Expected damages of retrofitted bridges with RC jacketing

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Abstract. The bridge infrastructure in many countries of the world consists of medium span length structures built several decades ago and designed for very low seismic forces. Many of them are reinforced concrete structures that according to the current code regulations have to be rehabilitated to increase their seismic capacity. One way to reduce the vulnerability of the bridges is by using retrofitting techniques that increase the strength of the structure or by incorporating devices to reduce the seismic demand. One of the most common retrofit techniques of the bridges substructures is the use of RC jacketing; this research assesses the expected damages of seismically deficient medium length highway bridges retrofitted with reinforced concrete jacketing, by conducting a parametric study. We select a suite of twenty accelerograms of subduction earthquakes recorded close to the Pacific Coast in Mexico. The original structures consist of five 30 m span simple supported bridges with five pier heights of 5 m, 10 m, 15 m 20 and 25 m and the analyses include three different jacket thickness and three steel ratios. The bridges were subjected to the seismic records and non-linear time history analyses were carried out by using the OpenSEEs Platform. Results allow selecting the reinforced concrete jacketing that better improves the expected seismic behavior of the bridge models.

1. Introduction
Many of the medium span length bridges constructed around the world are old structures with more than 40 years. Most of them are non-seismically designed bridges and they have to fulfill the current seismic regulation codes. Moreover, the continuous increments in load live amplitudes and dead loads derive in cracks formation in the structural elements that conduct to the necessity of evaluate the safety of these structures.

Although one of the main challenges in earthquake engineering is to mitigate the risk of existing structures, most of the regulation codes in the world are oriented to the project of new structures, without explicit mention of the criteria to be taken on the retrofit techniques of existing bridges. The rehabilitation and seismic retrofitting of a bridge involve a set of parameters in addition to the purely technical aspects that influence the decision to intervene structure. It must be evaluated at least the structural capacity level, the seismic demand on site, the importance of the bridge structure in the context of the road network, the path redundancy, environmental restrictions, the expected life service, the importance of the structure and the viability of access among other variables.

The interruption of a bridge service has serious effects in the transportation systems. Immediately after a disaster, the closure of roadway can prevent emergency operations. In addition to the direct costs among the different possibilities of rehabilitation and strengthening, there are indirect costs,
which often exceed the direct costs, caused by the economic and social impact of transit disruption. Indirect costs depend on the time the bridge remains unused, the economic importance of traffic on the road, the traffic delay caused by the use of alternative routes, among other parameters. Even though there are many rehabilitation techniques, this study deals with the reinforced concrete jacketing because of its spread use around the world to retrofit bridge piers.

2. RC jacketing
Bridges located in seismic prone areas have usually important displacement demands in piers. The seismic design of old bridges considered low seismic coefficients that conduct to small lateral forces. Additionally the small lap splice length in longitudinal reinforcement of the piers and the small amount of transverse reinforcement conducted to several failures [1]. One column pier bridges are particularly affected by insufficient lap splice length in the zone of plastic hinge formation (Figure 1).

![Figure 1. Fukae viaduct collapse during the Kobe earthquake in Japan.](image)

The traditional techniques for the rehabilitation of the bridge substructures increase their resistance and involve Reinforced Concrete (RC) or steel jacketing (Figure 2). Several studies analyze the most important parameters related to RC jacketing in analytical approaches and experimental tests [2-5].

![Figure 2. RC jacketing in a column and shear wall](image)

The jacket, in addition to provide lateral confinement, improves the bending and shear capacity of the original column. An important aspect to be considered in all jackets is that this capacity increase at the base of the column causes additional demands on the existing foundation. The bending overstrength at the column ends can induce failures in other regions of the element with insufficient shear or bending capacity and a brittle failure can occur. For this reason, it is common to leave a gap between the jacket and the foundation.

3. Seismic demand
The seismic records to conduct the nonlinear analyses were selected from the Mexican base of strong motion data that contains earthquakes recorded in the period of 1960-1999. The database provides information of 527 strong motion stations in Mexico. We chose a suite of 20 seismic records from the subduction zone, one of the most important seismic sources in the country. Table 1 shows some characteristics of the earthquakes, including the epicenter location, the magnitude and the focal depth.
Table 1. Earthquakes’ epicenter from the subduction seismic source.

| Date       | Earthquake ID | Lat. N | Long. W | Focal d (km) | Magnitude (Mw) |
|------------|---------------|--------|---------|--------------|----------------|
| 09/19/1985 | 850016        | 18.08  | 102.94  | 15           | 8              |
| 09/21/1985 | 850018        | 17.62  | 101.82  | 22           | 7.5            |
| 02/08/1988 | 880004        | 17.49  | 101.16  | 19.2         | 5.8            |
| 04/25/1989 | 890024        | 16.60  | 99.4    | 19           | 6.9            |
| 05/15/1993 | 930005        | 16.47  | 98.72   | 15           | 6              |
| 10/24/1993 | 930009        | 16.54  | 98.98   | 19           | 6.6            |
| 09/14/1995 | 950001        | 16.31  | 98.88   | 22           | 7.3            |

Table 2 presents the information of the 20 seismic records selected to perform the analyses of the bridges. The table displays the station name, the record ID, the station location, the peak ground acceleration (PGA) and the distance between the epicenter and the seismic station.

Table 2. Seismic records selected to perform the analyses.

| Earthquake | Station ID | Lat. N | Long. W | PGA (gals) | Distance (km) |
|------------|------------|--------|---------|------------|---------------|
| 850016     | AZIH8509.191 | 17.60  | 101.46  | 153.93     | 166.17        |
|            | FICA8509.191 | 17.65  | 99.84   | 69.18      | 331.62        |
|            | PAPN8509.191 | 17.33  | 101.04  | 154.95     | 218.24        |
|            | PARS8509.191 | 17.34  | 100.21  | 109.82     | 300.44        |
|            | SUCH8509.191 | 17.23  | 100.64  | 103.12     | 261.66        |
|            | UNIO8509.191 | 17.98  | 101.81  | 165.29     | 120.76        |
|            | VILE8509.191 | 17.65  | 99.84   | 69.18      | 331.62        |
|            | ATY8509.211  | 17.21  | 100.43  | 79.66      | 154.27        |
|            | PAPN8509.211 | 17.33  | 101.04  | 242.69     | 88.9          |
| 850018     | PARS8509.211 | 17.34  | 100.21  | 625.78     | 173.12        |
|            | SUCH8509.211 | 17.23  | 100.64  | 85.98      | 132.47        |
|            | MAGY8802.081 | 17.38  | 100.58  | 102.09     | 62.91         |
| 880004     | PARS8802.081 | 17.34  | 100.21  | 246.91     | 101.46        |
|            | ACAP8904.251 | 16.84  | 99.91   | 104.39     | 60.58         |
|            | COYC8904.251 | 16.97  | 100.08  | 85.08      | 83.39         |
|            | OCT8904.251  | 17.25  | 99.51   | 201.16     | 72.93         |
|            | PARS8904.251 | 17.34  | 100.21  | 117.11     | 119.55        |
| 930005     | VIGA9305.152 | 16.76  | 99.234  | 67.31      | 63.59         |
| 930009     | MSAS9310.241 | 17.01  | 99.46   | 119.05     | 72.58         |
| 950001     | VIGA9509.141 | 16.76  | 99.24   | 100.35     | 62.55         |

The PGA of the seismic records was scaled with the results of a seismic hazard assessment of the country that included the contribution of all seismic sources. The uniform hazard spectra for sites close to the pacific coast and five return periods (Figure 3) were obtained. The scale factor was the required
value to match the maximum amplitude of the response spectra of each accelerogram to the maximum amplitude of the uniform hazard spectrum for a 2500 return period.

Figure 3. Uniform hazard spectra for a site close to the pacific coast in Mexico.

4. RC bridge models
We select typical RC bridges of medium span length that can be typically found in many countries. In México, this type of structures is distributed all around the country; all of them have similar superstructure and substructures as described in the next section.

4.1. Bridges’ characteristics
The bridges are composed of five simple supported 30 m long spans with four possible pier heights, 5 m, 10 m, 15 m, 20 m and 25 m. Most of the bridges designed in the 70’s have piers with small longitudinal reinforcement ratios product of gravitational designs. In this study, the longitudinal reinforcement ratio for columns was of 0.5%.

The models are composed of reinforced concrete slab resting on prestressed concrete AASTHO type IV girders (Figure 4). At each span end and at intermediate span length, there are diaphragms to provide lateral stiffness to the superstructure. The substructure consists of frame type piers with four circular columns of constant transverse section and wall type RC abutments. The girders are supported on elastomeric bearings located on top of the bent cap and over the abutments.

The circular columns were strengthened by using three possible jacket thicknesses of 10 cm, 15 cm and 20 cm and three reinforcement ratios of 0.5%, 1% and 1.5%. The parameter combination (heights, jacket thickness and reinforcement ratio) produced 50 bridge models, five original structures and 45 retrofitted models. The bridges were subjected to the 20 seismic records in longitudinal and transverse directions, which conducted to 2000 nonlinear time history analyses.

4.2. Seismic analysis
3D models of the bridges were created using the SAP2000 software [6]. The girders, diaphragms, pier columns, and cap beams were modeled using frame type elements with six degrees of freedom in each node. The 0.20 m thick slab was formed using shell type finite elements and link-type elements were used to model the neoprene bearings and gap elements for the expansion joints.

The frame elements were divided in sub-elements to distribute the mass in several nodes. The columns diameter and the cap beam width depend on the pier height (from 5m to 25m) and they are in the range of 0.85 m - 1.50 m and 1.25 m - 1.90 m, respectively. These cross sections fulfill the AASHTO regulation requirements for a load combination design for gravitational loads.

The nomenclature used to describe the bridge models identifies the pier height, the jacketing thickness and the jacketing reinforcement ratio, e.g. P25_E10_R0.5% corresponds to the 25 meters high bridge with a 0.10m jacket and 0.5% of reinforcement ratio.
The nonlinear analysis were performed with the OpenSEES program. We use a concentrated plasticity model assigning plastic hinges at both ends of the bridge columns. Figure 5 displays typical moment-curvature relationships for an exterior and interior columns of the 10 m and 25 m high models. The curves correspond to the 0.10 m thick jacket and 0.5% longitudinal reinforcement ratio.

The parameters of interest in columns to correlate them with the expected damages were: rotations, curvatures and lateral displacements. The proposal of Dutta and Mander [8] relates drift ratios with the column expected damages for a non-seismically and seismically designed bridges (Table 3). To identify the damage state, we use the drift limits of the non-seismically case.

A maximum column response for each seismic record was obtained. The mean values of rotation and drift demands for the 5 m high retrofitted bridges is presented in Table 4. The table displays the
longitudinal and the transverse demands of the bridge models; the first row presents the demands in the original bridges and the following rows are the results of the retrofitted models.

Table 3. Drift ratios and expected column damages [4].

| Damage state       | Description           | Drift limit* | Drift limit** |
|--------------------|-----------------------|--------------|---------------|
| No damage          | First yield           | 0.005        | 0.008         |
| Slight damage      | Cracking, spalling    | 0.007        | 0.010         |
| Moderate damage    | Loss of anchorage     | 0.015        | 0.025         |
| Extensive damage   | Incipient column collapse | 0.025     | 0.050         |
| Complete damage    | Column collapse       | 0.050        | 0.075         |

* Non-seismically designed  
** Seismically designed

Table 4. Rotation and drift ratio demands in longitudinal and transverse directions of the 5m high bridges.

| Bridge model       | $\theta_{\text{long}}$ | Drift long | $\theta_{\text{transv}}$ | Drift transv |
|--------------------|------------------------|------------|--------------------------|--------------|
| P05                | 0.0589                 | 0.0641     | 0.0232                   | 0.0253       |
| P05_E10_R0.5%      | 0.0395                 | 0.0448     | 0.0278                   | 0.0311       |
| P05_E10_R1.0%      | 0.0319                 | 0.0415     | 0.0179                   | 0.0205       |
| P05_E10_R1.5%      | 0.0335                 | 0.0394     | 0.0237                   | 0.0282       |
| P05_E15_R0.5%      | 0.0360                 | 0.0411     | 0.0180                   | 0.0198       |
| P05_E15_R1.0%      | 0.0326                 | 0.0381     | 0.0221                   | 0.0266       |
| P05_E15_R1.5%      | 0.0339                 | 0.0389     | 0.0219                   | 0.0234       |
| P05_E20_R0.5%      | 0.0404                 | 0.0411     | 0.0211                   | 0.0198       |
| P05_E20_R1.0%      | 0.0285                 | 0.0328     | 0.0144                   | 0.0156       |
| P05_E20_R1.5%      | 0.0279                 | 0.0324     | 0.0141                   | 0.0153       |

As expected, the longitudinal direction had larger demands than those demands in transverse direction of analysis. The pier deformation in longitudinal direction is single curvature whereas in transverse direction a frame type behavior is presented with a double curvature shape. The rotation ratio between the original models and the retrofitted models is in the range of 1.46 to 2.11 in longitudinal direction and in the range of 0.83 to 1.65 in transverse direction. In general, the rotation demands decrease faster when the reinforcement ratio changes from 0.5% to 1.0% than they change with an increase of the reinforcement ratio from 1.0% to 1.5%. It is also evident that the change of the jacket thickness (from 0.10 to 0.20 m) is less important than the change in the longitudinal reinforcement (from 05% to 1.5%) for the rotation and drift demands in both directions of analysis. Figure 6 shows the rotation demands of the 5 m and 15 m high bridges in longitudinal direction.

The horizontal dot line is the rotation demand of the non-retrofitted bridge models and the other three lines are the rotation demands of the retrofitted models. The three jacket thicknesses in the 5 m high bridges reduce notably the pier rotation demands and the results are similar regardless the thickness value. As mentioned before, changing the reinforcement ratio from 0.5% to 1.0% decreases more the demand than in the range of 1.0% to 1.5%.
Higher bridges reduce the efficiency of the RC jacketing. The 15 m high model (Figure 6) displays similar rotation demands of the original and the retrofitted models with the smallest reinforcement ratio and 0.10m jacket thickness. However, slightly smaller rotation demands are presented with the other two jacket thickness. Even though the influence of the reinforcement ratio is similar to the results of 5 m high bridge, the drop of the rotation demands in the range of 0.5% to 1.0% is not now quite different than the drop between 1.0% and 1.5%.

Table 5 displays the column rotation demands for the 15 m and 25 m high pier bridges. The rotation demands continue decreasing with the height increase of the bridges. Now, the rotation ratio between the original models and the retrofitted models is in the range of 1.01 to 1.45 in longitudinal direction and in the range of 0.79 to 1.01 in transverse direction of the 15 m high bridges. These values are in the range of 0.87 to 1.24 and 0.70 to 1.01 in the 25 m high models. Even though the general trends mentioned in the former paragraph are similar, the contribution of the RC jacketing to reduce the seismic response of the bridges is reduced.

Finally, Figure 7 displays the drift demands in the longitudinal direction and the limit states of behavior of the 5 m, 15 m, 20 m and 25 m high bridges. The horizontal discontinuous lines are the limit states of behavior, the dot line is the response of the original models, whereas the rest of the lines are the rotation demands of the retrofitted models. According to the drift demands, the original 5 m high model is located in the complete damage zone while the rehabilitated bridges are in the moderate limit state of behavior. The bridges with higher piers present less expected damages reducing the limit state of behavior to the zone of moderate and in a few cases extensive damages.

Table 5. Rotation demands in both directions of the 15 m and 25 m high bridges.

| Bridge model | P15 | P25 |
|--------------|-----|-----|
|              | $\theta_{\text{long}}$ | $\theta_{\text{transv}}$ | $\theta_{\text{long}}$ | $\theta_{\text{transv}}$ |
| Original     | 0.0249 | 0.0104 | 0.0168 | 0.0079 |
| E10_R0.5%    | 0.0247 | 0.0132 | 0.0176 | 0.0089 |
| E10_R1.0%    | 0.0222 | 0.0103 | 0.0161 | 0.0085 |
| E10_R1.5%    | 0.0205 | 0.0122 | 0.0161 | 0.0085 |
| E15_R0.5%    | 0.0233 | 0.0108 | 0.0186 | 0.0115 |
| E15_R1.0%    | 0.0205 | 0.0115 | 0.0191 | 0.0109 |
| E15_R1.5%    | 0.0183 | 0.0131 | 0.0194 | 0.0110 |
| E20_R0.5%    | 0.0200 | 0.0119 | 0.0172 | 0.0078 |
| E20_R1.0%    | 0.0178 | 0.0132 | 0.0140 | 0.0092 |
| E20_R1.5%    | 0.0172 | 0.0128 | 0.0136 | 0.0090 |
5. Conclusions

The results of the analyses allow concluding that small reinforcement ratios combined with jacket thickness in the range of 0.10 m to 0.15 m are the most efficient way to reduce the seismic response of the bridges. The height of the pier resulted also an important parameter to be considered to select the best retrofit technique. The nonlinear analyses showed that the efficiency of the RC jacketing is reduced with the pier height increase, showing important benefits in bridges with short length piers.

The seismic responses of the retrofitted models with piers higher than 5 m were similar regardless the jacket thickness and the reinforcement ratio. It was also observed that changes in the reinforcement ratio modify more the seismic response than the variations of the jacket thickness.

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