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Development of Analytical Fragility Curves to Assess Status of Existing Two-Span RCC Bridge in Nepal

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Abstract

This paper presents the seismic vulnerability of a typical two spanned RCC T-girder bridge in Nepal with a span length of 25m and a single circular pier. The bridge is modeled on finite element analysis-based program OpenSees. Ten different earthquake motions are considered for the non-linear time history analysis of the bridge with the drift ratio taken as the parameter to check the damage states (DS) and the performance limits. The pier of the bridge is taken as the most critical component and the fragility curves are developed considering the non-linear deformation concentrated in the pier only. The study suggests that for the ground motion with spectral acceleration of 0.5g, the bridge is susceptible to moderate damage at forty nine percent probability of exceedance, to extensive damage at twenty six percent probability of exceedance and complete damage at eighteen percent probability of exceedance.

Keywords

Seismic Vulnerability Assessment, Fragility curves, OpeenSees, RCC T-grider bridge

1. Introduction

Bridges are the prominent components of the road transportation network whose failure results in tremendous economic losses for the nation. The losses may be direct losses for the re-construction or repair and retrofit of the damaged bridge and indirect losses due to obstruction of road networks and restriction of economic activities. Failure of bridges due to earthquake exacerbate the situation as it can cause the obstruction of important roadways and hinder emergency vehicles from reaching the crisis zone. With immense resources and capital at stake, a rational decision-making criterion is required for pre and post disaster management for the numerous bridges in the nation. Fragility curves can become such rational criteria as the potential losses due to the earthquake can be predicted beforehand and the retrofit prioritization of the bridges can be planned [1]. Moreover, the curves can also help to prepare the robust retrofit method and develop the plans for the post-event action. [2]. Fragility curves are emerging tools for the seismic vulnerability assessment of highway bridges [1]. They are conditional probability statements that give the probability of a bridge reaching or exceeding a particular damage level for an earthquake of a given intensity level. The fragility curves can be developed through various methods: empirical method, experimental method and analytical method. The empirical method is subjective and provides different curves for different inspection teams. The experimental method is limited and uncommon due to the requirement of sophisticated larger laboratories. Analytical method can be done through linear static, non-linear static, linear dynamic and non-linear dynamic procedures. Although nonlinear time history analysis is the most computationally expensive method, the reliability of this method for the generation of fragility curves is high and is greatly preferred [3]. Non-linear time history analysis method is selected for this study. OpenSees finite element (FE) platform was utilized with scripts written in Tcl programming language. The developed OpenSees model is also validated with the available experimental data. The validated model is extended to analyze a typical two spanned RCC T-girder bridge of Nepal. A total of 9 near-fault earthquakes are selected for non-linear time-history analysis and a representative fragility curve for the selected bridge type is developed.

Bridge piers are one of the most vulnerable components of the bridge as they tend to enter the nonlinear deformation range under severe earthquake loads [4]. This paper formulates the analytical fragility curve for the bridge pier of a typical two spanned RCC T-girder bridge of Nepal in order to assess the vulnerability of the pier. Damage States (DS) and Intensity Measure (IM) are to be quantified to generate the fragility curves. IM is the representative of the seismic loading on the structure with which seismic demand is estimated and compared with the estimated capacity of the structure to establish the fragility curves. DS is the level of damage the structure is expected to undergo under the representative IM as per the conditions of the site of the bridge.[5] recommended the PGA as the optimum IM to describe the severity of the earthquake ground motion. [6] recommended the drift ratio as the demand parameter for the piers and suggested threshold values for different damage states through experiment on real-size bridge pier as shown in Table1.

Table 1: Proposed Damage States Using Column Drift Limits

Damage State	Description	Drift Limits
Almost no damage	First yield	0.005
Slight damage	Cracking and spalling	0.007
Moderate damage	Loss of anchorage	0.015
Extensive damage	Incipient column collapse	0.025
Complete damage	Column collapse	0.050

[7] suggests the probability function that the given structure exceeds the limit damage state for given ground motion is provided by equation 1.

$$P\left(\frac{LS_i}{GMI}\right) = 1 - \phi\left(\frac{\lambda_{CL}^{i} - \lambda_{\frac{D}{GMI}}}{\beta_{\frac{D}{GMI}}}\right) \tag{1}$$

Where, $P\left(\frac{LS_i}{GMI}\right)$ is the probability of exceeding a particular limit state given ground motion intensity

$\phi()$ is the standard normal cumulative distribution function

λ_{CL}^{i} is ln (median drift for a particular limit state, i)

$\lambda_{\frac{D}{GMI}}$ is ln (calculated median demand drift given the ground motion intensity from the best fit power-law line)

$\beta_{\frac{D}{GMI}}$ is the demand uncertainty.

The two parameters $\lambda_{\frac{D}{GMI}}$ and $\beta_{\frac{D}{GMI}}$ are given by equation 2 and equation 3.

$$\lambda_{\frac{D}{GMI}} = ln a_1 + a_2 ln(GMI) \tag{2}$$

$$\beta_{\frac{D}{GMI}} = \sqrt{\frac{\sum_{k=1}^{n}[ln(GMI_k) - \lambda_{\frac{D}{GMI}}(GMI_k)]^2}{n-2}} \tag{3}$$

2. Selection of Ground Motion for Nonlinear Time History Analysis

A total of ten earthquake motions are selected for non-linear time-history analysis. Table2 shows the details of the chosen earthquakes with PGAs from 0.17g to 0.90g. Figure1 shows the response spectrum of the selected earthquake ground motions.

Table 2: List of earthquake motions

Earthquake ID	Earthquake Name	PGA (g)
1	Kobe	0.77
2	Tabas	0.90
3	Lomaprieta Losgatos	0.70
4	Lomaprieta Lexdam	0.67
5	Cmendocino	0.62
6	Erzincan	0.42
7	Landers	0.69
8	Rinaldi	0.87
9	Oliveview	0.72
10	Gorkha	0.17

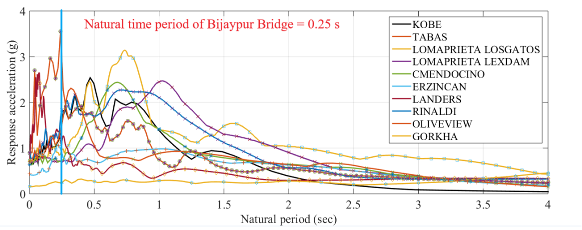

Figure 1: Response spectrum of the selected ground motions

3. Finite Element MOdeling of the Bridge

Finite element method is used to perform the analysis of the bridge. The bridge is analytically modeled in the object-oriented finite element method-based framework OpenSees [8] developed by the Pacific Earthquake Engineering Research Center at the University of California, Berkeley. The FE model was created with scripts written in Tcl programming language.

OpenSees has an extensive library of material properties. This paper uses Steel02 Material – Giuffre-Menegotto-Pinto Model with Isotropic Strain Hardening to represent the reinforcing elements and Concrete02 Material – Linear Tension Softening materials from the OpenSees library. RCC bridge pier is made up of three materials i.e., unconfined concrete, confined concrete and steel reinforcement bars. The unconfined concrete lies on the outer side of transverse reinforcement bars while the confined concrete refers to the concrete inside the transverse reinforcement bars. Concrete02 material is used for both confined and unconfined concrete.

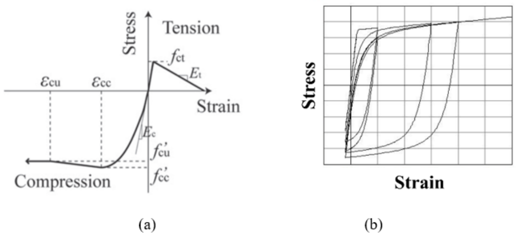

Figure 2: Adopted constitutive material models: (a) Concrete02, (b) Steel02 with isotropic hardening

3.1 Validation of the OpenSees model

[9] conducted an experimental test on cantilever column with four cyclic loading histories. For validation purpose of the bridge model in this study, the cantilever column from the experimental test is modelled in OpenSees validated against the cyclic loading history. The output of the analytical model is then compared with the experimental test result. The material characteristics for Concrete02 are:

uniaxialMaterial Concrete02 $matTag $fpc $epsc0 $fpcu $epscu $lambda $ft $Ets

where

$matTag = 1,2 for confined and unconfined concrete.

$fpc = -42.51, -32.7 N/mm2 for confined and unconfined concrete

$epsc0 = -0.00297 , -0.003 for confined and unconfined concrete

$fpcu = -8.502, -6.54 N/ mm2 for confined and unconfined concrete

$epsU = -0.01487, -0.01 for confined and unconfined concrete

$lambda = 0.1

$ft = 4.578 N/ mm^2 and

$Ets = 2289 N/ mm^2

For Steel02 material model, the material parameters are:

uniaxialMaterial Steel02 $matTag $Fy $E $b $R0 $cR1 $cR2 $a1 $a2 $a3 $a4

where

$matTag = 3

$Fy = 565.4 N/ mm2

$E = 20000 N/ mm2

$b = 0.01

$R0, $cR1, $cR2 = 18, 0.925, 0.15.

$a1, $a2, $a3, $a4 are isotropic hardening parameters (optional).

Material Properties

Concrete Strength: 32.7 MPa

Transverse Steel: Yield Stress: 434.4 MPa

Longitudinal Steel: Yield Stress: 565.4 MPa **Geometry**

Diameter: 457.2 mm

Length: 2438.4 mm

Configuration: Cantilever

Loading Axial Load: 231.3 kN

Longitudinal Reinforcement

Diameter: 19 mm

Number of Bars: 12

Reinforcement Ratios: 0.0198

The loading history in the first experimental test is as shown in Figure3. The force-displacement curve obtained from the experiment and the analytical model in OpenSees is plotted in Figure4. The numerical results show good correlation with the test data for the estimated values for base shear, drift, the cyclic hysteresis and the energy dissipation. The same modeling technique is extended to the real scale two-span RCC bridge.

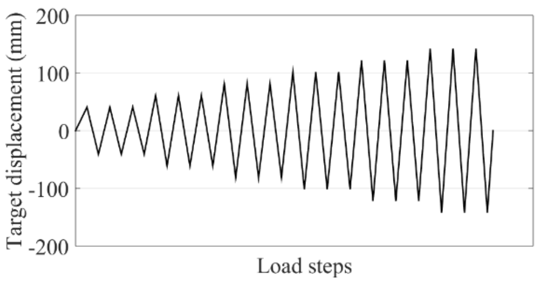

Figure 3: Loading history of experimental test (Moyer & Kowalsky, 2003)

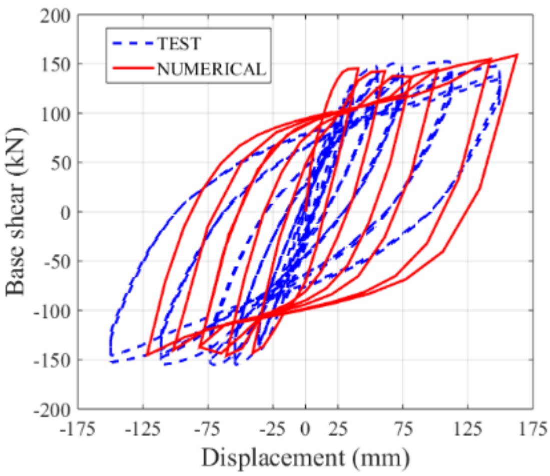

Figure 4: Comparison of experimental result and analytical result

3.2 Real scale two-span RCC bridge

The bridge selected is a typical two spanned RCC T-beam girder bridge in Nepal. The two spans are 25m each in length supported on two abutments on each end and a pier of height 7.8m at the middle. The girders are simply supported resting on abutment on one side and the pier on the other. The foundation is not modeled and the pier is assumed to be fixed on the ground. The study considers the damage on the pier only, thus the abutments are also not modeled. As the girders are simply supported on the abutments and pier, the load from the girder is directly transferred to the pier as axial compression load. Thus, the two spanned bridge is modeled as the cantilever pier with axial load as per the load distribution of the girder. The material characteristics for Concrete02 are:

uniaxialMaterial Concrete02 $matTag $fpc $epsc0 $fpcu $epscu $lambda $ft $Ets

where

$matTag = 1,2 for confined and unconfined concrete.

$fpc = -39.0, -30.0 N/mm2 for confined and unconfined concrete

$epsc0 = -0.00285 , -0.003 for confined and unconfined concrete

$fpcu = -7.8, -6.0 N/ mm2 for confined and unconfined concrete

$epsU = -0.01424, -0.01 for confined and unconfined concrete

$lambda = 0.1

$ft = 4.2 N/ mm2 and

$Ets = 2100 N/ mm2

For Steel02, material model the material parameters are:

uniaxialMaterial Steel02 $matTag $Fy $E $b $R0 $cR1 $cR2 $a1 $a2 $a3 $a4

where

$matTag = 3

$Fy = 500.0 N/ mm2

$E = 20000 N/ mm2

$b = 0.01

$R0, $cR1, $cR2 = 18, 0.925, 0.15.

$a1, $a2, $a3, $a4 are isotropic hardening parameters (optional).

Material Properties

Concrete Strength: 30.0 MPa

Transverse Steel: Yield Stress: 500.0 MPa

Longitudinal Steel: Yield Stress: 500.0 MPa
Geometry

Diameter: 1950 mm

Length: 7880 mm

Configuration: Cantilever

Loading Axial Load: 3894.1675 kN

Longitudinal Reinforcement

Diameter: 32 mm

Number of Bars: 38

Reinforcement Ratios: 0.010233

The section of the pier is divided into various fibers as shown in Figure5.

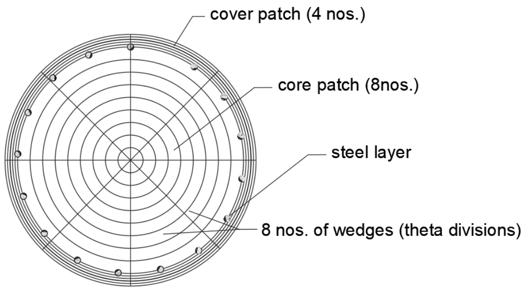

Figure 5: Fiber section of the pier

The line diagram of the bridge and corresponding model in OpenSees for analysis is shown in Figure6.

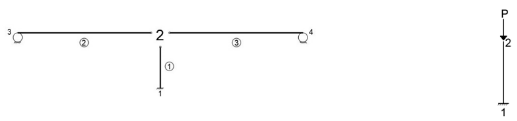

Figure 6: Line diagram of two spanned bridge and its corresponding line model

4. Results and Discussion

The pushover curve from the static nonlinear analysis of the bridge is shown in Figure7.

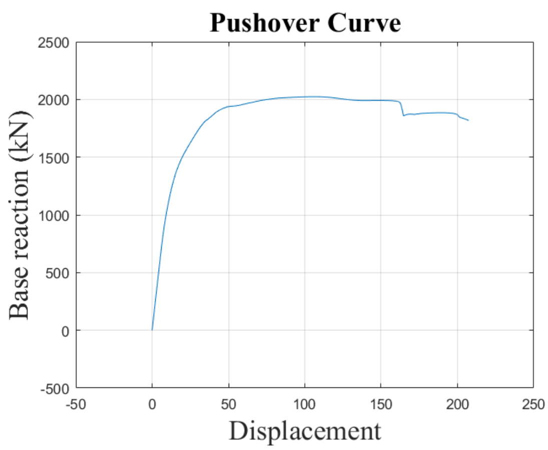

Figure 7: Pushover curve of the pier of the bridge

The first yield occurred at the drift value of 0.01246 corresponding to the base shear of 1212 kN. The natural time period of the bridge is 0.25 seconds. The non-linear time history analysis is done scaling the earthquakes from 60 percent to 200 percent of the peak ground acceleration for each selected earthquake records. Figures8-11 show the representative time histories for the Erzincan and Kobe earthquake records and the corresponding drift time histories of the bridge pier.

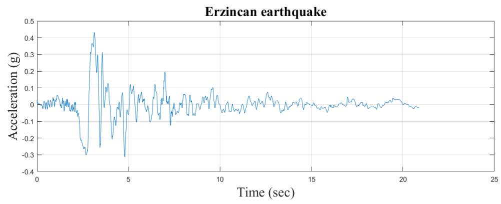

Figure 8: Time history of Erzincan earthquake

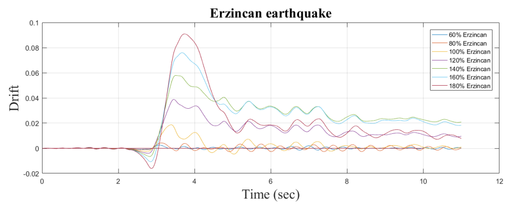

Figure 9: Drift vs time of the bridge due to Erzincan earthquake at different scale factors

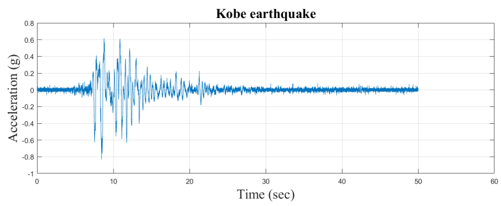

Figure 10: Time history of Kobe earthquake

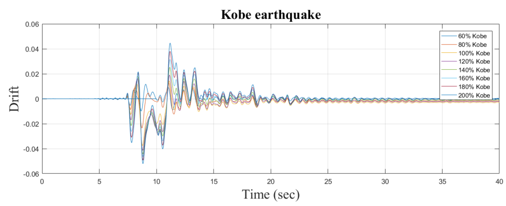

Figure 11: Drift vs time of the bridge due to Kobe earthquake at different scale factors

The ln(drift) and ln(PGA) is plotted in the graph and the ln(median demand drift for the given earthquakes) is used from the best fit power law line. Using the drift limit for the stated damage state and the median demand drift from the plot, the fragility curves for the various damage states are prepared.

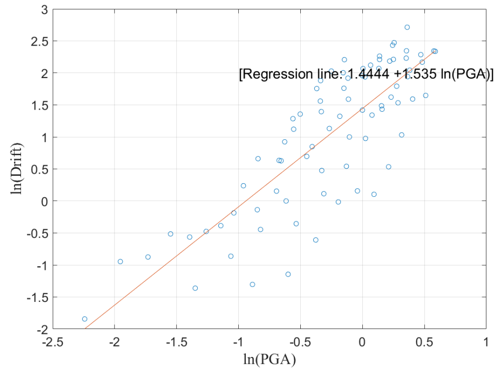

Figure 12: Linear regression analysis for median demand drift

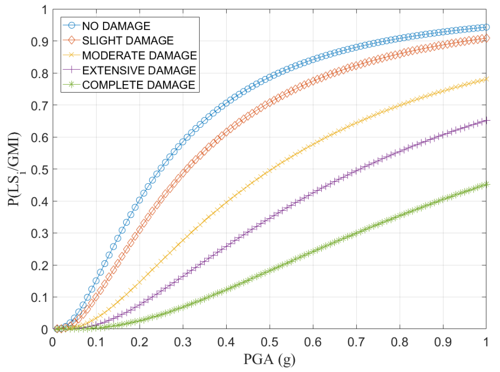

Figure 13: Fragility curves of pier of Bijaypur Bridge for various damage states

The fragility curves give the probability of reaching or exceeding the stated damage state in the given PGA. The curve shows that at 0.5g PGA, the probability that the bridge suffers slight, moderate, extensive and complete damage is 0.71, 0.49, 0.34 and 0.18 respectively while at 1g PGA, the probability that the bridge suffers slight, moderate, extensive and complete damage is 0.92, 0.78, 0.65 and 0.44 respectively.

5. Conclusion

This paper analyzed the seismic vulnerability of a typical two spanned RCC T-girder bridge considering the pier as the most critical component of the bridge. The bridge was analyzed with non-linear time history analysis using ten different time histories of the earthquakes with different frequencies and amplitudes. The fragility curves with the probability of exceeding defined damage states at different PGA are developed in order to assess the seismic vulnerability of the bridge. The study suggests that for the ground motion with spectral acceleration of 1g, the bridge is susceptible to moderate damage at seventy eight percent probability of exceedance, to extensive damage at sixty five percent probability of exceedance and to complete damage at forty four percent probability of exceedance.

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