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MULTI-HAZARD (BLAST AND SEISMIC) RESISTANCE OF VARIOUS TYPES OF BRIDGE COLUMNS

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Abstract. *Various types of bridge columns known (or expected) to have satisfactory ductile seismic performance have been tested to investigate their blast resistance while also meeting multi-hazard demands. A multi-hazard engineering perspective is defined here as the search for a single design concept which can satisfactorily fulfill the possibly conflicting demands of multiple hazards. Concrete-Filled Steel Tube (CFST) columns, Concrete-Filled Double Skin Tube (CFDST) column, and Modified Steel Jacketed Column (MSJC) designs were proposed and shown to develop ductile flexural blast response. In contrast, comparable reinforced concrete columns and conventionally jacketed columns failed in a brittle manner in direct shear. CFDST were also subjected to cyclic inelastic testing and exhibited a ductile response, suggesting an adequate seismic performance.*

INTRODUCTION

There is a growing concern that bridges piers must resist blast forces as a result of terrorism. In the USA, this concern is justified as threats have been received targeting bridges across the nation. In this context, highway bridges seem more accessible and vulnerable than landmark bridges which are closely monitored. In many instances, the destruction of a highway bridge can have profound effects on the economic circuit those infrastructures support.

At the same time, bridges are exposed to other hazards beyond terrorist focused issues alone. In particular, seismic resistance of bridge piers is a major consideration in earthquake-prone regions. To better integrate these diverse threats with conflicting demands on structural systems, there has been a growing trend in the engineering community to find integrated solutions for the design of infrastructures against various hazards, namely multi-hazard engineering. Multi-hazard engineering is here defined as the search for a single design concept which can satisfactorily fulfill the demands of multiple hazards. The properties that might be desirable to resist one hazard may have detrimental effects to resist other hazards. Therefore, multi-hazard engineering addresses problems of infrastructures from the system perspective by establishing optimized solutions that can provide protections against multiple hazards (e.g. Agrawal et al. 2009).

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Towards that goal, the blast and seismic behavior of a series of different bridge column designs was numerically and experimentally investigated. A summary of these experimental findings is presented in this paper. Analytical studies are presented elsewhere (Fujikura and Bruneau 2008, 2011; Fouché and Bruneau 2014).

Note that, in all cases, the assumed blast scenario considered here is the detonation of explosives located inside a small vehicle below the bridge deck at close distance to the column (as illustrated in Figure 1), but for the ductile column designs proposed by the authors (that behaved satisfactorily for the scenario threat), intensity of blast forces was increased beyond that scenario for the purpose of investigating the ultimate failure modes of the columns. Also, in all cases, the bents were laterally supported by a reaction frame that also served to simulate the boundary condition and rigidity at the top of the bent beam that the deck of a full-scale bridge would have provided (Figure 2).

All blast tests were performed at the U.S. Army Corps of Engineers Research and Development Center's Facility in Vicksburg, Mississippi. Due to constraints with the maximum possible blast charge weight that could be used at the test site and specimen cost considerations, test specimen dimensions were set to be 1/4 scale of the prototype bridge piers.

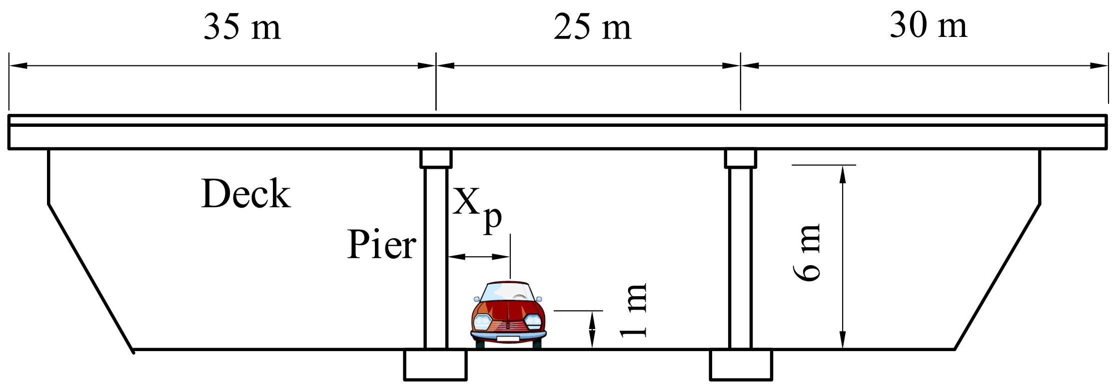


Figure 1: Schematics of prototype bridge and assumed blast scenario

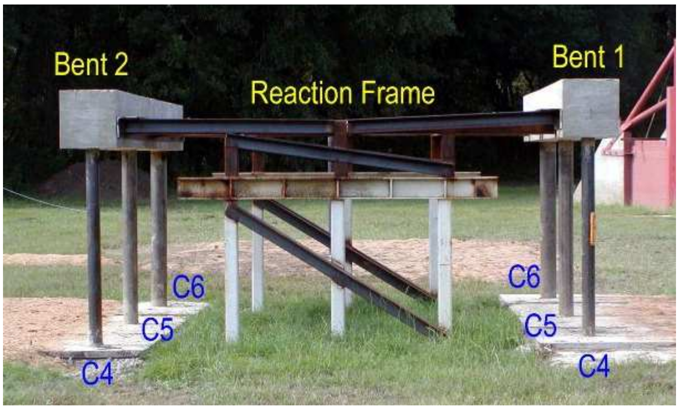


Figure 2: Test setup

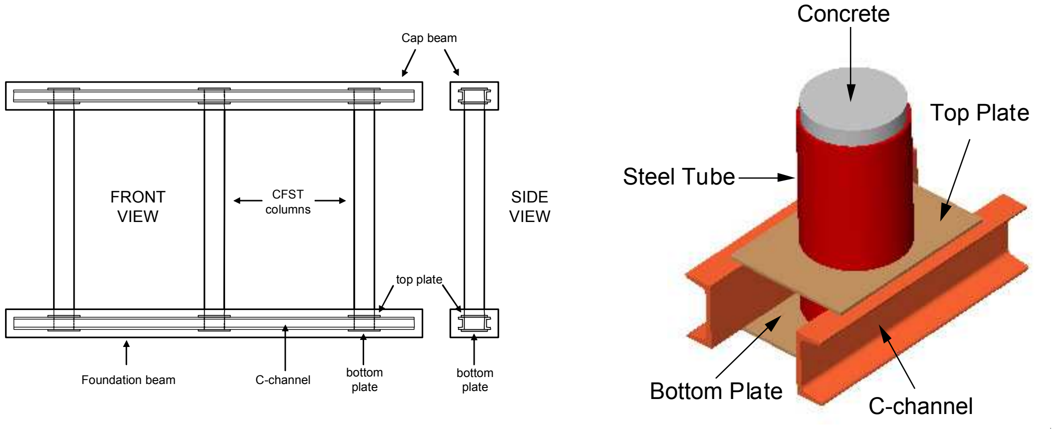
CONCRETE-FILLED STEEL TUBE (CFST) COLUMNS

Preliminary analyses and existing literature (e.g., Winget et al. 2005) indicate that the combined effects of spalling and cratering, and possibly breaching, control the design of substructure reinforced concrete members subjected to close-in blast loading. In this context, spalling and cratering occurs when the concrete disengages at the back and front side of the member, respectively, and breaching is punching through the full concrete thickness (UFC 2004). The performance of concrete members under blast loading could be substantially improved if these behaviors could be somehow prevented. Note that alternative all steel solutions have other challenges (e.g., Krishnappa et al 2014, Warn and Bruneau 2009) and have not been considered in this specific application.

In that perspective, encasing concrete in a steel shell would seem to be an adequate approach to provide blast-resistant piers. The addition of steel jackets has been shown to be a viable strategy for the seismic retrofit of concrete bridge pier columns (Priestley et al. 1996). However, here, using such a jacket alone was calculated to be insufficient to provide adequate resistance to the large (direct) shear forces that develop at the base of piers subjected to blast loads. As such, using a fully composite concrete-filled steel tube extending continuously into the footing was found to be a more appropriate solution. Therefore, the proposed multi-hazard resistant pier concept (Fujikura et al. 2007, 2008) is a multi-column pier-bent with concrete-filled steel tube (CFST) columns. Tests carried out by Marson and Bruneau (2004) for the purpose of seismic applications showed that CFST columns subjected to cyclic loading exhibit good energy-dissipation capabilities and stable hysteretic behavior up to a drift level equal to 7 %.

A possible implementation of this concept is schematically shown in Figure 3a. The foundation beam consists of concrete-embedded C-channels linked to the columns through steel plates. This connection concept is schematically illustrated in Figure 3b. This type of foundation beam performed successfully in the tests by Marson and Bruneau (2004) in that it allowed the composite columns to develop their full moment capacity. Conceptually, the channels are designed to resist the full composite strength of the columns, and the concrete at the foundation beam does not need any reinforcement for strength purposes (fiber concrete is however recommended to prevent cracking of the concrete and subsequent water infiltration into the footing).

The steel for all circular columns tested here (namely HSS 4.0x0.125, HSS 5.0x0.125, and HSS 6.0x0.125 shapes) was specified to be ASTM A500 Grade B. Coupons were cut out from the specimens after the blast tests. Since the columns were partially damaged due to the tests, coupons were cut off from sides of the columns that were subjected to less strain (and presumably remained elastic). The measured yield strengths of the steel tubes were 357 MPa, 254 MPa, 419 MPa and the measured Young's moduli were 188,041 MPa, 178,793 MPa, 196,179 MPa for columns of 4", 5", and 6" diameter, respectively. The compressive strength for the concrete used in the CFST was obtained from compression tests of concrete cylinders. Average 28-day unconfined compressive concrete cylinder strengths were 42.0 MPa and 30.0MPa for column and cap-beam, and footing, respectively. Concrete compressive strength of the circular columns on the day of blast load testing was predicted by the relationships proposed by ACI Committee 211 (1992) since cylinder tests were not conducted on the test day. The predicted compressive strengths on the test day ranged from 43.2 MPa to 43.5 MPa.



(a) (b)

Figure 3: Multi-column pier-bent made up of CFST columns: (a) General description; (b) Details of column-to-foundation beam connection

Figure 4a shows a typical column for a charge located at mid-height. Many pits and a notch were observed on the surface of the column around the height of the blast charge, as seen in Figure 4b. These marks can be attributed to debris impacts, particularly to the disk attached at the mid-height of the blast charge container as it hit the column during the explosion, but these minor local effects were of no impact on the results. No spalling of the concrete was observed at the cap-beam and foundation-beam as a result of the blast pressures. Inspection of core concrete after removal of half of the steel shell (Figure 4c) showed that cracks occurred at column mid-height on the tension side. In addition, some cracks developed at both the top and bottom of the column on the tension side of the negative moment region due to the rigid boundary conditions. It should be added that although the cap-beams were not fixed to the reaction frames, the rotation of the cap-beam was partly restrained by the torsion resistance of the cap-beam and the other two columns in the pier-bent.

(a) (b) (c)

Figure 4: Column subjected to charge at mid-height: (a) column deformation; (b) column surface around mid-height; (c) core concrete

Standoff distance and charge weight for some tests were arbitrarily modified in an attempt to achieve the onset of fracture of the steel tube. Values were chosen based on the previous experimental results and calibration of simple analysis models with the observed maximum rotations of the columns (Fujikura et al. 2007 and 2008). Figure 5a shows such a column after its test. Buckling of the steel tube was observed near the height where maximum deformation occurred as seen in Figure 5c. The fracture of the steel was observed halfway around the base of the column as shown in Figure 5b. The crater into the foundation reached the embedded C-channel connection.

Overall, the CFST bridge columns exhibited a satisfactory ductile behavior under blast loading. The foundation connection concept applied in this experiment allowed to develop the composite strength of CFST column under blast loading. Comparison of the results from the blast tests with the results predicted by various analysis methods are provided elsewhere (Fujikura et al. 2007, 2008; Fujikura and Bruneau 2012).

By sequencing the tests results as a function of increasing charge, the progress of damage along a typical column is presented in Figure 6 for the blast charge located at low height. Although results presented in these figures are for columns having different diameters, they provide useful information on how the deformation of a column relates to the extent of damage. Results are presented corresponding to these damage states, namely; (1) plastic deformation, (2) on-set of fracture of the column and (3) post-fracture of the column. In each case, column deformations are shown along with the rotation at supports and maximum deformation, and the crack patterns of core concrete are sketched based on the observation of the core concrete performed after the test. Figure 6(3) shows the deformations obtained at the onset of fracture. For that case, this limit state was observed to develop at a plastic rotation angle of approximately 0.297 rad (17.0 deg) at the bottom support. It can be speculated that the plastic rotation angle for that limit state would have been similar for the other columns. Figure 6(4) shows the case for which the steel tube fractured fully. In this case, the complete fracture first occurred at the bottom end of the column. After it fractured under the applied pressures, the column behaved as a cantilever suspended from the top. Therefore, it developed the curvature in the direction reversed to what was observed for the other columns. Then, it eventually fractured at the top as this column was projected outside of its setup under the blast forces. One could approximate the plastic rotation that occurred when the top ruptured, to be 0.327 rad (18.7 degree).

(a) (b) (c)

Figure 5: Specimen after extreme blast (charge at credible threat height): (a) column deformation (b) foundation (c) buckling.

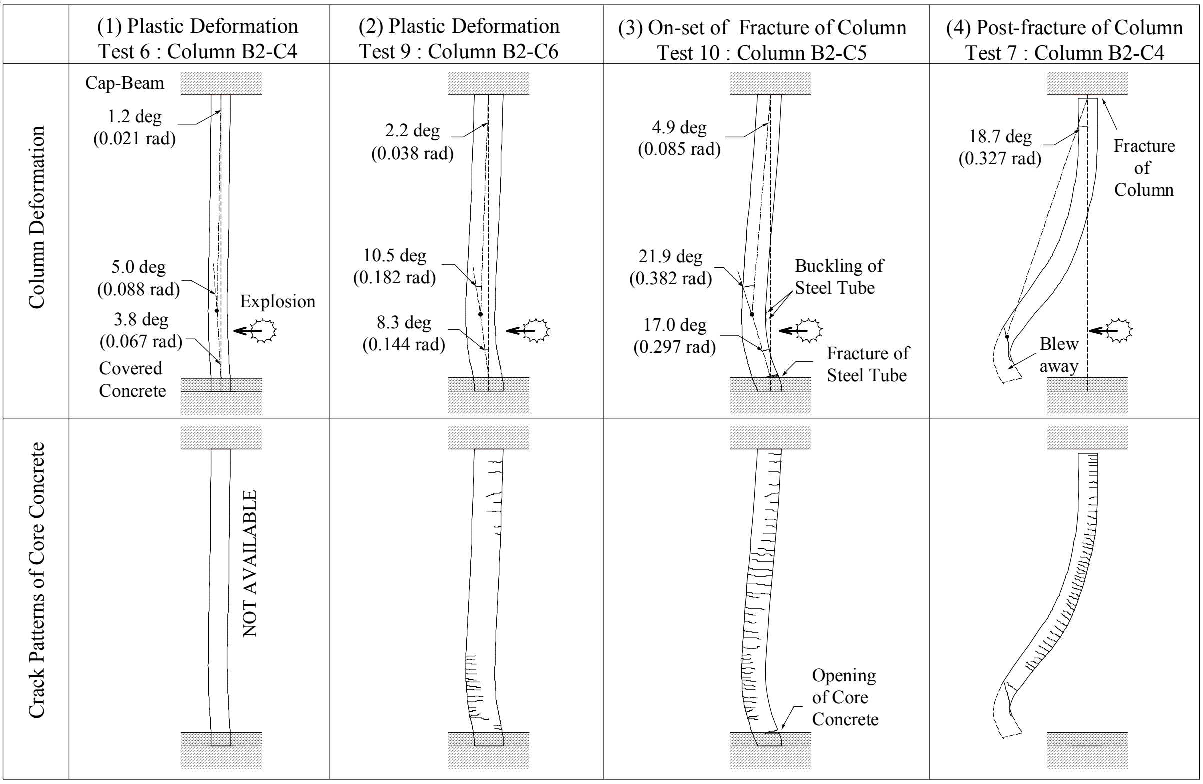


Figure 6: Damage progression in column (blast at credible threat height)

DUCTILE REINFORCED CONCRETE COLUMNS AND JACKETED COLUMNS

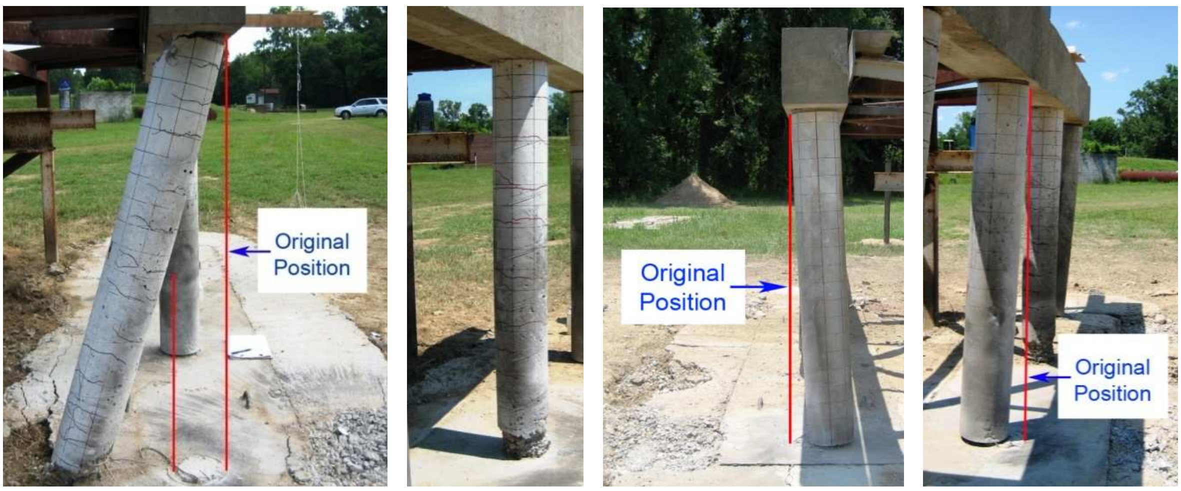
While CFST columns performed excellently in a multi-hazard perspective, they have not been commonly used in bridge engineering practice. Questions arose as to whether conventional columns designed to perform satisfactory under seismic excitations would possess adequate blast resistance. Furthermore, in many parts of the United States, particularly in California, reinforcement detailing requirements in effect prior to the 1971 San Fernando earthquake have resulted in RC columns that exhibited non-ductile behavior during earthquakes. Many methods have been used to retrofit such non-ductile columns. One of the most popular methods is steel jacketing which has been commonly used on the west coast of the United States. A column retrofitted with a steel jacket visually resembles a CFST column, but is typically discontinuous at the column top and base in order to avoid undesirable overload of the adjacent members (i.e., footing or cap beam) due to composite action that would significantly increase the flexural strength of the column (Buckle et al. 2006).

Therefore, tests were conducted to investigate the blast resistance of commonly used bridge columns, namely seismically ductile RC columns and non-ductile RC columns retrofitted with steel jackets to make them ductile, detailed in accordance to recent codes of practice.

Three different design codes or guidelines for bridges, namely AASHTO (2004), MCEER/ATC-49(2003) and CALTRANS (2003, 2006), were used for design of the ductile reinforced concrete columns of the prototype bridge pier bent. The columns were designed to be in compliance with all three (although differences were small, the more stringent requirements were taken as governing in all cases).

Then, non-ductile reinforced concrete columns were designed retrofitted by adding a steel shell that provides confinement to the concrete and allows plastic hinges to develop at the top and bottom of the column. The steel jacket was designed in accordance with the procedure developed by Chai et al. (1991) (as reported in Buckle et al. 2006).

Figure 7 shows the photo of each column after testing, and Figure 8 illustrates the corresponding deformation and damage of each column from side view. The post-test deformation of the column (from which residual plastic rotation can be calculated) was obtained by attaching a string at the original positions of top and bottom of the column and using a measuring tape to record the distance to the column. In Figure 8a and 8b, crack patterns observed after tests are shown along the height of the RC columns. The columns did not exhibit a ductile behavior under blast loading, but rather failed in shear at the base of the column as shown in Figures 7 and 8. More details are provided in Fujikura and Bruneau (2008, 2011).



(a) (b) (c) (d)

Figure 7: Columns after tests: (a) side view of Column RC1, Test 1 (b) front view of Column RC2, Test 2 (c) side view of Column SJ2, Test 3 (d) side view of Column SJ1, Test 4

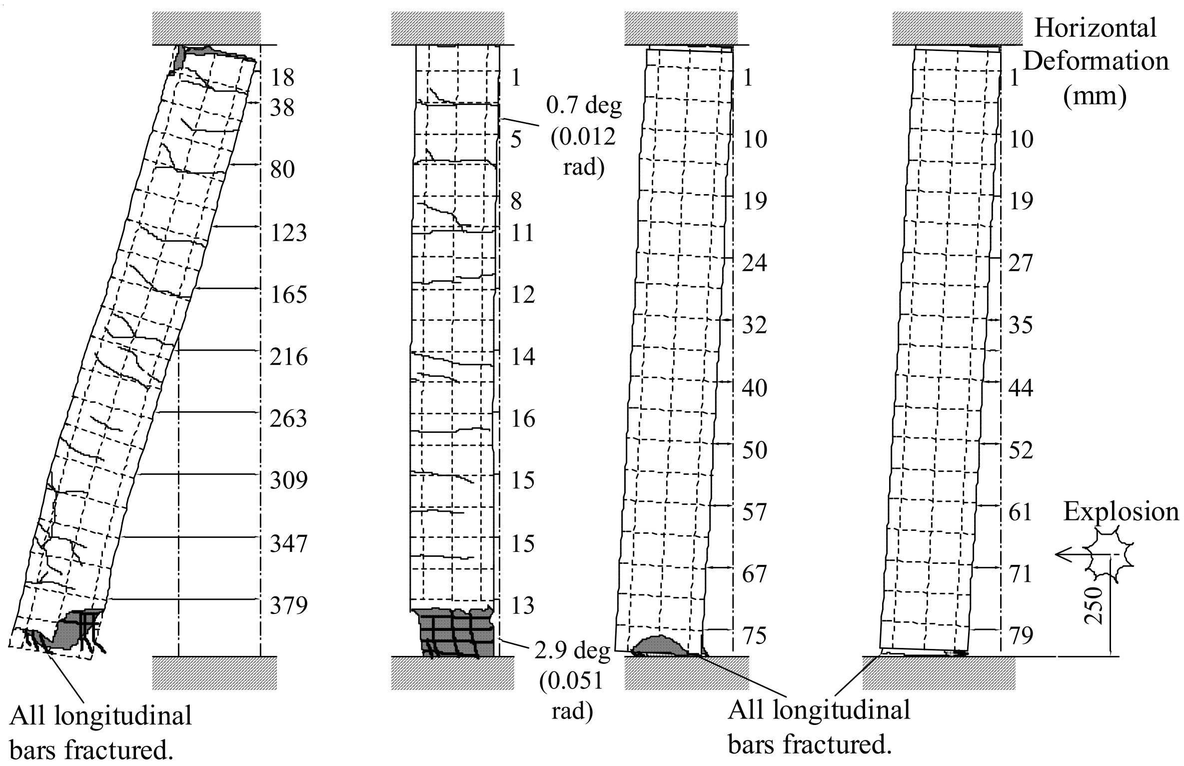


Figure 8: Sketches of columns after tests from side view: Left to right, (a) Column RC1, Test 1 (b) Column RC2, Test 2(c) Column SJ2, Test 3 (d) Column SJ1 Test 4

CONCRETE-FILLED DOUBLE SKIN STEEL TUBE COLUMNS

A Concrete-Filled Double Skin Steel Tube (CFDST) is formed by two concentric steel tubes separated by a concrete core as shown on Figure 9. That configuration seeks to draw upon the benefits in strength, toughness and stiffness derived by placing the steel at the periphery of a filler material (as described, for example, in Montague 1975). The outside skin at the periphery of the section provides strength and stiffness, the inside skin enhances ductility, and the concrete in between provides local and overall stability to the system. That synergy between the tubes and the core results in a more redundant section which completely fails only when all three layers of materials fail. In this section, concrete material failures such as spalling, breaching or scabbing under shock load are controlled. This helps preserve the stability of the section and guarantee enough (axial) residual strength for the section at ultimate conditions.

Note that the designs considered had sections that combined inside and outside tubes ranging from moderately ductile (MD) to highly ductile (HD). ASTM A513 steel tubes with a minimum yield of 290 MPa and self-consolidating concrete with a minimum strength of 34.5MPa were specified for the CFDST.

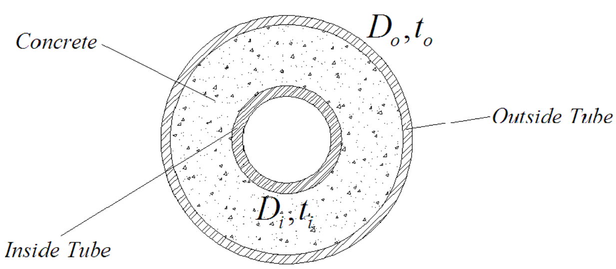


Figure 9: Concrete Filled Double Skin Tube

MODIFIED STEEL-JACKETED COLUMNS

As shown above, while wrapping a concrete column with a steel jacket is widely accepted as a cost effective retrofit technique for column of seismically deficient bridges, the merits of this technique do not translate into improved blast performance for bridge column. Direct shear failure under blast load have been observed at the gaps between the jacket and the surrounding footing and cap beam when exposed to blast. By controlling this undesirable failure mode without hindering the initial role of the jacket, a multi-hazard resistant system is obtained.

Structural steel collars were placed around the gaps and tied to the adjacent elements with post-installed anchors (as shown in Figure 10) to help increase the shear strength locally. A non-stick interface material was also inserted between the collar and the column to allow only smooth contact, thus increasing only shear strength while leaving flexural strength of the column virtually unchanged (as intended by the initial jacketing design).

The materials properties for the MSJC were the same as in Fujikura and Bruneau (2008). A 9.5mm A36 steel plate was used for the base of the collar, whereas the lip of the collar was cut from an A53 steel tube (216mm in diameter and 8mm-thick).

Figure 10: Steel Collar: Components (Left and Middle) and Construction (Right)

BLAST TESTS OF CFDST AND MSJC COLUMNS

The CFDST and MSJC column bents were tested under blast overpressures. Measured plastic rotations after the blast tests were well in excess of 2 degrees for CFDSTs and MSJC (Figures 11 and 12, respectively). For reinforced concrete columns, blast-induced plastic rotations of 2 to 4 degrees would lead to “moderate to heavy damage” (UFC, 2008) suggesting that repairs or complete replacement would be necessary. However, comparable test results for both CFDSTs and MSJCs showed that repair would be needed for aesthetical reasons only, and that the deformed columns would be able to perform their functions. For more severe blast charges, plastic rotations in excess of 4 degrees were accompanied with moderate to important local denting of CFDST section and, in one case, seam failure of MSJC’s jacket. For near contact charge condition (defined here as a scaled distance below 0.5ft/lb$^{1/3}$), fracture of the outside tube of the CFDST occurred, but the inner tube prevented complete failure of the columns.

Figure 11: Plastic Deformation of CFDST under Blast

Figure 12: Global (Left) and Local Deformation (Right) of MSJC after Blast Test

CYCLIC INELASTIC TESTS

Cyclic inelastic tests of seismically detailed reinforced concrete columns, jacketed concrete columns, and CFST have been numerous and are reported extensively in the existing literature. Cyclic inelastic tests of CFDST are much fewer. Therefore, such tests were conducted on CFDST of similar construction as those that were subjected to blast testing above, to simulate behaviour expected during earthquake excitations.

All specimens in the seismic test program developed strengths that exceeded their theoretical plastic moment (calculated following simple plastic theory, as described in Bruneau et al. 2011), and exhibited ductile behavior up to failure. In all cases, yielding preceded buckling of the outside tube. Yielding first occurred at a drift level of about 1.5%, local buckling around at least 3%, and failure at well beyond 7.5%. Stiffness reduction was gradual, and no substantial reduction in strength was observed until failure. In general, hysteretic loops for the specimens were stable and exhibited pinching proportional to the severity of local buckling (Figure 13).

Note that testing continued until little residual strength remained, to allow quantifying the rate of strength degradation that would be needed to be able to perform analytical studies such as those described in FEMA 356 or FEMA 695 (although such analyses are not part of the current scope of work).

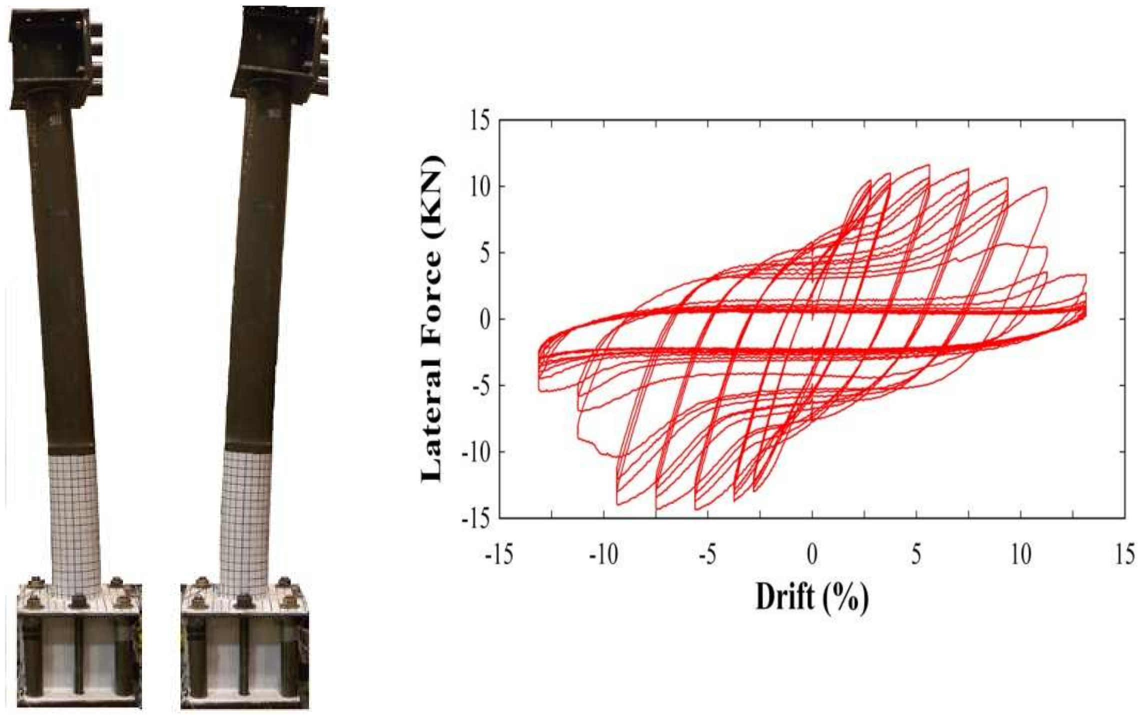


Figure 13: Specimen at Maximum Strength (Left) and Typical Hysteretic Loop (Right)

CONCLUSIONS

Concrete-filled steel tube (CFST), concrete-filled double skin steel tube (CFDST), and Modified Steel Jacketed Columns (MSJC) were subjected to blast, as well as in some cases cyclic inelastic tests, to investigate their applicability as candidate multi-hazard systems for bridge applications. Satisfactory behavior was obtained in all cases. Large ductile flexural deformations were achievable during both seismic and blast tests. In contrast, seismically ductile reinforced concrete columns and jacketed reinforced concrete columns both failed in direct shear, in a non-ductile manner. This suggests that CFST, CFDST, and MSJC columns can be effective alternatives to regular reinforced concrete construction and conventional jacketed columns in bridge piers, providing both seismic as well as blast resistance.

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