Technical Note

Post-Fire Axial Load Resistance of Concrete-Filled, Double-Skin Tube (CFDST) Stub Columns

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ABSTRACT

This technical note reports findings on a series of squash tests to investigate the effects of a significant fire loading history on the axial load strength of concrete-filled, double-skin tube (CFDST) stub columns. Axial loading tests were conducted on two stub columns that were previously subjected to the first 60 minutes of the standard ASTM E119 (ASTM, 2012) fire. Results were compared to the resistance of an identical virgin stub column. Comparisons indicated an average reduction of 28% in the axial load strength of stub columns when subjected to the mentioned fire loading history.

Keywords: steel tube column, fire loading, ASTM E119, double skin, axial strength.

INTRODUCTION

Concrete-filled, double-skin tube (CFDST) columns have been shown to perform well under both single- and multi-hazard conditions (e.g., when subjected to inelastic cycling loads to simulate seismic effects, when subjected to blasts, and when subjected to subsequent cyclic and fire loading to simulate cascading effects—or fire and cyclic loading to simulate earthquakes happening on a building subjected to a fire years earlier) in several past studies (Zhao and Grzebieta, 2002; Han et al., 2004; Yang and Han, 2008; Lu et al., 2010; Fouche and Bruneau, 2010; Imani et al., 2014a; 2015). This has made CFDST columns appealing from a multi-hazard perspective. However, one specific aspect that needed to be considered in the referenced studies investigating the resilience of CFDST columns was whether the squash strength of CFDST columns was adequate solely from a post-fire perspective because any permanent effects from a prior fire loading would need to be known to ensure proper functioning of these columns through the life of a structure after exposure to a fire (as one of many limit states that may need to be evaluated in post-fire conditions to determine if repair or replacement is warranted). For that reason, this study examines the post-fire axial load resistance of CFDST stub columns following exposure to a standard ASTM E119 fire (ASTM, 2012). In addition, and to a lesser extent, fire test results were used to study the effect of ventilation holes on temperature distribution and concrete moisture content loss in CFDST stub columns.

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SPECIMENS

Three CFDST stub columns, hereafter referred to as SC1, SC2 and SC3, were built with identical height (i.e., 1 ft) and cross-section, as shown in Figure 1. The outer and inner tubes were 8-in.- and 5-in.-diameter, respectively. Concrete was poured between the two steel tubes and, as recommended in AISC Design Guide 19 *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003), small (⅛-in.) vent holes were drilled into the outer tube to allow pressure relief during fire. Stub columns SC1 and SC2 were built with a total of two and three ventilation holes. The first two holes were located on the round surface of the specimens at about 1 in. from the top and bottom plates. The third hole, in the case of SC2, was located at the opposite side of the other vent hole drilled near the top end. The intent of an additional hole was to crudely investigate whether additional ventilation and pressure relief can change the resulting temperature history and possible permanent fire effects on the strength of stub columns. Stub column SC3 was kept without holes and was to be tested without having been previously exposed to fire to serve as a reference specimen.

FIRE TESTS

Stub columns SC1 and SC2 were subjected to the first 60 minutes of the ASTM E119 (ASTM, 2012) fire curve in a natural gas furnace. No insulation was used at the top or bottom ends of the specimens. The standard curve and fire duration were selected to provide a common comparison base for different specimens in this study as well as similar past studies. Three thermocouples located at mid-height of stub columns were monitoring temperature variations at three different points through the cross-section (i.e., inner surface of the outer tube, half-width through concrete, and outer surface of the inner tube) for both stub columns. A hole was drilled on the outer tube at this location to allow for the exit of wires. Figure 2 shows SC1 and SC2 after the fire test. No axial load was applied to the specimens during the fire test, but based on results for corresponding slender columns tested simultaneously (Imani et al., 2014a), it is known that these stub columns would not have reached their compression limit state. No visible damage was detected on the stub columns after the fire test for the intensity and duration described earlier. Both specimens were left to gradually cool down to room temperature.

Figure 3 shows the time histories of temperature recorded by the thermocouples installed in the stub columns along with the average furnace air temperature during fire. Note that the actual test stopped at around 70 minutes and the recorders were kept operating for a few additional minutes. The recorded temperature curves generally show a smooth increasing trend, except for the ones recorded by the thermocouples installed on the surface of the inner tube for both SC1 and SC2 that show sudden fluctuations, especially during the first 20 minutes of the test. These fluctuations are speculated to occur due to the random arrangement of aggregates in the concrete of the specimens, causing moisture and pressure relief at certain times and affecting the thermocouple readings.

Comparison of temperature time histories for the two stub columns show that the temperature values for all of the three

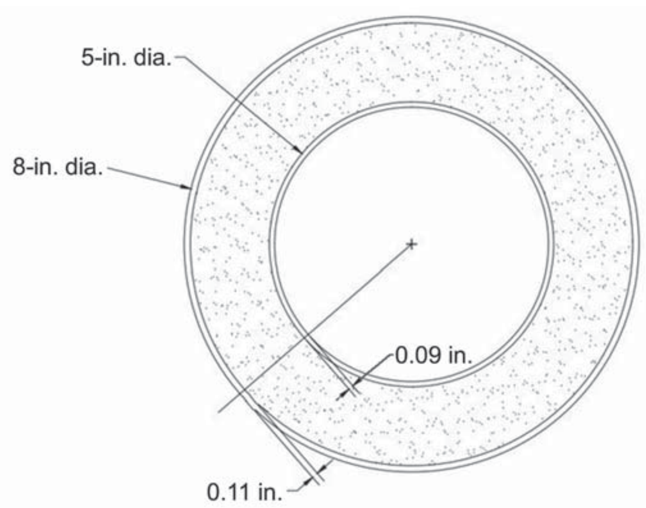
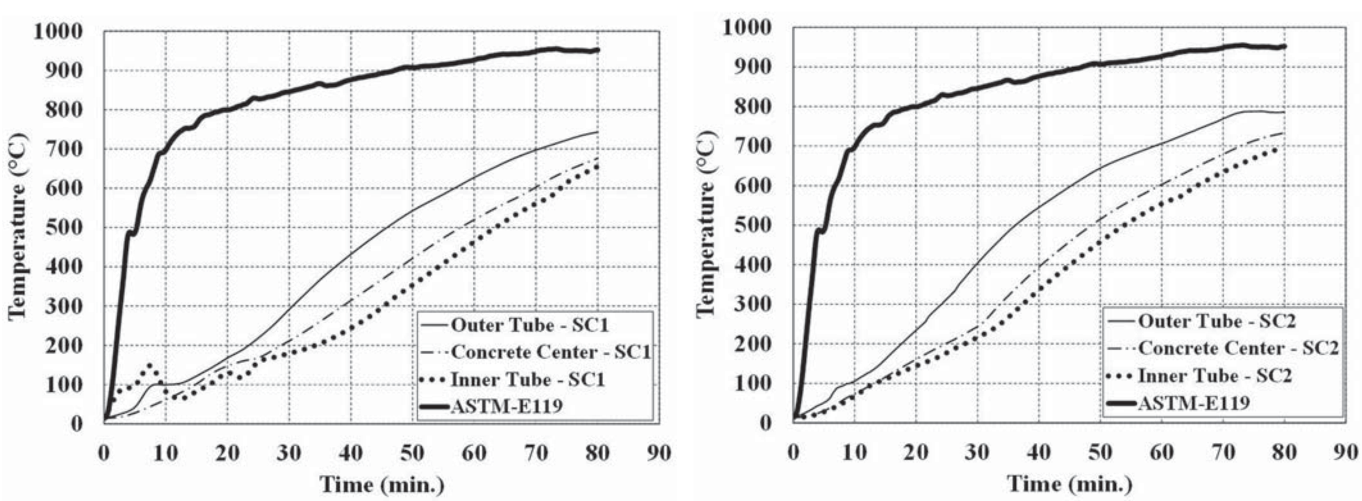


Fig. 1. Cross-section of the stub columns.

(a) (b)

Fig. 2. Stub columns after fire test: (a) SC1; (b) SC2.



(a) (b)

Fig. 3. Temperature data measured from the stub columns: (a) SC1; (b) SC2.

thermocouples installed in SC2 are about 50°C higher than the values for SC1. Because the difference in temperature is noticeable in values recorded on the outer surfaces of columns and from the beginning stages of the test, it is inferred that the reason for this difference is likely related to the different locations of the stub columns at the bottom of the furnace. Note that nine thermocouples were installed at different heights inside the furnace, and the average of readings from all of those was used to control average temperature to follow the standard fire curve. Beyond this, recommended additional experimental research may be informative to study the possible effect of vent holes on temperature distribution within the specimens.

Stub columns were also used to study the effects of fire tests on the moisture content of concrete by comparing the measured relative humidity (RH) of the two stub columns tested in the fire (i.e., SC1 and SC2) with that of the third one, SC3, which was kept intact in the lab. Relative humidity is the amount of water vapor present in a volume of air at a given temperature compared to the maximum amount that the air could hold at that temperature, expressed as a percentage. Relative humidity of the three stub columns were measured based on the standard ASTM F2170 (ASTM, 2011). An electronic probe was inserted into holes drilled into the concrete (after running through the steel) to measure the RH values. Measurements gave post-fire RH values of 29% and 20% for SC1 and SC2, respectively. These values were about half of the RH value of 59% for SC3, which wasn't subjected to the fire test (RH was measured about 6 months after casting). Relative humidity of 59% is within the range of 50 to 75% defined by ASTM E119 (ASTM, 2012) for concrete dried at room temperature. The effect of the additional vent hole is seen in the difference of RH values for SC1 and SC2. Note that the holes required for the RH tests were drilled after the fire tests at locations, which were not close to the original vent holes of stub columns to avoid capturing local effects.

SQUASH LOADING TEST

To investigate the permanent effects of an approximate 1-hour-long fire exposure on the squash load of CFDST columns (ASTM, 2012), an axial loading test was conducted on the SC1, SC2 and SC3 stub columns. The squash tests were conducted using the 2,200,000-lb uniaxial loading testing facility of Taylor Devices Inc. in Buffalo, New York. Axial load was applied to 12×12×¼-in. cap plates welded to both ends of each stub column. All tests were terminated upon observance of significant loss in axial load capacity, which was typically accompanied by severe local buckling of the steel. Figure 4 shows photos of all three failed stub columns.

Figure 5 shows axial load versus axial contraction results for all three stub columns (SC1, SC2 and SC3). The initial phase of all three curves (i.e., axial contraction below 0.025 in.) nearly follows a similar path, suggesting that the fire loading history, in the absence of structural loads, has caused no significant permanent effects on the elastic stiffness of SC1 and SC2.

However, the figure demonstrates differences in maximum strength and post-maximum strength for the three specimens. Stub columns SC1 and SC2, which were both subjected to fire previously, had maximum axial load values of 334 and 339 kips, respectively. These values are about 28% less than the 471-kip squash load obtained for the reference specimen not subjected to fire (SC3).

Cylinder tests showed an average f_c' of 9.7 ksi for concrete (normal weight, density: 150 pcf) from the specimen that was not tested in fire. Steel tubes were manufactured from ASTM A513 (ASTM, 2015) type 1 steel with nominal yield and tensile strength values of 32 ksi and 45 ksi, respectively. Steel coupon tests resulted in an average f_y of 50 ksi for the outer tube and 44 ksi for the inner tube (Imani et al., 2014b). The theoretical squash load can be calculated as:

$$f_y(A_{st,inner} + A_{st,outer}) + 0.85 f_c' A_{concrete} \tag{1}$$

(a) *(b)* *(c)*

Fig. 4. Failed stub columns: (a) SC1; (b) SC2; (c) SC3.

where A refers to the cross-sectional area described by the subscripts (i.e., inner/outer steel tubes and concrete between them) and f_y is the yield strength of steel taken from the coupon tests. Applying Equation 1 to the CFDST section geometry shown in Figure 1 with the room temperature values of f_c' and f_y (as previously mentioned) results in a theoretical squash load value of 428 kips. The theoretical calculated value is within 10% of the test result (i.e., 471 kips).

According to Eurocode 4 (CEN, 2005) specifications, the residual compressive strength of concrete heated to maximum temperature of θ_{max} and subsequently cooled down to ambient temperature of 20°C can be calculated as follows:

$$f'_{c,\theta max,20} = \phi f'_c$$

$$\phi = \left\{ \begin{array}{ll} k_{c,\theta max} & 20 \leq \theta_{max} < 100 \\ 1.0 - \left[\dfrac{0.235(\theta_{max} - 100)}{200}\right] & 100 \leq \theta_{max} < 300 \\ 0.9k_{c,\theta max} & \theta_{max} \geq 300 \end{array} \right\} \tag{2}$$

where $k_{c,\theta max}$ is a reduction factor that is provided in Eurocode 4 tables for normal- and light-weight concrete and f_c' refers to the compressive strength of concrete at room temperature. Considering $\theta_{max} \approx 700$°C for the case in hand (see Figure 3) and a $k_{c,\theta max}$ factor of 0.3 (from Eurocode 4 tabulated values), the residual compressive strength is calculated to be about 27% of the initial (i.e., room temperature) f_c' value. For steel, although it is considered to almost fully recover after cooling down to room temperatures from maximum temperature levels seen in Figure 3, Eurocode conservatively recommends a 10% reduction factor to be applied to the room temperature yield strength value.

Using the residual values of strength for steel and concrete calculated with the average measured strength values at room temperature and the reduction factors mentioned earlier for the case in hand, a theoretical reduction of 40% is achieved for the squash load of stub columns. As such, test results reveal that the strength reduction factors specified in Eurocode 4 might be slightly conservative (28% reduction in test versus the calculated 40%) but can provide a reasonable estimate of the post-fire conditions. A similar reduction is seen in the post-peak strength plateau reached in both SC1 and SC2 curves, where SC1 and SC2, respectively, show 18 and 24% reductions in axial force from the recorded value of 305 kips for SC3 (i.e., the specimen not subjected to fire).

CONCLUSION

Experimental observations presented in this study indicated that a fire loading history, albeit causing no significant visible damage, can induce permanent effects on structural integrity of CFDST stub columns. More specifically, an average 28% reduction was observed in the axial squash load strength of two stub columns previously exposed to a fire loading history as opposed to a reference specimen not subjected to fire. Moreover, a simple comparison between two specimens with different number of vent holes demonstrated that more ventilation accelerates concrete moisture loss without significantly affecting the temperature distribution during fire. However, more research is needed to further validate this observation.

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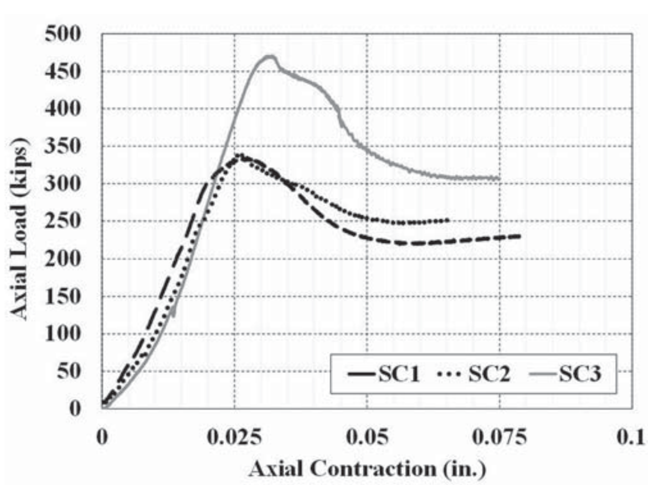

Fig. 5. Axial force vs. axial contraction curves from squash tests.

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