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Seismic performance evaluation of retrofitted multi-column bridge bents

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ABSTRACT: There are many important RC bridges in Canada, which do not meet the seismic standards. In this study a three column bridge bent has been considered, which was designed in the pre-1965 with inadequate seismic detailing. Several retrofitting provisions have been considered in this study to improve the seismic performance of a gravity load designed bridge under seismic forces where the different retrofitting techniques include steel jacketing, CFRP jacketing and steel bracing. Nonlinear pushover analyses have been performed for the original and retrofitted frames. An artificial ground motion record typical for Vancouver, B.C. has also been used to evaluate the dynamic response of these structures. Based on the analyses seismic demand/capacity ratio, drift ratio, ductility, has been estimated. On the basis of these results the best retrofitting technique has been proposed for such a gravity load design multi column bridge bent.

1 INTRODUCTION

In Canada highway bridges play an important role to build a smooth and fast communication system in between cities and across the country. There are many reinforced concrete (RC) bridges in Canada which were designed and constructed before 1965 without proper reinforcement detailing against seismic loads, and mostly designed for gravity loads only. Improper reinforcement detailing of bridges may lead to non-ductile catastrophic failure as observed during a number ofearthquakes for instance, 1995 Hanshin–Awaji (Kobe), and 1999 Kocaeli (Turkey) earthquakes. Besides, there are many bridges which have been seismically designed but not capable to meet the increasing traffic demand. The replacement of these bridges will be a costly undertaking, where alternatively if the bridges could be retrofitted/strengthened for current seismic and traffic demand; it could save a substantial amount of money for the Canadian economy. Various rehabilitation techniques are available to upgrade the seismic performance of existing RC structures. The major techniques for structural rehabilitation of RC bridges include encasing columns and beam column joints using steel, FRP or RC jackets or by adding new structural elements, such as steel bracings (Elfath and Ghobarah, 2000). Steel bracings are commonly used in Mexico, Japan, and other places for the rehabilitation of non-ductile RC buildings. Various researchers (Canales and Vega, 1992; (Kawamata and Masaki, 1980; Jones 1985; Goel and Lee, 1990; Yamamoto and Umemura, 1992; Maheri and Sahebi, 1995) proved experimentally that the steel bracing for RC frames significantly improve the strength as well as the stiffness of the RC frame. Miranda (1991) analytically proved the efficiency of steel bracings in improving the seismic performance of low rise RC buildings. Priestley et al. (1996) presented various seismic rehabilitation techniques of reinforced concrete column using steel, concrete, and fiber-reinforced polymer (FRP) composite jacketing. The seismic retrofit design guidelines for reinforced concrete column with FRP jackets was proposed by Seible et al (1997). Nanni et al. (1999) strengthened the piers of an RC bridge using FRP jackets and tested the pier up to failure. Tsai and Lin (2001) and Tsai et al(2002) has proposed octagonal shaped steel jacket for rectangular reinforced concrete column. Their test results indicate that proportioned octagonal steel jackets improve the ductility and cyclic strength of bridge columns lacking in flexural and shear strength. Tsai and Lin (2001), Harries at al. (1999), Sun et al.(1993)showed that the rectangular steel jacket for rectangular reinforced concrete column cannot efficiently provide lateral confinement. Sun et al. (1993) showed that elliptical steel jacket is more efficient for rectangular RC column.

The objective of this research is to compare the performance of a pre-1965 designed multi column bridge bent retrofitted with different rehabilitation techniques, specifically FRP jacketing, steel jacketing and steel bracing. Finite element analyses have been carried out to determine the strength, stiffness, and ductility of various retrofitting schemes and the results have been compared to choose the best possible option among the three techniques. Nonlinear dynamic time history analyses were also performed to determine the capacity/demand ratio in terms of base shear and drift.

2 BRIDGE BENTS DETAILS

To evaluate the performance of the retrofitted multi-column bridge bent, the northbound lanes of the South Temple Bridge is considered in this study. The bridge was built in the year 1963 and had several deficiencies in the amount and seismic detailing of the steel reinforcement; in addition, the steel reinforcement was significantly corroded in the bent cap. Pantelides and Gergely (2002) retrofit this bridge by using CFRP and performed both experimental and analytical verification. The bent consists of three columns and a bent cap, as shown in Figure1. The bents support eight steel girders and a concrete deck spanning between the two bents, with a 21.87 m (71.73 ft) span. Reinforcement details of the column, bent cap, and joints are shown in Figure2. The column transverse reinforcement was not sufficient in the lap-splice region, and the longitudinal reinforcement had inadequate anchorage in the pile caps and the bent cap. There were no transverse hoops in the bent cap joints; and tie spacing in the plastic hinge regions of the columns was insufficient. The steel reinforcement in the bent cap had significant levels of corrosion; however, there was no obvious sign of corrosion in the columns. Each steel girder carriesa gravity load (including live load) of 240 kN (53.95 Kip). The yield strength of the reinforcing steel was 275 MPa (40 ksi). The compressive strength of the concrete was 21 MPa (3 ksi).

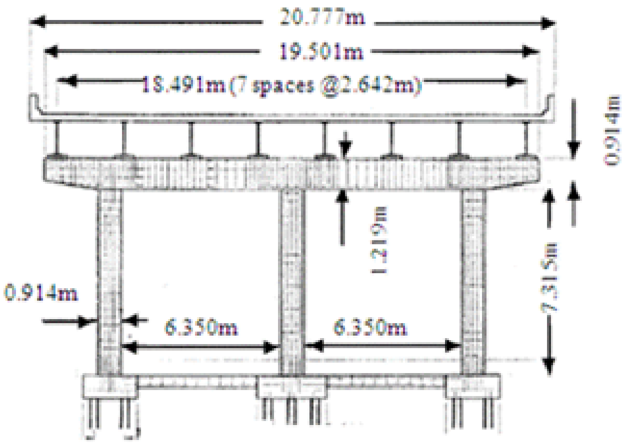


Figure 1 South Temple Bridge bent dimensions (Adopted from Pantelides and Gergely(2002))

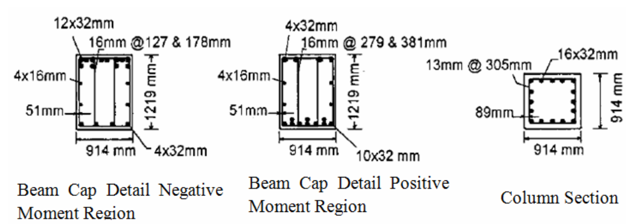


Figure 2 South Temple Bridge bent reinforcement details (Adopted from Pantelides and Gergely (2002)

3 RETROFITTING TECHNIQUES

Different retrofitting techniques have been considered in this study specifically CFRP composite jackets, steel jacketing and steel bracing for multicolumn bridge bents. Design procedures for the considered techniques are given below:

3.1 *CFRP composite jackets:*

Here the CFRP composite jacket retrofitting technique has been implemented from Pantelides and Gergely(2002) The material is a carbon fiber/epoxy resin composite with 48,000 fibers per tow unidirectional carbon fibers. The number of tows per 25.4 mm (1 inch) of sheet (pitch) was 6.5, and the width of the carbon fiber sheets was 457 mm (18 inch). The properties of the ambient temperature-cured CFRP composite were determined according to the ASTM D 3039 specifications (ASTM 1996) where the design parameters include modulus of elasticity =65 GPa (9425ksi); tensile strength =628 MPa (91ksi); ultimate axial strain =10 mm/m; layer thickness =1.32 mm; and fiber volume fraction=35.

3.2 *Steel jackets*

Secondly, elliptical steel jacket has been utilized, which has been found effective for retrofitting of rectangular reinforced bridge columns (Sun et al (1993)). According to FHWA-HRT-06-032 (2006) the gap between the steel jacket and the column is grouted with a pure cement grout, after flushing with water. . The shell thickness and plastic hinge length has been calculated as per FHWA-HRT-06-032 (2006)and calculated value for thickness is 6.68mm (0.26 inch). A conservative estimate for thickness is 10mm and the plastic hinge length is 1200 mm (4 ft). The yield strength for shell element is considered as 400 MPa (58ksi).

3.3 *Steel bracing*

Steel cross-bracing has been considered as another potential technique for retrofitting the deficient multi-column bridge bent. The bracings have been designed as per CSA S16-09 (2009) standards to carry only axial forces i.e. both tension and compression. As per CSA guidelines the slenderness ratio for the bracing member shall not be greater than 200. The bracing member has been designed considering moderately ductile member. To satisfy the buckling criteria a HSS 178x178x6.4 (HSS 7x7x0.25) section has been chosen as a bracing member. This section has least radius of gyration of 69.9. So, four HSS 178x178x6.4 (HSS 7x7x0.25) section has been added in the original model to develop the steel bracing retrofitting model. The yield strength for bracing member is considered as 275 MPa (40ksi).

4 FINITE ELEMENT MODELING

Finite element (FE) analysis has been performed to determine the seismic performance of the original and retrofitted multi-column bridge bents. The bridge bent is assumed to be located in the southwestern corner of the province of British Columbia, Canada (Seismic site classification type "C") on very dense soil and soft rock with un-drained shear strength of more than 100 KPa (14.5 psi). Nonlinear static and dynamic time-history analyses have been performed on the bridge bents using a FE package (SeismoStruct). The FE program is capable of predicting large displacement behavior of structures taking into account both geometric nonlinearities and material inelasticity. The fibre modeling approach has been employed to represent the distribution of material nonlinearity along the length and cross-sectional area of the member. 3D beam elements have been used for modeling the beam and column where the sectional stress-strain state of the elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres in which the section has been subdivided following the spread of material inelasticity within the member cross-section and along the member length.

To develop the analytical model in Seismostruct, Menegotto- Pintosteel model (Menegotto and Pinto 1973) with Filippou isotropic strain hardening property is used for reinforcing steel material. The yield strength of steel is 275MPa (40ksi), strain hardening parameter 0.5% and modulus of elasticity 2x105MPa (29000ksi) has been considered here. For concretenonlinear variable confinement concrete model, Madas and Elnashai (1992)with compressive strength 18.7MPa (2.71 Ksi) and tensile strength 1.7 MPa has been used. The bridge model geometry is generated using Seismostruct(2010). Then static nonlinear pushover analyses have been performed for the bridge model and the results were then compared with the experimental and analytical results obtained from Pantelides and Gergely (2002). Figure3 presents the comparative study of pushover response curve for the previous experimental and analytical result on the south temple bridge bent along with the results obtained from the present analytical model. The present analysis provided better results compared to those of previous analytical results (Pantelides and Gergely 2002) and could simulate the initial stiffness, post elastic stiffness, and ultimate load carrying capacity accurately when compared to the test results.

To develop the analytical model for bridge bent retrofitting with CFRP composite material, Trilinear FRP model (FIB (2006)) with tensile strength 628MPa (91ksi), initial stiffness $6.5x10^4$MPa (9425 ksi) and post-peak stiffness -5.0x105MPa(72500 ksi) has been selected. The retrofitted parts of the bridge bent have been modeled in Seismostruct with jacketed rectangular section. Then static nonlinear pushover analysis has been performed to verify theanalytical model and compare the results to those of Pantelides and Gergely (2002). The comparison of the analyses results is shown in Figure4. The results show that the present analytical model had similar initial stiffness to that of the tested specimen whereas the analytical model presented by Pantelides and Gergely(2002) exhibited much lower stiffness than the experimental result. However, both models overestimated the lateral load carrying by about 14%.

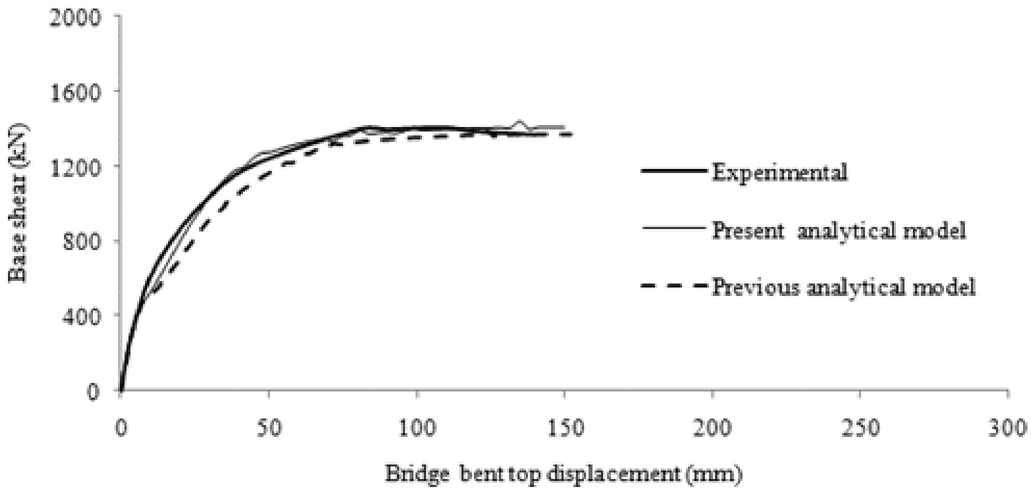


Figure3: Pushover response curve for original bridge bent

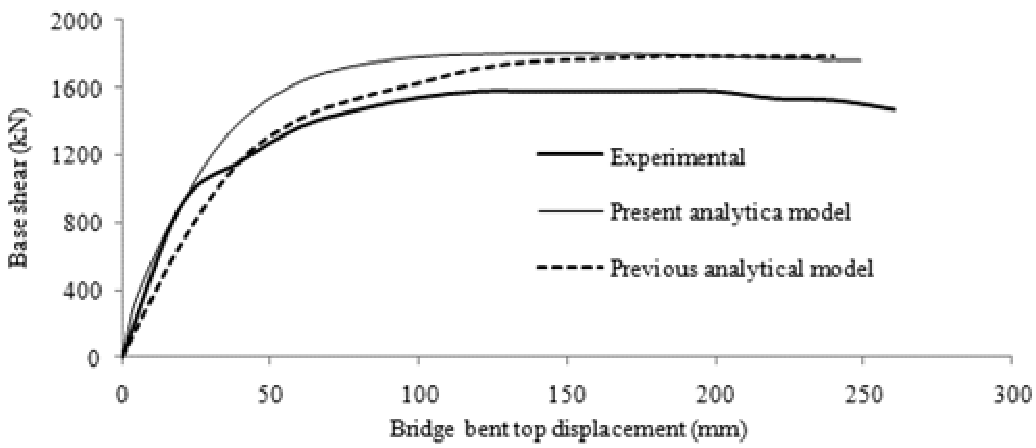


Figure4: Pushover response curve for bridge bent retrofitting with CFRP

However, the present model could predict the lateral drift corresponding to the capacity accurately. Figure3 shows that the experimental base shear reached its capacity at a drift of 120 mm (4.72 inch) whereas the present model reached its base shear capacity at the same drift of 120 mm (4.72 inch).

5 NON-LINEAR STATIC PUSHOVER ANALYSIS

Pushover analysis has been performed for each bridge bent considering a 2-D frame. The girder load including live load of 240 kN(53.95 Kip) was applied as a permanent load at each girder location and for the pushover analysis incremental load was applied in the form of displacement. The pushover response curve is shown in Figure5, for different retrofitting techniques like steel jacketing, steel bracing, CFRP jacketing and the original bridge bent. From Pushover responses curves it is observed that the lateral capacities for the bent retrofitted with steel bracing is the maximum and bridge bent without retrofitting is the minimum. The lateral capacity for CFRP and steel jacketing is very close to each other.

Figure6 presents the ductility for bridge bent retrofitted with different retrofitting techniques. Ductility is defined as the ratio of the maximum bent top displacement to the bent top yield displacement. The maximum ductility is achievedfor the steel bracedbridge bent, which is 7.8. The original bridge bent experienced the least amount of ductility of 2.7. For CFRP and Steel jacketing the ductility value was 3.3 and 4.1, respectively. The bridge bent retrofit with steel bracing is 2.89 times more ductile than the original bridge bent without retrofitting condition whereas bridge bent retrofitted with CFRP and steel jacket is 1.22 and 1.51 times more ductile than the original bridge bent without retrofit. From this result it can be concluded that the bridge bent retrofit with steel bracing is more ductile than any other retrofitting techniques.

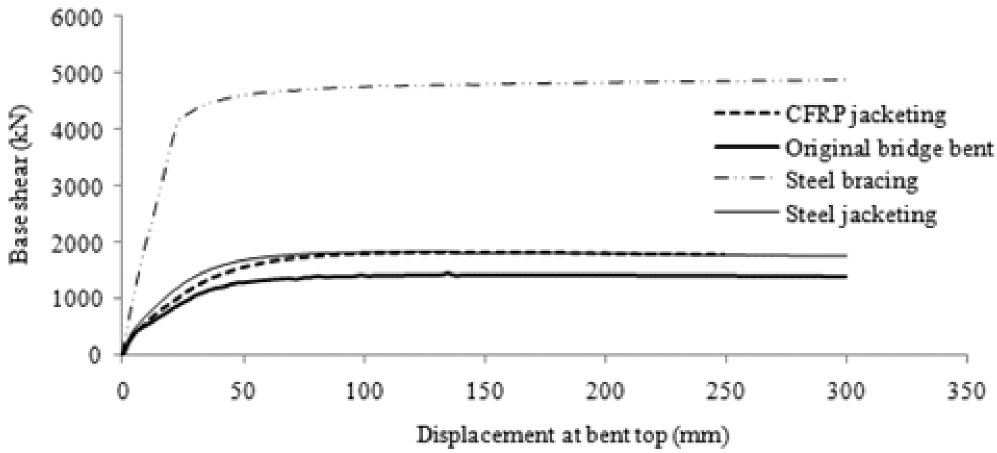


Figure5: Pushover response curve for bridge bent

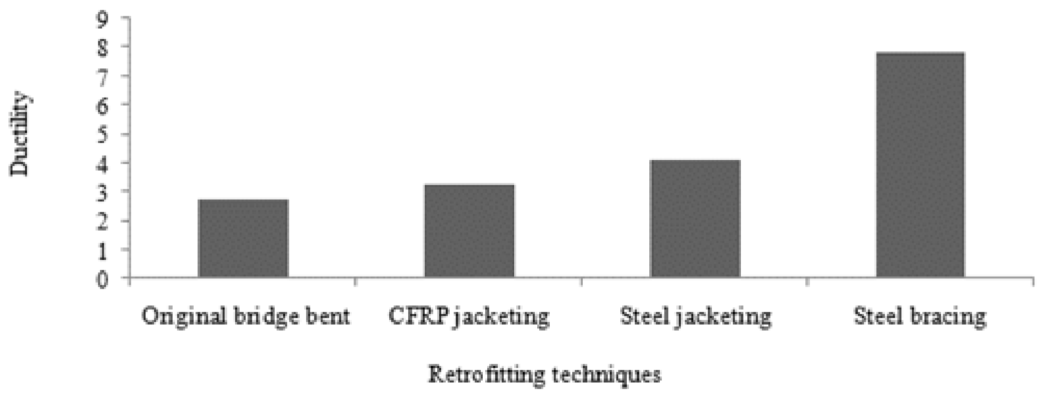


Figure6: Ductility for bridge bent.

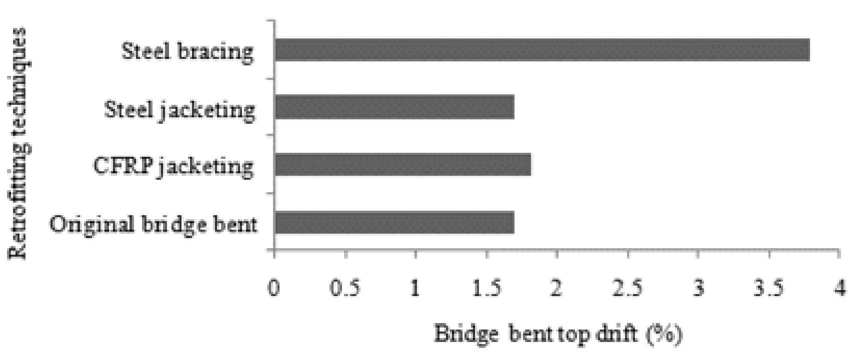


Figure7: Top drift (%) for bridge bent retrofitted with various techniques.

Figure7 presents the top drift (%) for bridge bent retrofitted with various techniques. The drift (%) for different bent has been calculated by dividing the maximum bent top displacement by the height of the bridge bent and multiplied by hundred. The maximum bent top displacement was found from the nonlinear static pushover analyses. The maximum bent top displacement is the value of the pushover curve corresponding to the maximum base shear. From Figure7 it is observed that the bridge bent retrofitted with steel bracing allows more displacement before failure. The bridge bent retrofit with CFRP and steel jacket are allowed same bent top displacement before failure. From this result it can be observedthat the bridge bent retrofitted with steel bracing are subjected to more top displacement before failure.

6 DYNAMIC TIME HISTORY ANALYSIS

For dynamic time history analyses an artificial earthquake ground motion data has been used, which was developed by Atkinson (2009) for Vancouver city. Figure8 shows the spectral acceleration for the time history data used to analyze the bridge bent and the spectrum that is proposed in NBCC-2005. The dynamic analyses were carried out for all bridge bents. The dynamic time history analyses results are used to compute the demand capacity ratio in terms of base shear, axial capacity and in terms of bent top displacement. The capacity of the structure is obtained from static nonlinear pushover analyses and the demand of the structure is obtained from the nonlinear dynamic time history analyses.

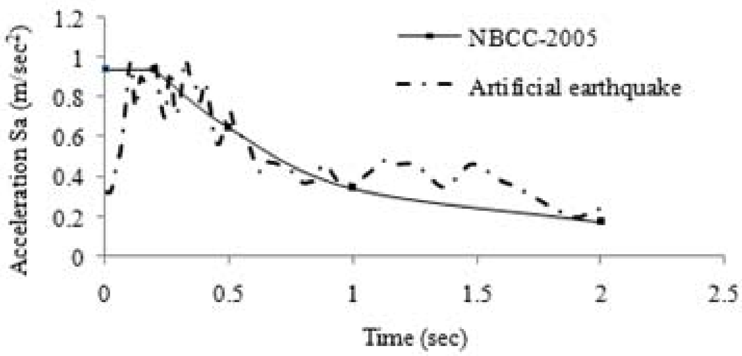


Figure8: Variation of spectral acceleration with period of structure

The hysteretic behaviour in terms of base shear versus bent top displacement of bridge bent without retrofitting, retrofit with CFRP jacketing, steel jacketing and steel bracing are presented inFigure9. The base shear demand is defined as the average maximum base shear obtained for earthquake loading and the capacity is defined as the amount of base shear obtained from pushover analyses.

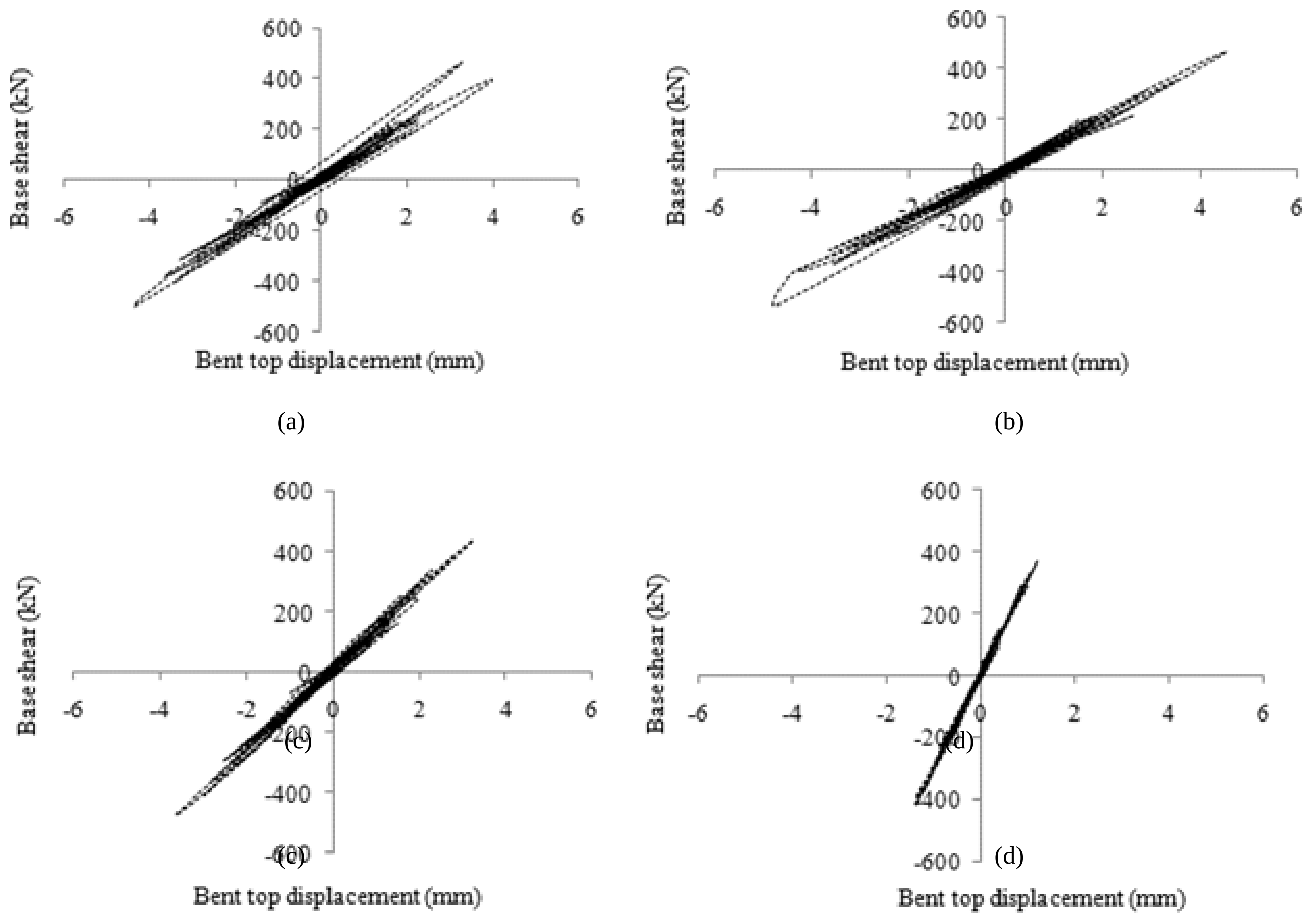
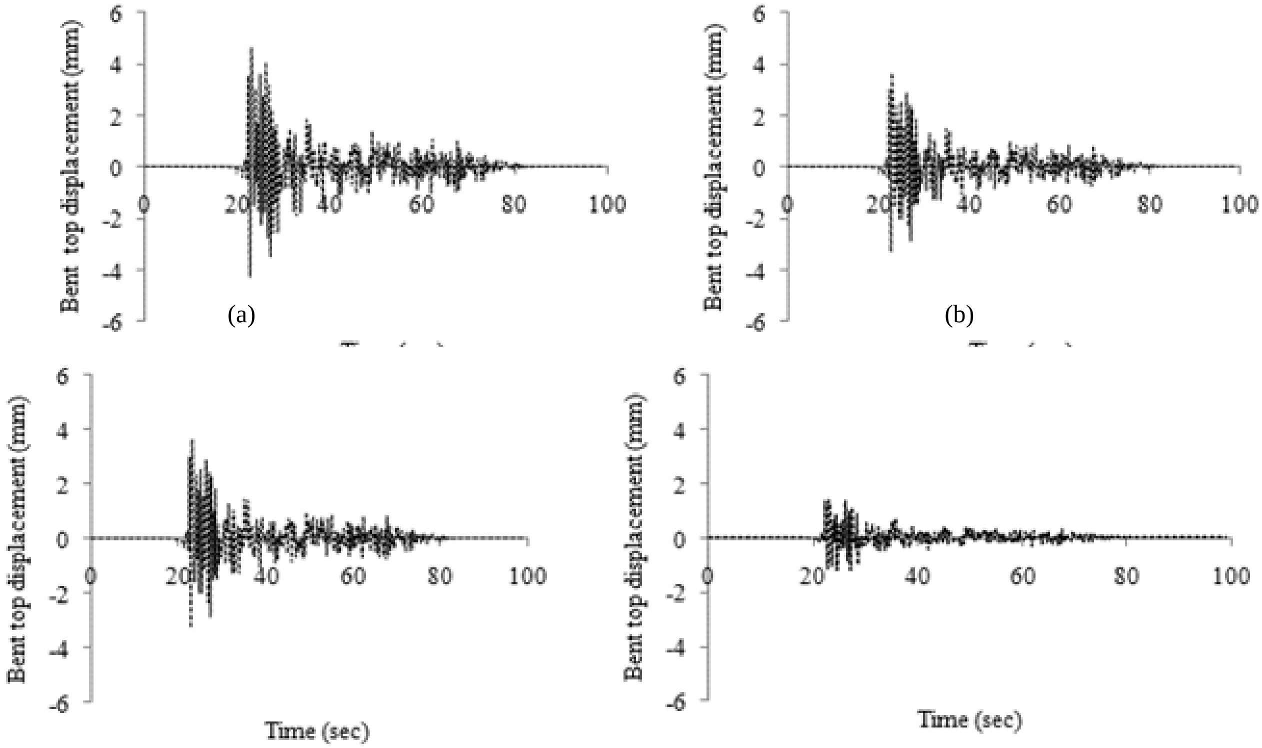


Figure 9: Base shear versus roof displacement hysteresis for bridge bent, (a) without retrofitting. (b) retrofit with CFRP jacketing, (c) retrofit with steel jacketing and (d) retrofit with steel bracing.



(c) (d)

Figure 10: Bent top displacement Spectrum for bridge bent, (a) without retrofitting. (b) retrofit with CFRP jacketing, (c) retrofit with steel jacketing and (d) retrofit with steel bracing.

Bent top displacement time histories for bridge bent without retrofitting, retrofit with CFRP jacketing, steel jacketing and steel bracing under earthquake loading are presented in Figure10. From the base shear versus bent top displacement hysteresis figure it can be concluded that the bridge bent without retrofitting condition is subjected to more displacement and base shear whereas the bridge bent retrofitted with steel bracing is subjected to minimum bent top displacement and base shear.

From bent top displacement spectrum it is observed that the bent top displacement is the maximum for bridge bent without retrofitting and minimum for bridge bent retrofitted with steel bracing. Drift capacity is defined as the global drift of the bent, which is obtained from pushover analyses. The drift demand is defined as the average maximum bent top drift when subjected to an earthquake load.

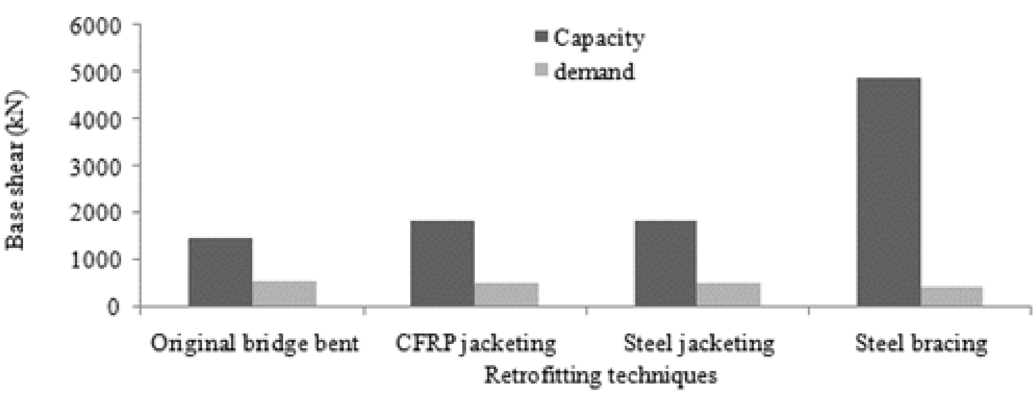


Figure 11: Base shear capacity and demand.

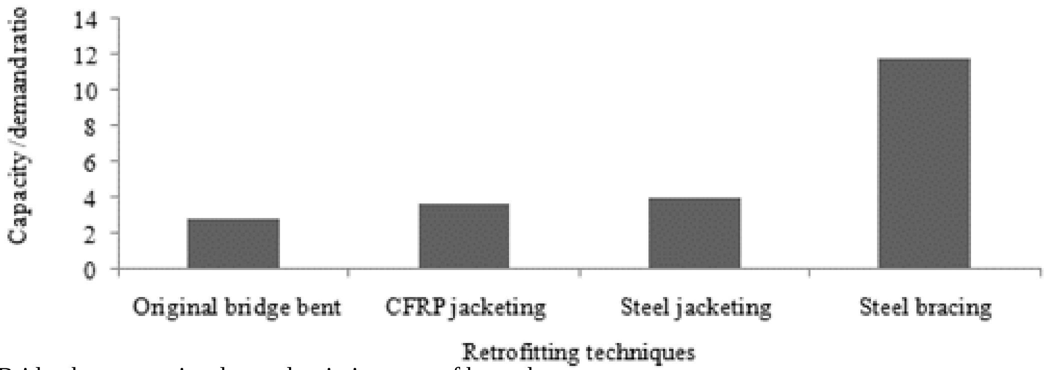


Figure12: Bridge bent capacity-demand ratio in terms of base shear.

The base shear capacity-demand for bridge bent retrofitted with different techniques is given in Figure11. From Figure11 it is observed that the capacity for bridge bent retrofitted with steel bracing is the maximum and seismic demand is the minimum. It is also observed that the base shear capacity of the original bridge bent without retrofitting is minimum whereas its seismic demand is maximum. The capacity-demand ratio is depicted in Figure12. From this Figure it is observed that the capacity demand ratio in terms of base shear for bridge bent retrofitted with steel bracing is 11.73 whereas the capacity demand ratios are 2.73, 3.66 and 3.9 for the original bridge bent without retrofitting, CFRP retrofitted bridge bent, and steel jacketing retrofitted bridge bent, respectively.

The bent top horizontal drift capacity-demand ratio for bridge bent retrofitted with different techniques is illustrated in Figure13. From Figure13 it is observed that the capacity for the bridge bent retrofitted with steel bracing is maximum whereas its seismic demand is minimum. It also shows that the bent top drift capacity is minimum for the original unretrofitted bent whereas its seismic demand is maximum. The capacity-demand ratio is presented in Figure14 where the best performance is provided by steel bracing.

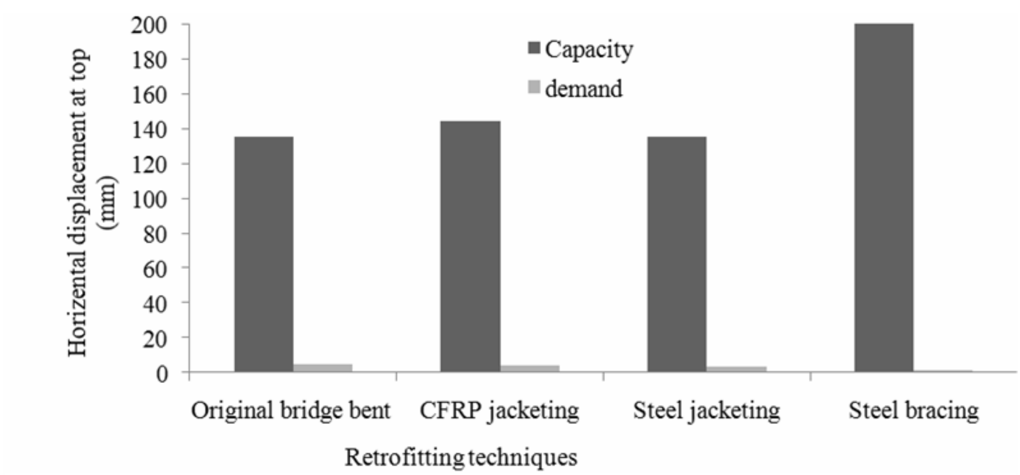


Figure13: Bridge bent top displacement capacity-demand.

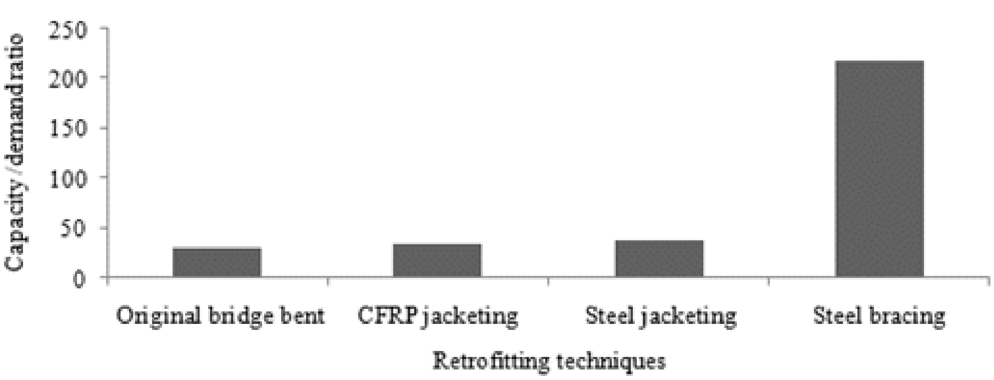


Figure14: Bridge bent top displacement capacity-demand ratio in terms of displacement.

7 CONCLUSIONS

The performance of a multi-column bridge bent has been evaluated in this study using different retrofitting schemes for instance, CFRP jacketing, steel jacketing and steel bracing. A three column bridge bent has been considered here which is a part of the northbound lanes of the South Temple Bridge constructed in 1962.

In this study, the performance of the bridge bents using different retrofitting techniques has been evaluated based on ductility, allowable bridge bent top drift (%), base shear capacity-demand ratio and bridge bent top displacement capacity-demand ratio. Here, ductility is defined as the ratio of maximum bent top displacement before global collapse to the global yield displacement of the bent top. From this result it is concluded that the multi column bridge bent becomes more ductile if steel bracing is used as a retrofitting technique. Steel jacketed bridge bent is more ductile compared to that of CFRP jacketed bridge bent. The allowable bridge bent top drift (%) is defined as the maximum bent top displacement before collapse to the height of the bent and multiplied by hundred. This value is also obtained from nonlinear static pushover analyses. The results show that the steel bracing system allows more bent top displacement before collapse. The base shear capacity is obtained from the nonlinear static pushover analyses and the demand is obtained from the dynamic time history analyses. The base shear capacity demand ratio is the maximum for bridge bent retrofitted with steel bracing system. The base shear capacity-demand ratio for bridge bent retrofitted with steel jacketing is more than the bridge bent retrofitted with CFRP jacketing. The bent top drift capacity is obtained from the pushover analyses and the demand is obtained from the dynamic time history analyses. On the basis of this indicator the most preferred retrofitting technique is again steel bracing system and the second preferred option is the steel jacketing system.

From the above discussion it is clear that the most desired retrofitting technique is the steel bracing system for multi-column bridge bent because the steel bracing system performed better than any other retrofitting systems considered in this study.

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