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Effect of Steel Jacketing Thickness on Seismic Performance of Bridge

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Abstract:

The bridges of Nepal are getting older and old bridges were designed without considering the seismic forces, hence have low seismic capacity. The capacity of these bridges needs to be enhanced and should perform well during seismic events. The jacketing technique can be used to upgrade the bridge's structural performance. This research paper mostly focuses on the seismic performance enhancement after the application of steel jacketing on the bridge pier. The variable that has been varied is the thickness of the steel jacket. The capacity of a bridge pier is evaluated using displacement-controlled nonlinear static analysis (Pushover). The time history load is applied to the structure to determine the seismic demand of the bridge. The quantification of the enhancement of the bridge structure after the application of Steel Jacketing is evaluated by plotting the fragility curve. The modeling of the bridge is done in CSI Bridge V20.2.0. The different damage state is defined using the strain values obtained from the pushover analysis. For different damage states, the capacity and demand value are used to obtain the probability of exceeding at different PGA levels. The fragility curve is developed using the First Order Second Order Method (FOSM). From the study, it is found, that the vulnerability of the bridge after the use of jacketing for extensive damage state, the probability of failure reduced from 28.22% to 14.24% at 1.0g PGA. Similarly, the vulnerability of the bridge after the use of jacketing for collapse damage state, the probability of failure decreased from 16.93% to 7.22% at 1.0 PGA.

Key words: Seismic Performance; Fragility curve; Damage state; Pushover analysis; Time history analysis; Steel Jacketing.

1. Introduction

Nepal lies in a highly diversified zone with higher Himalayan ranges in the North and flat plain Terai in the South. There are a large number of rivers within a 200 km distance. Bridges play a major role in the transportation system of Nepal, which saves traveling time, fuel, and energy. As per the Department of Roads (DoR) of Nepal, there are more than 2020 bridges already constructed (2022). Most of them are reinforced concrete bridges. Bridges are essential in ensuring that road traffic can flow continuously throughout the year.

Bridges built before 1970 in Nepal did not have sufficient seismic resistance as they lacked the inclusion of ductility provisions in their design. As a result, those bridges have low seismic resistance and are vulnerable to significant damage even from moderate earthquakes (Patil and Pawar 2018). For the consideration of structural performance under severe seismic situations, the concepts of capacity design and ductility design are required (Priestley et al 1996). Even functional damage to bridge structures during severe earthquakes is undesirable because of the immense significance of bridge structures in the transportation system. In order to evaluate the seismic performance of structures constructed in compliance with the allowed stress, non-linear analysis approaches such as the capacity spectrum approach or response history analysis should be applied. For developing countries like Nepal, the retrofitting method should be both technologically and economically viable. Hence, this study evaluated Steel Jacketing as a retrofitting technique for an existing structure.

Jacketing is a technique used to strengthen the existing bridges in order to increase the capacity of the bridges, which helps to confirm to the current bridge design standards. Bridges are a very important and crucial element of the transportation facilities. If timely inspection and maintenance are not provided then the transportation chain could be disturbed for a longer time frame. The damages that could occur in the bridge due to seismic force can be prevented by seismic jacketing prior to seismic events. Due to inadequate shear capacity, a lack of transverse steel and confinements, improperly lapped longitudinal steel, and early termination of longitudinal steel, there is a higher risk of failure in the pier, abutments, or foundations. The efficiency of bridges is negatively impacted by these flaws. Among the bridge’s structural components, the piers are the most vulnerable elements under the action of seismic loads. To improve the piers bending capacity, different techniques are employed to retrofit the structures, RC jackets, steel jackets, CFRP jackets, external prestressed cables, or even the inclusion of new elements (Navarrete et al. 2016).

Past earthquakes such as the 1971 San Fernando Earthquake, the 1994 Northridge Earthquake, the 1995 Great Hanshin Earthquake in Japan, and the 1999 Chi-Chi earthquake in Taiwan have demonstrated that bridges are vulnerable to earthquakes (Hsu and Fu 2004) (Mitchell, et al. 1995, Maragakis and Jennings 1987). Since bridges are one of the most critical components of highway systems, it is necessary to evaluate the seismic performance of the bridges which helps in making correct decisions.

2. Objectives

1. To determine the seismic response of the existing bridge pier excited by different ground motion time histories.
2. To develop fragility curves for both existing and steel jacketed bridge pier.
3. To quantify the seismic performance of jacketed (Steel Jacketing) bridge pier excited by different ground motion time histories.

3. Methodology

3.1 Bridge Description

An existing bridge in Nepal was selected for the research purpose. The bridge lies in the Kathmandu district over the Bagmati River. The as-built drawing of the bridge is collected from DoR, Government of Nepal. Figure 1 represents the cross-section of the selected bridge. The selected bridge is the RCC T-girder Bridge which is 75m long with two numbers of spans each 37.5m long. The total width of the bridge superstructure is 11m wide which has a footpath of width 1.750m on each side and a 7.50m carriageway in the middle. The superstructure consists of four longitudinal prestressed rectangular main girders which is 1900 x 750 mm and five cross girder each of size 1600 x 400 mm at an equal end of spans in each of the spans. The thickness of the deck slab is 200mm. All the components of the superstructure are constructed monolithically using M45-grade concrete.

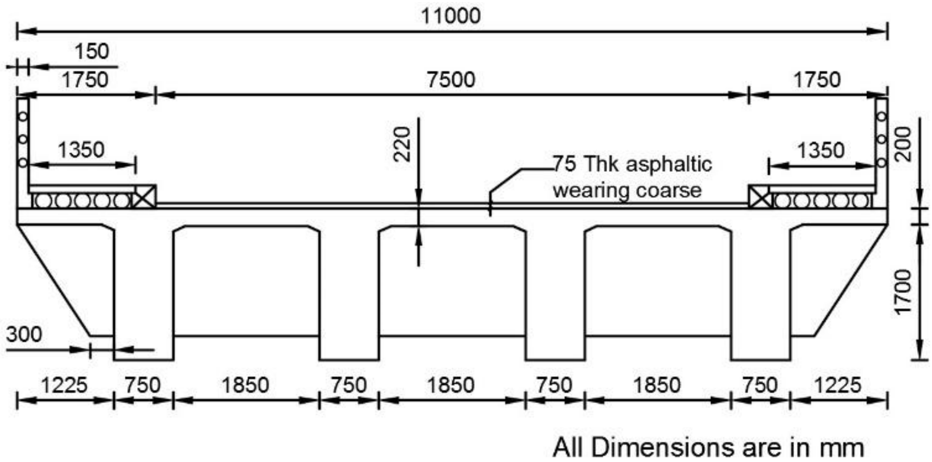

Figure 1 Cross-section of the bridge

The connection between the superstructure and substructure is done using Polyterrafluoroethylene (PTFE) POT bearing. The PTFE POT-fixed bearing is placed at bents which restricts translation in all directions and allows rotation. Similarly, PTFE POT-free bearing is placed at an abutment which restricts translation in the longitudinal direction while it allows translation in the transverse and vertical direction and allows rotation. The size of the bearing is 0.8 x 0.6 x 0.075 m. The substructure comprises of cantilever retaining wall as an abutment and one hammerhead single-column bent pier. The diameter of the pier is 2.7m with a clear height of 6.247m. The dimensions of the pier cap are 10.5m x 4.5m x 2m. The pier column has 36 numbers of 32 mm diameter longitudinal reinforcement and 12 mm diameter bars at 100 mm and 150 mm C/C transverse reinforcement. The material used in the pier and abutment is M25 grade concrete and Fe500 reinforcement.

Most of the old bridges constructed before 1990 AD have a longitudinal reinforcement ratio of 0.005 (Navarrete, et al. 2016). Hence the model with a longitudinal reinforcement ratio of 0.005 in pier has been used and is denoted with the name “existing” in the analysis. Similarly, the steel jacketed models are denoted with “Steel 10 mm” for the jacket thickness of 10 mm of steel Fe 345 and “Steel 20 mm” for the jacket thickness of 20 mm of steel Fe 345.

The footing of the pier rests on a group of sixteen piles and that of the abutment in a group of twelve piles. The diameter of the pile is 1.0m and 22 m deep. The M25 grade concrete and Fe 500 rebar have been used.

3.2 Bridge Modelling

A three-dimensional Finite Element Model of the bridge is modelled in CSI bridge V20.2.0. (Computer and Structures Inc: CSI Bridge 2020). The as-built drawings provided by DoR are used to define geometric dimensions and material properties accurately. The (Mander, Priestley, and Park 1998) Model is used to represent the stress strain relationship for confined and unconfined concrete. The effect of confinement increases the compressive strength and ultimate strain of the concrete.

The moment-curvature relation is the plot of unit angle cross-sectional deformation in radians due to bending or both bending and axial load to the structure. The moment curvature relation gives a clear idea about the stiffness, ductility properties, and strength of the cross-section of the column.

Mathematically,

$$\text{Curvature } (\square) = \frac{d\theta}{dy} = \frac{d2y}{dx2} = \frac{1}{\rho}$$

$$\text{Curvature } (\square) = \frac{M}{EI}$$

where,

 = Curvature

EI = Flexural Stiffness

Moment curvature relationship can be evaluated by two methods, analytically and experimentally. In this research, the moment-curvature relation is obtained using an analytical method. The relationship is automatically plotted in the FEM software (CSI Bridge) when the section is defined by the section designer. The moment-curvature relation for different models for our research is plotted in Figure 2. The plot shows that the moment capacity of the section increases with the application of RC jacketing. Furthermore, it increases on increasing the ductility percentage of the jacketing. Hence, the strength and stiffness of the bridge pier gets increased

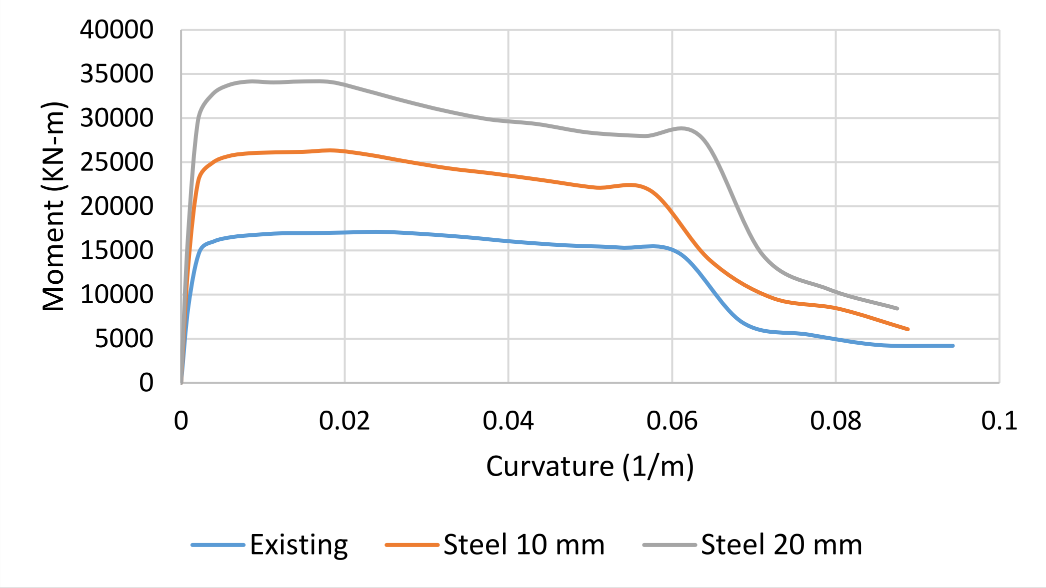

Figure 2 Moment Curvature Relationship

The figure-3 represents the spline model of the selected model in CSI Bridge. In the FEM software, the displacement-controlled pushover analysis is carried out to determine the capacity of a bridge pier in the form of base shear versus pier displacement plot. Non-linear direct integration time history analysis is performed to determine the response of the structure in terms of top pier displacement using recorded accelerograms.

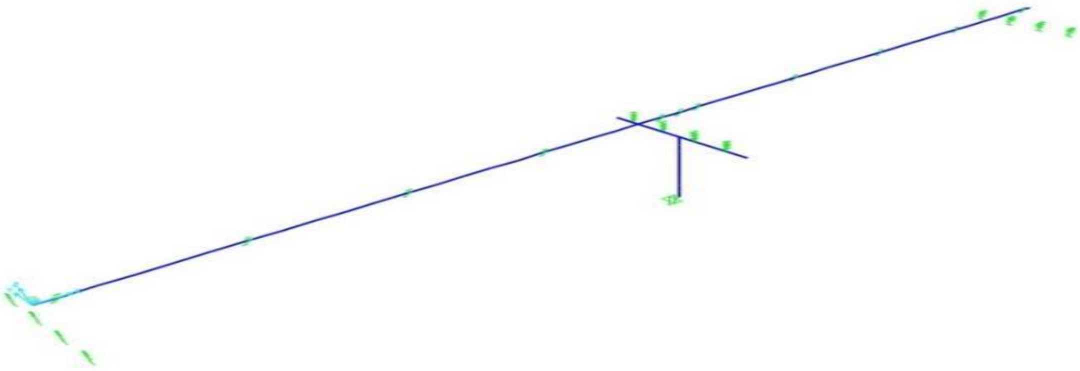

Figure 3 Global Finite Element Model

All the loads and forces were transferred to the substructure through PTFE POT bearing which has been modelled as the linear link element. The bearing positioned over the pier cap and abutment is represented by an idealized, linear elastic bearing model. The bearing over the hammer head pier cap is restrained along three directions but is made free in rotation. Similarly, bearing over the abutment is restrained along the longitudinal direction but is free along the transverse and vertical directions and is free in rotation.

The components of the superstructure are longitudinal girder, cross girder, and deck slab. In the FEM software, modeling of the deck slab is done as an elastic shell element and girder as linear 3D frame elements. During the seismic events, the response of the bridge girder is expected to be in the elastic range (Mander, et al. 2007). The model can be simplified to the Beam stick model for analysis purposes. In the simplified beam stick model the superstructure is represented by a single beam element which has the equivalent property of the entire deck. Bents are modeled as 3D beam-column frame elements. The software automatically maintains the connection between the bridge components to ensure the continuity of the model using rigid link elements. The Caltrans fiber section is defined by the section designer of CSI Bridge which helps to depict the nonlinear characteristic of the pier during seismic loading. The defined fiber section is assigned at the plastic hinge formation zone, at the top and bottom of the pier near fixity.

The length of a plastic hinge can be evaluated by using the following relation suggested by FHWA-1995 ((FHWA) 1995)

$$L_p = 0.08 * h + 0.022 * f_y * D_b$$

Where h is the height of bridge pier (m), f_y is the yield stress of longitudinal rebar (MPa) and D_b is the diameter of the longitudinal rebar (m). The hinge length is compared to the twice value for the minimum hinge length, described as $L_p = 0.022 * f_y * D_b$, and the larger value is used. After the hinge lengths have been determined, the hinges are placed on the bent columns at each end of the column at distances from each end equal to 1/2 the hinge length.

The column of the pier is jacketed using Steel of Fe 345. The thickness of steel is the parameter that has been varied. In the FEM model, it is assumed that there is a perfect bond between the original pier and the Steel jacket (Navarrete, et al. 2016). The retrofitted cross section consists of two confined cores, longitudinal reinforcement layer and outer steel jacketed layer. This hypothesis allows the estimation of the column's capacity and behavior with the available models for traditional RC members in flexo-compression, including the expression used to estimate the plastic hinge length (Paulay and Park 1975).

3.3 Selection of Ground Motion Data

The nonlinear response of the structure relies on the structural modelling technique and ground motion characteristics. Precautions should be taken while choosing the ground motion data, as it directly affects the evaluation process. The ground motion data are selected on the basis of magnitudes, fault types, and spectral content. Table 2 represents the earthquake selected and the station for the selected earthquake.

Ground motion parameters might be acceleration, velocity, and displacement or all combined. Acceleration is recorded and other parameters are derived from it. Actual ground motion near to the site should be selected for better results. However, actual earthquake records in Nepal are not available remarkably. The earthquake which resembles to the site should be considered. For this, the target spectrum is defined according to IS 1893(part I): 2016. The earthquake data is taken from the PEER Earthquake Database (NGA West 2) (PEER Ground Motion Database 2022). The downloaded data are the raw input data for time history. It is matched using Seismomatch software considering the target spectrum. The output from the Seismomatch software gives matched time history data which is used for nonlinear time history analysis.

Table 1: Selected Ground Motion data

RSN	**Event (years)**	**M_w**	**Station**	**Fault Mechanism**
6	Northridge -01 (1994)	6.95	EL-centro Array #9	Strike-slip
1083	Imperial Valley -02 (1940)	6.69	Suland-Mt. Gloason	Normal + Reverse

			Ave	
1108	Kobe, Japan (1995)	6.90	Kobe University	Strike slip
1131	Kocaeli (1999)	7.51	Gebze	Strike slip
22	EL Alamo (1950)	6.80	EL Centro Array #9	Strike slip
3548	Loma Prieta (1989)	6.93	Los Gatos – Lexington Dam	Reverse Oblique
879	Landers (1992)	7.28	Lucerene	Strike slip

3.4 Probabilistic Fragility Function

Fragility curve gives the conditional probability of attainment or exceeding of a damage state for a given intensity "X" of ground excitation. The quality of the fragility curve relies mostly on, the modeling of the bridge structure, selection of ground motion, and definition of damage state. According to Melchers (2001), the fragility curve can be represented by a lognormal cumulative distribution function and is given by;

$$P_f = \Phi\left[\frac{ln ln \frac{Sd}{Sc}}{\sqrt{\beta d^2+\beta c^2}}\right]$$

where, ȸ () is the standard normal distribution function, S_c is the median value of structural capacity defined for the damage state, S_d is the seismic demand response due to imposed ground motion parameter into the structure, βc is the lognormal standard deviation of the structural capacity, βd is the lognormal standard deviation for the demand.

$\sqrt{\beta d^2 + \beta c^2}$ Represents the composite logarithmic standard deviation which includes the uncertainties and randomness for both capacity and demand. Its value is defined in (Federal Emergency Management Agency: HAZUS Technical Manual 2003) as 0.55 for slight and moderate damage state and 0.7 for extensive and collapse damage state.

For all seismic intensity levels, the capacity of the bridge pier for a given damage type is constant. It is defined using nonlinear static analysis in accordance with a predefined strain limit. The strain limits are taken from the Mander model for material nonlinearity from the section designer of CSI Bridge (Mander, Priestley, and Park 1998). The guideline HAZUS- MH 2003 is been used for the qualitative definition of bridge pier damage state (Federal Emergency Management Agency: HAZUS Technical Manual 2003). The quantitative definition of damage state is carried out using the qualitative definition and ductility displacement is obtained accordingly (Basnet and Suwal 2019). The strain value for each damage state as mentioned in Table 3 is used to calculate the median capacity of the bridge pier for all conditions separately. The obtained values are tabulated in Table 4 and these values depict the capacity of the bridge pier and are used for the development of fragility functions. Similarly, the seismic demand of models is assessed from linear regression analysis of response data from nonlinear time history analysis using a power model

$$S_d = aIM^b$$

Multiplying both sides by natural log,

$$\ln (S_d) = \ln(a) + b\ln (IM)$$

where, a and b are regression coefficients, IM is the seismic intensity measure.

Displacement ductility (μ_d) = Δ_T / Δ_y (Caltrans: Seismic Design Criteria 2004)

where, Δ_T is maximum top pier displacement and Δ_y is the rebar first yield displacement.

Table 2 Damage state according to displacement ductility. (Basnet and Suwal 2019)

Damage state	Qualitative Definition (HAZUS-MH-2.1)	Description	Rebar strain	Concrete strain
Slight	Minor spalling of column	First yielding of extreme rebar	0.0025	-
Moderate	Spalling in column	Maximum compressive strain at cover concrete =0.002	-	0.002
Extensive	Column degradation without collapse	Maximum compressive strain at core concrete	-	0.0028
Collapse	Column collapsing	Ultimate compressive strain at core concrete	-	0.0076

Table 3 Bridge median capacity ratio in terms of displacement ductility for different conditions

Damage State	Existing	Steel 10 mm	Steel 20 mm
Slight	1.66	1.62	1.68
Moderate	3.3	3.24	3.31
Extensive	6.2	6.16	6.2
Collapse	8.15	8.10	8.18

4. Results and Discussions

Free vibration analysis of the bridge structure was carried out in FEM software along the transverse direction using the linear elastic analysis method. The obtained fundamental time period of vibration with a modal participation factor which is more than 90% in the transverse direction is given in Table 4.

Table 4 Transverse Vibration period of Bridge

Condition	Transverse Vibration Period (s)
Existing	0.67791
Steel 10 mm	0.67659
Steel 20 mm	0.6241

On increasing steel thickness in Steel Jacketing, the lateral stiffness and bending capacity of the bridge pier increases which eventually decreases the time period of the bridge structure. The nonlinear pushover analysis is carried out to obtain the capacity characteristic of the bridge pier in CSI Bridge V20.2.0. Displacement controlled pushover analysis method was used where the pier was displaced in the transverse direction up to 350 mm displacement. The nonlinear characteristics of the bridge pier are captured with the help of fiber PMM hinges which were assigned near the point of fixity of the pier, top and bottom ends. The load is applied monotonically in the transverse direction. In this research, the structure was pushed up to 350mm for all model cases. The damaged state of the bridge pier was obtained from the pushover curve with the help of predefined stain yield values of reinforcement, unconfined concrete, and confined concrete for all bridge model cases. The obtained capacity displacement curve is analyzed and plotted as shown in Figure 3. The pushover curve depicts that the base shear reaction increases along with top pier displacement as the thickness of steel in steel jacketing increases. From the pushover curve data, we can clearly observe that on applying Steel Jacketing the base shear reaction of the structure increases. It gets increased by 13.72% and 27.67% for Steel 10 mm and Steel 20 mm respectively. The increase in base shear of the bridge pier is due to an increase in lateral stiffness and bending capacity due to the addition of a layer with certain structural characteristics.

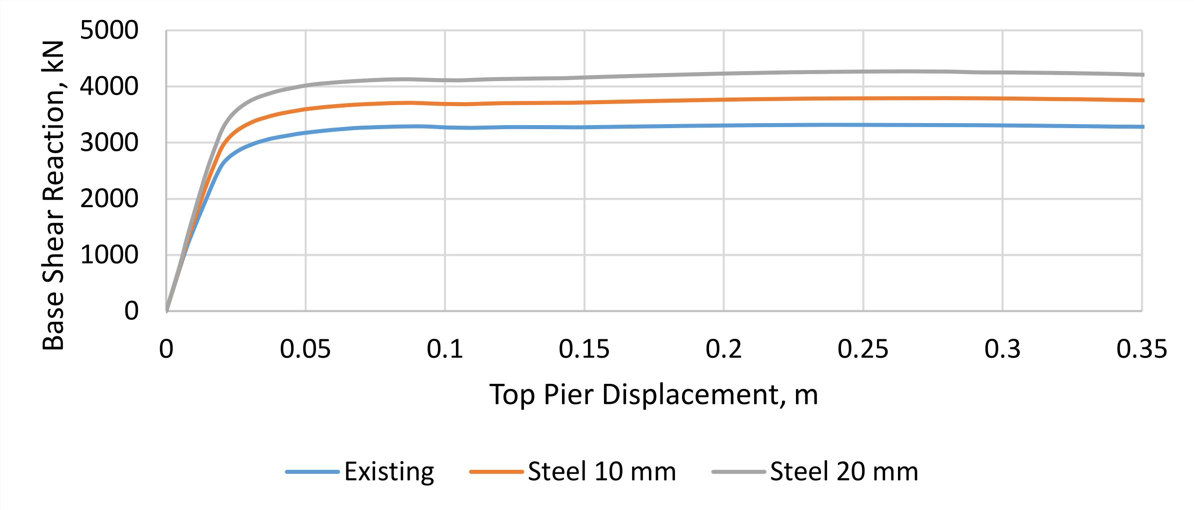

Figure 4 Pushover Curve

In this research, seven ground motion data were selected as listed in Table 2 and its component were applied in both transverse and longitudinal directions. The top pier displacement for each intensity level from 0.1g to 2.0g at an interval of 0.1 for each ground motion was recorded. While scaling Peak Ground Acceleration (PGA), the linear scale method was used. The displacement ductility for each data recorded for each intensity level was calculated using CALTRANS 2004 formulae. Linear regression analysis on the logarithmic value of displacement ductility and Pga is carried out to obtain the median demand line. The fragility curve is developed using the damage state defined earlier and the median demand line. Figure 5 shows the fragility curve of the bridge pier with no jacketing 'Existing' Model. It depicts 95.36%, 65.57%, 28.22%, and 16.93% probability of failure for slight, moderate, extensive, and collapse damage states respectively at 1.0g PGA. The above data shows the probability of failure for different cases at 1.0g PGA.

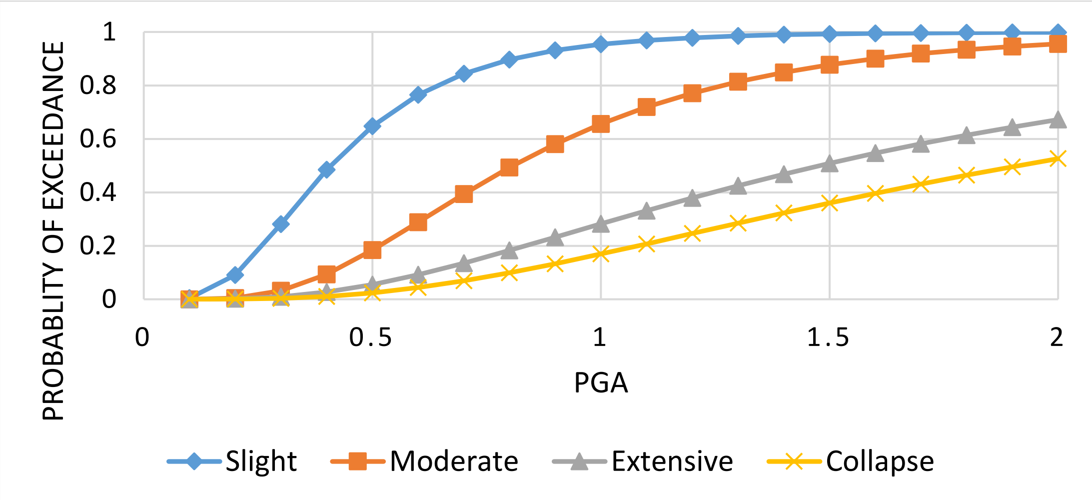

Figure 5 Fragility function for Existing Model

Similarly, 91.95%, 55.63%, 20.99%, 11.54% probability of failure of Steel 10 mm for slight, moderate, extensive and collapse damage state respectively at 1.0g PGA and is shown in Figure 6.

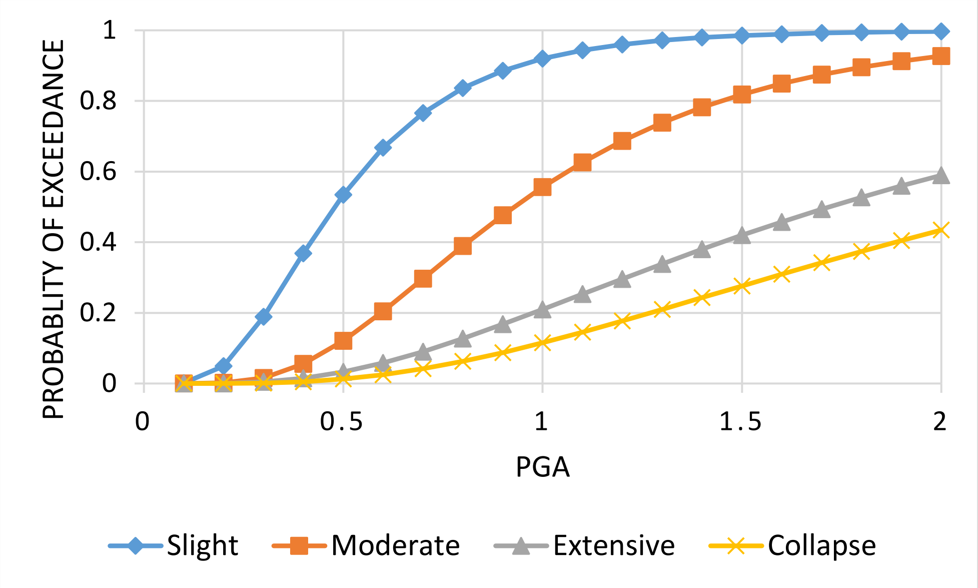

Figure 6 Fragility function for Steel 10 mm Model

Similarly, for Steel 20 mm the probability of failure is 84.96%, 41.51%, 14.25%, 7.21% for slight, moderate, extensive and collapse damage state respectively at 1.0g PGA and is shown in figure 7.

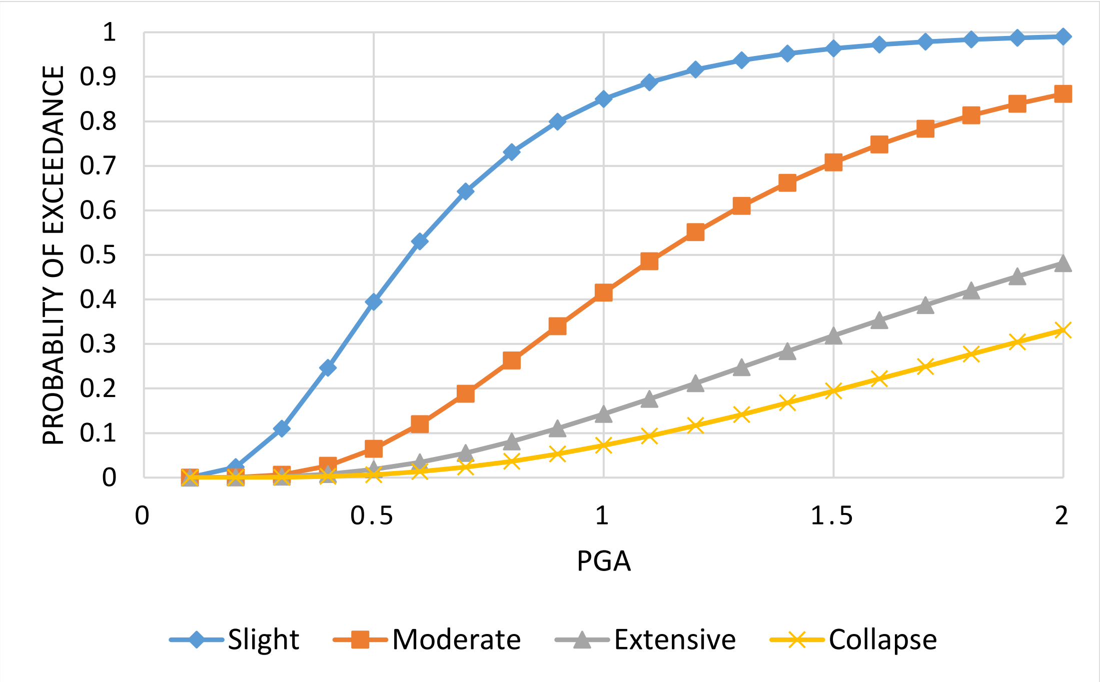

Figure 7 Fragility function for Steel 20 mm Model

5. Conclusion and Recommendations

Fragility curves are plotted for the two-span continuous hammer head bridge pier jacketed with Steel jacketing with varying steel thickness using an analytical method. These curves help to quantify the improvement of seismic performance after the use of Steel Jacketing at different ductility percentages. The major conclusions are:

- The transverse vibration period of the bridge decreases with the application of Steel Jacketing. The time period of the bridge decreases with the increase of steel jacket thickness. As the structure is retrofitted with a Steel Jacket the stiffness of the pier increases, due to which the time period of the structure decreases.

- From nonlinear static analysis, the capacity of the bridge increases with an increase in steel jacketing thickness. The base shear reaction of the structure increased by up to 27.67 % on the application of a steel 20mm thickness jacket.
- From nonlinear dynamic analysis, the displacement ductility of bridge pier decreases along transverse direction in great extent with increase of steel thickness in Steel Jacketing.
- So, the probability of exceedance for collapse damage state decreased as the thickness of steel in the Steel Jacket increases. It decreased from 16.93% to 7.21% for collapse damage state for the 'Existing' model to steel 20 mm model at 1.0 PGA level. As the thickness of steel in the steel jacket increases, the stiffness of lateral load resisting increases and the bending capacity of the pier increases. Hence, increases the performance of the bridge.

Thus, this study concluded that the performance of the two-span bridges with single-bent column can be increased by the application of Steel Jacketing. For further study, the seismic performance of the abutment, the effect of the isolation system, soil structure interaction at the abutment and foundation, bent column P-delta effect, and bridge pounding effects of a bridge can be done.

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