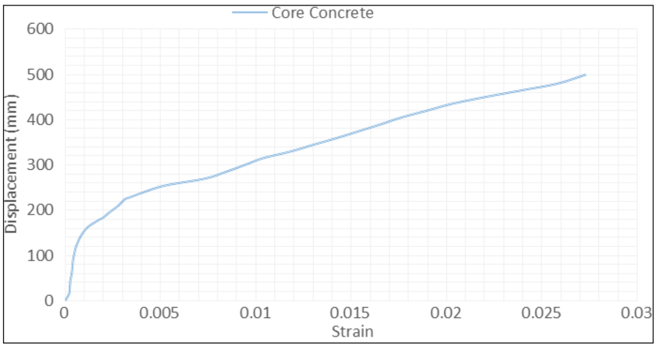

**Figure 2:** Fragility Curve for Multicolumn Bridge Pier



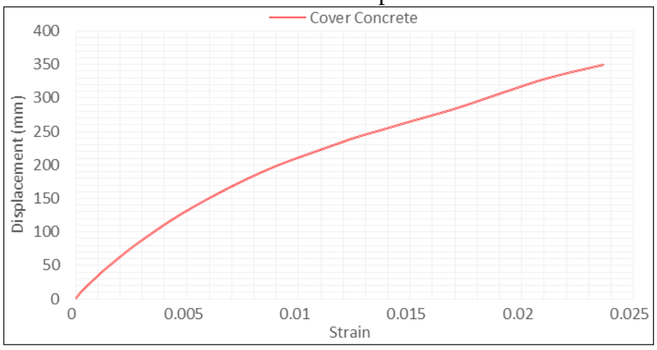

(a) Displacement versus strain for cover concrete of hammerhead pier



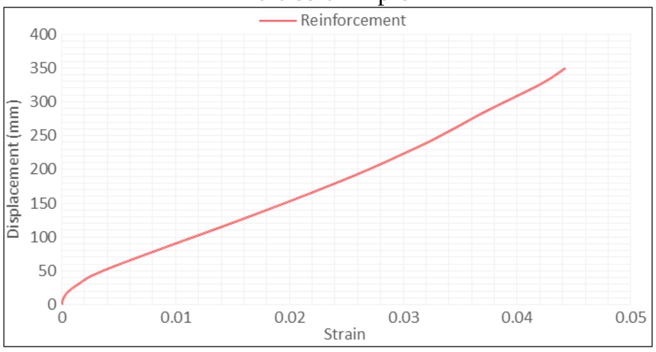

(b) Displacement versus strain for reinforcement of hammerhead pier



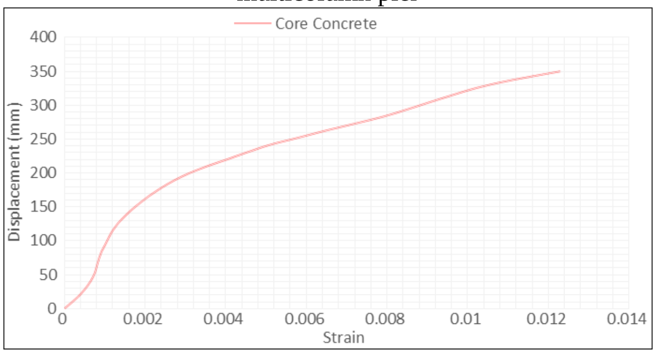

(c) Displacement versus strain for core concrete of hammerhead pier




(d) Displacement versus strain for cover concrete of multicolumn pier




(e) Displacement versus strain for reinforcement of multicolumn pier




(f) Displacement versus strain for core concrete of multicolumn pier

## 4. Conclusion

The fragility curves can be used in finding the probability of failure of the bridge piers corresponding to input peak ground acceleration of desired return period. The following major conclusions are drawn from the current research.

- The capacity analysis and response analysis of the pier section shows that the top level displacement is more in case of hammerhead pier than multicolumn pier.
- Seismic vulnerability of the bridge pier can be quantified with the help of fragility curves. The curves when read along with seismic hazard map of bridge location will provide the probability of failures for different damage states namely slight, moderate, extensive and complete. For 0.45g PGA, the probability of failure corresponding to slight, moderate, extensive and complete damage are 71.2% ,12.7% , 4.6% & 3.4 % for hammerhead bridge piers; 56.3%, 9.1% 3.5 % & 2.1 % for multicolumn.
- From the point of view of probability of failure multicolumn pier has less risk than hammerhead pier.

## 5. Other Recommendations

The authors would like to acknowledge all the faculties, the staff of department of Civil Engineering, Pulchowk Campus Tribhuvan University for their help and support.

## References

[1] Basoz, N. and Kiremidjian, A.S. (1998), *Evaluation of bridge damage data from the Loma Prieta and Northridge, California earthquakes*,

[2] Bommer, J., Acevedo, A. and Douglas, J. (2003). "The selection and scaling of real earthquake accelerograms for use in seismic design and assessment", *Proceedings of ACI International Conference on Seismic Bridge Design and Retrofit, American Concrete Institute*.

[3] Caltrans, S. (2004), *Caltrans Seismic Design Criteria*, Version

[4] Cornell, C.A., Jalayer, F., Hamburger, R.O. and Foutch, D.A. (2002), "Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines", *Journal of Structural Engineering*. **128**(4), 526-533.

[5] Datta, T.K. (2010), *Seismic analysis of structures*, John Wiley & Sons

[6] FEMA, H.-M. (2003), "MR3 Technical Manual", *Multi-hazard Loss Estimation Methodology Earthquake Model*.

[7] GOSWAMI, R. and Murty, C. (2003), "Seismic shear design of rc bridge piers: Part I: Review of code provisions", *Indian concrete journal*. **77**(6), 1127-1133.

[8] Hazus, E.L.E.M. (1997), "Technical manual", *National Institute of Building for the Federal Emergency Management Agency, Washington (DC)*.

[9] Karim, K.R. and Yamazaki, F. (2001), "Effect of earthquake ground motions on fragility curves of highway bridge piers based on numerical simulation", *Earthquake engineering & structural dynamics*. **30**(12), 1839-1856.

[10] Kibboua, A., Naili, M., Benouar, D. and Kehila, F. (2011), "Analytical fragility curves for typical Algerian reinforced concrete bridge piers", *Structural Engineering and Mechanics*. **39**(3), 411-425.

[11] Kurian, S.A., Deb, S.K. and Dutta, A. (2006). "Seismic vulnerability assessment of a railway overbridge using fragility curves", *Proceedings of Fourth International Conference on Earthquake Engineering. Taipei, Taiwan*.

[12] Liao, W.I. and Loh, C.H. (2004), "Preliminary study on

the fragility curves for highway bridges in Taiwan", *Journal of the Chinese Institute of Engineers*. **27**(3), 367-375.
[13] Mander, J.B., Priestley, M.J. and Park, R. (1988), "Theoretical stress-strain model for confined concrete", *Journal of structural engineering*. **114**(8), 1804-1826.
[14] Nielson, B.G. and DesRoches, R. (2007), "Seismic fragility methodology for highway bridges using a component level approach", *Earthquake Engineering & Structural Dynamics*. **36**(6), 823-839.
[15] Priestley, M.N., Seible, F. and Calvi, G.M. (1996), *Seismic design and retrofit of bridges*, John Wiley & Sons
[16] Ranganathan, R. (1990), *Reliability analysis and design of structures*, Tata McGraw-Hill
[17] Shinozuka, M., Moore, J., Gordon, P., Richardson, H., Chang, S. and Cho, S.-b. (1998), *An integrated model of highway networks and the spatial metropolitan economy: towards a general model of how earthquake losses affect the economy*, US. National Center for Earthquake Engineering Research (NCEER)
[18] Werner, S., Jernigan, J., Taylor, C.E. and Hwang, H. (1995), "Seismic vulnerability assessment of highway systems", *NCEER Bulletin*. 9(4), 1-11

## Author Profile

**Bhabuk Raj Aryal** received the Bachelors of Civil Engineering and Msc in structural Engineering from Institute of Engineering, Tribhuvan University in 2012 and 2016. He is now working in the consulting field in Nepal.