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Finite-Element Analysis of the Seismic Response of Controlled Rocking Steel Bridge Piers

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ABSTRACT

This paper aims at investigating the seismic response of a controlled rocking steel bridge pier which is capable of not only overcoming the problems associated with its traditional counterpart, but also offering an accelerated construction that can significantly reduce costs before construction and repairs after an earthquake. The seismic damage is limited due to reduced plastic straining in the column as an outcome of gap opening mechanism. While the majority of damage can be confined within external sacrificial elements, the rocking pier is able to exhibit re-centering capability, high ductility, and stable hysteretic response. However, the self-centering ability and lateral load/deformation capacity can be undesirably affected if local buckling occurs in the column due to its thin wall thickness. A continuum finite element (FE) is generated and its ability to capture local buckling is validated. Lateral cyclic analyses of rocking steel piers with various diameter-over-thickness ratios are carried out to investigate the re-centering property. The use of link elements in simulating buckling restrained energy dissipating steel bars can remarkably increase the efficiency of the FE model while yielding to almost the same results as the continuum approach. A simplified macro model is developed to conduct nonlinear response history analyses under different earthquake records and obtain seismic displacement demands on the rocking pier. The FE models are then analyzed under the acquired demands to gain insight on the local behavior of the column and its effects on the global behavior. It is observed that the seismic response is similar to that of cyclic analysis in terms of lateral load capacity while resulting in more residual displacements due to the presence of small displacement demands between large excursions.

Keywords: Steel bridge pier, Rocking, Finite element (FE), Local buckling, Energy dissipater.

INTRODUCTION

Piers in a bridge, as front liners in resisting seismic forces, are susceptible to damage during an earthquake. Since it is not economical nor practical to design a pier to remain completely elastic and damage-free in an earthquake, conventional seismic codes prescribe the use of inelastic capacity of steel. However, past earthquakes have shown that bridge piers designed based on the conventional seismic codes experience destructive damages which result in crippling bridge serviceability [1].

There has been a growing interest in the rocking systems and that is rooted in historical evidences. The rocking phenomenon has been repeatedly reported in earthquakes since more than a century ago. The common point among these reports is that apparently unstable structures that experienced rocking survived while other seemingly stable structures were severely damaged [2-4]. The controlled type of rocking systems, wherein the motion mode is limited to uplifting, is capable of swinging along the earthquake and reverting to its original position afterwards. The essence of the self-centering and rocking structures is to debond structural components to avoid the generation of highly stressed regions and following that, damage to the primary components. In such systems, lateral deformation capacity is provided through gap opening rather than the material dislocation as in their traditional counterparts. Hence, the nonlinearity in their behavior is due to the change in the contact status in lieu of the material nonlinearity. Under lateral loading, a gap forms at the connection interface and after load removal, the post-tensioning forces in the tendon close the gap and a self-centering behavior is achieved. Lateral displacements can be limited by using energy dissipaters, and a flag-shaped behavior can be obtained.

This paper is an extension to a previous study in which a controlled rocking steel bridge pier was proposed and numerically and analytically investigated [5]. The pier consists of a tubular steel column, a post-tensioned tendon, and supplemental energy-dissipative elements. As opposed to its traditional counterpart, the rocking pier is able to undergo large lateral displacements and revert to its upright position after being exposed to lateral loads without experiencing significant damages. The FE method is employed and then validated to simulate local buckling of the column. The models of the rocking piers are analyzed under lateral cyclic loading with a given attention to the diameter-over-thickness ratio of the column and base plate thickness as variables. Macro and continuum modeling approaches for simulating buckling restrained energy dissipating steel bars are studied. The effect of local buckling in seismic response of the rocking pier is examined. For such an undertaking, a simple macro model is developed to obtain displacement demands under different earthquake ground motions.

DESCRIPTION OF BENCHMARK BRIDGE

The details of the studied bridge are shown in Figure 1. It is a major bridge with two 33 m long spans located in Vancouver, BC. The pier consists of a hollow circular column, 15 high-strength seven-wire strands posttensioned up to 30% of their ultimate strength, and a base plate. The column has a single rocking configuration which takes place at the interface of the base and foundations plates. The height of the pier from the rocking interface to the centroid of the superstructure is 6900 mm. The amount of initial posttensioning force was chosen so that the tendon remains essentially elastic under seismic demands at the location of the bridge. The calculated weight of the superstructure is 122 kN/m which results in a dead load of 5033 kN and a seismic weight of 8052 kN. Four buckling-restrained energy dissipaters were evenly placed around the tube. The energy dissipater comprises of a steel bar, fused down to a diameter of 20 mm for 500 mm, confined within a steel tube and epoxy as the gap filler.

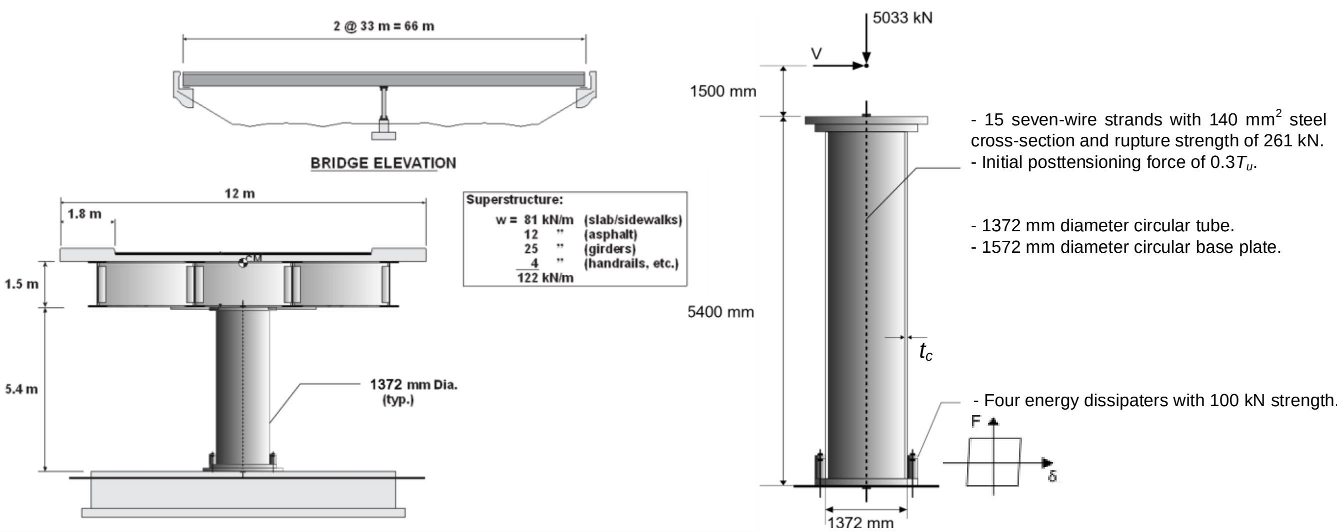

Figure 1. Details of the considered bridge.

LOCAL BUCKLING VALIDATION

The objective of the presented validation study herein is to confirm the ability of the FE model to adequately predict yielding and local buckling responses of circular steel columns with moderately large diameter-over-thickness ratios. A tested specimen (No.8) from Nishikawa et al. [6] was chosen. The specimen consists of a hollow circular section with an outer diameter of 900 mm and a wall thickness of 8.7 mm. The height of the column, from the fixed base to the top of the cap plate, is 3173 mm. The constant vertical and quasi-static lateral loads were applied using an actuator positioned at a height of 3403 mm from the base. SS400 steel with a yield point of 290 MPa and a Young's modulus of 206 GPa, was selected as the material for the components.

To simulate the cyclic behavior of the specimen, three-dimensional FE model is developed using ANSYS Mechanical APDL [7]. A nonlinear kinematic hardening model with three back stresses was employed. The SOLID185 element which has eight nodes, each having three transitional degrees of freedom, was used to mesh the piers. To model contacts, four-node surface-to-surface contact elements (CONTA173), and TARGE170 elements were used. In order to trigger local buckling, the perturbed geometry was established by adding a sum of the first ten mode shapes extracted in the preliminary buckling analysis. The amount of vertical load, as reported by Nishikawa et al. [6], was 0.124 of the axial yield load of the section. As in the actual test, the cyclic lateral loading was applied as a multiple of yield displacement, which increased step-by-step. For the boundary condition, the displacement and rotation of the nodes at the base of the pier were constrained to be zero in all directions. A detailed explanation of the FE simulation procedure, including material model, element mesh, contacts, initial imperfection, and analysis can be found in Rahmzadeh et al. [5]. Comparison between the hysteresis responses obtained from the FE analysis and test along with the final state of the specimen is shown in Figure 2.

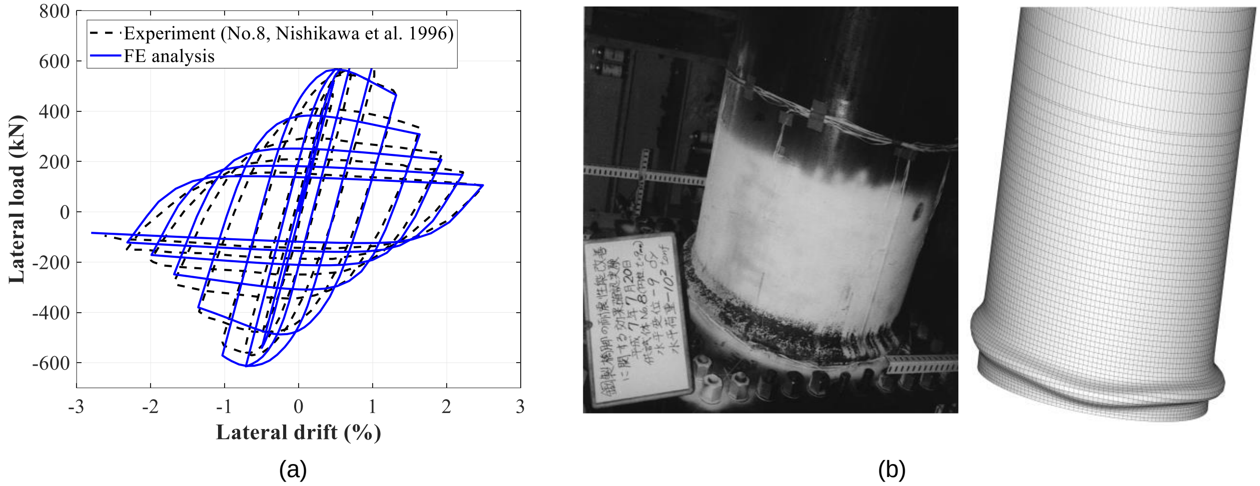

Figure 2. (a) Test-FE response comparison, (b) Final state of the specimen.

FE CYCLIC ANALYSIS

In order to simulate the rocking mechanism in the FE model, the column was placed upon the foundation plate that is anchored to the foundation. High-strength strands that run through the holes made in the cap, base and foundation plates were placed at the center of the column section and anchored at the foundation. A trilinear kinematic model with a strain-hardening ratio of 5% and a cutoff value at the ultimate strength was chosen for the strands. The yield strength was assumed to be 85% of the tensile strength. Contacts were defined between the cable nut and cap plate and the interface between the base and foundation plates. All contacts were defined as "*Standard*" except the welded components to which a "*Bonded*" contact was assigned. To apply the posttensioning force on the strands, first, the PSMESH command was used to create a pretension section and then the section stressed using the SLOAD command. The components of the FE model are illustrated in Figure 3. The energy dissipaters can be modeled either using continuum or macro elements as demonstrated in Figure 3. Since this study is focused on the column, the energy dissipaters were simulated using the latter modeling approach. Besides, the number of elements significantly decreases in the second case which in turn results in much less computationally demanding simulations.

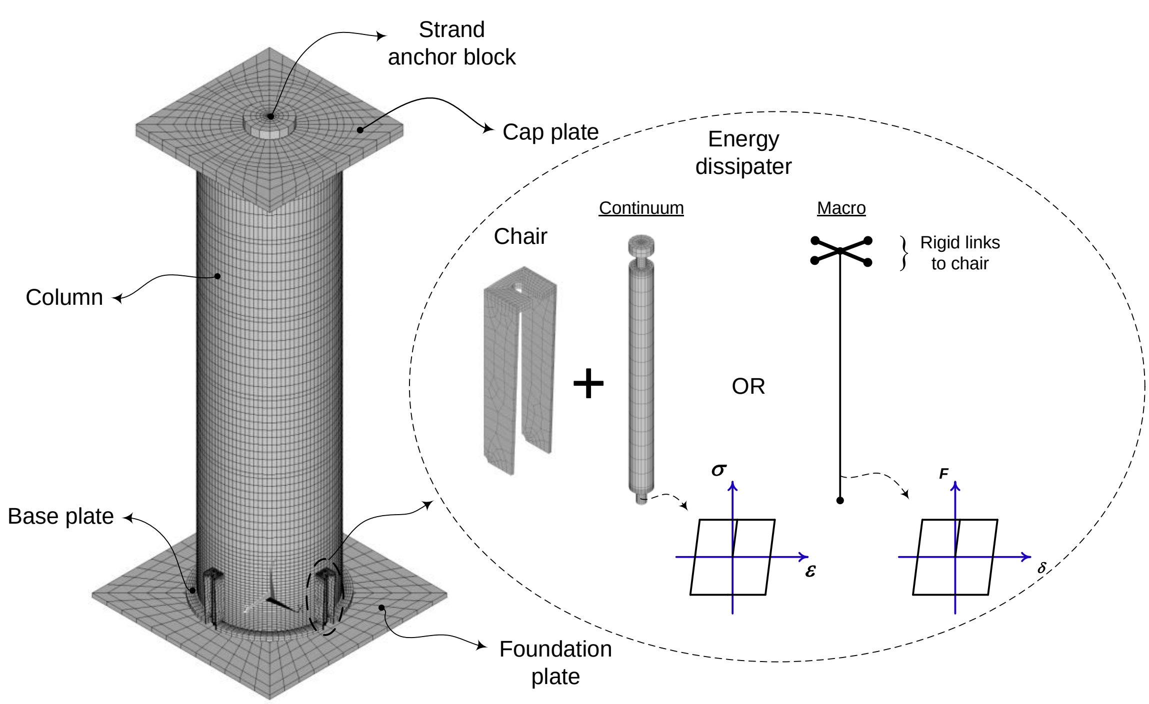

Figure 3. Developed FE model components.

The details of each specimen are listed in Table 1. The main variable is the diameter-over-thickness ratio of the tube which affects the re-centering behavior. To investigate the effect of the base plate on the post-uplifting response, three different thicknesses were also considered. The developed models were analyzed under incrementally increasing lateral cyclic drifts at a height corresponding to the centroid of the superstructure.

Table 1. Specimen matrix.

Specimen	*D/t* [1]	t_{BP} [2] (mm)
RP1-DT42-BP75	42	75
RP2-DT42-BP50	42	50
RP3-DT42-BP25	42	25
RP4-DT54-BP50	54	50
RP5-DT86-BP50	86	50
RP6-DT108-BP50	108	50

[1] Diameter-over-thickness ratio of column.
[2] Thickness of base plate.

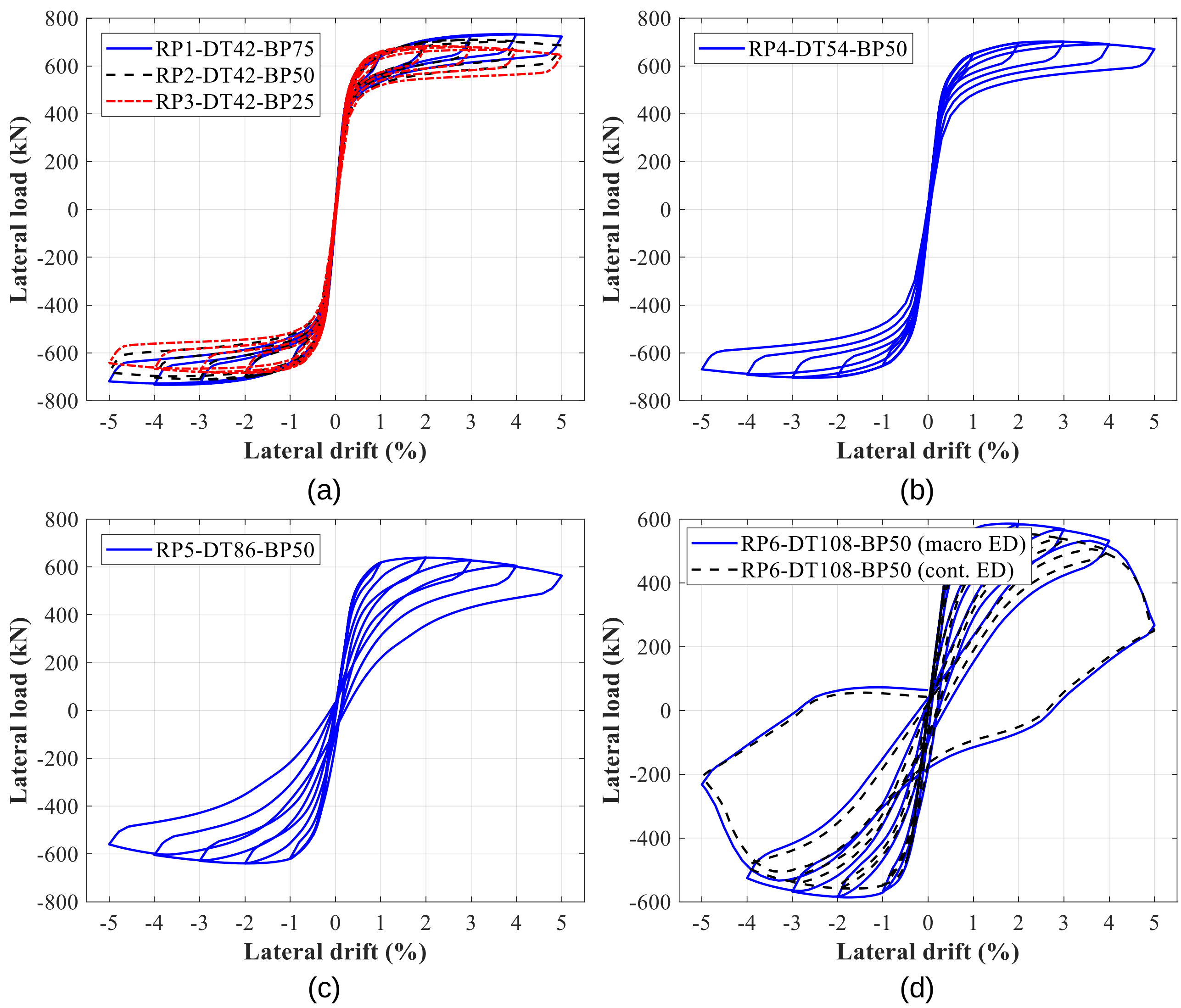

Figure 4. Lateral cyclic response of the specimens.

The analysis results are shown in Figure 4. Figure 4a depicts the influence of the base plate thickness on the lateral response; the thicker the base plate, the less softening in post-uplifting stiffness at large drifts. The re-centering property is independent of the base plate thickness. As the diameter-over-thickness ratio increases, its effects on energy dissipation and re-centering properties become more evident. The energy dissipation increases due to bulging of the tube wall near the rocking interface. Owing to the unsymmetrical buckled tube, the specimens with large diameter-over-thickness ratio could not fully revert even though the strands remained elastic (e.g., Figures 4c and 4d). The response of the models with continuum and macro energy dissipaters is compared in Figure 4d. As inferred by this figure, the modeling technique used for the dissipaters does not affect the response as long as it yields in the same uniaxial behavior. The difference between the two can be ascribed to the fact that even if the material behavior in the continuum case is elastic-almost perfectly plastic, transition between yielding and post-yielding uniaxial behavior of the energy dissipater is smooth and gradual while in the counterpart case it can be sharp. However, the difference is small and the simpler, more computationally effective model is deemed acceptable for evaluating column local buckling effects.

NONLINEAR TIME HISTORY ANALYSIS

Error! Reference source not found.a demonstrates the details of the developed two-dimensional macro model to perform nonlinear time history analysis. The column and cable are modeled with a linear beam element connected to a node at the top, and different nodes at the bottom. A rigid link was employed to simulate the base plate as well as the distance between the top of the column and the centroid of the superstructure. Springs with nonlinear elastic-perfectly plastic behavior were used to model the energy dissipaters. In order to simulate gap opening/closing, gap elements with infinite stiffness in compression and zero stiffness in tension were placed at the edges of the base plate and fixed at the base.

Three different seismic ground motion records, i.e. from crustal, in-slab subduction and interface subduction earthquakes, were selected and scaled to match the CSA S6-14 [8] design spectrum for site class C. The details of each earthquake record are given in Table 2.

Table 2. Earthquake records.

ID	Event	Year	Station	Magnitude	R (km)	Component
V307	Northridge	1994	Brentwood VA Hospital	6.7	23	195 deg.
V318	El Salvador	2001	4359c	7.6	76	270 deg.
V326	Tohoku	2011	AKTH191	9.1	155	EW

The macro model was exposed to the three abovementioned earthquake ground motions and the displacement of the superstructure centroid was recorded for each case to be used in the FE analysis (Figure 5b).

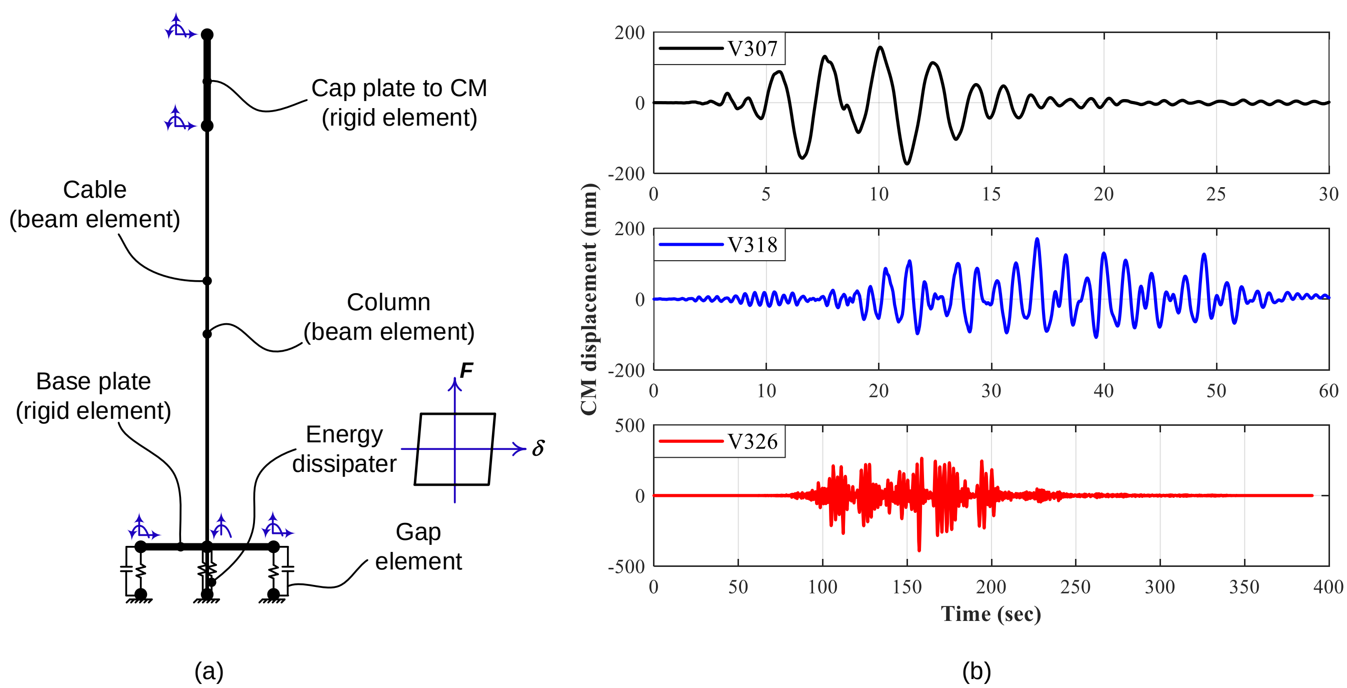

Figure 5. (a) Details of single rocking pier macro model, (b) Displacement of superstructure centroid under considered earthquake records.

FE SEISMIC ANALYSIS

Figure 6 shows the response of the specimens under the displacement demands obtained for the considered earthquake records. Specimens with diameter-over-thickness ratios of 42 and 54 behaved in the same way and fully re-centered in all records, indicating that local buckling did not occur in either of them. This conclusion can be drawn from cyclic analysis conducted before in which these two specimens had the same hysteresis responses. The lateral load capacity for these specimens based on the cyclic analyses was 700 kN. They reached the same amount of capacity under earthquake records at the same drift. The same correlation between the cyclic and seismic responses can be seen for the other specimens. However, residual drift wise, a meaningful deviation between the two analysis types exists. For instance, specimen RP5-DT86-BP50 showed a residual displacement of 8 mm at a drift of 2% under cyclic analysis, while this quantity at the same amount of drift was 8.5 mm and 8.8 mm under records V307 and V318, respectively. Under record V326, the specimen had a residual displacement of 16.5 mm after experiencing 5% drift whereas the cyclic analysis resulted in a value of 12 mm for the same amount of drift. This deviation can be attributed to the small cycles that exist between the large excursions in an earthquake record which add up to the out-of-plane deformation of the tube wall. In terms of strength degradation due to the development of local buckling, the same pattern as the cyclic analyses can be observed in seismic responses. It should be noted that the comparison between the cyclic and seismic analyses is valid as long as the energy dissipaters behave consistently in both.

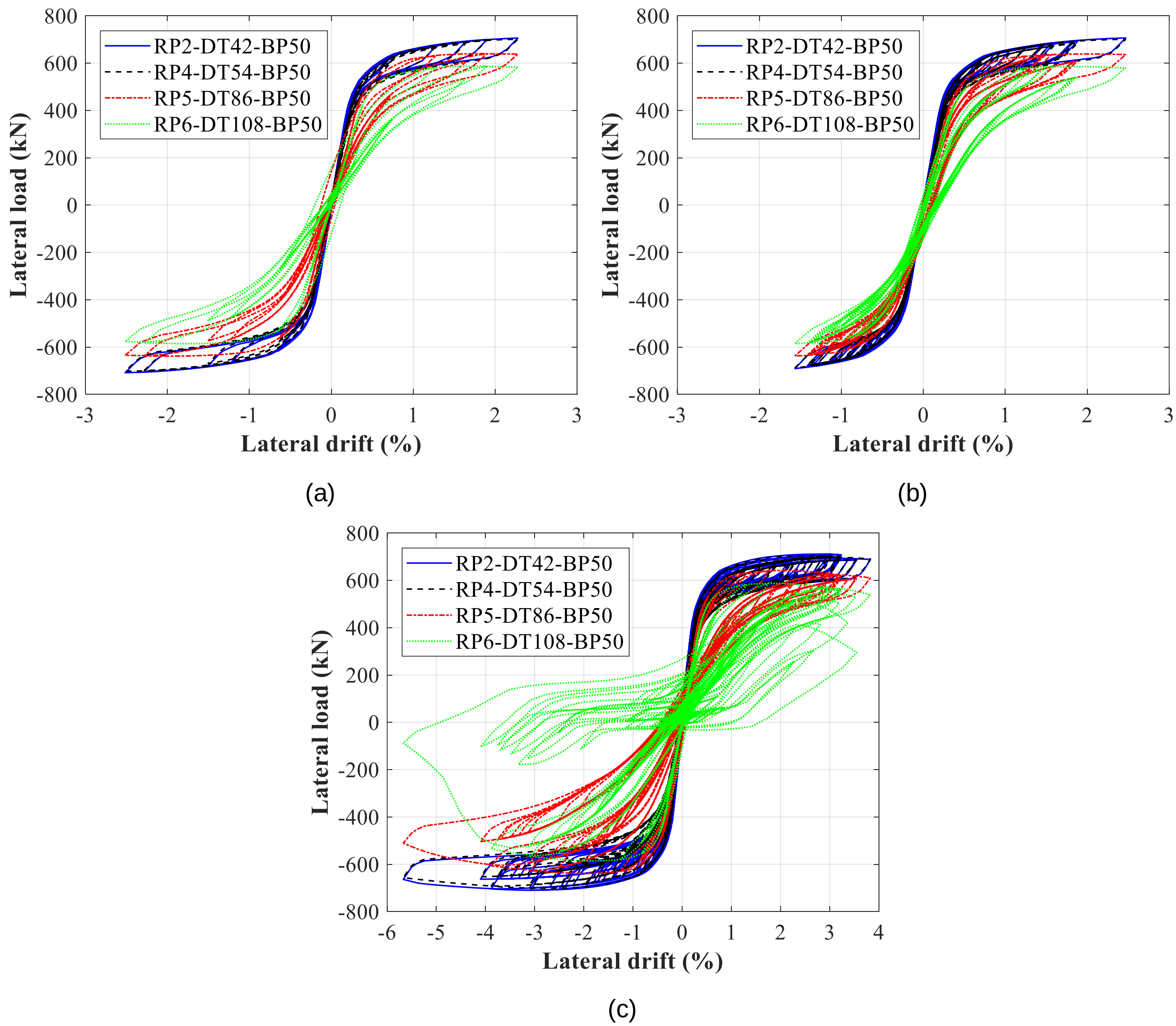

Figure 6. FE seismic analysis results for (a) V307, (b) V318, and (c) V326 records.

CONCLUSIONS

An unbounded post-tensioned steel bridge pier has been numerically studied. The pier incorporates a posttensioned high-strength tendon to limit the motion mode to uplifting along with sacrificial elements to dissipate energy. To verify the capability of the FE model in capturing local buckling, a validation study was carried out. The model well predicted the results of the previously conducted experiment. Diameter-over-thickness ratio of the tube and base plate thickness were considered as two variables in cyclic analysis. The base plate thickness does not affect the re-centering property, however, it influences the post-uplifting stiffness by changing the lever arm of the compressive force at the rocking interface. Due to unsymmetrical development of local buckling near the base, the specimens with large diameter-over-thickness ratios could not fully re-center although the tendon remained elastic. Two modeling techniques in simulating the buckling restrained steel bars were studied. Macro model yields in almost the same results as the continuum approach with much less computational time. Crustal, in-slab and interface subduction earthquake records were imposed on a simple macro model of the rocking pier, and displacement demands were recorded. An investigation of the response of the FE models under the obtained seismic displacement demands demonstrated a good correlation with that of cyclic analysis. The pattern of the lateral load capacity and its degradation was the same for both, whereas the residual displacement was higher for the considered seismic records. Numerical studies presented in this paper should be extended to investigate piers with a double rocking configuration.

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