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SEISMIC RETROFITTING OF REINFORCED CONCRETE BRIDGE BENTS UTILIZING HYSTERETIC DAMPERS

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Abstract

Typical reinforced concrete bridge bents constructed in the 1950 to mid-1970 in the Pacific Northwest of the United States were designed and built with minimum seismic considerations. As a result of this design practice, bridge bents have numerous deficiencies making them highly susceptible to damage following an earthquake. In this study, the performance assessment of a representative reinforced concrete bridge bent retrofitted using hysteretic dampers is investigated by performing nonlinear time history analysis. A refined numerical model is presented and validated in order to perform the nonlinear analyses. Special attention is put on assessing the bridge bent subjected to subduction zone ground motions. Buckling restrained braces (BRBs) is the main hysteretic damper investigated in this study. Other potential hysteretic devices for retrofitting RC bridge bents are also presented such as eccentrically braced frames (EBFs) and U-shaped dampers. The results of this study demonstrate the effectiveness of utilizing hysteretic dampers for achieving high displacement ductility of the retrofitted structure, while also controlling the damage of the existing vulnerable reinforced concrete bridge bent up to the required design performance level. The potential of improving the overall seismic behavior and the design performance levels with hysteretic dampers offers structural design professionals a viable method for performance driven retrofit of reinforced concrete bridge bents.

Keywords: Bridge seismic retrofit; reinforced concrete bridge bents; hysteretic dampers.

1. Introduction

Bridges in the United States and in other parts of the world, including Chile, are seismically designed based on the AASHTO requirements [1]. Under the AASHTO code requirements bridges are primarily designed to meet a standard performance level based on a life-safety approach, which means the bridge has a very low probability of collapse when subjected to earthquakes that are most likely to occur over the life of the structure. Nowadays, the seismic design philosophy of bridges is trending to a performance-based seismic design, in which different performance levels need to be satisfied under representative seismic hazards [2]. For seismic retrofit of bridges the Federal Highway Administration (FHWA) released a performance-based guideline in 2006 [3], which uses a multi-level performance criteria. Following the FHWA steps, a few Departments of Transportation in the United States have adopted the performance-based seismic design criteria in their manuals and regulations. This is the case of the Oregon Department of Transportation (ODOT), which for both new and existing bridges adopted a two-level performance criteria [4]. The two-level performance criteria adopted by ODOT comprises a "Life Safety" criteria under a 1000-year event (7% probability of exceedance in 75 years) and an "Operational" criteria under a 500-year event (14% probability of exceedance in 75 years). The inclusion of this additional lower level performance criteria aims to ensure the serviceability of the brige.

Typical RC bridge bents built prior 1970 in the Pacific Northwest of the United States are highly vulnerable to seismic events since they were not designed with a seismic criteria in mind, as a consequence, numerous bridges have inadequate reinforcing steel detailing, inadequate transverse reinforcement in columns, cap beams and joints, insufficient confinement and lap splices within expected plastic hinge zones. In order to overcome these deficiencies, retrofit measures for RC components aim to improve the component ductility, shear strength and provide confinement by "Jacketing" the component through the use of various materials such steel and fiber-reinforced polymers [5], [6], [7]. This method has now been implemented on a large number of deficient bridges throughout seismic regions and have helped preventing bridge failures during major seismic events [8]. Although, these retrofit techniques can prevent the collapse of a bridge they do not limit neither significant damage nor excesive displacements that can lead to not satisfying the operational performance criteria. A seismic retrofitting method that may help to satisfy the operational criteria and has been widely used in RC frames of buildings but not in RC bridge bents consists of adding new structural elements such as steel bracing. Current research into seismic retrofitting with steel bracing mostly involves adding supplemental damping devices in an effort to control excessive displacements and dissipate energy through the device itself. Among the damping devices that have been proposed for seismic retrofitting of structures can be found friction dampers, viscoelastic dampers and hysteretic dampers.

Many potential hysteretic devices such as buckling-restrained braces (BRBs), eccentrically braced frames (EBFs), U-shaped dampers and energy dissipating braces (EDBs) can be used for retroffiting RC bridge bents as shown in Fig. 1. The buckling-restrained braces (BRB) is one of the most promising hysteretic devices because of its versatile design, easy implementation and wide use in the structural engineering community. El-Bahey and Bruneau [9], and Bazaez and Dusicka [10] have shown that this retrofit method can be an efficient way to dissipate energy and limit displacements during seismic events. The objective of this study is to further investigate retrofitting deficient RC bridge bents with buckling-restrained braces by comparing performance limits to demands from crustal ground motions and subduction ground motions. Moreover, numerical modelling of the retrofitting technique is presented and validated with experimental results

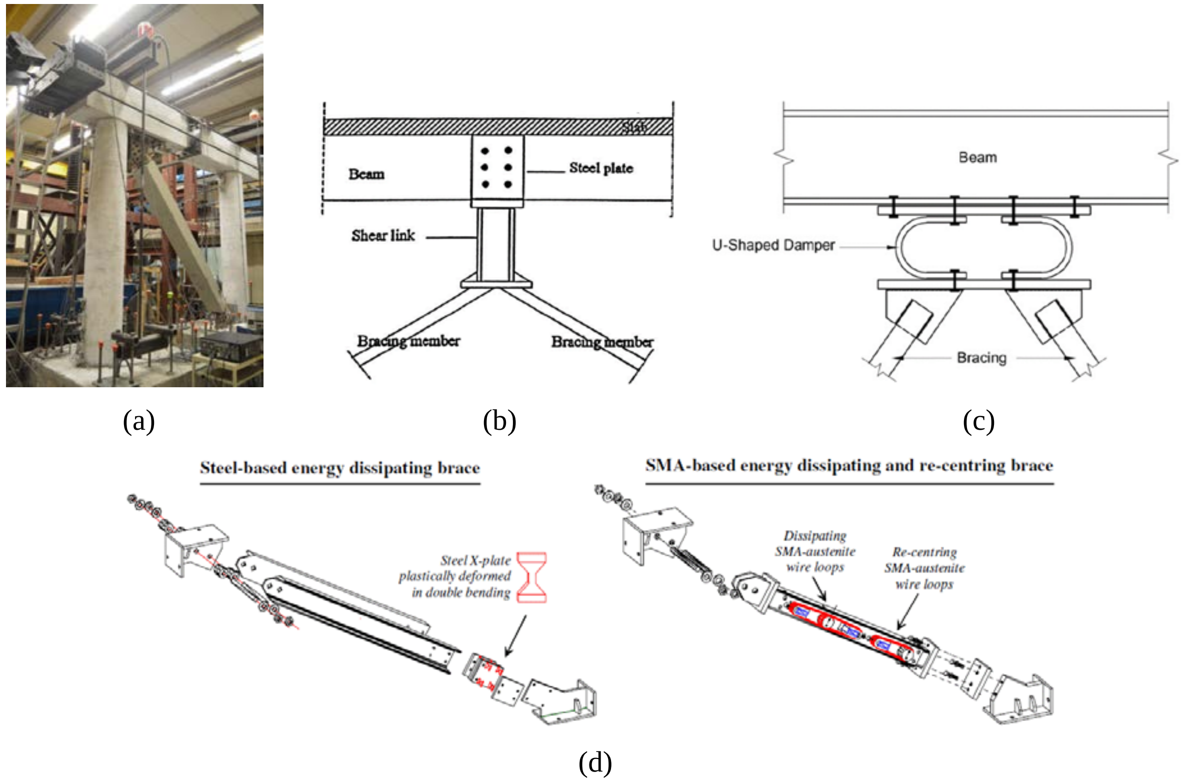


Fig. 1 – Potential hysteretic dampers for seismic retrofitting of RC bridge bents. (a) BRBs [10], (b) EBFs [11], (c) U-Shaped dampers [12], (d) EDBs [13]

2. Implementation of Hysteretic Dampers

Various research efforts on structural damage control concepts had taken place in order to develop more effective and reliable retrofit strategies for existing structures subjected to strong ground motions. One alternative that has attracted the focus of current investigations is the use of structural fuses. This structural fuse retrofit concept consists of integrating replaceable components in the main structural system in such a way to restrict the damage undergoing for the primary structure after a damaging earthquake [14]. In the case of hysteretic dampers, these are designed as replaceable elements in order to take the earthquake-induced energy and dissipate it through nonlinear hysteretic behavior [9]. The main objective of the design concept is that the retrofitted bridge bent remains elastic for the 500-year seismic event or incurs minor inelastic excursions under severe seismic events [15]. This objective is achieved by reducing the displacement demands of the as-built bent for the 500-year event (δ_{500}) and 1000-year event (δ_{1000}) to displacement demands below the yield displacement of the as-built bent (δ_y^B) for the case of the retrofitted bent under the 500-year event ($\delta^R{}_{500}$) and 1.4 times the yield displacement of the as-built bent for the case of the retrofitted bent under the 1000-year event ($\delta^R{}_{1000}$) as shown in Fig. 2. In an ideal fuse concept design the structure should remain perfectly elastic. However, the authors believe that that is an over-conservative assumption. Therefore, a maximum displacement of 140 percent the yield displacement is recommended in an effort to limit the damage of the as-built bent and still provide for a system ductile response.

Important parameters in the fuse contribution are the yield displacement of the fuse (δ_y^{Fuse}), the lateral force at that displacement (V_y^{Fuse}), the initial stiffness (K_i^{Fuse}) and any over-strength factor affecting the hardening of the fuse. The design of the hysteretic device is reduced to iterate until the stiffness of the fuse system (K_i^{Fuse}) is determined as follows:

$$K_i^{Fuse} \geq \max\left(\frac{R_d S_{a-500} mg}{\delta_y^B}, \frac{R_d S_{a-1000} mg}{1.4 \cdot \delta_y^B}\right) - K_i^{Bent} \tag{1}$$

where, R_d is a displacement magnification for short period structures as per AASHTO [1], S_a is the spectral acceleration given by the respective response spectrum (500-year or 1000-year event), m is the inertial mass of the system, g is the acceleration due to gravity, δ_y^B is the effective yield displacement of the as-built bent and K_i^{Bent} is the stiffness of the as-built bent. In the case of a diagonal buckling-restrained brace, a preliminary stiffness of the fuse, neglecting the contribution of the non-yielding sections, can be computed using Eq. (2).

$$K_i^{Fuse} = \frac{E_s A_{sc}}{L_c} \cos^2(\theta) \tag{2}$$

Where, E_s is the steel modulus of elasticity, A_{sc}, is the area of the steel core, L_c is the length of the steel core and θ is the angle between the brace and a horizontal reference. Therefore, the area of the steel core and its length can be determined replacing Eq. (2) in (1). The final implementation and design is recommended to be in coordination with a BRB manufacturer.

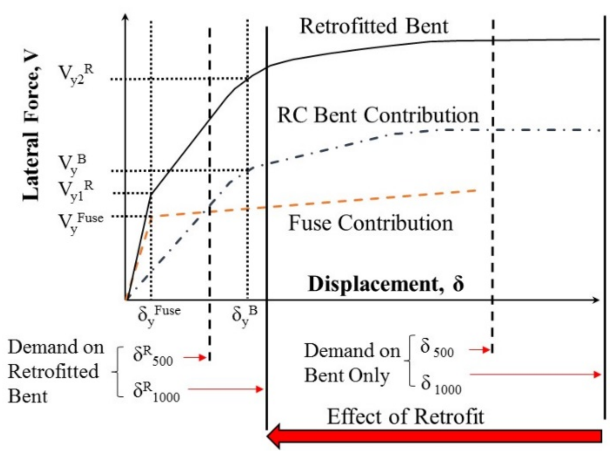
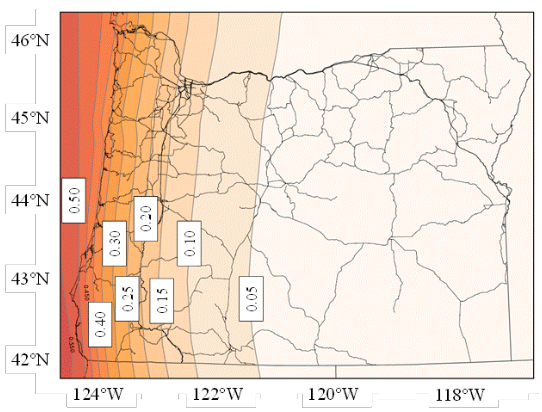


Fig. 2 – Fuse concept design strategy

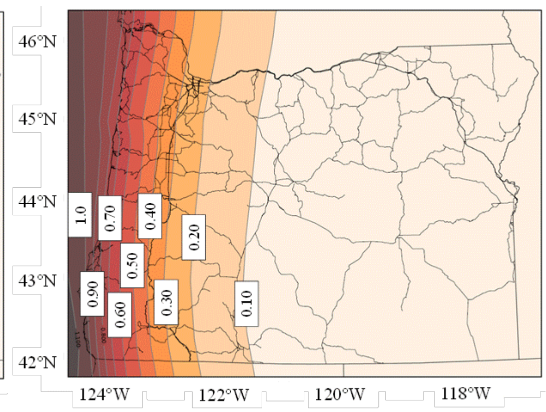
3. Seismic Hazard and Ground Motions

The seismic hazard posed by the Cascadia subduction zone (CSZ) in the Pacific Northwest of the United States had been largely underestimated due to the believe that the Cascadia Subduction zone was a quiet fault. However, with the studies carried out by Atwater et al. [16] and Goldfinger et al. [17] we now know that a megathrust subduction earthquake is likely to strike the coasts of the Pacific Northwest. In order to overcome this problem and have representative seismic demands, deterministic seismic hazard maps to represent the 500-year event (operational criteria) were developed using the full CSZ rupture model, logic tree and attenuation equations presented in the 2014 update of the United States National Seismic Hazard Maps [18]. The maps generated for the state of Oregon are shown in Fig. 3 for peak ground acceleration (PGA), 0.2 sec and 1.0 sec. The maps present acceleration values for a B/C NEHRP soil category (V_{s30} = 760 m/s).

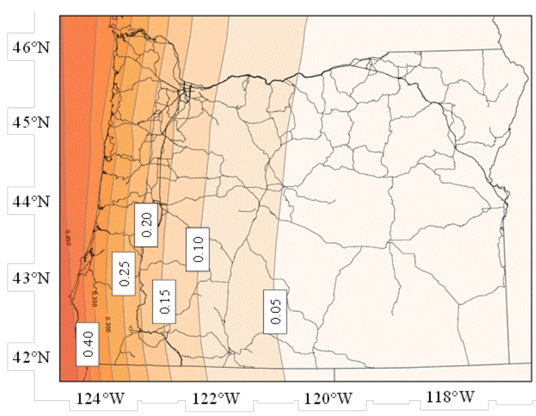
In this study six unscaled ground motions were used to evaluate the performance of a typical RC bridge bent and represent the 1000-year event. The ground motions were selected from crustal and subduction sources. Ground motions recorded at the Capitola station during the 1989 Loma Prieta earthquake were selected to represent the crustal source. Four subduction ground motions were selected from the 2010 Maule, Chile and 2011 Tohoku, Japan earthquakes as presented in Table 1. Ground motions with differnet NEHRP site classes were used with the aim of representing different soil conditions. Table 1 also shows the duration of the ground motion records. Even though, there is not a wide consensus in the duration definition of a ground motion, in this study two duration metrics were used, namely, bracketed duration and significant duration. Bracketed duration was taken as the time interval from the first to the last occurrence of an acceleration of 0.05 g. Significant duration was taken as the time interval from 5% to 95% of the total energy. From Table 1 and Fig. 4 it can be observed that the subduction ground motions have longer duration than the crustal ground motions. This longer duration implies more inelastic cycles imposed in the structure [19] and may lead to more damage and risk of collapse [20], [21].



(a)



(b)



(c)

Fig. 3 – CSZ 2014 full rupture acceleration values for (a) PGA, (b) 0.2 sec, (c) 1.0 sec

Table 1 – Ground motion records used in this study

Earthquake	Station	Mw	Site Class	Component	PGA (g)	Bracketed Duration (sec)	Significant Duration (sec)
Loma Prieta (Crustal)	Capitola	6.9	D	00 (X)	0.53	25	12
				90 (Y)	0.44	20	13
Maule, Chile (Subduction)	Curico	8.8	C	EW (X)	0.41	91	52
				NS (Y)	0.47	91	50
Tohoku, Japan (Subduction)	Kakuda	9.0	E	EW (X)	0.36	140	130
				NS (Y)	0.32	208	227

Fig. 4 shows the 5% damped response spectrum of the six ground motions used in this study. Moreover, three response spectrum representing the 500-year event (operational criteria), the 1000-year event (life safety criteria), and the maximum considered earthquake (MCE) are presented in Fig. 4. The 1000-year event was based on the 84th percentile of the response spectrum of numerous bridges in the state of Oregon in an effort of representing the seismic demand of a vast number of bridges.

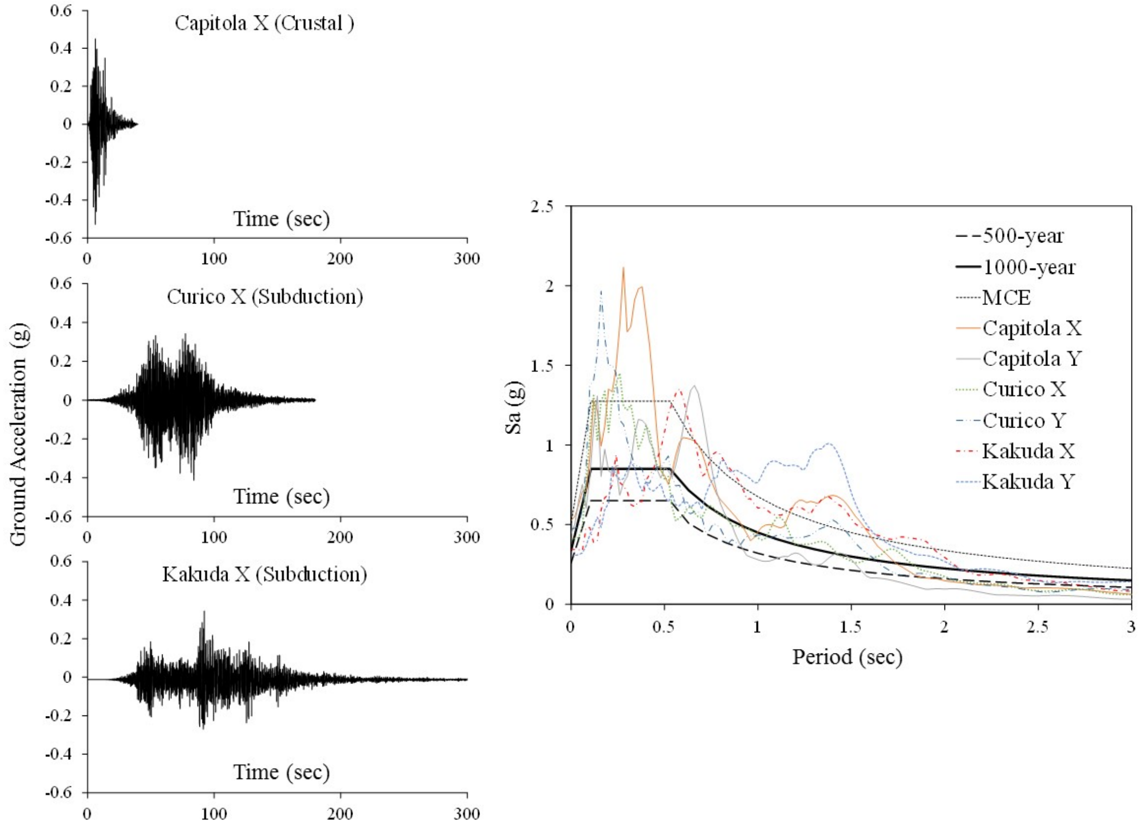


Fig. 4 –Ground motion records and response spectrum for 5% damping

4. Prototype Bridge Bent and BRB Design

The representative reinforced concrete bent corresponds to an existing RC multi-column bridge bent constructed in the 1950 to mid-1970 in the state of Oregon. As many of the bridge structures built at that time in the Pacific Northwest, the bridge substructure was designed and built with minimum seismic considerations. This resulted in inadequate transverse reinforcement, no seismic detailing, and lap-splices of length from 20 to 40 times the diameter of the longitudinal reinforcing steel (d_b) in expected plastic hinge zones.

The prototype bridge bent consists of two circular columns per bent, a rectangular cap beam and rectangular pile cap footings. The column longitudinal reinforcement ratio is $\rho_L = 1.2\%$, which is just above the minimum required by AASHTO. The provided column shear reinforcement and confinement does not meet code requirements since Ø13 mm circular hoops spaced at 305 mm were provided. The cap beam has premature termination of top reinforcement and low quantity of bottom steel reinforcement at the ends of the span, which might result in the formation of premature hinges in the cap beam at the column face. Moreover, lap splices of 39 d_b can be found at the base of columns and no seismic detailing was specified. The specified material properties for the prototype bridge were 22.8 MPa as compressive strength of concrete at 28 days and Grade 40 (276 MPa) steel. Typical details for the representative two column RC bridge bent is illustrated in Fig. 5.

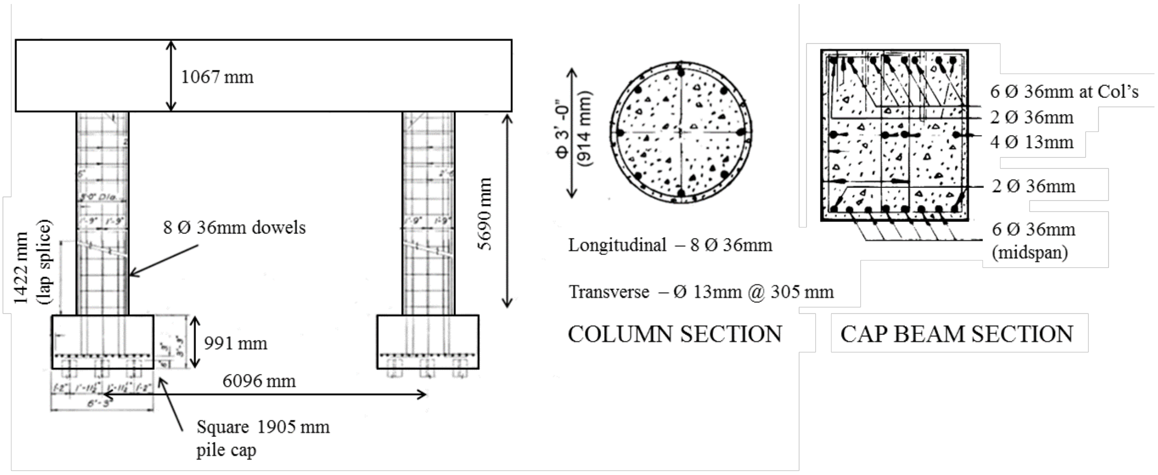


Fig. 5 –Prototype RC bridge bent

In order to retrofit the deficient representative bridge bent through the inclusion of buckling restrained braces, the following considerations were made; (1) The BRB length and angle are limited by the dimensions of the representative bridge bent, which has a span length, L = 6096 mm and column height, H = 5690 mm. A brace angle (θ) of 47.8 degrees was considered appropriate for this application. (2) An inertial mass of 318 kN-s2/m (317.5 Ton) was calculated from the superstructure dead load. (3) An expected compressive strength of concrete equal to 34 MPa, which corresponds to 1.5 times the nominal strength, and an expected yield stress of 331 MPa for the reinforcing steel were considered. For the brace, a yield stress of 290 MPa was considered appropriate for this application. (4) The yield displacement of the as-built bent was computed taking into account flexural, shear and bond slip deformations resulting in a displacement of 29 mm. Using Eq. (1) and (2) one possible BRB design would require a steel core area of 2200 mm^2 and a core length of 1524 mm.

5. Numerical Model

The prototype bridge bent presented in this study was modeled using a distributed-plasticity method. In this method, the nonlinear behavior of the structural components such as columns and beams is distributed along the length of the member through the use of fiber-based elements and integration points. OpenSees [22] was utilized for the numerical simulations due to its widely use in earthquake engineering research.

A schematic of the model is depicted in Fig. 6, where force-based beam-column elements with six integration points between nodes were used to represent the columns and the cap beam. The Concrete02 with linear tension softening uniaxial material was used in this study to model both confined and unconfined concrete. The longitudinal reinforcing steel was modeled using the Hysteretic uniaxial material since it is able to capture the pinching behavior observed in columns with lap splices and is also capable of simulating strength and stiffness degradation. Fiber-based models can accurately capture the flexural behavior of a component. However, deformations due to shear and bond slip are not considered unless additional elements or stiffness modifiers are introduced into the model. In this study, shear deformations were introduced in the model through the use of the section aggregator command, in which a constant shear modulus equal to $0.2E_c$ was used following the recommendations of Elwood and Eberhard [23]. Bond slip deformations at the base and top of column and at the ends of the cap beam were modeled through the use of a zero length section following the model proposed by Ghannaoum [24].

The cyclic behavior of buckling-restrained braces was modeled using the Steel4 and the fatigue material as proposed by Zsarnóczay [25], and was calibrated with subassemblage tests of buckling-restrained braces that were carried out in the iSTAR laboratory at the Portland State University as shown in Fig.6. The numerical simulation that uses the Steel4 plus the fatigue material agrees well with experimental results. A benefit of incorporating the fatigue material into the model is that BRB failure can be artificially captured in the model.

The BRB in the model of the prototype bridge bent had an effective length of 6636 mm and two rigid links that represent the end connections (gusset plates).

Refined modeling techniques calibrated with experimental results is essential to achieve reliable displacement demands in structures subjected to strong ground motions. For that reason, the numerical model of the retrofitted bent was calibrated with the experiments reported in Bazaez & Dusicka [10]. The results of the calibration are shown in Fig. 7, which shows a well agreement in initial stiffness, stiffness degradation, maximum strength, strength degradation, and dissipated energy between the numerical and experimental results.

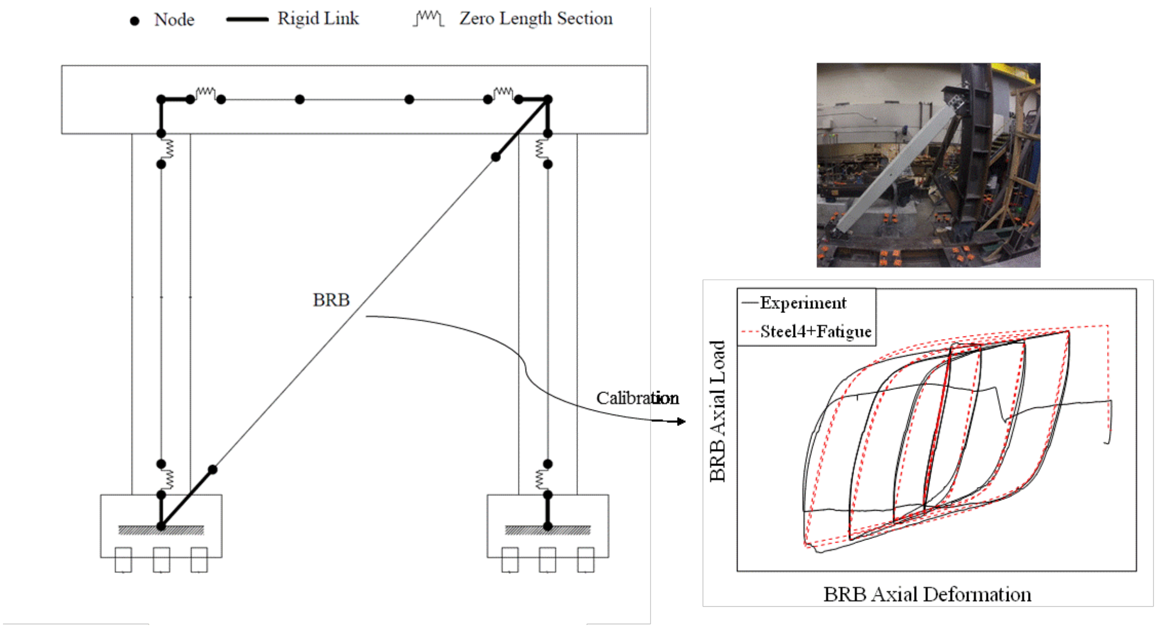


Fig. 6 –Numerical model

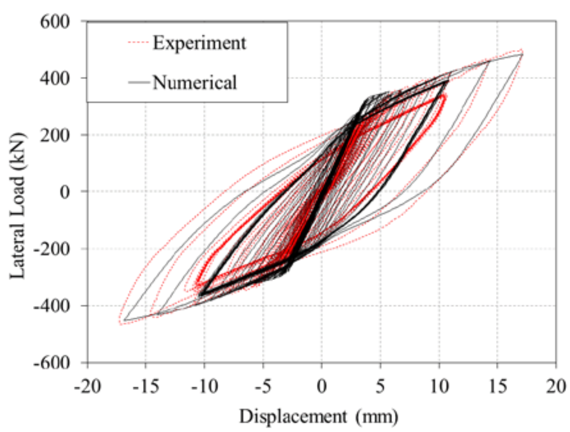
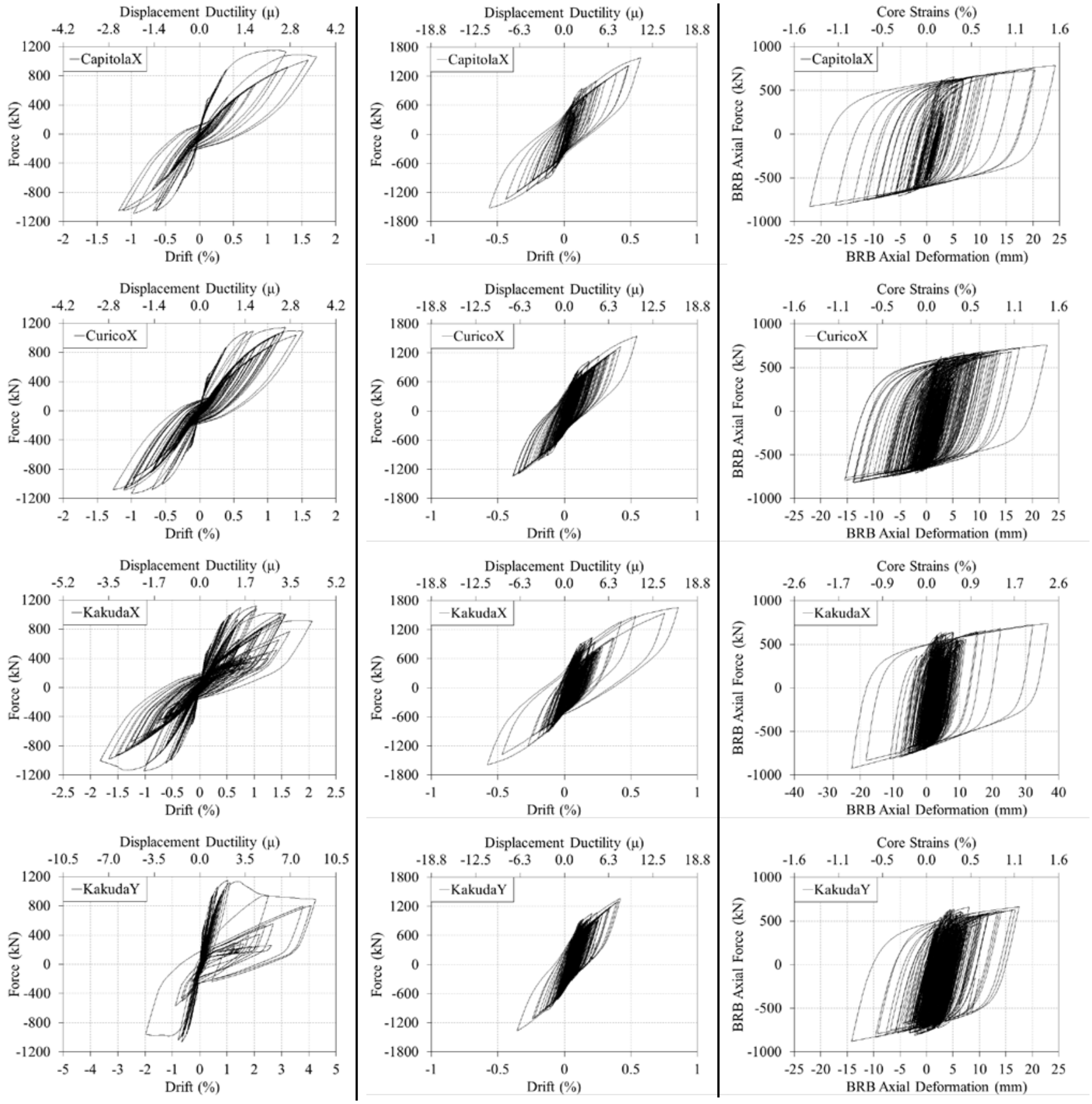


Fig. 7 – Experimental vs Numerical results

6. Nonlinear Time History Results and Discussion

Nonlinear response analysis has been widely used to assess the performance of structures for the earthquake engineering community. In this study, the set of ground motions and the numerical model presented in previous sections were used to perform such analyses. Fixed based supports were assumed at the base of the columns for this stage of the analysis. Fundamental periods around 0.5 sec and 0.26 sec were computed for the As-built and the retrofitted bent, respectively.

The damping of RC bridge structures is a complex subject and methods to effectively represent it for nonlinear analysis without introducing spurious damping has been studied and discussed by others [26]. In this study, a 5% tangent-stiffness elastic damping was assigned to the RC elements in order to represent damping in the initial stages of response. No Rayleigh damping was used for the BRB, instead energy dissipation through hysteretic damping was assumed to be well captured by the numerical model.



(a) (b) (c)

Fig. 8 –System and component responses. (a) As-built bent, (b) retrofitted bent, (c) BRB

The results of the analysis (Fig. 8) show that the As-built bent exhibits strength deterioration as well as hysteretic pinching, which is expected in inadequately detailed reinforced concrete elements. The retrofit hysteresis on the other hand exhibit hysteretic behavior with much reduced strength degradation or pinching. The subduction ground motions, namely, Curico and Kakuda, subjected both the As-built bent and the retrofitted bent to many more inelastic cycles of small amplitude as compared to the crustal ground motion. Consequently, the As-built bent subjected to the CapitolaX ground motion did not show clear signs of deterioration from one cycle to the next, which is known as cyclic deterioration, but instead it showed in-cycle deterioration. On the other hand, the Kakuda ground motions subjected the As-built bent to many inelastic cycles causing an increase in cyclic deterioration. Further, based on the numerical results the KakudaY ground motion would cause severe damage and even collapse of the bent. This result can be a consequence of the long duration and distinctive site condition (NEHRP class E) of the Kakuda ground motions. Comparing the behavior of the retrofitted bent and the as-built bent, it can be observed that for the selected ground motions, the retrofitting technique was able to effectively reduce the drift demands and provide a ductile response. Ductility demand ratios over 8 were calculated for the retrofitted bent in all the cases.

Maximum drift comparisons are summarized in Fig. 9, comparing the performance between the As-built and retrofitted bent responses. Retrofitting the As-built bent with buckling-restrained braces was an effective measure to reduce the drift demands more than 1%, consequently limiting the damage of the As-built bent components. A comparison with the two-level performance criteria required by the state of Oregon, which are based on strain limitations for concrete and steel, is also depicted in Fig. 9. This figure demonstrates that an operational performance level of response can be achieved for ground motions targeting the 1000-year event, and even for the MCE event if we take into account that for the retrofitted bent, which has a fundamental period of 0.26 sec, the demands generated from the Capitola and Curico ground motions would be close to the demands calculated from the MCE response spectrum shown in Fig. 4.

Interestingly, the retrofit technique was more effective for the case where the As-built bent had the greatest demand, i.e. for the KakudaY ground motion. This is most likely caused by the shift in fundamental period and consequently the reduction in the demands for shorter period structures in that record as shown in Fig.4, and can imply that when soil effects are expected to amplify the demands for medium and long period structures, retrofitting them with a stiff and ductile hysteretic damper can be an effective retrofit option.

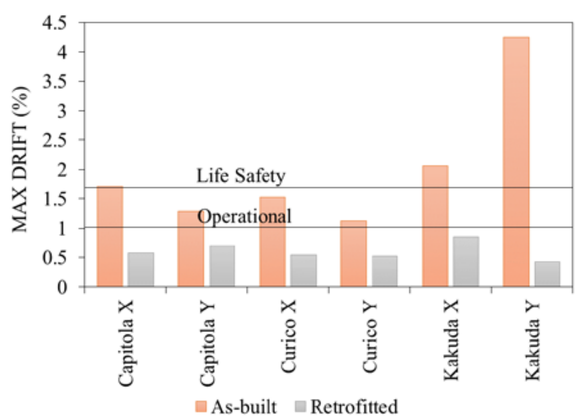


Fig. 9 –System response comparison

7. Summary and Conclusions

Hysteretic dampers for the retrofit of existing RC bridge bent were investigated and potential hysteretic devices were presented. Implementation requirements based on a structural fuse concept were briefly discussed. Seismic demands to reflect a full CSZ rupture were depicted in form of maps and were utilized to represent the seismic hazard for the operational performance criteria. A numerical model to predict the behavior of the retrofitted system was developed and validated with large-scale experimental results. The validated numerical model was used to carry out nonlinear time history analyses and used to assess the performance of a typical deficient RC bent built in the Pacific Northwest. The results from the nonlinear time history analyses demonstrated the efficiency of the retrofit technique to achieve an operational performance level even for ground motions with demands comparable to the MCE. For all the selected ground motions, drift demands were significantly reduced by adding the hysteretic device, leading to operational performance levels of the retrofitted bents. While more rigorous analyses are needed to reach broader conclusions, the results indicated that larger reductions in drift demands can be expected in RC structures retrofitted with hysteretic devices that are subjected to increased acceleration demands in the medium and long period range due to soil effects.

8. Acknowledgements

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