Experimental Evaluation of Seismic Pounding at Seat-Type Abutments of Horizontally Curved Bridges

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SUMMARY:
The expansion gap closure in seat-type bridge abutments during strong earthquakes results in seismic pounding. This pounding significantly affects the behaviour of the bridge and yet there are limited studies focusing on this impact. As a part of the Federal Highway Administration (FHWA) funded project, a 2/5 scale curved bridge model was constructed to be tested on the four shake tables in the University of Nevada, Reno Large Scale Structures Laboratory. One of the six configurations of the bridge model was designed to study the seismic pounding at the abutments with an equivalent nonlinear backfill soil. An abutment configuration was designed for the experiment to investigate the abutment impact accounting for the nonlinearity of the backfill soil. In this configuration, the superstructure was forced to impact a backwall supported by nonlinear springs with initial stiffness similar to that of typical embankment soil. The preliminary experimental results presented in this paper demonstrate that the closure of the expansion gap significantly influences the global response of the bridge system. Experimental measurements of the impact forces is planned to be used to calibrate numerical impact models.

Keywords: Curved Bridges, Large Scale, Experimental Testing

1. INTRODUCTION

Seat-type abutments consist of a footing, stemwalls, seat, and backwall. The superstructure is supported by bearings on the abutment seat. The backwall retains the backfill above the abutment seat so that the backfill of the approach embankment is not in contact with the superstructure. A gap between the abutment backwall and the superstructure provides a stress relief during thermal loadings. This type of abutment is commonly used for long span, highly skewed, or highly curved bridges to avoid large or unbalanced stresses in the superstructure and embankment backfill soil under temperature loads.

However, during significant seismic events the expansion gap is forced to close resulting in seismic pounding between the bridge superstructure and abutment backwall. Large inertia forces generated in the superstructure mobilize the active pressure in the backfill soil behind the abutment backwall which can result in nonlinear soil behaviour. Damage to seat type abutments caused by seismic pounding has occurred in several recent earthquakes. Severe damage to highway bridges in the 1971 San Fernando earthquake occurred due to expansion gaps closing (Jennings, 1971). In the 1994 Northridge earthquake, considerable impact damage was observed at the expansion joints of the I5/SR14 interchange located near the epicenter (EERI, 1995a). During the 1995 Kobe earthquake, seismic pounding was cited as a major cause of bearing support damage and may have contributed to the collapse of bridge superstructures (EERI, 1995b). Significant damage to shear keys, bearings, and anchor bolts in the 1999 Chi Chi earthquake was the result of pounding of the expansion joints (EERI, 2001).

Inspired by the damage to bridge abutments observed during recent earthquakes, several researchers have approached the problem from different angles. Analytical studies by Maragakis (1985) and Maragakis and Jennings (1987) concluded that the damage to skew bridges at the abutments during strong earthquake motions is a direct result of excessive in-plane motion of the bridge deck created by impact of the bridge deck with the abutments. Simple analytical bridge models including impact were developed by Maragakis et al. (1989) to identify important parameters for modeling the response of short bridges including the effects of abutment gap closures. Additional, analytical modeling of seismic pounding has led to the development of two basic approaches to model impact – a) forced-based and b) stereo mechanical. The force-based contact element approach employs a high stiffness linear spring, sometimes combined with a damper, to prevent the overlap of impacting bodies when in contact (Maison, 1990; Jankowski, 1998; Pantelides, 1998). This method is limited by the determination of the exact value of the spring stiffness to be used. In addition, using springs with large stiffness can overestimate the impact forces and lead to numerical convergence issues. The alternative stereo mechanical approach uses momentum balance and the coefficient of restitution to modify velocities of the colliding bodies after impact (Papadrakakis, 1991; Athanassiadou, 1994; Malhotra, 1998). This method cannot be easily implemented using existing commercial software which poses a problem for practicing engineers to account for this phenomenon in the design of bridges.

In addition to investigating the impact phenomenon, several research studies have focused on evaluating an appropriate stiffness and capacity of abutment backfill soil. Experimental research conducted at the University of California, Davis (Maroney, 1995) provided the basis for current Caltrans design/analysis provisions in the Seismic Design Criteria (SDC) (Caltrans, 2010). Large scale static testing of abutment backfill soil behind the abutment backwall was conducted to generate realistic values of backfill soil stiffness and capacity (Maroney, 1995). Additionally, Siddharthan et al. (1997) developed a simple methodology to evaluate the nonlinear translational spring stiffness of bridge abutments.

As a part of a Federal Highway Administration funded project, a complete experimental investigation of a large-scale three-span curved bridge is currently in progress. A part of this study is targeted at observing the effect of seismic pounding and the performance of seat-type abutments during strong ground motions. Results from the experiment will be used to develop and calibrate an appropriate forced-based contact element which can be used by designers. This paper presents the details of the experimental setup designed to evaluate the performance of seat-type abutments.

2. CURVED BRIDGE EXPERIMENTAL MODEL

The two-fifth scale model used in the experimental investigations is a three-span, steel I-girder bridge with high degree of curvature with a subtended angle of 104° (1.8 radians). The overall geometry of the prototype and the model is summarized in Table 2.1. The superstructure comprises of a reinforced concrete deck that is composite with three steel I-girders. The reinforced concrete deck is 83 mm thick with 19 mm haunch. The girders are built-up sections consisting of 16 mm by 229 mm flange plates and 10 mm by 660 mm web plate. The cross-frames between the girders are spaced 1.83 m throughout the length except at the middle of the bridge where two cross-frames are spaced 1.89 m. Figure 2.1 shows the as-built bridge model inside the large scale structures laboratory at the University of Nevada, Reno.

Table 2.1. Overall Geometry of the Curved Bridge

	Prototype	Model
Total length (m)	110.5	44.2
Span lengths (m)	32-46.5-32	12.8-18.6-12.8
Centerline radius (m)	61	24.4
Total width (m)	9.15	3.66
Girder spacing (m)	3.4	1.37
Column height (m)	6.1	2.44
Column diameter (m)	1.52	0.61

(a) View over the Deck (b) View Underneath

Figure 2.1. Bridge Model inside the Laboratory

The bents are single columns with a drop cap. The column diameter in the model is 0.61 m (1.52 m in the prototype) with 1% longitudinal steel ratio. The specified concrete strength was 38 MPa and the steel reinforcement is A706 Gr. 60 steel.

The weight of the model superstructure is 563 kN. A total added mass of 833 kN was added to account for the scale effect. The added mass was comprised of steel and lead plates distributed on the bridge deck and on top of the bent cap. Figure 2.2 shows the bridge model with the added mass distributed on the deck and on the bent cap.

(a) Added Mass on Deck (b) Added Mass on Bent Cap

Figure 2.2. Added Mass on Bridge Deck and on Bent Cap

The curved bridge and its components were designed based on the 2008 AASHTO LRFD Bridge Design Specifications (AASHTO, 2008). The design spectrum was based on a rock site in Reno, Nevada. The peak ground acceleration was 0.472 g, the short-period spectral acceleration is 1.135 g, and the 1-second spectral acceleration was 0.41 g.

The ground motion recorded at Sylmar Station during the 1994 Northridge Earthquake was used as input motion. In the experiment, the ground motion was increased incrementally in terms of the equivalent Design Earthquake (DE). The equivalent DE was determined by scaling the amplitude of the major component of Sylmar record such that its spectral acceleration at 1.0 second is equal to the 1-second spectral acceleration of the Reno design spectrum. This scale factor is equal to 0.475 and will be applied to both the major and minor components of the Sylmar record. Since the bridge model scale was 0.4, the time scale factor applied to the ground motion was 0.632 which is equal to the square root of 0.4.

Although the focus of the study presented in this paper is on the seat abutment performance and abutment pounding, the bridge model is used to experimentally study the seismic performance of this bridge in a variety of different configurations including: i) conventional superstructure, columns, and bearings without live load, ii) conventional system with live load, iii) fully isolated superstructure on conventional columns, iv) partially isolated superstructure with ductile cross-frames on conventional columns (hybrid isolation), v) conventional superstructure and columns with abutment interaction and simulated backfill, and vi) conventional superstructure on rocking columns. The findings of these experimental studies, coupled with analytical investigations, will be used to develop a set of seismic design guidelines for horizontally curved steel bridges.

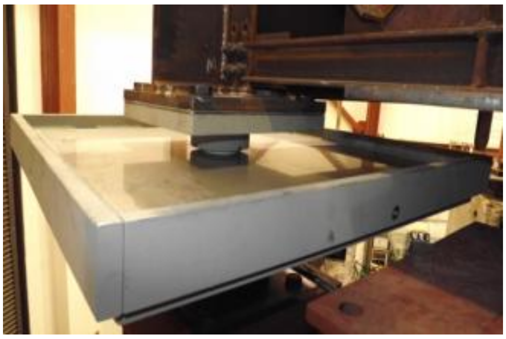
3. DESIGN OF EXPERIMENTAL SEAT-TYPE ABUTMENT SYSTEM

The ends of the bridge model are supported on seat-type abutments. The seat supports, commonly used in bridge construction, are used to accommodate the thermal expansion without engaging the abutment backfill. This, in turn, eliminates the high stresses that would otherwise be present in the superstructure when the bridge is rigidly held at the abutments. However, during large seismic events, there is a possibility that the joint gap between the end of the bridge and the backwall would close and high-acceleration seismic pounding would occur. The following sections describe the experimental setup used to investigate the seismic performance of seat-type abutments in horizontally curved bridges.

3.1. Superstructure Support

The bridge boundary conditions at supports are:

- At Bents – only shear transfer between the superstructure and the bent cap using the pin bearings shown in Fig. 3.1b.
- At Abutments – free translation in the tangential direction and restrained in the radial direction at earthquake levels up to 75% design earthquake (DE) using a combination of slider bearing and shear pin. At earthquake levels beyond 75% DE, the abutments are free in both horizontal directions since the shear pin was designed to fail at this level.



(a) Slider Bearing @ Abutments

(b) Pin Bearing @ Piers

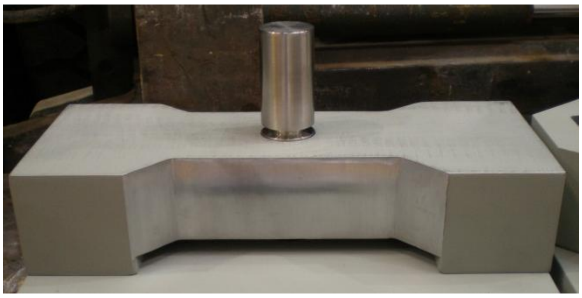
Figure 3.1. Types of Bridge Bearings at Support Locations

The bearings located at the abutments and pier support locations are shown in Fig. 3.1. The slider bearings at the abutments consist of a sliding pot bearing with a Teflon surface which is attached to the underside of the girder, and a stainless steel sliding plate on which the slider is free to move with a low coefficient of friction. The slider bearing can accommodate a displacement range of 914 mm (± 457 mm) in tangential and radial directions. Moreover, there is no vertical restraint at the ends of the bridge.

The pin bearing used at the piers was able to accommodate 0.075 radians of rotation freedom about its horizontal axes and was free to rotate about its vertical axis. The bearing's compression load capacity was 445 kN, the lateral load capacity was 222 kN, and the tensile capacity was 178 kN.

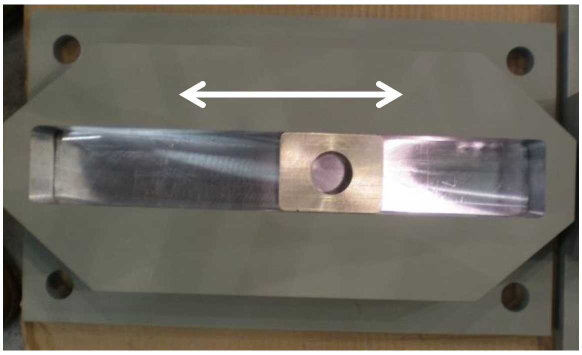
3.2. Shear Keys

Caltrans' design philosophy is such that abutment shear key provide transverse restraint for wind and low seismic loads but fail under strong ground motions to prevent damage to the abutment foundation and pile system. With this in mind, a set of sacrificial shear keys were designed to fail during the 75% DE level. This design facilitates the experimental assessment of the influence of a sudden change in boundary conditions during the response of the bridge.

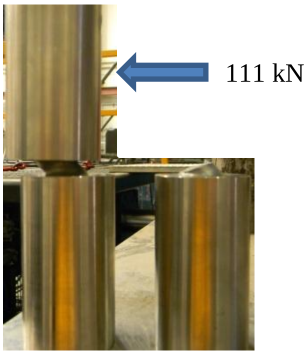


(a) "Dog-Bone" and Shear Pin

(c) Shear Key Assembly



(b) Trough and Brass Sliding Sleeve



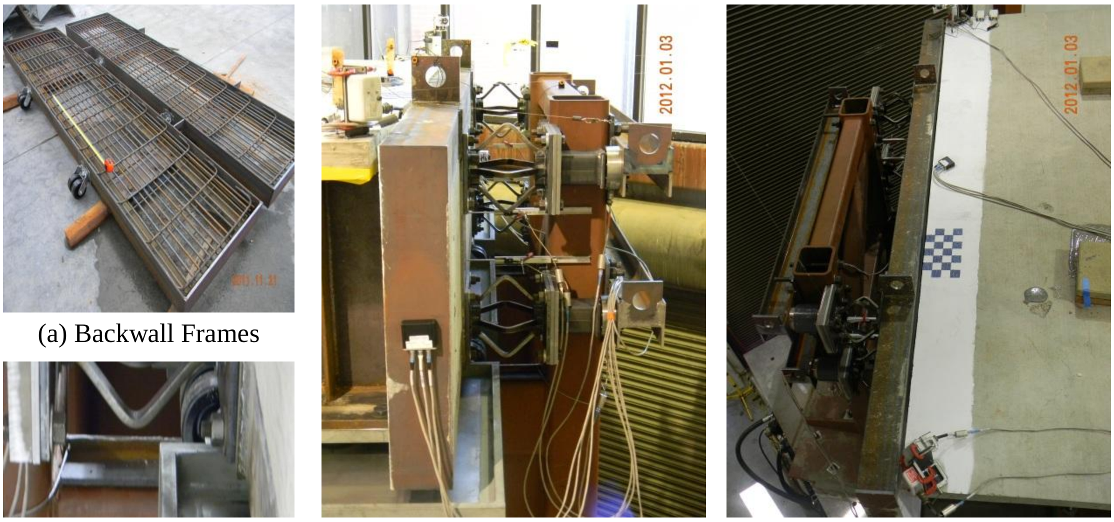
(d) Deformed Shear Pins

Figure 3.2. Shear Key Assembly

Figure 3.2 shows the shear key attached to the bottom chord of the abutment cross-frames. The shear key assembly is comprised of 4 components: 1) the shear pin, 2) the sleeve, 3) the "dog-bone", and 4) the trough. The top component that is attached to the cross-frame bottom chord consists of the "dog-bone" and stainless steel shear pin. Half of the shear pin is inserted into a hole at the center of the "dog-bone", as shown in Fig. 3.2a. The other half of the shear pin is inserted into the brass sleeve which is allowed to slide freely within the trough oriented along the tangential degree of freedom and attached to the abutment structure (Fig. 3.2b). The final shear key assembly is shown Fig. 3.2c. The pin was designed to fail in shear (Fig. 3.2d) at a transverse force equal to 111 kN, which is approximately equivalent to100% of the dead load carried at the abutment. This was achieved by making a small groove in the pin to reduce its area and ensure it fails in shear.

3.3. Backwall

The abutment backwall was designed to provide a realistic contact surface to enable simulating the impact between the bridge abutment and the superstructure. The backwall is reinforced concrete within a 3.66 m x 0.84 m x 0.15 m steel frame (Fig. 3.3a). It spans the width of the deck and weighs 17.8 kN. The concrete strength of the backwall was 40 MPa. The backwall is supported on two casters which travel along a rail oriented in the tangential direction of the bridge. The rails restrain movement of the backwall in the radial direction (Fig. 3.3b). Four nonlinear soil springs are employed in parallel to represent the passive resistance of the soil against tangential movement of the backwall. The initial gap between the end of the superstructure and the backwall shown in Fig. 3.3d was 19 mm. It was determined from temperature load analysis of the prototype and was scaled down to the model. Using a reinforced concrete wall facilitates the development an experimental calibration of the damping and energy loss during the impact. Also, any local damage of the backwall, i.e. crushing or cracking of the concrete, would be observed.



(a) Backwall Frames

(b) "T-Rail" Radial Restraint

(c) Radial View of North Abutment Backwall

(d) Above View of South Abutment Gap

Figure 3.3. Abutment Backwall Assembly

3.4. Embankment Soil

Modelling the nonlinear soil behavior behind the backwall of a seat-type abutment is an important part of this investigation. Since soil itself was not an option due weight and space restraints on the shake tables, a set of nonlinear compression-only springs, shown in Fig. 3.4a, were developed to simulate the passive resistance of the soil backfill. These devices are made of pre-bent steel plates which are forced to yield into a buckled shape, as shown in Fig. 3.4b. Extensive analytical modeling and prototype testing of the devices was completed to accurately obtain their desired performance. Finally, each soil spring consisted of four 20 mm wide by 13 mm thick legs pre bent to a 90° angle. The height of the device was 254 mm. Four of these soil springs would work in parallel to provide the total passive soil resistance to each backwall.

The targeted soil properties of stiffness and capacity were calculated according to the provisions of Caltrans SDC (Caltrans, 2010). Using the set of Caltrans equations below, an appropriate stiffness was calculated.

$$K_i \approx \frac{28.7kN/mm}{m} \tag{3.1}$$

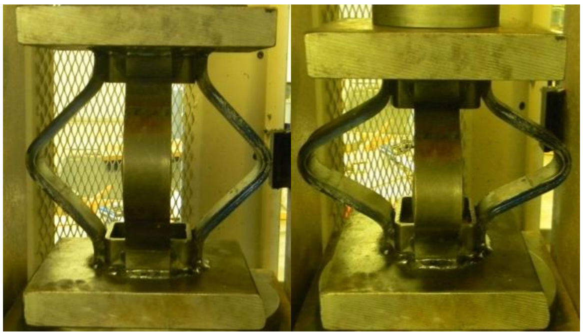
$$K_{abut} = K_i \times w \times \left(\frac{h}{1.7m}\right) \tag{3.2}$$

$$P_{bw} = A_e \times 239kPa \times \left(\frac{h}{1.7}\right) \tag{3.3}$$

In which, K_i is the initial stiffness for the embankment fill material meeting the requirements of Caltrans Standard Specifications. For fill not meeting these requirements the value of K_i may be taken as 14.35 kN/mm/m. K_{abut} is the stiffness of an abutment with effective width, w, and height, h. Based on an assumed elastic-perfectly-plastic force displacement relationship the passive pressure force is

P_{bw}, which is calculated based on the effective area of the backwall, A_e, the maximum passive pressure obtained from full scale abutment testing, 239kPa, and the height proportionality factor.

Based on these equations the recommended stiffness was determined to be 108.2 kN/mm and the ultimate passive resistance is 832 kN. However, it was determined that these properties would introduce a very large demand on the shake tables. To avoid compromising the shake tables, the largest allowable capacity of the soil springs was set to 178 kN which is 20% of the target capacity. Also, in sizing the soil spring device for this capacity, the stiffness that can be achieved is 42.0 kN/mm. Although this is 40% of the target stiffness, it is still within the allowable range of soil stiffness recommended by Caltrans for embankment fills not meeting Caltrans Standard Specifications. Thus, the experimental results represent a weak soil condition behind the backwall but the kinematics of the backwall pounding was still properly modelled.



(a) Undeformed soil spring

(b) Deformed soil spring

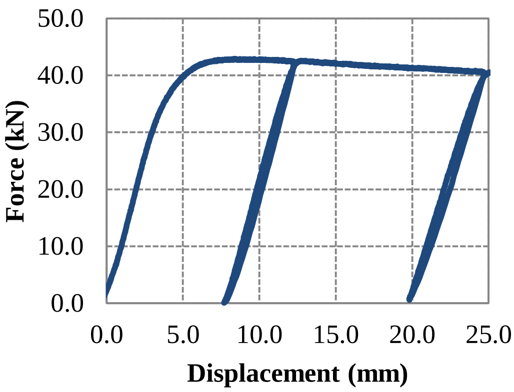
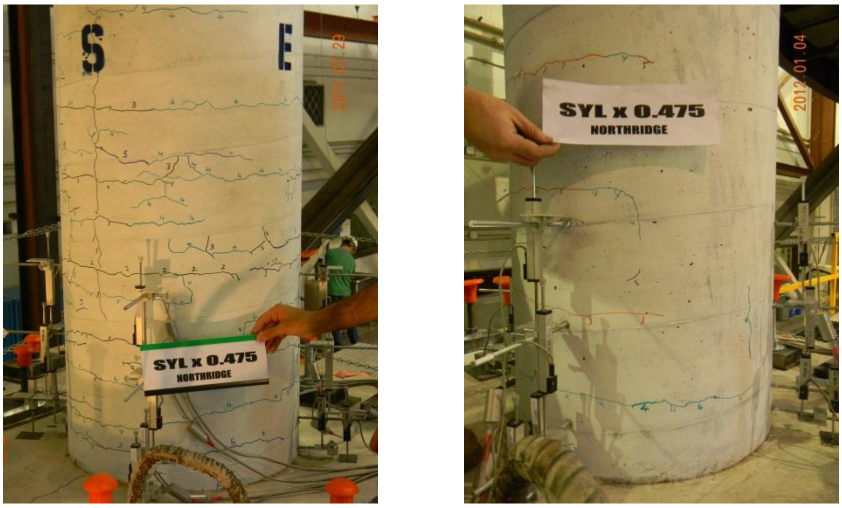


Figure 3.4. Soil spring component tests

4. PRELIMINARY EXPERIMENTAL RESULTS

The response of the conventional configuration was used as a benchmark and compared to the response of the backwall configured bridge. Several response parameters are compared including the displacement demand on the columns, column shear demands, and displacement demand at the abutments. In addition to the comparative results, response parameters unique to the backwall configuration such as backwall hysteresis and expansion gap closure are also investigated.

A visual comparison of the cracking observed in the south column of the two configurations, Fig. 4.1, shows that the conventional configuration had significantly more cracking after the design level earthquake shaking table run. This corresponds with the reduction in column displacement and shear demands in the abutment pounding configuration. This is illustrated by the shear demand acceleration relationships presented in Fig. 4.2. The deck displacements at the abutments were also reduced in the abutment pounding configuration, Fig. 4.3. It should be noted that the shear keys failed during the 75% DE run and the 100% DE run for the conventional configuration and abutment pounding configuration, respectively, which is apparent in Fig. 4.3b. The backwall system performed as desired and was successful in capturing the nonlinear behaviour of the backfill. Figure 4.4 shows the change in the initial expansion gap prior to each level of shaking as well as the backwall hysteresis, which is the sum of all four parallel soil springs. One of the unique observations obtained from these experiments was the unexpectedly large radial force imparted on the backwall by the superstructure while the gap was closed in the 150% DE run after the shear keys had failed. This force was generated when the girders dug into the backwall concrete (Fig. 4.5a) and the inertia forces of the bridge were acting radially. This unexpected level of force caused failure of the caster system of the south abutment backwall during the 150% DE run (Fig 4.5b).



(a) Conventional (b) Pounding

Figure 4.1. Observed column damage comparison after 100% DE

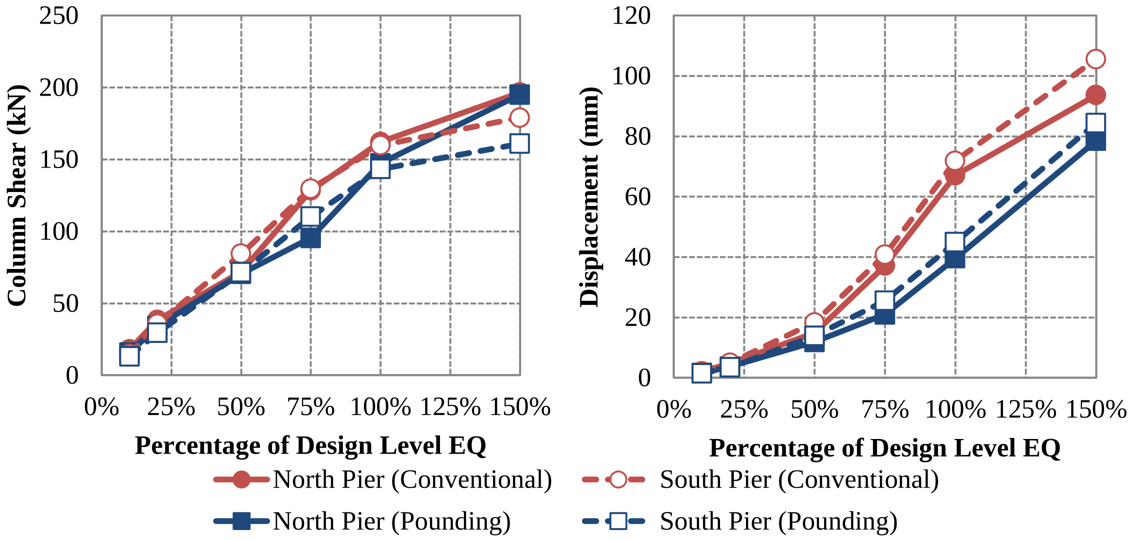


Figure 4.2. Column shear and displacement demands

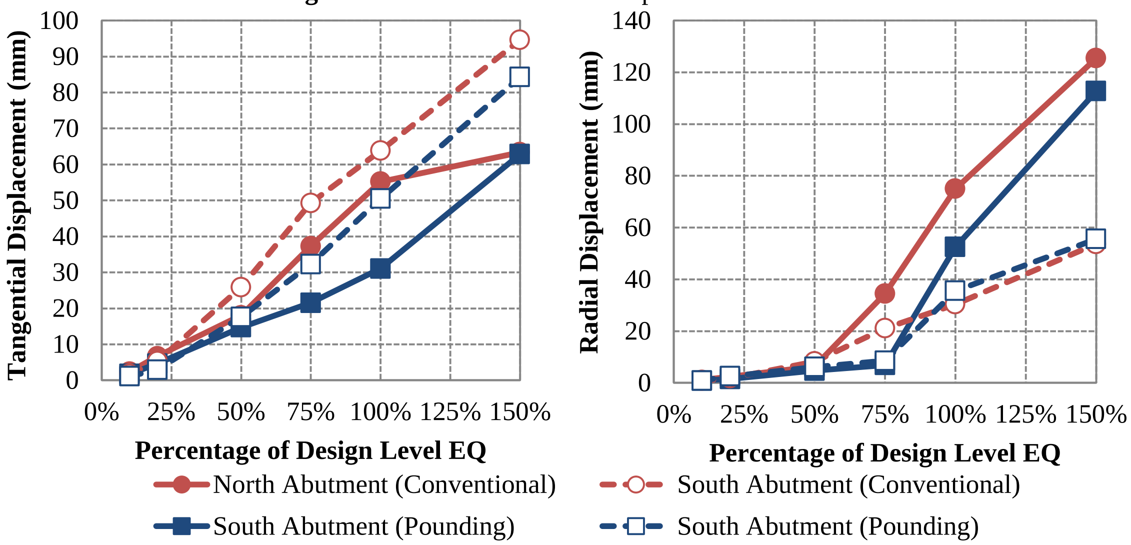
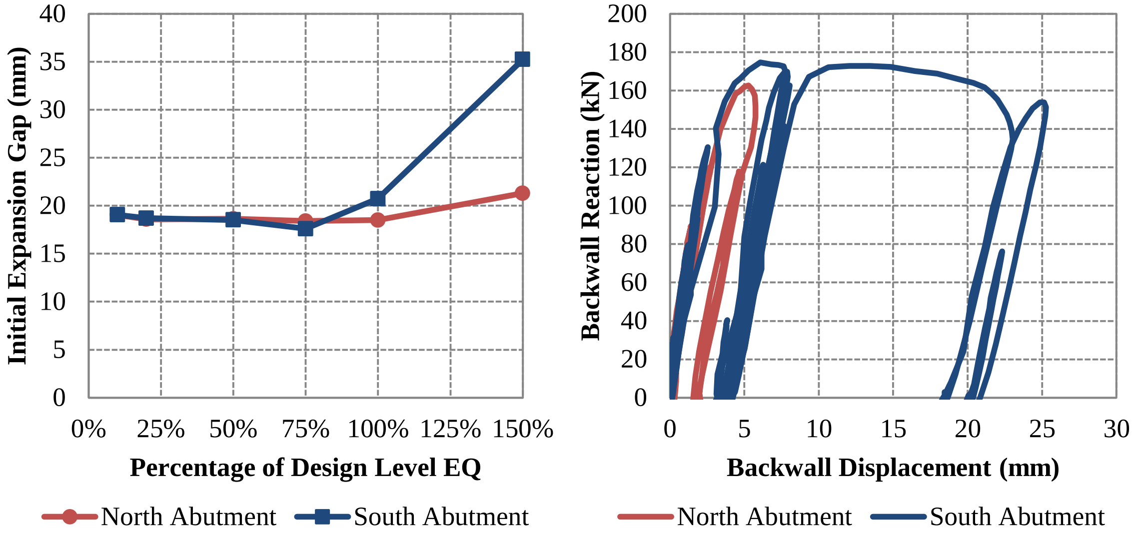


Figure 4.3. Single soil spring achieved force-displacement relationship



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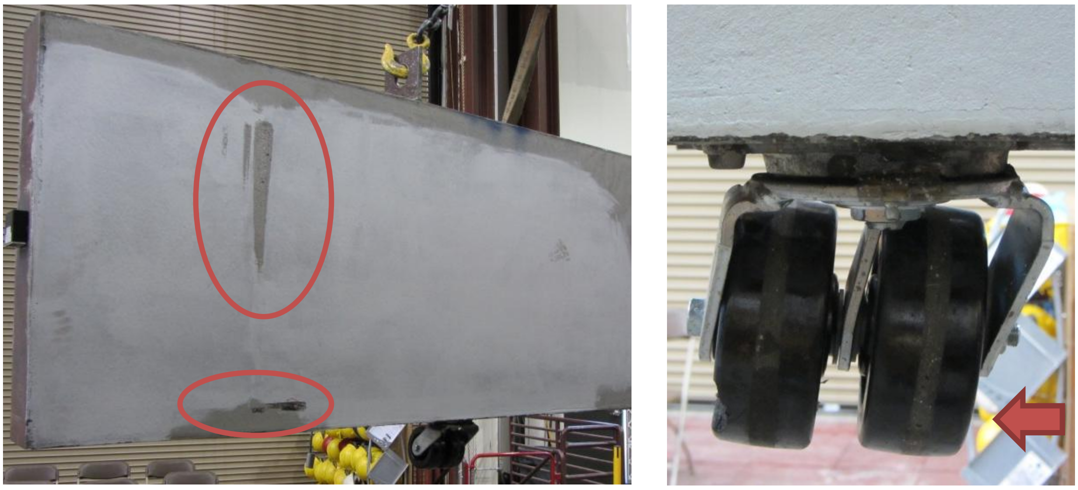

(a) Damage due to Girder Embedment into Backwall

(b) Damage to Caster

Figure 4.5. Observed Damage to South Abutment Backwall

5. SUMMARY AND CONCLUSIONS

The experimental set up for the evaluation of the seismic performance of seat-type abutments in a highly curved bridge was presented. The abutment setup includes slider bearings which allow uplift at the ends of the bridge, a shear key that is active only in the radial direction and was designed to fail during an earthquake equivalent to 75% of the design earthquake, and a backwall system with a set of nonlinear soil springs representing the soil passive resistance. This setup was used to experimentally study the seismic performance of seat-type abutments, abutment backwall pounding, and its effect on bridge response. A preliminary assessment of the experimental results suggests that engagement of the abutment backwalls during seismic events reduces the demands on the columns. The demand on the backwall system was sufficient enough to cause significant yielding of the backfill soil springs. The results from this experiment are currently being investigated in detail. The experiment results will be utilized to calibrate analytical models which, in turn, can be used by bridge designers to assess and design bridges with seat-type supports. This study is one of the six experimental investigations being conducted at University of Nevada, Reno which will be used to produce a set of seismic design guidelines for horizontally curved bridges.

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