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A novel rocking steel bridge pier system with enhanced seismic performance

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ABSTRACT: Past studies in the last few decades have shown that reinforced and precast concrete self-centering bridge piers can help achieve superior seismic response for bridge structures. Such self-centering systems have been adopted inthe construction of new bridge structures in New Zealand and the United States. A joint research project has been recently initiated bythe University of British Columbia and Polytechnique Montréal to investigate the use of an alternative rocking tubular steel bridge pier solution for the seismic protection of bridge structures. In this keynote paper, details of the proposed system are introduced and described, and the results ofthe detailed finite element analyses of the rocking response of the pier are presented. The influence of the diameter-to-thickness ratio of the tube, end plate dimensions, and supplemental energy dissipating devices on the hysteretic response of the rocking column is also discussed. The lateral cyclic behavior isdescribed by examining results from quasi-static tests performed on scaled specimens. The paper also includes the results ofnonlinear responsehistory analyses conducted on bridge structure with tubular steel rocking piers to examine their overall seismic response and possible effects of rocking induced impacts on the column axial loads and flexural demands on the bridge girders. Finally, the developed software to run seismic simulations of such rocking steel bridge piers is presented.

1 INTRODUCTION

The occurrence of the 1994 Northridge and 1995 Kobe earthquakes was a turning point in structural engineering. Prior to these earthquakes, it was imagined that structural engineering had attained its goal by saving peoples' lives. However, tremendous economic losses (billions of dollars) were experienced by even developed countries due to severe damages to code-designed structures during past and recent earthquakes (Eguchi et al. 1998, Horwich 2000, Elnashai et al. 2010, Elwood 2013). In striving for this goal, seismic codes have incorporated capacity design principles in which provisions are set out to restrict inelastic deformations to deformation-controlled components, capable of high deformation capacity, and to design force-controlled components with adequate strength to remain elastic. However, the occurrence of the plastic hinge in the deformation-controlled components under moderate earthquakes or both deformation and force-controlled components under severe earthquakes results in permanent residual drift and may necessitate complete replacement of the structure.

Bridges are vital components of a country's infrastructure and transportation system and must remain functional even after rare destructive earthquakes. Past earthquakes have shown the vulnerability of bridge components, especially bridge piers, whichare the most critical component of the bridge system (Elnashai et al. 2010, Elwood 2013, Aydan 2008, JSCE 2011). Conventional bridge piers can undergo large deformation during a major earthquake, resulting in an accumulation of significant residual deformation. Any damage to the bridge pier may, in turn, necessitatemajor rehabilitation or even demolition, leading to significant economic loss (Aydan 2008, JSCE 2011). This has motivated researchers to develop innovative structural systems with enhanced seismic resilience. In this context, self-centering (SC) structures have gained popularity among researchers due to their ability to return to their original position, even after a strong earthquake excitation, with negligible residual drift. Comprehensive research is required to investigate new structural bridge systems that can withstand seismic demands with no or minimal damage. Extensive research works have been performed in recent decades to develop high-performance structural systems such as SC structures with the capability of minimizing damage and eliminating residual deformation. SC structures were first introduced for precast concrete structures (Priestley & Tao 1993). Subsequently, Ricles et al. 2001, Tremblay et al. 2008, and Chowdhury et al. 2019 applied the same concept into the steel moment connections, steel braced frames, and steel beam-column subassemblies, respectively. In SC connections, unbonded post-tensioned (PT) strands or bars are used to re-center the system,while supplemental energy dissipation elements are used to control the lateral displacement. This system exhibits a flag-shaped hysteresis behavior with negligible residual deformations (see Figure 1).

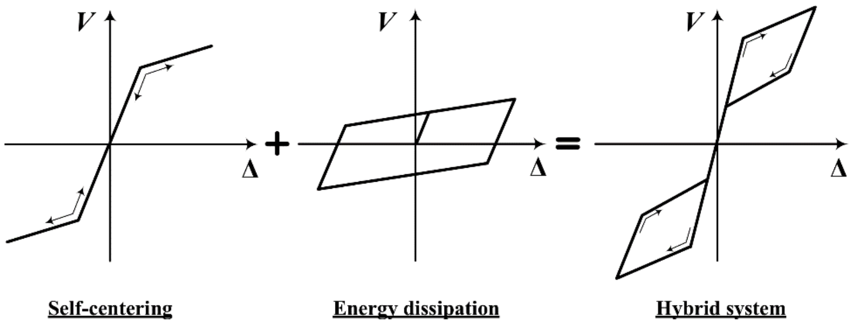


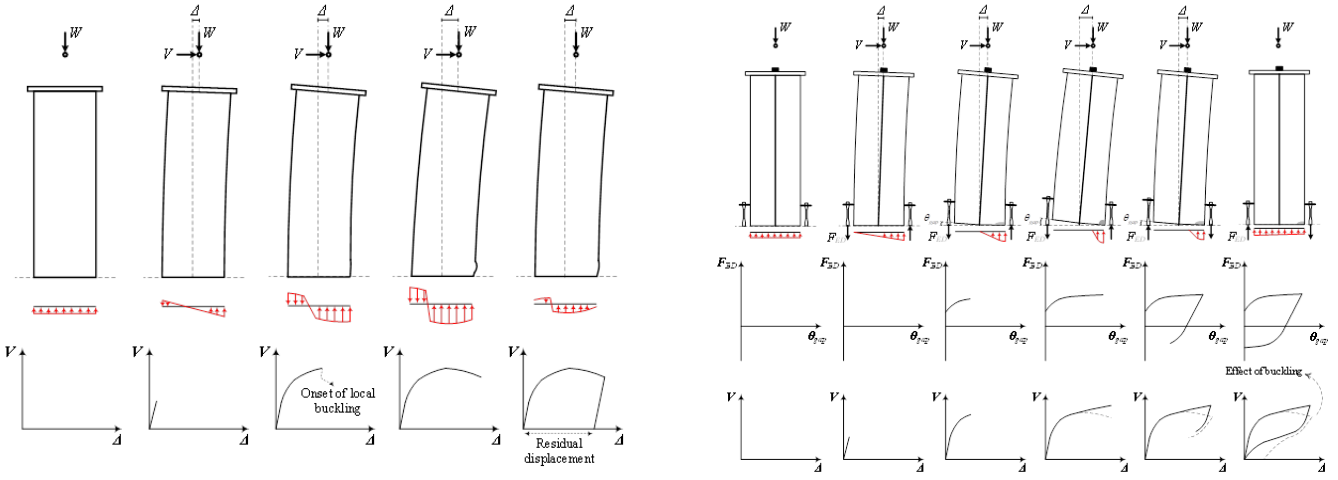
Figure 1. Flag-shape hysteresis behavior of the SC systems with energy dissipative elements.

Previous research has shown that residual deformation, bucking in the tube wall, and low deformation capacity in concrete-filled tubular sections due to the accumulation of plastic deformation over a short length are major drawbacks of conventional steel bridge piers (MacRae & Kawashima 2001). The present studythus builds upon the concept of a self-centering mechanism in steel bridge piers through experimental and numerical studies. So far, the rocking mechanism has been extended to concrete bridge piers either in the form of assembled segments(Kim et al. 2010, ElGawady & Sha'lan 2011) or as a whole (Palermo et al. 2007, Marriott et al. 2008, Rahmzadeh et al. 2018). The primary objective of the present research is to introduce a novel and cost-effective self-centering steel bridge pier with improved performance and develop a performance-based design guideline for such structural systems since this is absent in CSA S6 standard 2014.

This paper briefly summarizes the concept of rocking steel pier andvarious rocking configurations within a bridge system.The paper also describes Finite element (FE) analyses that were performed to study the cyclic behavior of such bridge piers, which was later validated with reduced-scale single-rocking steel columns equipped with energy dissipating (ED) devicesat the Applied Laboratory for Advanced Materials & Structures (ALAMS) at the University of British Columbia. This paper also highlights various FE modeling techniques that were used toinvestigate theseismic response of such rocking steel bridge piers.

2 CONVENTIONAL VS ROCKING STEEL BRIDGE PIERS

Figure 2a depicts the lateral behavior of a conventional fixed-base steel bridge pier. Prior to loading,conventional steel piers experience uniform stress distribution at the base.Upon loading, the applied moment decreases the compressive stresses at one edge. This reduction leads to the stress reversal at this edge. By further increasing the lateral load, the amplitude of stresses at both edges increases, resulting inthe yielding of extreme fibers. However, the response remains elastic. If the diameter-to-thickness ratio of the column is low enough, the plasticity spreads almost throughout the section by increasing the lateral load without developing local buckling. However, at some point, local buckling initiates due to the reduction of tangent modulus as well as the intensified local imperfections. Upon buckling, the compressive edge loses its load-carrying capacity, leading to lateral strength degradation of the pier. After the load removal, the column is not able to return to its plumb position because of excessive plastic deformations, which inevitably lead to non-recoverable damage. Such concentrated deformation/damage at the connection interface of the pier can be avoided by separating the pier from its base and introducing a rocking mechanism. Unbonded PT tendons and ED devices are used for re-centering the piers and controlling the excessive uplifting of the column. Here, residual displacements are minimized by confining the damage within the ED devices. However, the shape of the hysteresis response of the rocking system and its energy dissipation capacity vary depending on the type of ED devices being used. Figure 2b shows the lateral response of a rocking steel tubular bridge pier. The rocking steel pier will actas a fixed base conventional column before the application of the lateral load due to the gravity load and the initial post-tensioning forces. First, the pier will deform elastically under the lateral load, with a further increase in the lateral loadthe compressive stresses at the column edges will reduce. Further increasing the lateral loading will initiate the gap opening, and the pier will uplift from the base. Pier uplifting will, in turn, reduce the lateral stiffness and the ED devices will be activated. After unloading, the PT tendons and gravity load will help the pier to self-center,with the damage concentrated primarily within the ED devices, thus achieving a flag-shaped hysteresis behavior.



(a) (b)

Figure 2. Load-deformation response of a) fixed-base steel bridge pier,and b) rocking steel bridge pier.

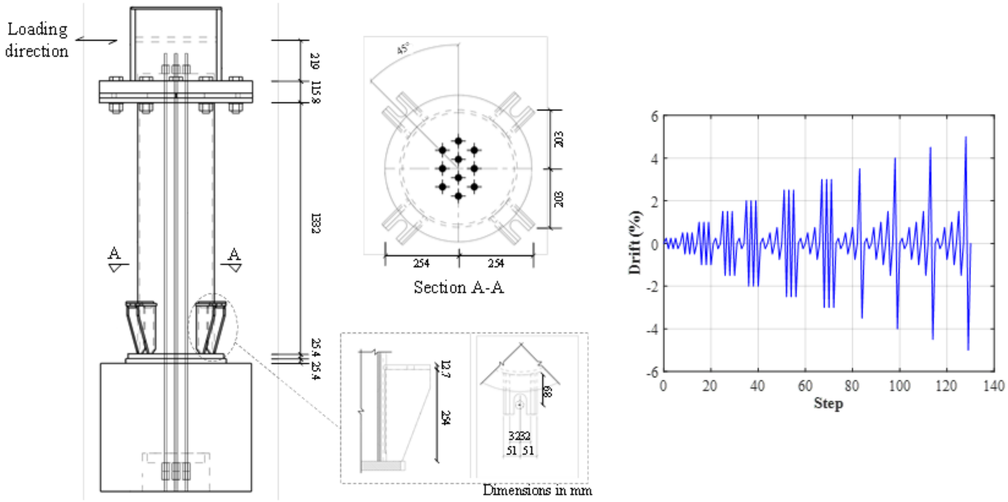
3 EXPERIMENTAL TEST PROGRAM ON ROCKING STEEL PIERS

3.1 *Test Specimens and Loading Protocol*

A prototype two-span bridge with a span length of 32.4 m is selected for designingthe test specimens. The calculated superstructure weight is 125 kN/m which results in a dead load of 5062.5 kN and seismic weight of 8100 kN for the equivalent cantilever pier. The hollow circular steel tube pier has a diameter of 1219.2 mm with a wall thickness of 28.6 mm. The pier isprestressed to 35% of its ultimate strength with high strength tendons(ASTM A416) with a steel area of 3800 mm^2. A50.8 mm thick circular base plate with a diameter of 1371.6 mm is used. The steel tube and plate grades have a yield strength of 385 MPa that comply with ASTM A252 Gr. 3and CSA-G40.21-13 Gr. 350AT, respectively. The height of the pier from the superstructure mass centroid tothe foundation top is 5100 mm. The energy is mainly dissipated through four buckling-restrained steel bars placed at an angle of 45° with respect to the longitudinal and transverse directions of the bridge.A continuum FE model of the considered pier was developed to perform pushover analysis to determineits lateral response. Then, the demand-capacity spectrum of the system was developed to check the performance requirements for different earthquake probabilities scaled to match the spectra for Vancouver site class C as perthe CSA S6 standard 2014. The design ensured that severe yielding and local buckling of the pierwill be avoided under any level of earthquake loading.The pier was then scaled to 1/3 specimen where ten 12.7 mm diameter seven-wire prestressing steel strands were used to applythe initial axial force including the dead load and prestressing force. The strands were anchored to a concrete base, which was post-tensioned to the strong floor. A diameter (d_c) of 406 mm and a wall thickness (t_c) of 9.5 mm were selected for the column where the d_c/t_c of the tube and initial axial load ratio (axial load/yield force) were kept the same as those of the original pier. The circular base plate had a diameter of 508 mm and a thickness of 25.4 mm. The height of the specimen from the rocking interface to the point of lateral load is 1692 mm. The energy dissipaters comprise a steel bar with a fuse length of 152 mm and a diameter of 10 mm, confined within a 32 mm diameter tube having a wall thickness of 3.2 mm, where the gap in between was filled with fiberglass resin.

Three specimens were tested where Specimen-1 was a column without a base plate and energy dissipaters, the purpose of which was a proof-of-concept. A base plate was added to the system in Specimen-2 to show its advantages (improving strength and post-elastic stiffness). The last Specimen-3 was incorporated with energy dissipaters to demonstrate an improvement inthe energy dissipation capacity of the system by the use of such sacrificial elements (see Figure 3a). All the tests were performed at the Applied Laboratory for Advanced Materials & Structures (ALAMS) at UBC.

In order to develop a loading protocol, a simplified macro model of the prototypepier was developed using commercially available FE program SAP2000.The column and PT tendons were modeled with linear beam elements connected to a top and bottom node. Gap type elements with zero stiffness in tension and infinite stiffness in compression were placed at the edges of the base plate and fixed at the base to capturethe gap opening/closing behavior at the rocking interface. This simplified model was able to predict the peak displacements of the rocking column with reasonable accuracy compared to the continuum FE model (Rahmzadeh et al. 2019). The detailed description of continuum FE modeling techniques is presented in the next section. Nonlinear responsehistory analyses were performed using an ensemble of 33 horizontal ground motions selected and scaled according to the 2015 National Building Code of Canada (NRCC 2017). The peak drifts from each record were collected and seeded into bins illustrating a range of drifts, which was used to establish the displacement-based loading protocol shown in Figure3b. The lateral displacement was applied at a rate of 10 mm/min corresponding to a low strain rate which isless than 0.00005 s^{-1}.



(a) (b) (c)
Figure 3. a) Rocking steel bridge pier Specimen-3 and b) displacement-based loading protocol.

3.2 *Test Results*

The hysteresis responses of the tested specimens under lateral cyclic loading are presented in Figure4. All the specimens exhibiteda self-centering response with negligible damage to the column as the main component due to the gap opening/closing mechanism limited the straining at the rocking interface. Figure 5 shows that the use of a base plate (Specimen-2) increases the post-uplifting stiffness and lateral strength of the rocking pier. This is due to the larger contact area at the rocking interface when the column uplifts. Also, the base plate can prevent the distortion of the column cross-section by distributing the shearing stresses over the entire section rather than a small portion of the section as in the case of Specimen-1 (Figure 4). The energy dissipation of the system, as well as its lateral strength, can be improved by using external sacrificial elements (Specimen-3). The amount of increase depends on the number and location of these elements with respect to the neutral axis at the connection interface.

4 FINITE ELEMENT SIMULATION

4.1 *Continuum Modeling*

A 3D continuum model of the rocking steel bridge pier was generated in ANSYS Mechanical APDL and verified with the experimental results of the lateral cyclic load test of Specimen-1. The components of the FE model are illustrated in Figure 5. Shell and solid elements were used to model the column and base plate, respectively. The column shell was extended inside the base plate and clamped to the mutual nodes of the base plate’s solid elements. A frame element was utilized to simulate the tendon, fixed at the bottom and tied to the column at the top using multipoint constraint elements. The tendon was extended above the column to the location of the lateral load application. To model the gap opening/closing mechanism, contact elements were used between the base and foundation plates. The energy dissipaters were simulated using frame elements with a bilinear model. This leads to less computational time, yet comparable accuracy compared to that of a detailed FE model (Rahmzadeh et al. 2019). Figure 5b illustrates the accuracy of the numerical model in predicting the stiffness and strength of the system. For the time history analysis, the rotational components of the mass centroid of the superstructure were neglected; hence, the mass has two translational degrees of freedom in the plane of ground shaking. Also, Rayleigh damping corresponding to 3% of critical in the first and third modes of the bridge was assigned. In order to prevent the erroneous damping forces, the stiffness proportional damping was only assigned to the material of the column.

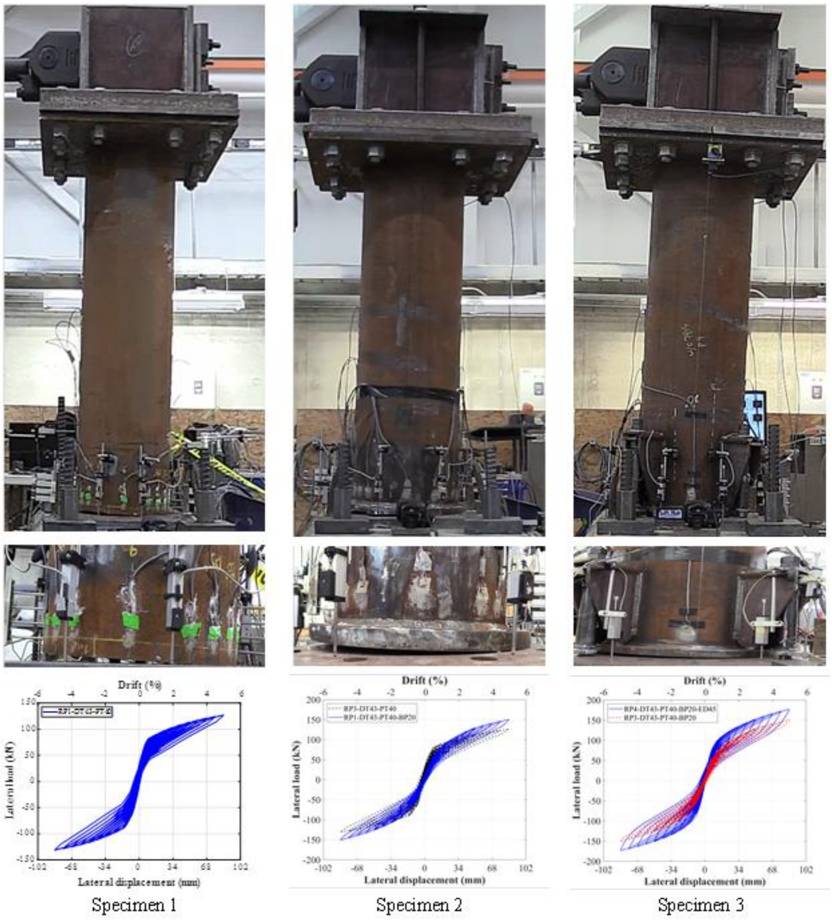


Figure 4. Test specimens and their hysteresis responses.

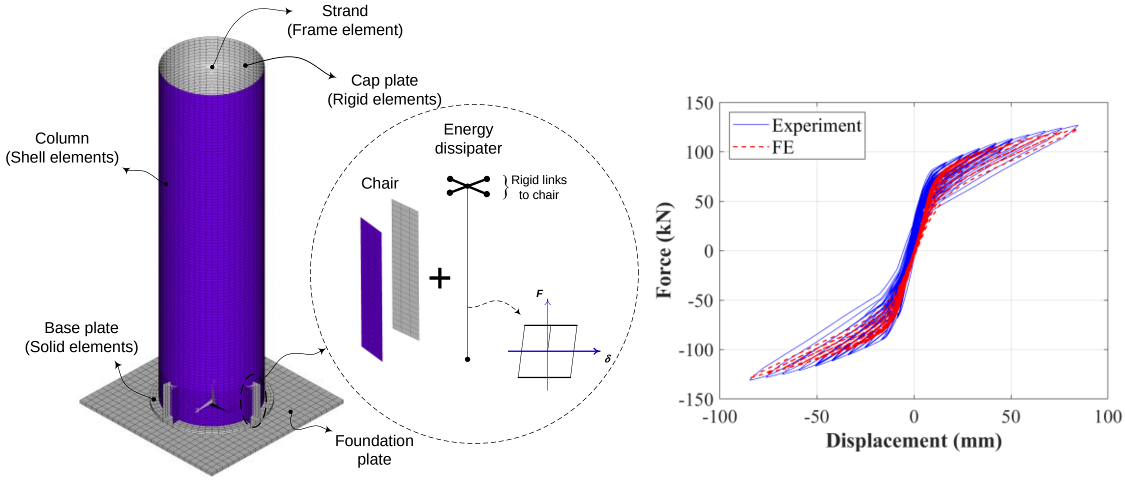
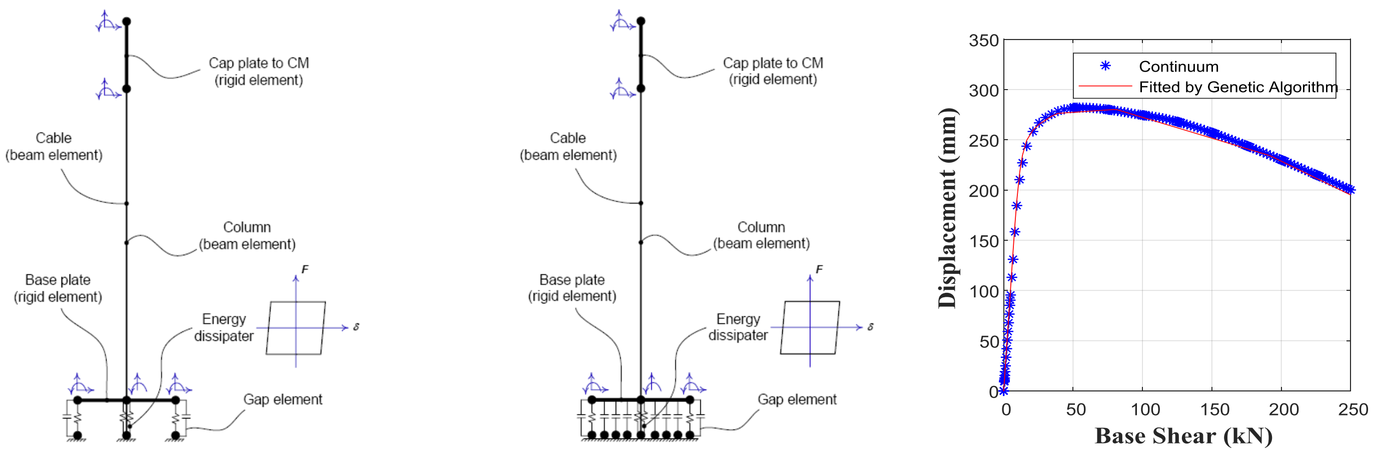


Figure 5. Developed continuum FE model and its verification.

4.2 *Macro Modeling*

Seismic response of rocking steel bridge piers can be modeled using different techniques. Since detailed FE models are highly complex and computationally expensive, they are not used in the practicing industry. Simplified FE models can be generated for rocking steel bridge columns based on frame and link elements that can be used by practicing engineers. Here, Opensees software (McKenna 2011) is used to develop such a simplified FE model where different elements have been used to model the components of the rocking system. The steel tubular column is modeled with an elastic beam element and the PTtendon is modeled with a linear truss element accounting for the entire diameter of the tendons. The co-rotational transformation was chosen to account for base rotation as the column uplifts. The initial post-tensioning force was applied as an initial strain in the material. Prestressing loss due to elastic shortening of the column is also considered. The base plate is modeled as a rigid element to ignore bending and axial deformation. In order to mimicthe gap opening/closing behavior, two types of spring elements are considered: a) two-springs method, and b) multi-springs method.In the two-springs method, the springsare modeled as rigid compression-only elements and placed at both ends of the rigid base plate (see Figure 6a). The stiffness of the spring is assignedby a large arbitrary value. In the other method, the springs are modeled as bilinear-elastic elementswhere the springs are evenly distributed below the rigid base plate (see Figure 6b). Their initial and the post-elastic stiffness values depend on the number of springs and thegeometric configuration of the steel pier.The purpose of using the compression-only element in both methods was to allow the gap opening when the base plate is subjected to uplift. The ED devices were modeled as nonlinear zero-length link elements. In the multi-springs model, three parameters are required to define the base spring including the initial stiffness, yield strength, and post-yield stiffness, which are the functions of the geometric configuration of the pier. Here, genetic algorigthm (GA) based optimization technique has been implemented to calculate the value of these parameters. A wide range of practical values of these parameters is considered to come up with different configurations of the bridge pier. Then, for each case, a static pushover analysis was performed using continuum FE analysis. For each set of geometric configuration, a set of spring parameters have been computed. About 5000 continuum models have been run to come up with spring parameters for each case. Then, a nonlinear regression analysis was performed to determine the optimum set of parameters. By predicting the multi-spring parameters of any given bridge pier configuration, its static pushover,and nonlinear response historyanalyses can be performed using a macro model. A sample calibrated pushover curve is shown in Figure 6c.



(a) (b) (c)

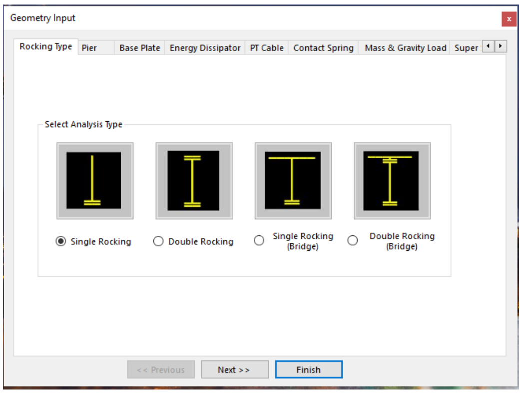
Figure 6. Macro models: a) two-springs model b) multi-springs model and c) calibrated pushover curve.

4.3 *Software Interface for Analyzing Rocking Bridge*

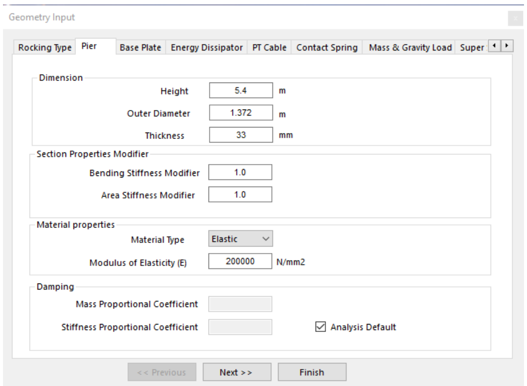
A software tool has been developed with an interface as shown in Figure 7 to analyze bridges with rocking piers. The main purpose of the software is to take all necessary user inputs from the engineer and prepare an input file to run the FE analysis using OpenSeessoftware. Besides, the input can be imported to any other popular structural analysis FE software packages, e.g., S-FRAME, SAP2000 etc. This tool can be effectively utilized to perform a series of nonlinear response history analyses, and parametric study at component and system level while performing a parametric study, the user can define the range (upper andlower limit) of different parameters (i.e. pier diameter, pier thickness, PT force) and the program will automatically generate all the necessary input files and run them in sequence. The different sample input forms to define a rocking pier in that software is shown in Figure 7. After running the analysis (time history or parametric), the program can produce necessary results either ingraphical or tabular format.

5 SEISMIC RESPONSE OF A ROCKING BRIDGE PIER

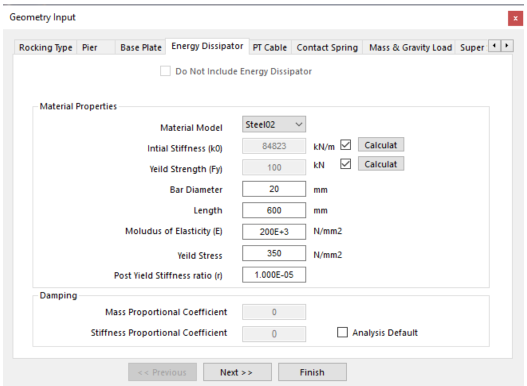
This section presents a case study on a bridge having a base rocking steel pier. The bridge is located in Vancouver, BC, Canada with soil Class C as defined in the CSA S6 2014 where the details are shown in Figure8a. The length of each span is 33 m. A 33 mm thick hollow circular steel column is used as the bridge pier post-tensioned up to 30% of the ultimate strength of the strands. The height of the pier is 6,900 mm from the base to the centroid of the superstructure. The total weight of the superstructure is 5,033 kN and the seismic weight is 8,052 kN. Four buckling-restrained energy dissipaters were evenly placed around the tube at the rocking interface. Each energy dissipater comprises of a steel bar, fused down to a diameter of 20 mm, confined within a steel tube and epoxy as the gap filler.Several representative horizontal ground motions were selected and scaled to match the design spectrum for Vancouver site class C following the National Building Code of Canada.(Figure8b).The ground accelerations were applied directly at the base of the pier while in the continuum model the acceleration records were integrated twice and the corresponding displacements were imposed to the base. In both modeling approaches, Rayleigh damping corresponding to 3% of critical in the first and third modes was assigned. The stiffness proportional damping was only assigned to the column element to avoid the development of excessive damping forces in the nonlinear link elements and rigid elements. The model did not include any gravity loads; a vertical acceleration of 1.0 g was imposed in all the analyses to generate gravity load effects from the structure masses. The second-order analysis was also performed to include the P-delta effects.



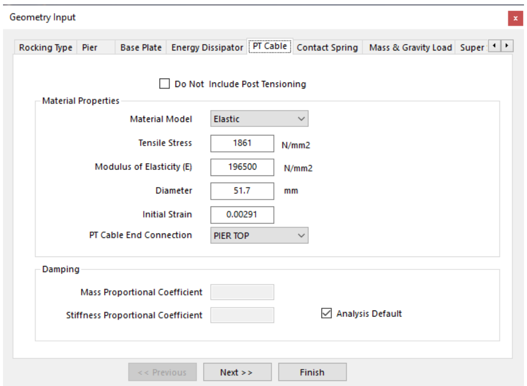
(Analysis options)



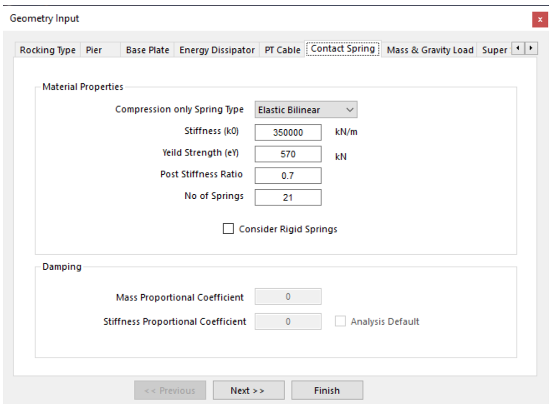
(Pier geometric inputs)



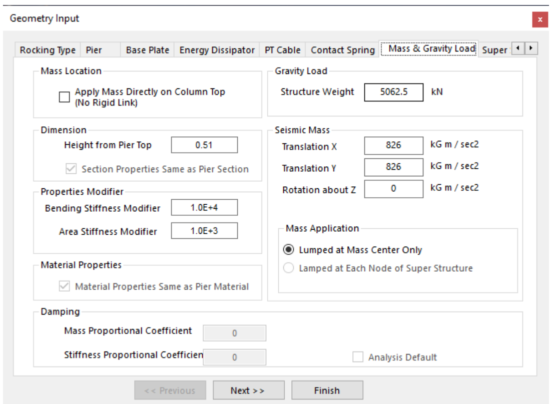
(Energy dissipator inputs)



(Post-Tension cable inputs)

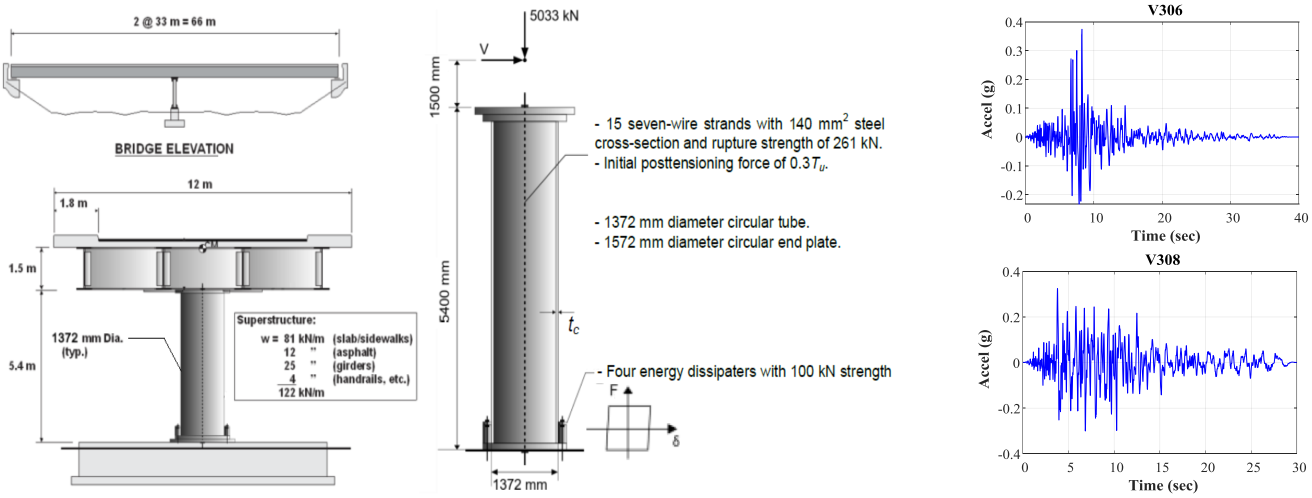


(Contact spring inputs)



(Mass definition inputs)

Figure 7. Various input forms for the software interface.



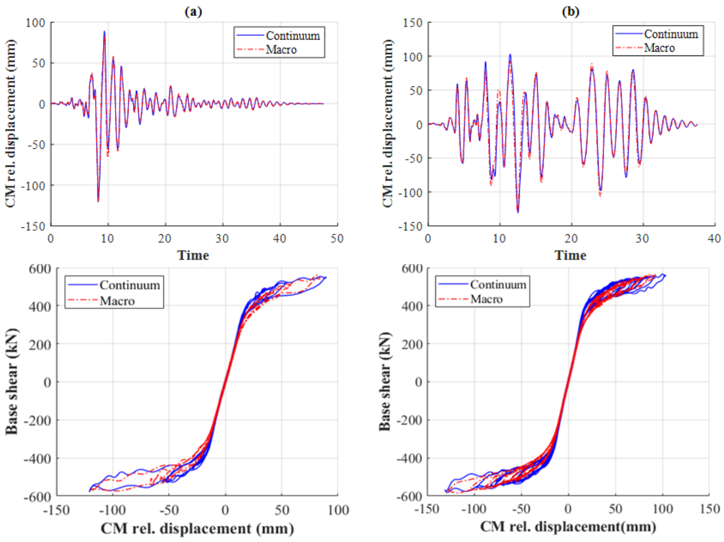
(a) (b)

Figure 8. a) Bridge with rocking steel piers and b) two sample ground motion records (V306, V308).

5.1 *Comparative Results*

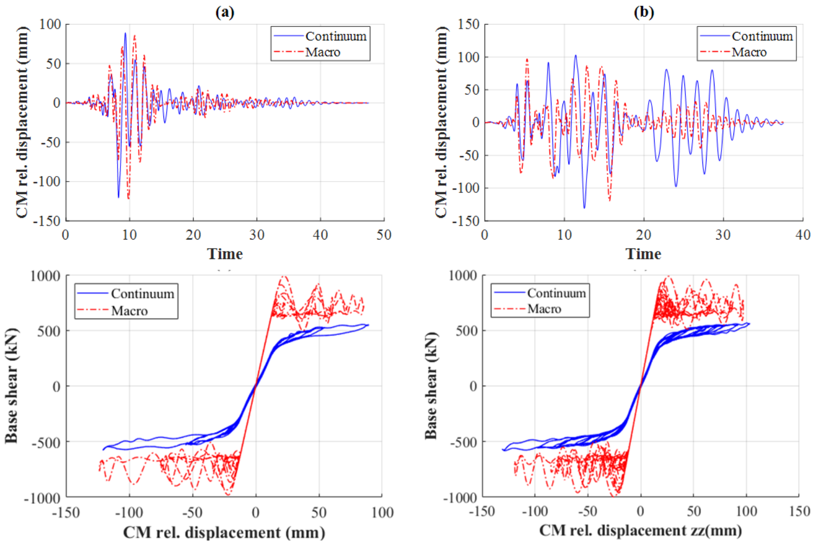
Figure 9 shows the displacement time history of the superstructure's centroid under the considered (a) record V306, and (b) record V308. As expected, the displacement demand in the continuum model is higher than that of the two-spring macro model in most cases due to the difference in the stiffness properties. The effects of vertical impact are more pronounced in the macro-model due to the larger lever arm of forces at the connection interface when the column returns to its original position after rocking. In the continuum FE model, the column exhibits a smooth transition between the pre and post uplifting conditions due to the flexibility of the column and bending of the base plate. The lateral resistance of the rocking model is overestimated by the macro-model as the model assumes a fixed position for the rocking point. The continuum FE model shows that the pivot point progressively migrates towards the center of the column due to elasticity of the column and localized yielding of the base plate in flexure, which reduces the level arm of the compressive force resultant. Although the two-springs macro model could not predict the load-displacement relation and stiffness close to the continuum model, all the results show self-centering behavior.

Figure 10 shows the comparative displacement time history of the superstructure's centroid and the load-displacement hysteresis of the rocking pier under the considered records. For each ground motion record, the simplified macro model using multi-springs was able to predict the response similarto that of the 3D continuum model.The results show that the multi-spring model can not only predict the initial stiffness accurately but also the lateral load resistance and the peak displacement of the rocking steel bridge piers.



(a) (b)

Figure 9. Continuum and two-springs macro models' displacement of superstructure centroid relative to the base and load-displacement hysteresis prediction under (a) record V306, (b) record V308.



(a) (b)

Figure 10. Continuum and multi-springs macro models' displacement of superstructure centroid relative to the base and load-displacement hysteresis prediction under (a) record V306, (b) record V308.

6 CONCLUSIONS

This paper introduced controlled rocking bridge piers built with steel circular tubularsections. Experimental investigations were performed on a 1/3-scale base rocking steel bridge piers to prove the concept of base rocking and its self-centering capability. A base plate welded to the column tube base could enhance the overall behavior of the system by increasing the lateral strength and post-uplifting stiffness. The results show that the addition of energy dissipaters can significantly improve the lateral capacity and energy dissipation capacity of the rocking bridge pier. The continuum FE model could accurately capture the lateral load response of a rocking bridge pier. In order to improve computational efficiency, the macro FE modeling technique with multiple springs was also introduced where the spring constants were calibrated using GA and regression equations from continuum FE models. A series of nonlinear response history analyses were performed to compare the displacement time history and load-displacement hysteresis results from the continuum, two-springs, and multi-springs model. The results show that the multi-springs model is more accurate in predicting the responses compared to the of two-springs model. A user-friendly software interface has been developed to take all necessary user inputs to model a bridge with rocking piers. This tool prepares an input file to run the FE analysis using OpenSEES software. This model can be also imported to any popular structural analysis software. Future research will look into various arrangements of the rocking pier including base and dual rocking system, various configurations of the cross beam, different types of energy dissipating elements, and development of performance-based design guidelines. Further experimental test programon rocking bridge pier and the bridge system will be also explored including bi-directional cyclic testing of rocking steel piers, real-time dynamic testing of scaledmulti-span rocking bridge, and multi-directional hybrid testing of rocking bridge piers.

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