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Finite element modeling and development of performance-based damage states for retrofitted multi-column bridge bent

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ABSTRACT: Being an inevitable element of civil infrastructure, transportation infrastructure plays a vital role in a country's economy. It provides communication within a country and beyond. As bridges ensure continuity in transportation infrastructure, their failure in earthquake can cost Canada $50 billion affecting post-seismic rehabilitation program. According to Ministry of Transportation and Infrastructure in British Columbia, bridges built before 1983 are vulnerable to earthquake because of not being designed for seismic effect. These bridges lack well-designed columns with proper ductility and high shear strength, strong steel bearings, wide seat, and pile caps with sufficient reinforcement. They have reinforced concrete multi-column bents that support the superstructure. In this research, detailed finite element models of as-built and retrofitted multi-column bents have been developed and validated against available experimental results. Four different jacketing techniques such as, carbon fiber reinforced polymer (CFRP), steel, engineering cementitious composite (ECC), and concrete have been considered in this study. Multi-column bents were retrofitted according to CSA S6-19. Using the validated numerical models, performance-based damage states are developed for the retrofitted bridge bents. This study will aid engineers to obtain material strain limits for retrofitted bridge currently unavailable in CSA S6-19.

1 INTRODUCTION

Being earthquake-prone, British Columbia (BC) endangers its transportation infrastructure that has 2555 bridges of which many do not meet current requirements of Canadian Highway Bridge Code (Siddiquee & Alam 2017). Besides, failure of transportation infrastructure including aging bridges in BC in an earthquake of magnitude 9 can incur a financial loss of $670 million (IBC 2013). Budget deficit, spillover, eco factors, and poor construction management have greatly influenced the transportation infrastructure so far. Moreover, increasing fund for transportation infrastructure has not been enough to *fulfill* the yearly cost of rehabilitating the existing bridges or to reduce the cumulative pending maintenance for many years. Thus, it will cost less to retrofit these bridges to satisfy requirements of CSA S6-19. Bridges requiring seismic retrofitting are classified into four types: Lifeline, disaster response route, economic sustainability route, and other bridges (Kennedy & Huffman 2005). Of them, lifeline and disaster response route bridges are the most important for post-seismic rehabilitation program. Furthermore, the failure of reinforced concrete multi-column bents supporting the superstructure during an earthquake can damage the whole bridge resulting in notable economic loss. There are different techniques to retrofit bridges. Of them, Steel and CFRP jacketing are the most popular for easy application. Besides, concrete and ECC jacketing can also be used though they are widely accepted for retrofitting buildings because of formwork difficult to be provided for bridges. Many researchers investigated the better performance of bridge retrofitted using these techniques. Of them, Roy et al. (2010) developed a performance-based approach to retrofit a three column bent using CFRP jacket. A new confinement model for RC column retrofitted with CFRP was considered in the design of the retrofitted bent that achieved ductility for considered earthquakes. Moreover, using a performance-based seismic design, Kanaji et al. (2008) retrofitted a long-span truss bridge that can survive maximum credible ground motion. They considered seismic isolation of decks and buckling restrained braces for longitudinal and transverse directions, respectively to achieve three levels of performance. Based on life cycle cost estimation, considering both retrofit cost and risk performance level for which main truss members and lateral braces were elastic and inelastic within a repairable level respectively was optimum. Furthermore, using performance-based seismic design, Pantelides &

Gergely (2002) retrofitted a rectangular three column bent with CFRP such that it can achieve adequate ductility. They suggested to upgrade CFRP retrofit design considering experimental results. Though CSA S6-19 has strain limits of concrete and steel as performance criteria for four performance levels: Immediate, limited, service disruption, and life safety, it lacks those of different jacket materials e.g. concrete, steel, CFRP, and engineering cementitious composite (ECC). Besides, CSA S6-19 does not specify material strain limits for retrofitted bridge. The objectives of this study is developing reliable numerical models for deficient reinforced concrete multi-column bents retrofitted with various retrofit options and performance-based damage states for those retrofit options in terms of material strains.

2 BRIDGE BENT DETAILS

South Temple Bridge bent, shown in Figure 1, was taken for this study (Pantelides & Gergely 2002).It has three columns and a cap beam. This bridge was not designed for earthquake. Based on Figure 2, the bent has the following seismic deficiencies: Inadequate transverse reinforcement in column plastic-hinge and lap-splice regions, no stirrups in column-cap beam connections, and inadequate embedment of column rebars into cap beam and pile caps. The bent supports eight equally spaced girders. Each carries a gravity load of 240 kN. Yield and compressive strengths of reinforcement and concrete are 275 MPa and 21 MPa respectively.

2.1 *Details of Retrofit Options*

There are various options to retrofit deficient bridge bents such as steel, CFRP, ECC, concrete, external prestressing with steel, precast concrete segment, and ferrocement jacketing, infill shear wall, etc. Of them, concrete jacketing is the cheapest and good for underwater application. Besides, it can increase lateral stiffness and strength, ductility, and shear strength (Rodriguez & Park 1994). However, it can upsize the member as opposed to steel and CFRP jacketing. Steel jacketing has been widely used to increase strength of deficient bridge(Chai et al. 1994). Though, thin steel jacket cannot improve ductility markedly like shear strength (Xiao & Wu 2003). On the other hand, CFRP is preferred to steel and concrete because it is noncorrosive and has high strength and high modulus of elasticity (Seible at al. 1997). It has poor fire resistance even if it can be applied easily. Moreover, ECC jacketing has gained popularity for its ability in limiting crack width and improving ductility and shear strength of deficient structure (Li et al. 2000). Retrofitting with external prestressing steel can improve ductility of column subjected to low shear (Coffman et al. 1993). Precast concrete segment can also be used to retrofit underwater column for easy construction, economy, and resistance to corrosion (Iverson et al. 1999). Ferrocement jacketing can improve lateral stiffness and strength, energy dissipation, and ductility and can ensure ductile flexural failure (Kumar et al. 2005).

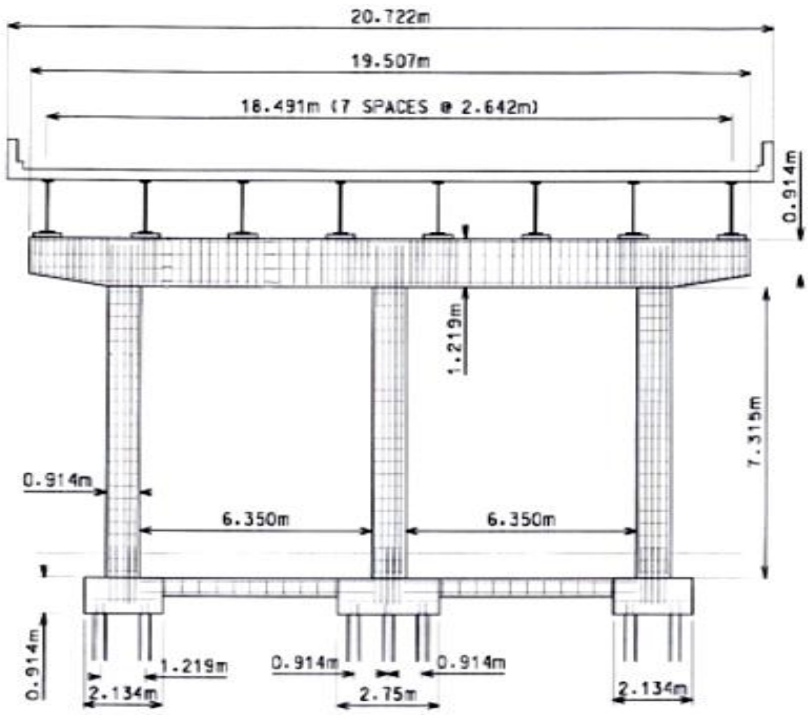

Figure 1. Bridge bent elevation. (Pantelides & Gergely 2002).

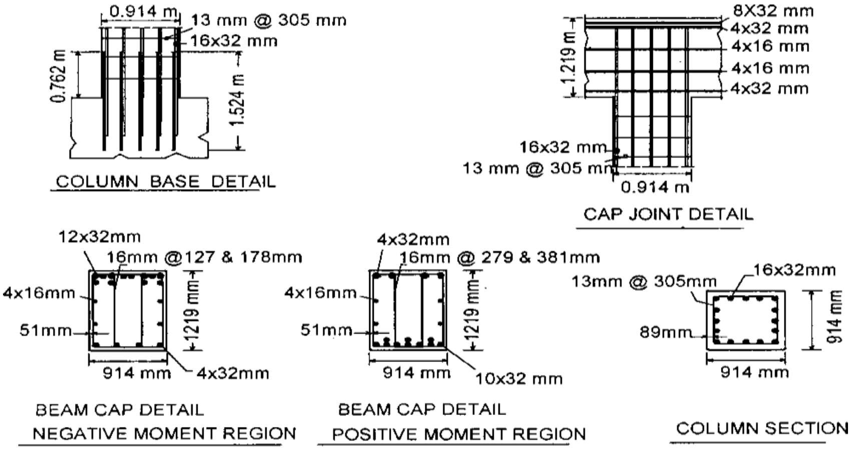

Figure 2. Column and cap beam cross-sections (Pantelides & Gergely 2002).

Infill shear wall can enhance lateral strength and ductility of deficient bridge bent quickly and efficiently too (Pulido et al. 2004).Four retrofit options: CFRP, steel, concrete, and ECC jackets were used to retrofit the bent considered in this study as they are readily available, economic, easy to be applied, and good in enhancing seismic performance. They were provided for a length of 914 mm along the column height at base and top of each column (Priestly et al. 1996). The bent was considered to be located in Vancouver to obtain design base shear of 1182 kN according to CSA S6-19. Tensile strength, ultimate tensile strain, and initial stiffness of CFRP used in this study were 628 MPa, 0.01, and 64730 MPa respectively obtained from Pantelides & Gergely (2002). CFRP jacket thickness was determined as 5.28 mm using Equations 1-2 considering an ultimate compressive strain of 0.0105 (Priestly et al. 1996).

$$\varepsilon_{cu} = 0.004 + \frac{1.25\rho_s f_{uj}\,\varepsilon_{uj}}{f_{cc}'} \qquad (1)$$

where ε_{cu} = ultimate compressive strain; ρ_s = volumetric ratio of confinement; f_{uj} = tensile strength of jacket material; ε_{uj} = ultimate tensile strain of jacket material; and f_{cc}' = compressive strength.

$$\rho_s = 2t_j\left(\frac{b+h}{bh}\right) \qquad (2)$$

where t_j = jacket thickness; b = column width; and h = column depth. Besides, ultimate tensile strain and yield strengthof steel jacket used in this study were 0.15 and 248 MPa respectively obtained from Priestly et al. (1996). Steel jacket thickness was determined as 10 mm using Equation 3considering an ultimate compressive strain of 0.0105 (Priestly et al. 1996).

$$t_j = \frac{0.18(\varepsilon_{cu} - 0.004)Df_{cc}'}{f_{yj}\,\varepsilon_{uj}} \qquad (3)$$

where D = jacket diameter; and f_{yj} = yield strength of steel. Moreover, concrete jacket thickness was determined as 160 mm using Equation 4 considering an ultimate compressive strain of 0.0105 (Priestly et al. 1996).

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}'} \qquad (4)$$

where f_{yh} = yield strength of transverse steel; and ε_{su} = ultimate tensile strain of transverse steel. Concrete jacket was reinforced with twelve 28-mm diameter rebars.10-mm diameter transverse reinforcement spaced at 150 mm and 10-mm diameter link bars through each column were used to provide lateral support for corner and middle longitudinal bars. Concrete, longitudinal bars and transverse reinforcement were considered to have the same properties of those of original column. Tensile strength and ultimate tensile strain of ECC used in this study were 6 MPa and 0.06 respectively obtained from Fischer & Li (2003). ECC jacket thickness was determined as 50 mm using Equations 1-2 considering an ultimate compressive strain of 0.0105 (Priestly et al. 1996).

3 FINITE ELEMENT MODELING

Using 3D inelastic displacement-based frame element, cap beam and columns were modeled using SeismoStruct 2020. This program can predict both stiffness and strength degradation of any frame structure subjected to static and dynamic loads considering both geometric and material nonlinearities. The fibre modeling method was used to consider nonlinearity of materials over the member length and cross-section. Every fibre's stress-strain curve was set up to model concrete with confinement effect and rebar. Menegetto-Pinto (1973), Mander et al. (1988), Trilinear FIB (2001,2006) and Han et al. (2003) constitutive models were used for steel, concrete, CFRP, and ECC respectively. The adopted modeling technique was validated against available experimental results on retrofitted bridge piers and bents.

3.1 *Model Validation with Experimental Results*

Pantelides and Gergely (2002) obtained the pushover curve, shown Figure 3, after a deficient bent had been subjected to cyclic load. After performing nonlinear static pushover analysis of the same bent modeled using SeismoStruct(2020), the pushover curve, shown in Figure 3, was obtained. Numerical model gave yield and maximum loads of 894 kN and 1268 kN, respectively which varied from those of experiment by 11% and 5%, respectively. Moreover, the authors obtained the hysteresis loops, shown in Figure 4, after a bent retrofitted with CFRP had been subjected to cyclic load. After performing quasi-static analysis of the same bent modeled using SeismoStruct (2020), the hysteresis response, shown in Figure 4,was obtained. Numerical model gave yield and peak loads of 976 kN and 2181 kN, respectively which varied from those of experiment by 2% and 1%, respectively. Vandoros & Dritsos (2008) obtained the hysteresis loops, shown in Figure 5, after a pier retrofitted with concrete had been subjected to cyclic load. After performing quasi-static analysis of the same pier modeled using SeismoStruct (2020), the hysteresis response, shown in Figure 4, was obtained. Numerical model gave cracking, ultimate, and failure loads of 127 kN, 143 kN, and 92.34 kN, respectively which varied from those of experiment by 11%, 4%, and 6% respectively. Furthermore, El Gawady et al. (2010) obtained the hysteresis loops, shown in Figure 6, after a pier retrofitted with steel was tested under cyclic load. After performing quasi-static analysis of the same pier modeled using SeismoStruct(2020), the hysteresis loops, shown in Figure 6, were obtained. Numerical model gave a peak load of 68 kN which varied from that of experiment by 3%.Gencturk & Elnashai (2011) obtained the hysteresis loops, shown in Figure 7, after an ECC pier was tested under reversed cyclic load. After performing quasi-static analysis of the same pier modeled using SeismoStruct 2020, the hysteresis loops, shown in Figure 7, were obtained. Numerical model gave yield, ultimate, and failure loads of 3.95 kN, 4.32kN, and 1.52kN respectively, which varied from those of experiment by 9%, 9%, and 6% respectively.

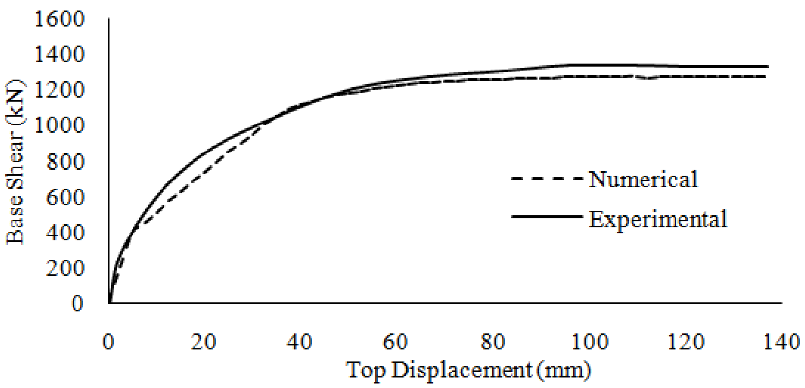

Figure 3. Experimental and numerical pushover curves of as built bridge bent (Pantelides & Gergely 2002).

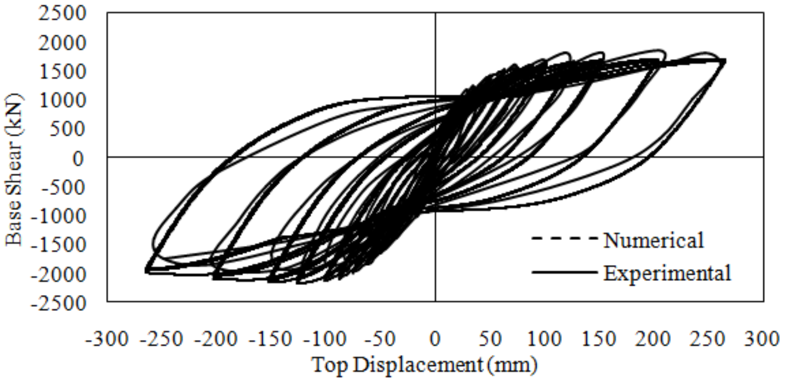

Figure 4. Experimental and numerical hysteresis loops of bent retrofitted with CFRP (Pantelides & Gergely 2002).

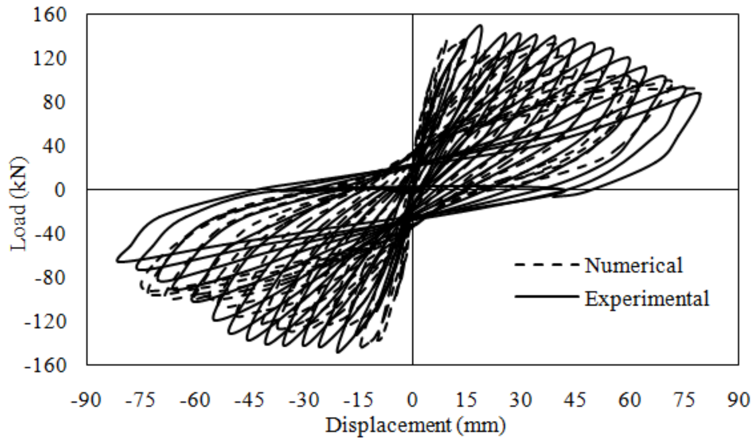

Figure 5. Experimental and numerical hysteresis loops of pier retrofitted with concrete. (Vandoros & Dritsos 2008).

4 RESULTS OF NONLINEAR STATIC PUSHOVER ANALYSIS

As CSA S6-19 does not have material strain limits as performance criteria for retrofitted bridge, performance-based damage states for various retrofit options in terms of material strains were developed in this study. 240 kN was applied as gravity load where girder is located. Nonlinear static pushover analyses were performed for as built and each retrofitted bents applying incremental load as displacement as they have been proven effective in predicting both strength and stiffness of bridge bent well (Billah & Alam 2014). Figure 8 shows push over curves of as built and retrofitted bent obtained from analyses. Different retrofitting techniques increased the lateral capacity markedly. Concrete and steel jackets performed the same in terms of lateral stiffness, and so did CFRP and ECC jackets. Though, all jackets exhibited almost the same lateral strength. Various retrofit options are compared at different performance-based damage states: Cracking of concrete, yielding of rebar, spalling of unconfined concrete, and crushing of confined concrete. Strain at which concrete cracks was calculated as 0.00013 dividing tensile strength of 2.75 MPa by modulus of elasticity of 21538 MPa. Besides, yield strain of steel was calculated as 0.00138 dividing yield strength of 275 MPa by modulus of elasticity of 200000 MPa. Spalling of unconfined concrete and crushing of confined concrete were considered to occur at compressive strains of 0.004 and 0.0105 respectively (Priestly et al. 1996).

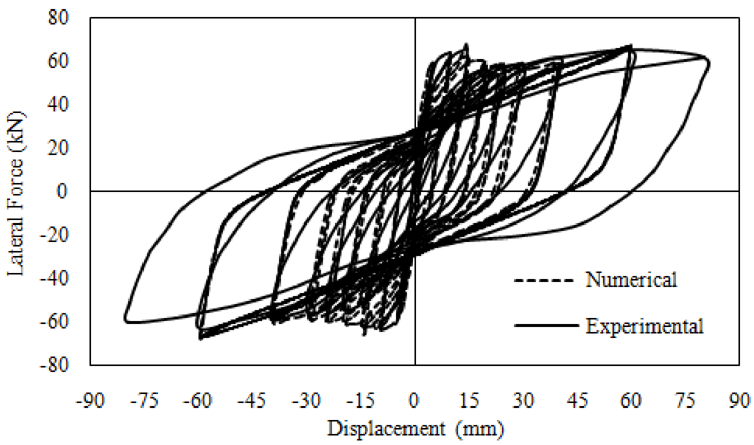

Figure 6. Experimental and numerical hysteresis loops of pier retrofitted with steel (El Gawadyet al. 2010).

The comparative performances of various retrofit options in terms of drift (%) and base shear (*V*) are shown in Table 1. For bridge bents retrofitted with steel, CFRP, and ECC, cracking started at the same drift, 50% more than that of bridge bent retrofitted with concrete. Furthermore, base shear at cracking for bent retrofitted with steel was 676 kN which was,19%, 29%, 36% more than those of bents retrofitted with ECC, concrete, and CFRP, respectively. Before yielding, bridge bents retrofitted with CFRP and ECC underwent the same drift that was 40% and 28% more than those of bridge bents retrofitted with steel and concrete, respectively. Before yielding, bent retrofitted with concrete encountered a base shear of 1241 kN which was1%, 3%, and 4% more than those of bents retrofitted with steel, ECC, and CFRP, respectively. Moreover, before spalling, bent retrofitted with ECC underwent a drift of 1.67% which was26%, 50%, and 54% more than those of bridge bents retrofitted with CFRP, concrete, and steel respectively. However, all jacketed sections encountered almost the same base shear before spalling. Before crushing, bridge bent retrofitted with CFRP sustained a drift of 2.32% which was 19%, 4%, and 2% higher than those of bridge bents retrofitted with steel, concrete, and ECC, respectively. Before crushing, though both concrete and CFRP jacketed sections experienced the least base shear, ECC jacketed section sustained the maximum base shear of 1560kNwhich is a little more than that of steel jacketed section.

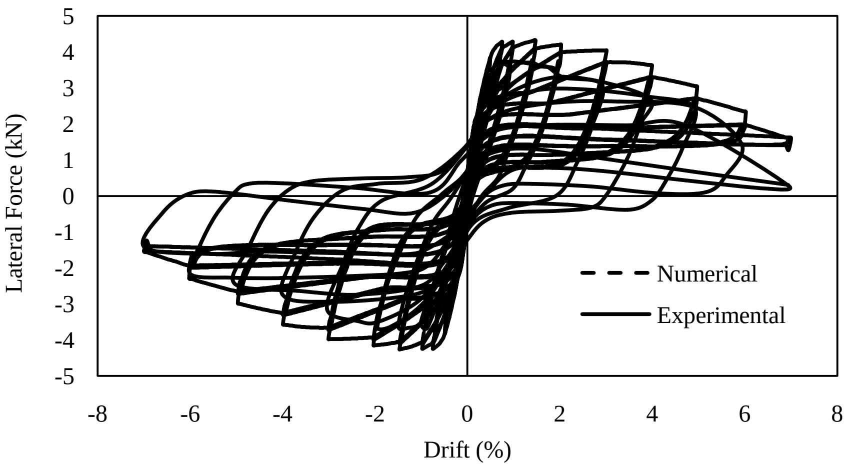

Figure 7. Experimental and numerical hysteresis loops of ECC pier (Gencturk & Elnashai 2011).

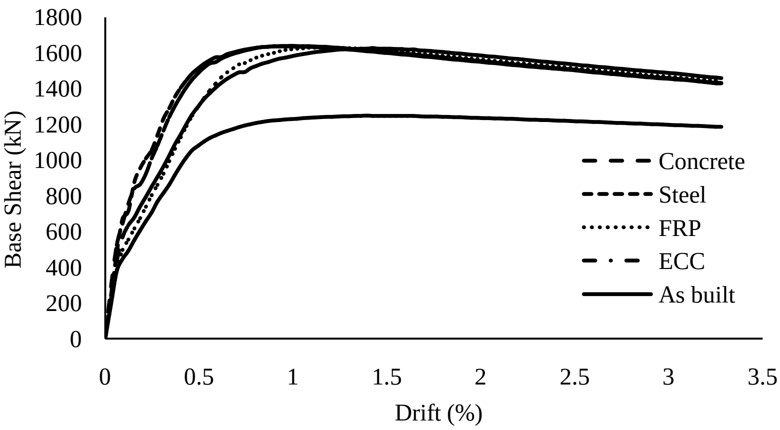

Figure 8. Pushover curves of as built and retrofitted bridge bents.

Table 1. Comparison of various retrofit options.

Techniques	Cracking		Yielding		Spalling		Crushing	
	Drift %	*V* kN	Drift %	*V* kN	Drift %	*V* kN	Drift %	*V* kN
Concrete	0.06	525	0.34	1241	1.11	1637	2.23	1532
Steel	0.09	676	0.31	1234	1.08	1634	1.95	1555
CFRP	0.09	497	0.43	1191	1.33	1626	2.32	1532
ECC	0.09	567	0.43	1204	1.67	1616	2.26	1560

5 CONCLUSIONS

This study aimed to develop reliable numerical models for deficient reinforced concrete multi-column bents retrofitted with various retrofit options: steel, concrete, CFRP, and ECC. Performance-based damage states for those retrofit options were also proposed in this study. The following outcomes are found:

- Numerical models can predict reliable lateral strength and stiffness of both as-built and retrofitted reinforced concrete bridge bents. Thus, they can be used instead of experiment to save significant time and money.
- Before cracking, multi-column bent retrofitted with steel encountered the maximum drift and base shear.

- Furthermore, before yielding, bridge bents retrofitted with CFRP and ECC underwent the maximum drift, whereas that retrofitted with concrete encountered the maximum base shear.
- Before spalling, multi-column bent retrofitted with ECC underwent the maximum drift, whereas that retrofitted with concrete encountered the maximum base shear.
- Moreover, before crushing, bridge bent retrofitted with FRP sustained the maximum drift, whereas that retrofitted with ECC encountered the maximum base shear.

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