BLAST RESISTANCE OF SEISMICALLY DESIGNED BRIDGE PIERS

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ABSTRACT :

The issue of protecting infrastructure against multiple extreme events is gaining interest in civil engineering. The authors previously presented the development and experimental validation of a multi-hazard bridge pier concept, i.e., a bridge pier system capable of providing an adequate level of protection against collapse under both seismic and blast loading. The proposed concept was a multi-column pier-bent with concrete-filled steel tube (CFST) columns. The columns turned out to be effective for blast loadings because breaching and spalling of concrete are prevented to occur in the CFST columns. While CFST columns perform excellently in a multi-hazard perspective, they have not been commonly used in bridge engineering practice. Questions arose as to whether conventional columns designed to perform satisfactory under seismic excitations would possess adequate blast resistance. Two commonly used systems to provide ductile performance of reinforced concrete (RC) columns in seismic regions were considered in this project; first, RC columns with closely spaced stirrups in compliance with latest seismic provisions, and second, non-ductile reinforced concrete columns retrofitted with steel jackets. This paper presents the findings of research to examine seismically resistant bridge piers that are designed according to current seismic knowledge and that are currently applied in typical highway bridge designs. A series of experiments were performed on 1/4 scale typical seismically detailed ductile RC columns and non-ductile RC columns retrofitted with steel jacketing. The standard RC and steel jacketed RC columns, known to exhibit satisfactory seismic behavior, failed in shear at the base of the column under blast loading.

KEYWORDS: Blast Resistance, Reinforced Concrete, Steel Jackets, Bridge Piers

1. OBJECTIVE

The issue of protecting infrastructure against multiple extreme events is gaining interest in civil engineering. This research focuses on the protection of highway bridges against two hazards, namely earthquakes and blasts. A similarity between seismic and blast events in relation to bridges is that they can both induce large inelastic deformations in key structural components. Since many bridges are (or will be) located in areas of moderate to high seismic activity, and because any bridge can be a potential target for terrorists, there is a need to develop structural systems capable of performing equally well under both events.

The authors previously presented the development and experimental validation of a multi-hazard bridge pier concept, i.e., a bridge pier system capable of providing an adequate level of protection against collapse under both seismic and blast loading (Fujikura et al. 2007, 2008). The proposed concept was a multi-column pier-bent with concrete-filled steel tube (CFST) columns. The columns turned out to be effective for blast loadings because breaching and spalling of concrete are prevented to occur in the CFST columns.

While CFST columns perform excellently in a multi-hazard perspective, they have not been commonly used in bridge engineering practice. Questions arose as to whether conventional columns designed to perform satisfactory under seismic excitations would possess adequate blast resistance. Therefore, the objective of this research is to investigate the blast resistance of the commonly used bridge columns, namely seismically ductile RC columns and non-ductile RC columns retrofitted with steel jackets to make these ductile, detailed in accordance to recent code of practice.

2. SEISMIC DESIGN OF PROTOTYPE BRIDGE PIERS

The prototype bridge pier chosen in this study is part of a typical 3-span continuous highway bridge described in Dicleli and Bruneau (1996). The span lengths are 35 m, 25 m and 30 m (total length = 90 m). The total gravity load on each pier is assumed equal to 4098 kN. The RC columns with diameter of 813 mm (32 in) and height of 6 m (19' 8") were reinforced with 16-#6 (19 mm nominal diameter) longitudinal bars, resulting in longitudinal reinforcement ratio, ρ_l, of 0.9 %. The transverse reinforcement of 1.0 % and 0.5 % was provided for plastic hinge regions and outside of plastic hinge regions, respectively, designed by current codes of CALTRANS (2003) and MCEER/ATC-49 (2003).

The bridge was assumed to be located in an area of moderate seismic activity. For analysis and design purposes, it is assumed that the corresponding pseudo-acceleration (S_A) response spectrum is given by:

$$S_A = \min\left\{ \left(1 + 18.75\,T\right) A_g \;,\; 2.50\,A_g \;,\; \frac{A_g}{T} \right\} \tag{2.1}$$

where A_g (peak ground acceleration) is assumed equal to 0.3 g, and T denotes natural period. The spectral shape of the response spectrum defined by Equation 2.1 is typical of rock or very stiff soil foundations. Calculated expected ductility demands in the longitudinal and transverse direction were 2.22 and 3.39, respectively.

The steel jacketed RC columns were reinforced with the same amount of longitudinal bars as RC columns and with transverse reinforcement of 0.35 % calculated to resist the developed column hinges form at the top and bottom of the column. Thickness of steel jacket was designed such that the steel jacket provided confinement stress of 2.07 MPa (300 psi), which is the value used in design methods developed by Chai et al. (1991). The resulting thickness of steel jacket was 4.3 mm (3/16").

3. EXPERIMENTS ON 1/4 SCALE MULTI-HAZARD BRIDGE PIERS

A series of tests was performed at U.S. Army Corps of Engineers Research Facility in Vicksburg, Mississippi. Due to constraints in the maximum possible blast charge weight that could be used at the test site, test specimen dimensions were set to be 1/4 scale of the prototype bridge piers. Details of experimental specimen are shown in Figure 1. The specimen consisted of two RC columns (RC1 and RC2) and two steel jacketed RC columns (SJ1 and SJ2), connected to a cap-beam and a footing.

The experimental specimen and setup is shown in Figure 2 and Figure 3, respectively. Those columns in the bent were subjected to successive blast tests. Note that the cap-beam was not fixed to the reaction frames as it was intended to allow rotating to replicate actual conditions in bridges. Summary of the pier tests are presented in Table 1 along with experimental observations. Exact values of charge weights and stand off distances were omitted for security reasons. Instead, these values were expressed by W and X respectively in Table 1. The maximum blast charge was limited to W due to the constraints at the test site. The standoff distances of Test 1 and Test 2 for RC columns were respectively determined to induce 4 degree and 2 degree of rotational angle at the bottom of the column. These calculations were done by simplified analysis using an equivalent SDOF system and energy conservation (see Fujikura et al. 2007, 2008 for details). The standoff distances of Test 3 and Test 4 for steel jacketed RC columns were same as the ones of Test 1 and Test 2, respectively.

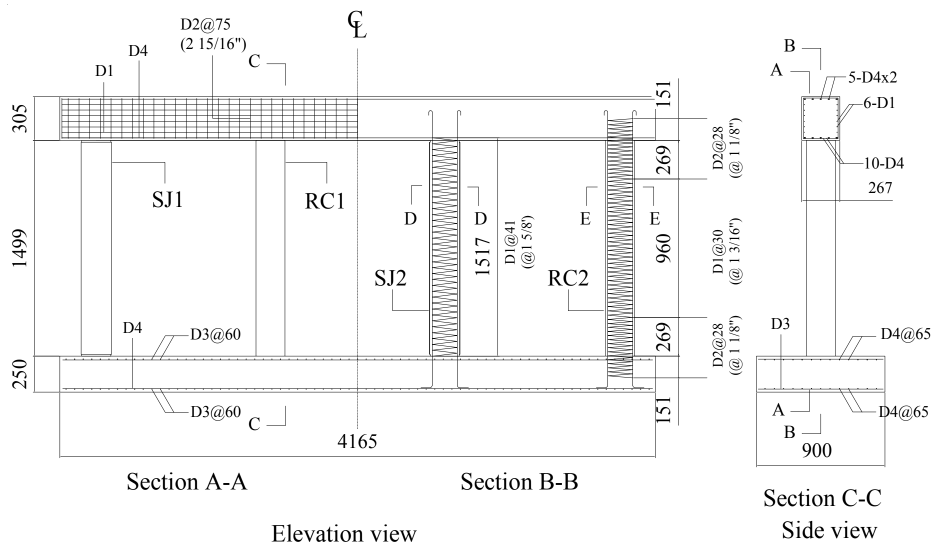

Figure 1 Details of Experimental Specimen

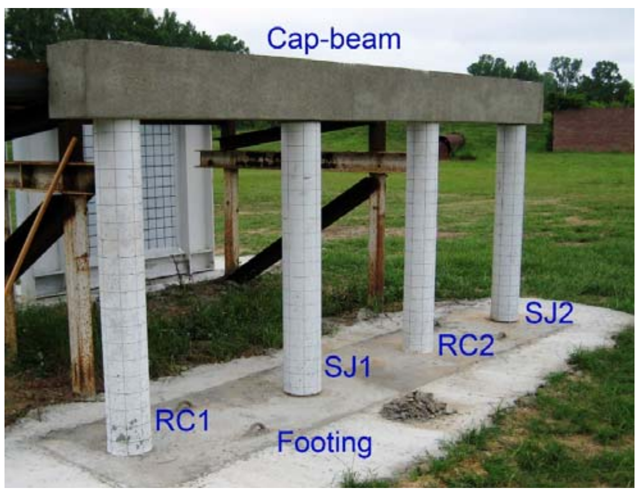

Figure 2 Experimental Specimen

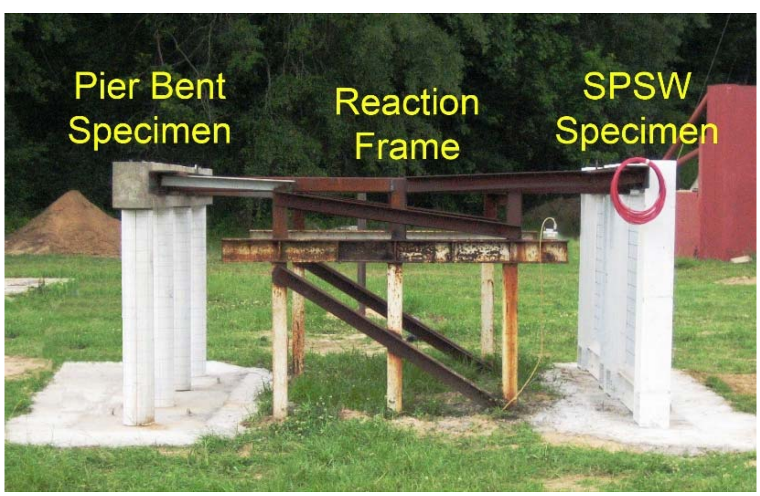

Figure 3 Experimental Setup

Table 1 Summary of Test Cases and Results

Test Num.	Column	Charge Weight	Standoff Distance	Charge Height (m)	Objective	Observations
Test 1	RC1	W	2.16 X	0.25	θ = 4 deg Collapse	Shear failure at bottom
Test 2	RC2	W	3.25 X	0.25	θ = 2 deg Minor Damage	Onset of shear failure at bottom
Test 3	SJ2	W	2.16 X	0.25	Same as Test 1	Shear failure at bottom
Test 4	SJ1	W	3.25 X	0.25	Same as Test 2	Shear failure at bottom

Figures 4 through 6 show, respectively, Column RC1, RC2 and SJ2 after the test. As shown in Figures 4 and 6, respectively, Column RC1 and SJ2 exhibited shear failures, often called direct shear failures, at the bottom of the columns and all longitudinal bars were fractured. The direct shear failure at the bottom of the column was also observed in Column SJ1 as similar as in Column SJ2. Spalling of the concrete was observed at the bottom of Column RC2 as shown in Figure 5. Figure 5 shows RC2 at the onset of the shear failure. The standard RC and steel jacketed RC columns were found not to exhibit a ductile behavior under blast loading, but failed in shear at the base of the column while the CFST columns were effective for blast loadings from previous experiments. The steel jacketed column visually resembles the CFST column, however there were gaps at the column top and base intended to prevent composite actions. This discontinuity of shear resistance at the boundary resulted in the direct shear.

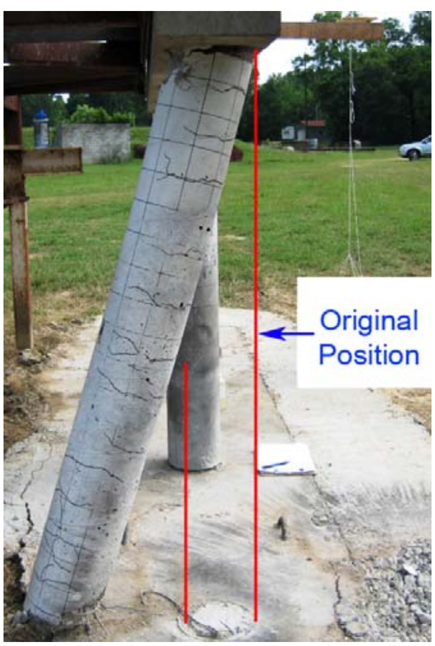

Figure 4 Column RC1 after test

Figure 5 Column RC2 after test

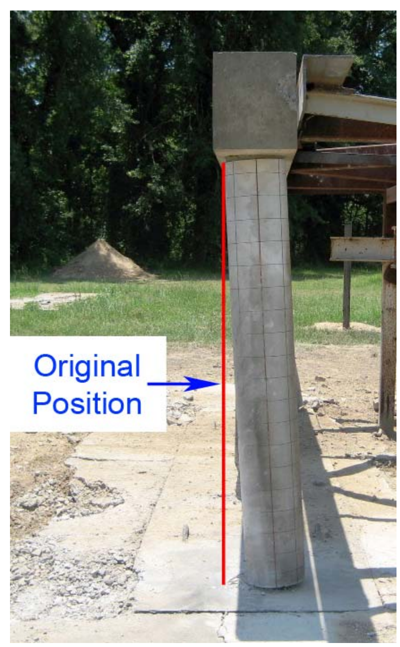

Figure 6 Column SJ2 after test

4. CONCLUSIONS

This paper has presented the findings of research to examine the blast resistance of bridge piers that are designed according to current seismic knowledge and that are currently applied in typical highway bridge designs. A series of experiments were performed on 1/4 scale typical RC columns and RC columns retrofitted with steel jacketing. The standard RC and steel jacketed RC columns were not found to exhibit a ductile behavior under blast loading, failing in shear at the base of the column rather than by flexural yielding as was the case with CFST columns. Reinforced concrete details by current seismic codes and steel jacketing, known to be effective to provide satisfactory seismic performance, were thus shown to be ineffective for the blast loading cases considered.

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