Canadian Conference - Pacific Conference on Earthquake Engineering 2023
Vancouver, British Columbia
June 25th – June 30th, 2023

Nonlinear Finite Element Modeling and Response Simulation of Controlled Rocking Tubular Steel Piers Prior to Shake Table Testing

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ABSTRACT

Research is currently being performed at the Polytechnique Montréal and the University of British Columbia to investigate the seismic response and design of rocking tubular steel piers to achieve superior bridge seismic performance. The system can be designed and detailed to obtain a rocking response at the column base, column top, or both to achieve a self-centering response. Circular steel tubes are typically used to allow for a bi-directional rocking response. Lateral displacements can be controlled by adding post-tensioned (PT) high-strength steel tendons or bars to enhance the system's recentering capacity and by introducing yielding or friction energy dissipation (ED) devices at the rocking interfaces. Steel rocking bridge piers are expected to sustain very limited structural damage and negligible residual deformation under design-level ground motions. A shake table test program will be conducted as part of the research project to validate the numerical models used to predict the dynamic seismic response of controlled rocking tubular steel piers. Testing will be conducted on the earthquake simulator of the Hydro-Québec Structural Engineering Laboratory at Polytechnique Montréal. The test specimens are 1:0.35 scaled models of a circular steel tube pier of a prototype bridge structure. The columns have welded base and cap plates. In the test setup, large steel plates are placed on the column specimens to impose vertical inertia forces from uplifting during rocking and horizontal inertia forces from lateral displacements of the bridge superstructure. Tests will be conducted on columns designed for rocking at their bases only and columns designed for rocking at their top and bottom ends. The effect of adding PT elements and energy dissipation devices will be investigated. The article presents detailed pretest three-dimensional finite-element (FE) analyses of the test columns performed with ABAQUS. For comparison and validation purposes, seismic analysis is also carried out with a simplified FE model, which is more computationally efficient in simulating the rocking response at the column ends. The purpose of these pretest analyses is to finalize the test program, including the verification of the geometry and physical characteristics of the test specimens and test setup and the selection of ground motion time histories. Nonlinear response history analyses (NLRHA) have been conducted with both numerical models using shallow crustal ground motions contributing to the seismic hazard in southeast British Columbia. Although the simplified FE model is computationally efficient, it over-predicts the lateral drift compared to the refined FE model. The results of the NLRHA are presented in terms of lateral displacement, horizontal and vertical accelerations of the pier top, PT bars force level, and uplifting profile. The simplified FE model, however, represents the general trend in lateral displacement and vertical acceleration of the pier top. This study gives valuable insight into the dynamic seismic response of the rocking steel column system before conducting the shake table testing.

Keywords: Steel bridge pier, Rocking, Finite element, Nonlinear dynamic analysis, and Model uncertainty.

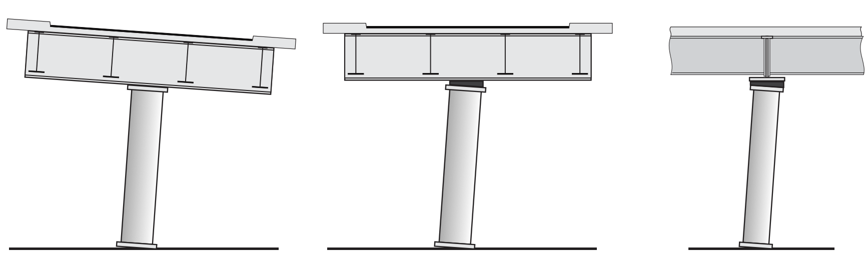
INTRODUCTION

In recent years, there has been an increasing emphasis on building seismically resilient infrastructure systems to withstand various natural disasters. A large portion of bridge infrastructure in Canada are structurally deficient which need immediate replacement or retrofitting (Canadian Infrastructure Report Card, 2019). According to the Insurance Bureau of Canada, the estimated loss resulting from an M9.0 earthquake in British Columbia would be \$75B and that resulting from an M7.1 earthquake in the Québec–Montréal–Ottawa corridor would be \$61B [1]. In this context, it is important to develop a resilient bridge infrastructure system that can withstand a large magnitude of ground motion. Self-centering rocking systems have gained popularity in recent decades due to their enhanced seismic performance and ability to reduce seismic demand on the structure. Rocking mechanisms mitigate the costly and time-consuming retrofitting measures associated with rehabilitating conventional bridge piers after an earthquake. Most previous studies on rocking isolation for bridges, though, have focused on concrete rocking piers [2-7]. A controlled rocking steel pier system has been recently proposed as a promising alternative for the seismic protection of bridge systems that allows Accelerated Bridge Construction (ABC) for bridge substructures in seismic areas. ABC has gained popularity among the bridge engineering community due to improved constructability, high-quality material, reduced onsite construction time, improved work-zone safety, and less impact on traffic. Several ABC techniques have been developed in the past for bridge substructures [5-7]. In the proposed system, the bridge pier comprises a tubular steel tube with welded circular end plates at the top and bottom ends of the columns, post-tensioned tendons to maintain positive lateral stiffness upon rocking, and energy-dissipating (ED) elements at the rocking interfaces to reduce the lateral displacements [8-11]. The overall design enables structures to be built more expeditiously.

Rahmzadeh et al. [10-11] experimentally and numerically demonstrated the proof-of-concept of the proposed system. They tested small-scale base rocking steel columns under reversed lateral cyclic loading conditions, concluding that the column diameter-to-thickness (d_c/t_c) ratio is the most critical design parameter and that a d_c/t_c ratio is less than or equal to 43 can help prevent buckling of the steel tube. They also observed that the base plate underwent significant bending due to the uplifting of the column. Trembley et al. [9] investigated the longitudinal seismic response of a double-rocking steel bridge pier system and demonstrated the self-centering capability of the system. To further implement the concept in practice, performing dynamic testing to validate the numerical modeling assumption is important. A shake table test program has been planned to experimentally validate the numerical models used to predict the dynamic seismic response of controlled rocking tubular steel piers. Before performing any large-scale dynamic testing, it is customary to evaluate the test system and predict the response of the structure under seismic loading condition. Numerical modeling is a popular tool of choice among structural engineers to predict the behavior of the structure. Different modeling techniques can provide different response predictions of the studied system. Rocking motion is a complex phenomenon, and validating a rocking motion through a numerical model is quite challenging [12-13]. Before performing a shake table testing on such a novel system, it is important to consider the modeling uncertainties and different modeling choices to predict the response of the rocking structure. Pretest analysis will give insight of the expected response of the test specimen, refining the design of the specimen and proper planning of the test program. The objective of the current study is to develop a refined numerical model representing the proposed shake table test setup and to perform nonlinear response history analyses to predict the expected seismic response of the rocking steel tubular bridge pier subjected to ground motions which will be used during the test program.

ROCKING MECHANISM IN SINGLE-COLUMN STEEL BRIDGE BENT

The Rocking mechanism has been implemented within a bridge system in a few notable projects in the past for enhanced seismic performance, including the South Rangitikei Railway Bridge in New Zealand [14], the North Approach of the Lions' Gate Bridge in Vancouver [15], the Rio Vista Bridge in California, the Deadman's Point Bridge in Cromwell, the Wigram-Magdala Link Bridge in New Zealand [16], and the Huangxulu Link Bridge in China [17]. Depending on the type of constructure, gap opening and closing behavior can be incorporated either in the base or at both ends of the bridge piers. Therefore, the bridge pier can have either single rocking (i.e., base rocking) or double rocking configurations. For the base rocking configuration, the cap beam is rigidly connected to the top of the bridge pier. However, it will develop a twisting of the super-structure during the uplift of the bridge pier as seen in Figure 1a. It can be avoided by providing a conventional pot bearing to accommodate the rotation between the super-structure and the bridge pier (Figure 1b-c). This study considered base rocking bridge pier cases while performing response history analyses.



(a) (b) (c)

Figure 1. Schematic of the base rocking mechanism in a steel bridge bent: (a, b) transverse and (c) longitudinal directions.

DESCRIPTION OF BENCHMARK ROCKING STEEL BRIDGE PIER

The bridge studied is a two-span straight highway bridge located on a class C site in Vancouver, British Columbia. It is a two-lane bridge with a total width of 11 m. The total length of the bridge is 46 m, and the two spans have equal length. Conventional elastomeric bearings are used at each abutment. The super-structure includes three continuous composite steel girders. The girders at the intermediate support are connected to an integral steel cross beam supported on a rocking circular steel column having an outer diameter of 1160 mm. The reference bridge pier is 7.2 m in height, including the end plate thickness, and is detailed to rock at the base along the longitudinal direction. The steel column is made from ASTM A252, Grade 3, steel pipes with a specified yield strength F_y = 310 MPa and Young's modulus E_s = 200000 MPa. The tube has a diameter-to-thickness ratio of 32, a value sufficiently low to prevent local buckling upon rocking [8-10]. The proto-type rocking steel bridge pier was designed to satisfy the new performance-based seismic design approach according to CSA S6-19 [18]. The weight of the superstructure is 122 kN/m, resulting in a dead load and total seismic weight of 3508 kN and 5612 kN, respectively. The PT element for each column consists of eight high-strength 36-mm diameter (1018 mm^2) dywidag threaded bars with ultimate tensile strength, f_{pu} = 1035 MPa.

SCALING OF THE BENCHMARK ROCKING STEEL BRIDGE PIER

The prototype bridge pier described in the previous section is scaled to 1:0.35 of the original size in order to satisfy the inherent limitations of the laboratory context. The scaling factors for the prototype bridge are considered to satisfy the classical similitude laws for dynamic analysis. The dynamic test program is designed to satisfy a length factor $S_L = \alpha = L_m/L_p = 0.35$ where m stands for the model, and p stands for the proto-type bridge pier. A scale factor for stresses $\beta = \sigma_m/\sigma_p = 1.0$, and forces $S_F = \alpha^2\beta = 0.122$ were adopted. In order to reduce the seismic weight, time was reduced by $\gamma = t_m/t_p = 0.522$, so the scaling factor for reduced seismic weight became $S_{SW} = \alpha\,\beta\,\gamma^2 = 0.095$. The reference bridge pier was scaled down using $\alpha = 0.35$, resulting in the first reference test specimen (i.e., CRBP4) with a total height of 2514.3 mm. The diameter and thickness of the welded circular base plate are 533 mm and 38.1 mm, respectively. After using the proper scaling factor (S_{SW}), the reduced seismic weight for shake table testing will be 533.1 kN (= 5612*0.095).

PROPOSED EXPERIMENTAL SETUP

As a part of the multi-institutional research project to investigate the seismic behavior of rocking steel tubular bridge pier, a large-scale shake table testing will be performed. The shake table testing aims to validate the numerical models used to predict the dynamic seismic response of controlled rocking tubular steel piers. Testing will be conducted on the earthquake simulator of the Hydro-Québec Structural Engineering Laboratory at Polytechnique Montréal. The plan dimensions of the shake table are 3.4 m x 3.4 m with a clear test height of 10 m. It is equipped with a very high-performance 500 kN capacity actuator. Figure 2 shows the 3D rendering of the proposed test setup. In the test setup, large steel plates will be placed on the column specimens to impose vertical inertia forces from uplifting during rocking and horizontal inertia forces from lateral displacements of the bridge superstructure. The total weight of the large steel plates is 538 kN which will act as the seismic weight during shake table testing, as shown in Figure 2. Two safety steel columns (not shown in the test setup) will be anchored to the shake table on both sides of the test specimen to support the steel plates in case of a non-expected failure of the tested column. Tests will be conducted on columns designed for rocking at their bases only (i.e., base rocking configuration) and columns designed for rocking at their top and bottom ends. The test specimen consists of a circular steel tubular column with a welded circular base plate and a cap plate, PT bar, foundation plate, and a reinforced concrete footing (RC) to represent the real bridge configuration.

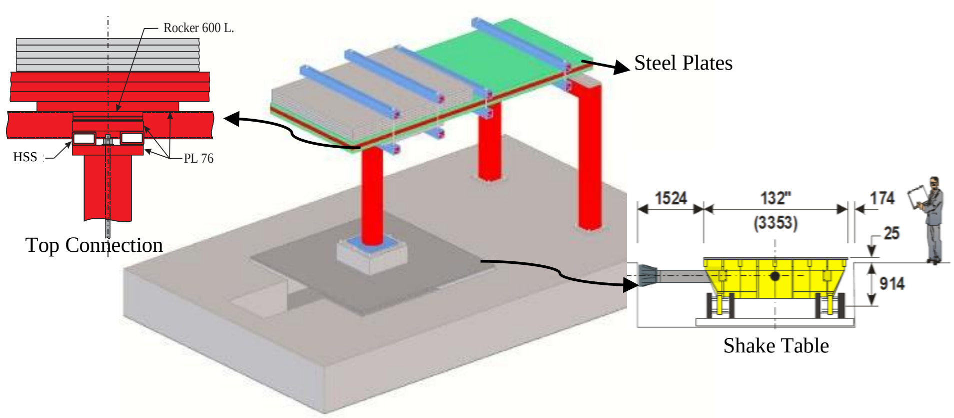


Figure 2. Experimental setup for shake table testing

FE MODELLING OF ROCKING STEEL BRIDGE PIER

A detailed numerical model of the full shake table test setup is developed using the FE software ABAQUS (2017). Various element types and modeling techniques are adopted to develop the refined FE model in order to achieve an appropriate balance between reasonable computational time and accuracy (due to the complex nature of the problem). The general-purpose four-node reduced integration shell element (S4R) is used to model the steel column. This shell element (S4R) uses thick-shell theory when the shell thickness increases, while it increasingly takes on the properties of a discrete Kirchoff thin-shell element as the thickness of the shell decreases. The foundation plate, base plate, and cap plate are modeled using the eight-node reduced integration and hourglass control linear brick element (C3D8R). RC footing and large steel plates on top of the column are also modeled using C3D8R solid elements. Shake table platen is modeled using discrete rigid elements (R3D4) available in ABAQUS as they are not expected to deform during testing. PT bars are modeled using the truss element (T3D2). An overview of the refined FE model of the shake table test setup is shown in Figure 3. A 'TIE' constraint is applied at the bottom surface of the foundation plate and the RC footing, as it will be embedded in the concrete foundation elements in the actual bridge. Although the welding between different parts is not modeled explicitly, the simplified 'TIE' constraint technique was used to simulate the welds between the column and base plate, and cap plate. Surface-to-surface contact was assumed based on the hard contact constraint. The default penalty method was used to simulate the contact interaction between rocking interfaces between the base and foundation plates. ABAQUS/Implicit solver was used for the nonlinear response history analysis. Rayleigh damping, corresponding to 3% of critical in the first two modes, was assigned to the model, with stiffness-proportional damping assigned only to the material of the test specimens. Data on residual stresses and geometrical imperfections of spirally welded A272 steel tubes are scarce. Residual stresses and geometrical imperfections were therefore omitted in this pretest analysis as they were not expected to significantly affect the column response upon rocking.

An isotropic material model was adopted for the column, baseplate, and cap plate. The engineering stress-stress response of a circular tube with a wall thickness of 9.53 mm measured in a previous study [10] was used for the material model. The coupon test results showed a continuous strain hardening with yield and ultimate stress values of 415 and 540 MPa, respectively. Similar modeling techniques have been adopted to develop the simplified FE model (see Figure 3). However, the simplified FE model ignored the top connection detail (as shown in Figure 2) and the contribution of RC footing while performing the nonlinear response history analysis.

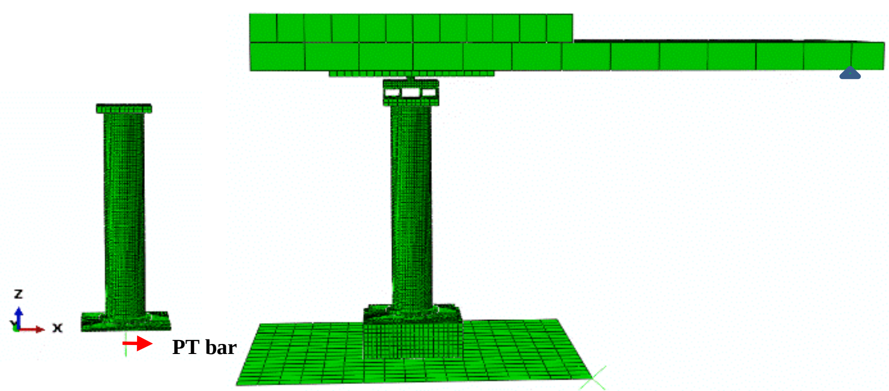


Figure 3. Schematic illustration of the FE model used in this study: Simplified FE Model (left) and Refined FE model (right)

LOCAL BUCKLING VALIDATION USING BENCHMARK EXPERIMENT

The developed FE modeling technique described in the previous section was validated using the benchmark experimental program performed by Nishikawa et al. [19] on large-scale circular hollow section steel columns under reverse cyclic loading condition. The main objective is to confirm the ability of the FE model to capture the yielding and local buckling of the steel tubular columns. Lateral load-deformation response obtained from the test program was used to compare the FE analysis results. Figure 3 shows the comparison between the benchmark test and the FE analysis results, and the final deformed configuration of the test specimen (No. 8) and the FE model. The difference between experimental and FE results in terms of maximum lateral load prediction in the positive and negative directions were found to be 2.2% and 4.8%, respectively. This clearly shows the capability of the FE model to accurately predict the cyclic response of the fixed-based circular hollow tubular column.

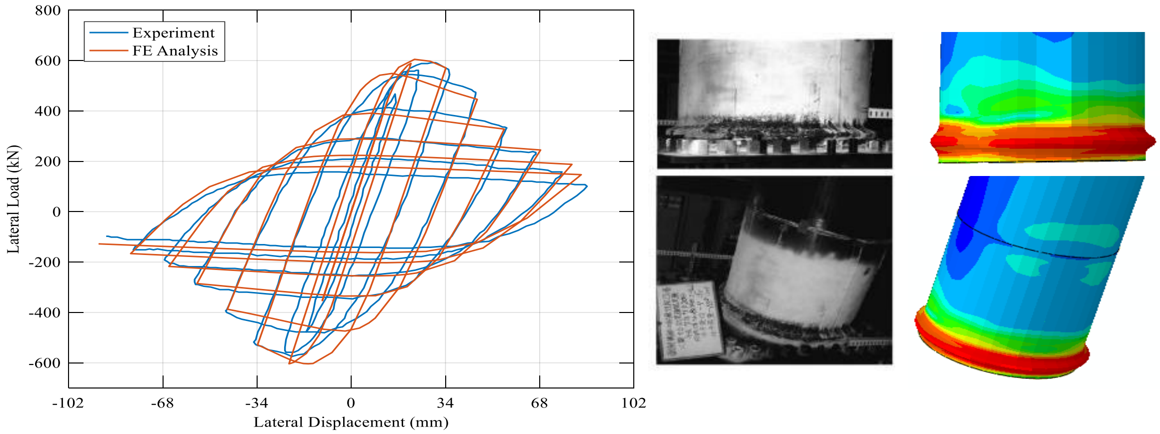


Figure 3. Lateral load-displacement response and final configuration of experimental and FE analysis for Specimen #8.

DESIGN OF SCALED ROCKING BRIDGE PIER SPECIMEN

The performance of the reference rocking steel column was verified through a nonlinear static pushover analysis, as shown in Figure 4. The same modelling techniques described in the previous section were adopted for the pushover analysis. The displacement demand was obtained from an earlier study by authors of the present work performed on a base rocking steel bridge pier using a simplified macro model [9-10]. The displacement demand for the 10% in 50 years probability and the 2% in 50 years probability were assumed to approximately be 1.45% and 4.5%, respectively, of the column height. The steel column and baseplate do not experience any yielding at 1.45% drift. Localized yielding of the baseplate and the tube wall initiates at 2.2% and 3.6% drift, respectively. No buckling of the tube wall was observed, even at 4.5% drift, meaning that the initial performance objective of the design was verified. We note that rocking steel columns should not experience yielding and buckling to have an influence on the self-centering response to seismic demand from 10% in 50 years and 2% in 50 years hazard level.

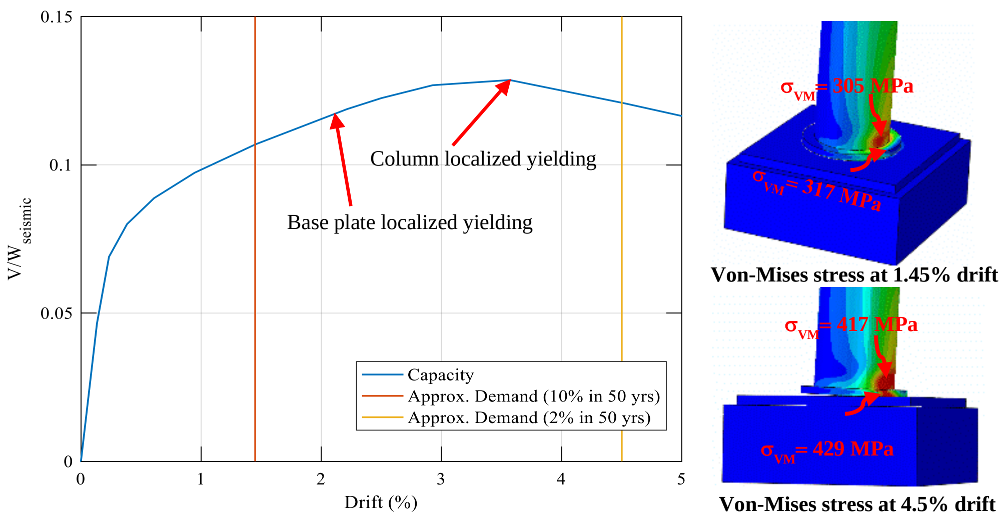


Figure 4. Lateral pushover response of reference test specimen

SELECTION AND SCALING OF SEISMIC GROUND MOTIONS

To evaluate the seismic performance of the rocking steel columns before shaking table testing, nonlinear response history analysis (NLRHA) is performed using an ensemble of representative ground motions selected and scaled per the guidelines of Commentary I of the 2020 National Building Code of Canada [20]. The ensemble comprises 2 suites of total of 22 ground motion records contributing to the hazard for the proto-type bridge site: 11 of them are shallow crustal and subduction deep in-slab, and the rest 11 are from subduction interface earthquakes. This paper presents the results from only three horizontal shallow crustal ground motion records. The selected ground motion records for NLRHA are listed in Table 1.

Table 1. Selected earthquake records

Record ID	Earthquake	M_w	Station	Component	Type	Scale factor
GM3 (RSN960)	1994 Northridge	6.69	Canyon Country	270°	Crustal	0.888
GM5 (RSN848)	1992 Landers	7.28	Coolwater	TR	Crustal	0.922
GM6 (RSN787)	1989 Loma Prieta	6.93	Palo Alto	360°	Crustal	1.499

NONLINEAR RESPONSE HISTORY ANALYSIS ON SCALED ROCKING BRIDGE PIER SPECIMEN

The seismic performance of three different test specimens was evaluated using the selected GM records. The considered cases are listed in Table 2. The considered parameters were column diameter-to-thickness ratio, d_c/t_c, the ratio of the area of the PT bar to that of the column cross-section (A_{pt}/A_c), and the axial force ratio (AFR_o), which is the ratio calculated as the gravity load plus the PT bar force divided by the yield strength of the column (i.e., $P_y = A_g F_y$). Previous experimental and numerical investigations have shown that the d_c/t_c ratio is the most important design parameter influencing the self-centering response of rocking steel bridge piers [8-11]. To investigate the effect of d_c/t_c ratio, three different cross-sectional classes for circular hollow tubular sections according to CSA S6-19 [18] were considered. The class sections were determined using F_y = 415 MPa.

Table 2. Selected analysis cases from the shake table test program

Column ID	d_c (mm)	t_c (mm)	d_c/t_c	A_{pt}/A_c	$f_{PT,o}/f_{PT,u}$	AFR_o	t_{bp}	e_{bp}	Cross-sectional Class
CRBP4 (reference)	406	12.7	32	0.06	0.35	11.8	38.1	63.5	Class 1
CRBP3	406	9.53	43	0.09	0.35	15.5	38.1	63.5	Class 2
CRBP1	406	6.35	64	0.13	0.35	23.2	38.1	63.5	Class 3

The seismic response of the reference column using the simplified and refined FE models under a crustal earthquake ground motion record is shown in Figure 5. As mentioned above, the seismic weight from the super-structure was lumped on top of the column in the simplified model, which ignored the effect of the foundation, shake table, and the top connection details, as shown in Figure 3. Compared to the more refined FE model, the simplified FE model was able to capture the seismic response pattern, but over-predicted the response quantities. Moreover, the vertical acceleration of the pier-top was found to be more sensitive in the simplified model compared to the refined model. This is mainly attributable to the fact that the superstructure mass was distributed along the length in the refined FE model, with the redistribution of the column axial force occurring in the right support (Figure 3), which cannot be seen in the lumped mass model. As such, the rest of this paper focuses on the results obtained from the refined FE models. Nevertheless, the simplified FE model may still be useful for conservatively predicting the displacement demand to avoid any unwanted failure of the test specimen during shake table testing.

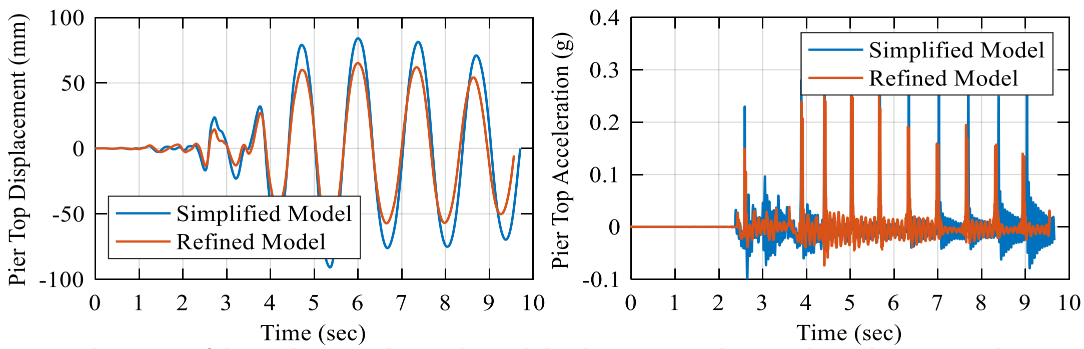


Figure 5. Time histories of the rocking steel piers lateral displacement and vertical acceleration at the pier top using simplified and refined FE models under a scaled GM

It should be noted that there is a significant difference in computational time between the simplified and refined FE models. NLRHA using the general-purpose FE software, ABAQUS, is highly computationally demanding. Therefore, the ground motion input was trimmed so that the simulation could be finished using a local machine. Even for 10 to 15 second input of ground motion, the simplified model took an average of 24 h to 36 h, while each refined model took between 72 h and 120 h, depending on the duration of the ground motion, the number of output responses requested from ABAQUS, the number of CPU cores used, the CPU parallelization, and the GPU acceleration. Furthermore, many of the analyses terminated early (i.e., before the total input ground motion duration had been simulated) due to insufficient memory space of the local machine, as the generated files exceeded the available physical memory of the local machine. In future work, Compute Canada's supercomputing facility will be used to perform the NLRHA to overcome this computational limitation.

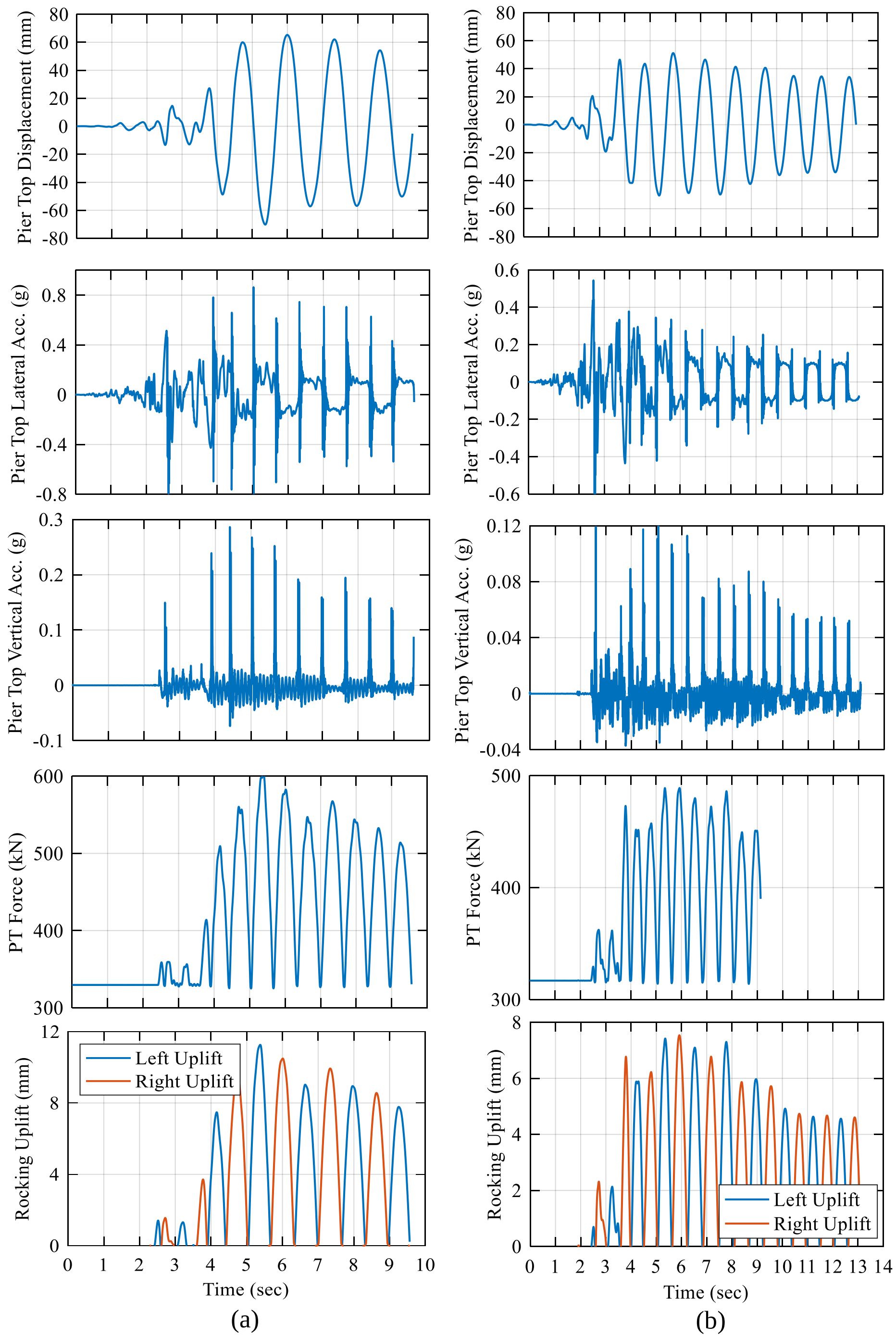


Figure 6. Response quantities for refined FE model with rocking steel pier: (a) CRBP4 (Class 1 section) and (b) CRBP3 (Class 2 section) under scaled Northridge earthquake record (100% Amplitude)

The results of the NLRHA performed on the CRBP4 and CRBP3 under a crustal GM record are presented in Figure 6. In both cases, the column d_c/t_c ratios were varied, resulting in different values of AFR$_o$ (i.e., 11.8% and 15.5%, respectively). The time history responses of five different response quantities are presented in Figure 6. As can be seen, the peak horizontal displacement was found to reach 70 mm (i.e., 2.8% of column height) for CRBP4 and 51 mm (i.e., 2% of column height) for CRBP3. The second and third pairs of plots show the variation of lateral and vertical acceleration responses, respectively, on the pier top. As can be seen, the lateral and vertical acceleration were found to reach 0.86 g and 0.29 g, respectively, for CRBP4. For the case of CRBP3, the peak lateral and vertical acceleration were found to reach 0.65 g and 0.12 g, respectively. It should be noted these peaks occur each time the pier uplifts due to rocking and then returns to the original position. The last set of plots in Figure 6 shows the rocking induced uplift profile of the column baseplate. As shown in the figure, the maximum values of uplifting of the edge of the baseplate for CRBP4 and CRBP3 were found to be 11.2 mm and 7.5 mm, respectively. If we assume the neutral axis depth is approximately 0.15d_c, the column lateral drift due to rocking can be approximately calculated to be 2.4% (= 11.2/(d_{BP} -0.15d_c)) for CRBP4 and 1.6% (= 7.5/(d_{BP} -0.15d_c)) for CRBP3, where d_{BP} is the diameter of the welded baseplate. The remaining drift values were due to the column lateral deformation and flexibility of the test setup. These findings need to be verified further using the shake table test results. Deformation of concrete footing and localized bending of baseplate are possible causes to be investigated during the shake table testing.

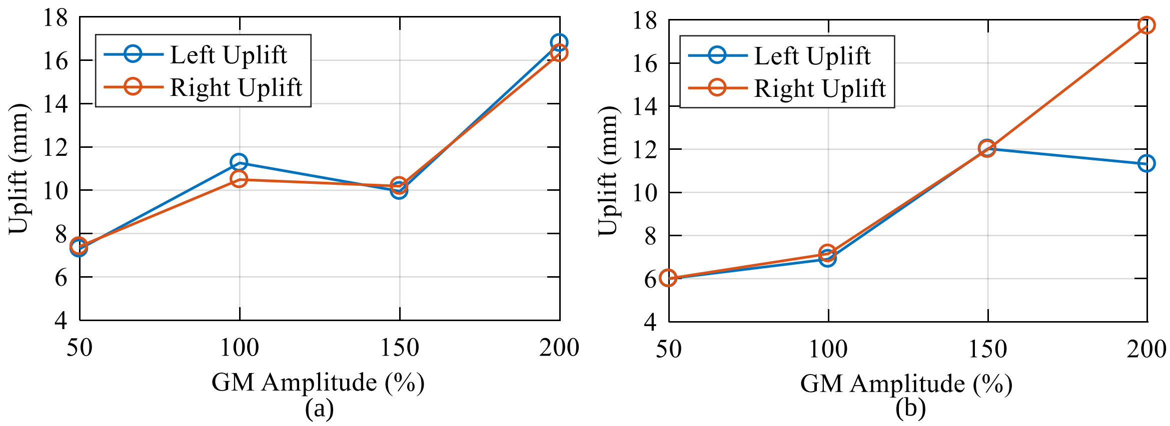


Figure 7. Displacement demand (rocking uplift) in rocking column specimens under increasing GM amplitude: (a) CRBP4 and (b) CRBP1

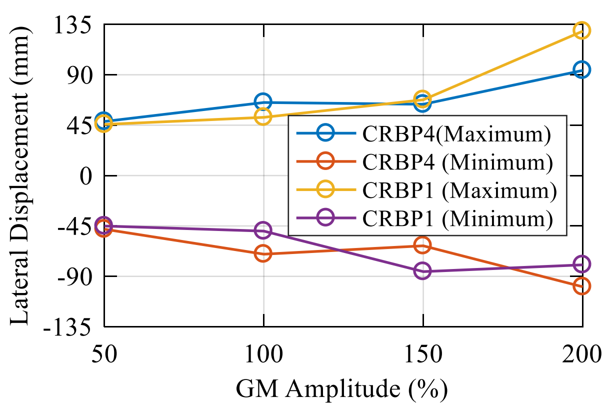


Figure 8. Lateral displacement demand in rocking column specimens under increasing GM amplitude: (a) CRBP4 and (b) CRBP1

Figure 7 and Figure 8 show the displacement demand for two test specimens under increasing ground motion (GM) intensity. As can be seen, rocking uplift and lateral drift demand generally increased as the GM amplitude increased. The lateral drift was higher than the drift associated with the rocking uplift, as explained in the previous section, due to the lateral deformation of the column. Moreover, the rocking column starts to experience local bulging in the tube wall due to increased GM intensity. This can be explained by the increased von-Misses stresses close to the rocking interface, along with Poisson's effect. This phenomenon was found to be more prevalent for Class 2 and Class 3 cross-sections with higher d_c/t_c and increased AFR$_o$ ratio.

Moreover, the CRBP1 specimen experienced more local buckling than did the CRBP4 specimen due to the former's very high d_c/t_c. This resulted in a different uplift profile for the CRBP1 specimen under 200% GM amplitude due to the loss of recentering. Furthermore, the increases in lateral drift and rocking uplift pattern were not linear due to the interaction of flexural deformation and buckling of the column, bending of the baseplate, and ground motion loading excursions. Both the column specimens experienced buckling in higher GM amplitudes, resulting in column shortening and strength degradation. Ultimately, CRBP1 could not recenter at 200% GM amplitude due to asymmetric local buckling at the compression side of the rocking interface, as seen in Figure 7b. An important consideration to underscore is that, under increasing GM amplitude, peak lateral displacement should not be the only criterion in evaluating the performance of rocking steel columns due to the complex nature of the problem and the localized phenomena that occur during the small cycles that exist between large cycles in a typical ground motion record. Due to space limitations, only the peak displacement demand is reported herein (see Figure 8). It should be noted that the FE modelling presented herein did not consider the effect of the combined hardening material model of steel in investigating the dynamic seismic response, although this might alter the peak response quantities and the overall stress distribution of the baseplate and tube wall.

SUMMARY AND CONCLUSIONS

This paper presents the results of the numerical seismic response of rocking steel tubular bridge piers subjected to different crustal ground motion records. Nonlinear FE models were developed using ABAQUS in order to investigate the dynamic seismic response of the system. A shake table test program has been planned to investigate the dynamic behavior of this novel bridge pier system. These pretest analyses will give valuable insights into the test program and the expected response of the test specimens. They will also help to establish the requisite degree of confidence in the test program to obtain the expected outcomes without any damage to the shake table or unexpected behaviour during testing. Two different FE modeling techniques were adopted to predict the seismic response of the rocking steel columns, with the reported response parameters being the lateral displacement of the column top, the horizontal and vertical acceleration of the column, the PT bar force pattern, and the uplifting profile of the rocking interface. The simplified FE models were found to overpredict the drift responses compared to the refined FE models, but to be capable of capturing the general trends with reasonable accuracy and in less computational time. The vertical acceleration of the pier-top was found to be sensitive to model complexity, mass redistribution, and flexibility of the steel plates as opposed to the lumped mass model. It should be noted that there was a slight variation in seismic weight between the simplified and refined FE models due to the weight of RC footing, foundation plate, and a few extra elements at the top connections (e.g., weight of HSS sections and small connecting plates). The column specimens were subjected to increased GM intensity in order to investigate their dynamic seismic response, and the results were reported in terms of lateral displacement and rocking uplift. Buckling in tube wall and bending in the baseplate under flexure were found to affect the self-centering and uplifting pattern of the rocking steel columns. However, it should be noted in this regard that the developed numerical models have to be calibrated after performing the shake table test, which is expected to take place at the end of this year. Future studies should investigate the effect of inherent damping in order to capture different energy dissipation phenomena during rocking motion and various damping related parameters on the dynamic response of a rocking steel bridge pier system. In addition, future numerical simulations should investigate the influence of initial imperfection, residual stress pattern for spirally welded tubular columns, and the effect of different material modelling in order to gain a better understanding of the behaviour and modeling uncertainty in predicting the key response quantities.

ACKNOWLEDGMENTS

This multi-institutional research project is sponsored by the Natural Sciences and Engineering Research Council (NSERC) of Canada under the Collaborative Research and Development (CRD) grant program, industrial partners including the Canadian Institute of Steel Construction (CISC), Rapid Span Structures Ltd., S-FRAME Software Inc., Emil Anderson Construction Inc., and Klohn Crippen Berger Ltd. The first author has received scholarships from the Fonds de recherche du Québec - Nature et Technologies (FRQNT) (Doctoral fellowship), the Centre d'études interuniversitaire des structures sous charges extrêmes (CEISCE) (International scholarship), and l'Association des Étudiants de Polytechnique (AEP).

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