IMPACT ON SEISMIC RESPONSE OF BRIDGE COLUMNS DUE TO SEASONAL FREEZING

L.M. Wotherspoon[1], S. Sritharan[2] and M.J. Pender[3]

[1] *Doctoral Candidate, Institute of Earth Science and Engineering, University of Auckland , Auckland, New Zealand*

[2] *Associate Professor, Dept. of Civil, Construction and Environmental Engineering, Iowa State University, Ames,IA, USA*

[3] *Professor, Dept. of Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand*

ABSTRACT :

Several significant seismic events have occurred during the winter months in areas that experience seasonal freezing. In a recent study, it has been shown that freezing temperatures can significantly influence the soil-foundation-structure interaction and structural response of bridge columns due to changes in material properties, especially those of soils. As part of this study, cyclic lateral load testing of identical full-scale column-foundation systems during summer and winter months has revealed the dramatic changes in response created by freezing temperatures. Using the measured material properties from the two tests, analytical models were developed and validated using the cyclic response of the test units. In this process, it was shown that the analytical models satisfactorily captured measured responses of a range of critical parameters, which differed greatly between the warm and frozen conditions. The analytical models were then extended to investigate the response of the column-pile-soil systems under seismic loads. This paper presents the validated analytical models and the expected effects of seasonal freezing on the seismic response of bridge columns subjected to earthquakes of different intensities.

KEYWORDS: Analytical model, soil-structure interaction, frozen soils, cyclic response, seismic response, bridge column

1. INTRODUCTION

Significant seismic events have occurred around the world during winter months in regions where cold temperatures causes ground freezing (e.g. Kobe, Japan, 1995 and Prince William Sound, Alaska, 1964). Using results from an outdoor test program at Iowa State University, a model capable of capturing the effects of seasonal freezing on the lateral load response of a soil-foundation-structure (SFS) system was developed using Ruaumoko (Carr 2005). Cyclic responses of a bridge column supported by a deep foundation were captured using a variety of recorded test outputs. This model was then extended to investigate the seismic response of a two span prototype bridge structure through the inclusion of the dynamic characteristics of the system. While the stiffness characteristics were taken from physical test data, the damping characteristics of the column/foundation and soil were assumed based on approaches found in the literature. Comparisons of the response of the system during frozen and unfrozen conditions were made over a range of seismic intensities.

2. BACKGROUND

An outdoor experimental program undertaken at Iowa State University examined the response of three large-scale bridge columns supported by cast in drill hole (CIDH) shafts embedded in glacial till. The soil was classified as low plasticity clay and the water table was at a depth of 8.2 m. Two identical test units were analysed during summer (SS1) and winter (SS2) months at ambient temperatures of 23°C and -10°C. The lateral load was applied to the column top of each test unit using a hydraulic actuator attached to a reaction column. SS1 and SS2 had 0.61 m diameter column and foundation shafts, with above-ground column length of 2.69 m and an embedded shaft length of 10.36 m. Test units were designed with a 2% longitudinal steel ratio along the entire column and shaft lengths, with 20, Grade 60, 19 mm diameter bars. A more complete summary of the test setup is given by Suleiman et al. (2006).

At the outdoor test facility, the temperature of the top 3 m of soil was monitored using a series of thermocouples spaced at 15.25 cm intervals. The temperature profile for the SS2 test indicated that the soil was frozen from the surface down to a depth of 0.76 m. An observation from the temperature profiles was that soil temperatures decreased linearly within the frozen soil layer (Sritharan et al. 2007). CPT tests were also undertaken to determine the soil characteristics with depth. In order to capture both the global and local responses of the test units, multiple output factors were recorded. Horizontal displacements of the top, middle and base of the column were recorded along with the gap opening at the ground level between the foundation shaft and the soil. Strain gauges were mounted to the longitudinal reinforcement down the length of the column and shaft in order to identify the peak moment location and the extent of the plastic region.

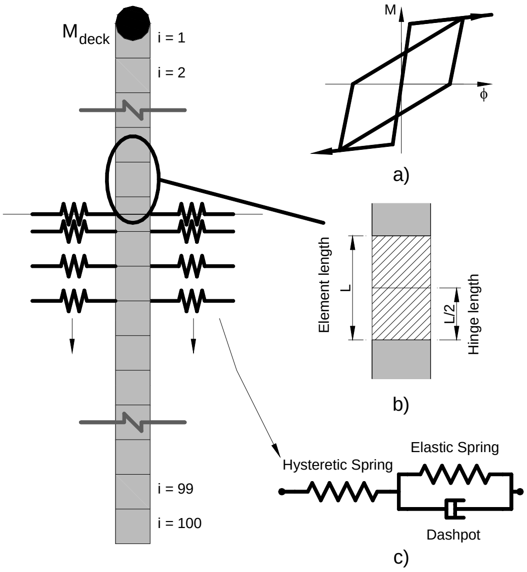

Figure 1 Seismic model layout used in Ruaumoko to represent the response of SS1 and SS2 a) column/foundation element detail; b) column/foundation moment-curvature characteristics; c) series radiation damping model

3. RUAUMOKO MODELS

A model suitable for the application of seismic loads and horizontal cyclic loads applied at the top of the column and is shown schematically in Figure 1. The column and pile shaft were divided into 100 segments represented by beam-column elements. Soil was modelled using non-linear spring and dashpot elements attached to the nodes at the end of each beam-column element. Springs were attached to both sides of the pile to represent the soil on each side and to model the attachment and reattachment of soil as it separates from the pile. The non-linear response of the column/pile was modelled by specifying the moment-curvature response envelopes at the ends of each beam-column member and allowing plastic action to develop over half the length of the member.

3.1 Column/Pile Properties

Using the measured properties of concrete and reinforcement in SS1, moment-curvature relationships of the column and foundation shaft sections were determined using concrete and reinforcement models proposed by Mander et al. (1988) and Menegotto & Pinto (1973) respectively. The frozen temperatures influenced the moment-curvature response of SS2 sections by enhancing the properties of the concrete and steel strengths and providing soil confinement beyond that provided by the transverse reinforcement (Sritharan et al. 2007).

Investigation after testing had commenced also revealed that the reinforcement cage in the SS2 unit had shifted during construction by 38 mm, which created an unsymmetrical cyclic response for this test unit. Moment curvature characteristics of the column/pile members were represented using the Modified Takeda hysteresis rule available in Ruaumoko, which is presented in Figure 1a (Otani 1971). To model the spread of plasticity along the column/foundation shaft, the hinges at each end of the beam-column elements were made equal to half the element length as indicated in Figure 1b, allowing plastic action to develop along the length of the column/foundation.

3.2 Soil Modelling

Using CPT and unconfined compression test data, p-y curves were developed to represent the properties of the soil stratum at the test site as described in Sritharan et al. (2007). For the soil to a depth of 0.76 m p-y curves were created from unconfined compression test data obtained for soil samples by following the procedure presented by Reese & Welch (1975). Below this level, p-y curves were created using the CPT data and procedures developed by Reese and Welch for stiff clay. This method was satisfactory for the SS1 test; however, modifications had to be made for the SS2 test to account for the effects of frozen soil. During SS2 testing a large tension crack opened up adjacent to the pile shaft perpendicular to the loading direction. Once opened, the crack increased with loading, suggesting it would have a significant impact on the behaviour. A method was devised to soften the response of the soil after the tension crack had developed using the relative sizes of the total gap and tension crack (Wotherspoon 2008).

In order to accurately model the development and growth of gaps adjacent to the pile, the force displacement characteristics of the soil springs on each side of the pile were represented using the compression-only bi-linear relationship in Figure 2. In the tensile range, the spring element provides no force resistance allowing it to move freely. If the spring yields in compression during loading, it will unload with the stiffness equal to the initial value and reach zero load at a non-zero displacement in the compressive range. If displacement continues in the tensile direction, no load will be carried, moving the point of gap initiation back from the origin in Figure 2b. The further the spring is pushed into the inelastic range, the larger this move will be, simulating the growth of the gap that typically forms adjacent to the pile shaft embedded in clay. Each soil spring was prestressed to account for the horizontal insitu soil stresses created by the soil overburden pressure. This defines the initial state of the spring prior to lateral loading, which is represented by a solid circle in Figure 2a.

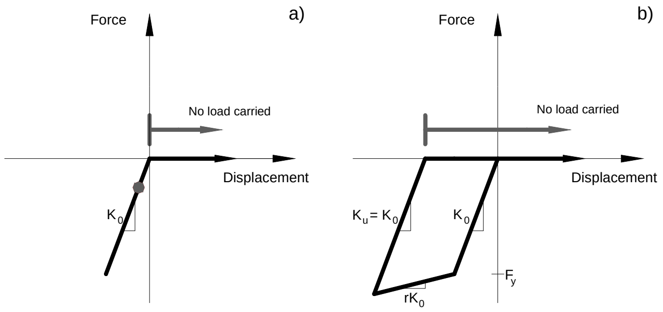

Figure 2 Compression only bi-linear soil spring hysteresis model for one side of foundation indicating displacement ranges where spring carries no force: a) prior to compressive yield; b) after compressive yield

3.3 Seismic Model

The seismic model was assumed to support a two-span bridge structure without any restraints at the abutments, with excitation directed along the transverse axis of the bridge. Between the abutments, the bridge was supported by one column/foundation unit in the middle with the same characteristics as the test units. Because of these assumptions, the column/foundation unit effectively acted as a single cantilever structure. The lumped mass attached to the top of the column to represent the inertia effect of the bridge superstructure was determined based on the assumption that the columns were designed to have 5% axial load ratio due to gravity effects. Using the specified concrete unconfined compressive strength of 27.6 MPa, the lumped mass at the column top was equal to 404 kN. For consistency only the SS1 characteristics were used to calculate this mass, which was then used in both the SS1 and SS2 models. The main differences between the column/foundation characteristics for the cyclic and the seismic models were the addition of the axial load from the superstructure and the use of a centred reinforcement cage for the SS2 model.

Spring and dashpot elements were used respectively to model the stiffness and damping characteristics of the soil surrounding the foundation. Soil stiffness characteristics from the cyclic models were used in the seismic models, incorporating the methodology used to represent the force-displacement characteristics of the frozen soil. The hysteretic damping of the soil was represented by the soil hysteresis model, and radiation damping was modelled using Ruaumoko dashpot elements. Spring and dashpot elements were attached along the length of the foundation shaft on both sides. At each depth, these were arranged in a series radiation damping configuration as illustrated in Figure 1c (Nogami et al. 1992, Novak and Sheta 1980). This term was coined by Wang et al. (1998) to describe a non-linear hysteretic element in series with a linear visco-elastic element. The spring closest to the foundation, labelled hysteretic spring in Figure 1c, models the detachment of the soil, the gap opening, and the non-linear compressive behaviour of soil. The outer spring, labelled elastic spring in Figure 1c, defines the elastic stiffness characteristics of each series radiation damping grouping. This configuration means that forces from the foundation must first travel through the hysteretic zone before being radiated away.

Dashpot characteristics for radiation damping in the seismic model were established based on the elastic theoretical solutions reported for vibration of a pile by Gazetas and Dobry (1984). Solutions for damping coefficients were dependant on the angular frequency, and a characteristic angular frequency (ω) was adopted using the fundamental period (T) of the soil-foundation-structure system ($\omega = 2\pi/T$). Kinematic interaction effects between the foundation and the surrounding soil were ignored in the analysis.

4. CYCLIC ANALYSIS RESULTS

Cyclic analyses followed the experimental loading sequences for each test unit, applying gradually increasing target displacements with no less than three cycles at each target displacement. These load sequences are summarised in Suleiman et al. (2006).

4.1 Force-displacement response

The force-displacement responses of the Ruaumoko and experimental results are compared in Figure 3. Figures 3a and 3b show the results obtained at the top and base of the SS1 column which confirm that the Ruaumoko model accurately captures the shape and the hysteretic nature of the SS1 soil-foundation-system at both locations. Figures 3c and 3d provide a similar comparison for SS2. Although satisfactory, they are not as closely matched as those observed for SS1. At the top of the column, the SS2 model hysteresis shape shows some good similarities with the test data and a small discrepancy seen between the measured and theoretical comparison is in the unloading regions. At the column base, the lateral displacements of the SS2 model were somewhat underestimated and overestimated in each respective direction of loading.

4.2 Column/foundation bending moment

The variation of bending moment down the column/foundation length was determined from analysis results and some critical information is compared to experimentally deduced values in Suleiman et al. (2006). The strain gauge data indicated that for the SS1 test unit the peak moment at 19.1 cm of lateral displacement occurred at almost 1.1 m below ground level and the plastic region in the foundation extended down to a depth of 2.5 m. Figure 4a shows the moment profile obtained from the SS1 cyclic Ruaumoko analysis for a top displacement of

19.1 cm. The peak moment was approximately 1.05 m below the ground surface and the plastic region extends to approximately 2.5 m below ground, both of which agree well with the SS1 test results. For the SS2 test, the peak moment at a lateral displacement of 9.4 cm displacement was determined to be at approximately 0.26 m below the ground level and the plastic region extended down to a depth of 0.9 m. Figure 4b shows that for the cyclic Ruaumoko model the peak bending moment was approximately 0.2 m below the ground surface and the plastic region extended to 1.1 m below ground, both which are again in good agreement with the test results.

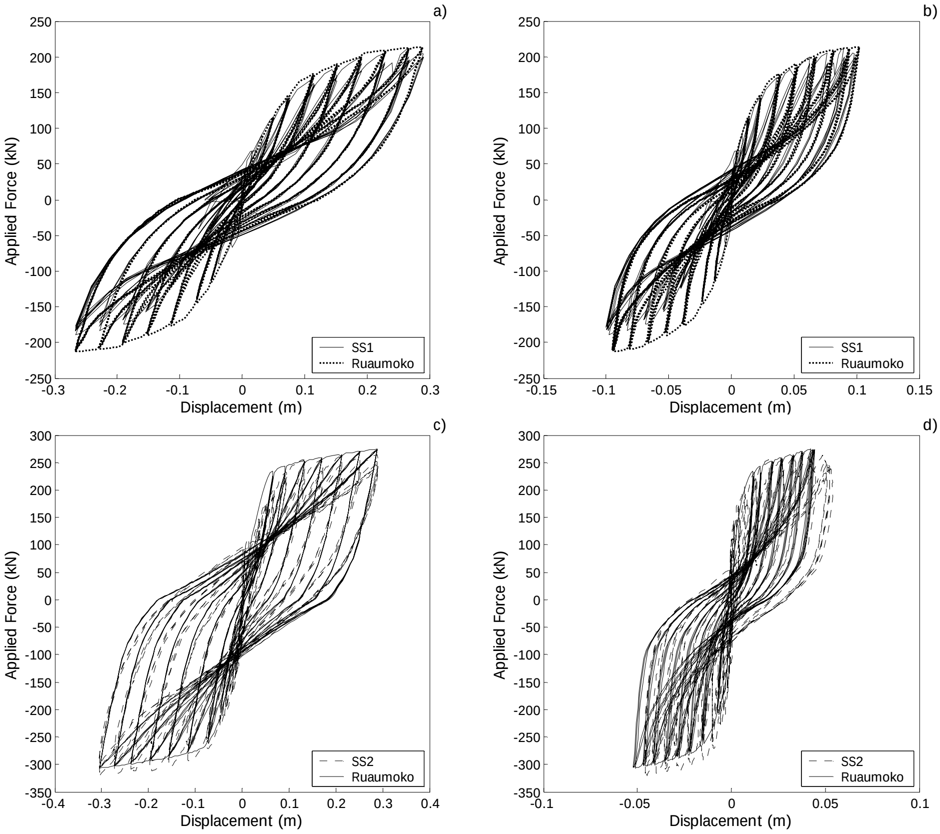

Figure 3 Cyclic force-displacement responses for a) SS1 column top; b) SS1 column base; c) SS2 column top; d) SS2 column base.

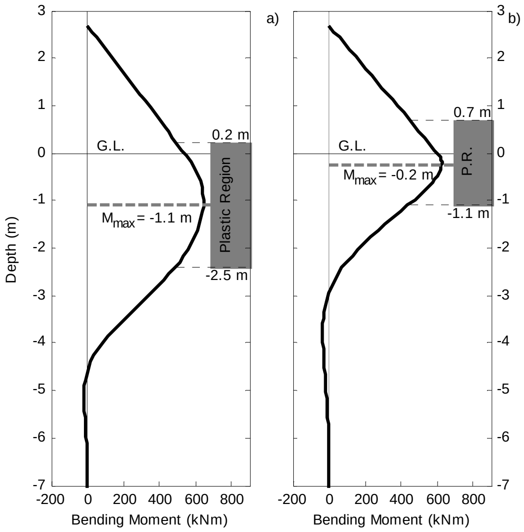

Figure 4 Bending moment profile along the length of a) SS1 at a column top lateral displacement of 19.1 cm; and b) SS2 at a column top lateral displacement of 9.4 cm

5. SEISMIC ANALYSIS RESULTS

Elastic analysis of the SS1 and SS2 seismic models determined that the fundamental period of the SS1 and SS2 units were 0.671 and 0.534 seconds, respectively. This corresponds to a 20% reduction in period due to the frozen condition and the corresponding effect on the stiffness of soil and column/foundation. These fundamental periods were used to define the scale factors for 500 and 2500 year return period events according to NZS 1170.5:2004 (SNZ 2004). The Tabas, Iran, 1978 event was used for both, with the peak ground acceleration (PGA) for the SS1 and SS2 models equal to 0.67g and 0.65g for the 500 year event, and 1.22g and 1.25g for the 2500 year event.

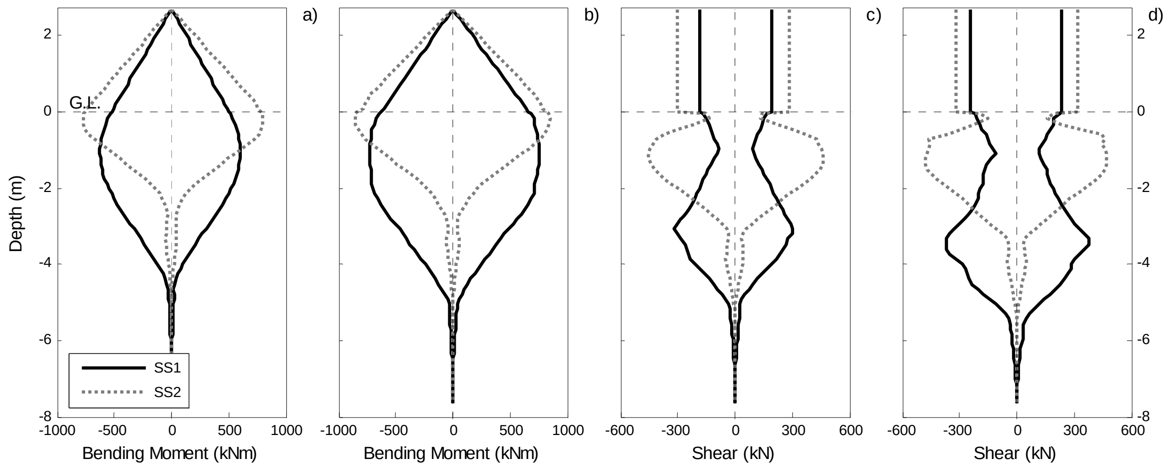

Figure 5 Envelopes of the SS1 and SS2 seismic models for a) bending moment 500 year return period; b) bending moment 2500 year return period; c) shear 500 year return period; d) shear 2500 year return period

5.1 Bending moment

Envelopes of the maximum bending moment down the length of the column/foundation for the SS1 and SS2 seismic models obtained for the 500 and 2500 year return period events are presented in Figure 5a and 5b. As the return period increases, the depth to maximum bending moment for the SS1 model increases slightly, while the depth for the SS2 model remains almost unchanged. For both return periods, the depth to the maximum moment for SS2 is much shallower than SS1 and the maximum bending moment is larger for the SS2 model. The 500 year return period results compared in Figure 5a show that the bending moment demand for the SS2 model was 30% larger than the SS1. Depth to the maximum moment reduced from 1.0 m for SS1 to 0.2 m for the SS2 model. No hinge developed in the SS1 model, while the plastic region for the SS2 model extended from 0.19 m above ground to 0.6 m below ground.

The 2500 year return period results in Figure 5b had similar characteristics to the 500 year return period data. The SS2 maximum moment was 12% larger than SS1, a reduction in the difference during the 500 year return period event as both column/foundations experienced non-linear responses. As both foundations developed plastic hinges, the rate of increase of bending moment decreased, allowing the peak moment values to move closer together. The plastic hinge region of the SS1 model extended from 0.19 m to 2.2 m below ground, and the SS2 model plastic hinge region was from 0.4 m above ground to 0.7 m below ground.

5.2 Shear

Figure 5c and 5d compare the maximum shear envelopes obtained for the same return period events as the bending moment profiles. The SS2 model had higher shear demands in the column as well as foundation for both return periods. The maximum shear depth for the SS1 model was deeper than the SS2 model, with the rate of increase of the maximum shear depth of the SS2 model significantly less than the SS1 model. For the 500 year event in Figure 5c, the shear in the SS2 column was between 45 and 60% larger than the SS1 column. Shear in the foundation was 43 to 55% larger for SS2, with the maximum shear depth reducing from 3.0 m to 1.2 m. Even though results for the two models were closer during the 2500 year event in Figure 5d, the maximum SS2 shear was still at least 25% larger than SS1 in both the column and the foundation. At the depth of maximum shear in the SS2 foundation, the shear demand was much larger than that of SS1 at the same depth. For the 500 year event the shear demand was 316% larger, while for the 2500 year event this value reduced to 200%. These are significant differences in demand for both return period events, and are important because one will assume shear is not critical at this location if the effects of frozen conditions are ignored.

5.3 Horizontal displacement

Envelopes of the maximum horizontal displacement of the two models for the 500 and 2500 year return period events are presented in Figure 6. Below ground displacements were significantly reduced due to the effects of frozen soil in the SS2 model, as indicated by the inset plots in both Figure 6a and 6b. The 500 year return period results in Figure 6a show that the SS2 column top displacement was between 54 and 78% of the SS1 peak displacement. At the ground level, these values reduced to between 22 and 30%, respectively. The displacement characteristics during the 2500 year return period earthquake were similar to the 500 year return period. Figure 6b shows that the SS2 column top displacement was between 43 and 76% of the SS1 displacement, while ground level displacements were between 14 and 24%.

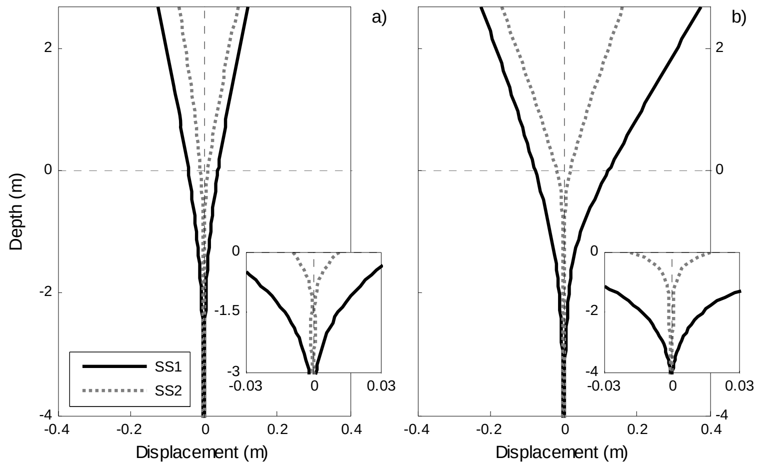

Figure 6 Horizontal displacement envelopes of SS1 and SS2 seismic models for a) 500 year return period; and b) 2500 year return period

The column top displacement capacities of the SS1 and SS2 units were defined at the first occurrence of a steel strain of 0.07% in the longitudinal reinforcement to account for the effects of low cycle fatigue expected under seismic loading. Using sectional analysis and monotonic pushover data, the resulting displacement capacities were equal to 42 cm and 15 cm for SS1 and SS2, respectively. For the 500 year return period event, the SS1 model reached 29% of the displacement capacity of the system, while the SS2 model reached 62%. For the 2500 year return period data, the fraction of displacement capacities of the two models moved closer together, with values of 90% for the SS1 and 109% for the SS2 model. A displacement demand above 100% of the capacity as in the latter case indicates fracture of the longitudinal steel reinforcement in the plastic region.

6. CONCLUSIONS

Using elements and hysteresis rules available in Ruaumoko, the cyclic responses of test column/foundation systems were captured successfully. Characteristics of both the summer and winter tests have been well represented using the Ruaumoko models and can highlight the effects of seasonal freezing temperatures. False confidence can be given to models that may capture the global response but are unable to represent localised characteristics. The use of multiple outputs to validate analysis results reinforces the accuracy of the models and indicates the importance of using more than one output variable. Results highlight the considerable impact of temperature on the response of the test units and the importance that must be placed on the range of possible temperature conditions when performing seismic design of bridges.

Consistent with cyclic responses, seismic analysis indicated that the maximum bending moment and shear values were larger in the SS2 model for both intensities. The difference in the maximum bending moment of the two models reduced from 30% during the 500 year event to 12% for the 2500 year event. For the 500 year event, the shear in the SS2 column was between 45 and 60% larger than that experienced by the SS1 column, and the peak shear in the SS2 foundation was 43 to 55% larger than SS1. During the 2500 year event, the maximum SS2 shear was 38% and 22% larger than SS1 in the column and foundation, respectively. The significant differences in the shear demand for both units at the depth of maximum shear in the SS2 foundation is important because one will assume that the shear is not critical at this location if the effects of seasonally frozen conditions are ignored. At this depth, the SS2 shear demand for the 500 and 2500 year events were 316% and 200% larger than the SS1 shear at the same depth. Even though the horizontal displacement of the SS1 model was larger than the SS2 for the 500 and 2500 year events, the SS1 model still had a larger reserve in terms of the displacement capacity. For the 500 year return period event, the SS1 model reached 29% of the displacement capacity and the SS2 model reached 62%. Increased values of 90% for the SS1 and 109% for the SS2 model occurred during the 2500 year event.

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