SEISMIC PERFORMANCE OF CIRCULAR CONCRETE FILLED TUBE COLUMNS FOR ACCELERATED BRIDGE CONSTRUCTION

Catherine Tucker$^1$ and Luis Ibarra$^2$

ABSTRACT

This study evaluates the seismic performance of circular concrete filled tube (CCFT) columns in accelerated bridge construction (ABC) projects. CCFT components are considered of interest for bridges subjected to seismic forces due to their efficient structural behavior under combined axial and bending loads: lateral stiffness of the steel tube is increased by the concrete and concrete confinement is provided by the steel tube. This paper addresses the ability of CCFT columns to perform adequately under gravitational and seismic loading before the concrete has reached its design strength. A reduced seismic hazard that accounts for this temporal condition is also implemented. The evaluation of performance is based on the probability of failure of the CCFT column.

For this research, a Caltrans bridge used in previous PEER studies is adopted. The performance of a proposed CCFT column is compared to the original circular reinforced concrete (RC) column. Numerical analyses using concentrated plasticity models in OpenSees were used for this evaluation. Experimental data was used to calibrate the deteriorating response of CCFT columns in OpenSees. The analytical model predicts the CCFT column’s behavior under monotonic, static cyclic, and dynamic (seismic) loading. Then, the model was adapted to consider the effects of partial strength concrete to assess the column behavior under the temporary condition of the concrete not having reached full strength. The study accounts for temporary conditions, such as concrete compressive strength lower than the design value, and reduced seismic loads. The results indicate that CCFT columns with partial design concrete compressive strength can be used for ABC because the relatively low decrease in strength is offset by the reduced seismic loads for this temporal condition.

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**Introduction**

This research evaluates the seismic performance of circular concrete-filled tube (CCFT) columns in accelerated bridge construction (ABC) projects. Current ABC usually uses precast concrete columns grouted to rebar connections at base and top, if intermediate columns are required. The bridge can be assembled in a few days, but the seismic performance objectives cannot be reached until the columns’ top and base connection grout reaches design strength. The advantage of CCFT columns is the use of standard bolted connection at the top and bottom of the column – capable of resisting design loads upon being bolted without the need to wait for curing design

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strength to be reached. The bolted connection also eliminates rebar congestion at the connection. Also, the materials needed to construct CCFT columns are readily available. The time the CCFT concrete filling takes to cure, and the column’s reduced capacity for that duration, poses a primary challenge when CCFT columns are considered for ABC. This study investigates whether a designation of temporary condition can be used to reduce the Design Basis Earthquake (DBE).

**Literature Review**

**Behavior of Concrete-Filled Tube (CFT) Columns**

In concrete-filled tubes (CFTs), the steel tube mainly resists flexure loads, while the concrete provides compression resistance and prevents buckling of the steel tube. CFT columns combine desirable characteristics of the base materials. CFT columns have an improved ductility as compared to traditional concrete [1]. Also, the strength of a CFT column is greater than that of either an equivalent concrete column or of an unfilled steel tube column [2]. The steel tube provides adequate confinement to the concrete, which has a higher Poisson ratio than steel at high loads. This situation prevents transverse expansion of the concrete.

CFT columns under axial compression exhibit elastic and elastic-plastic instability failure, as well as elastic instability failure. An, et al. [3] found that the failure mode of very slender circular CFT columns is elastic instability, and the ultimate strength is determined by the column’s flexural rigidity. The concrete core has two functions: to increase flexural stiffness and ultimate strength, and to prevent local buckling of the steel tube. The code-predicted ultimate strengths are conservative. Local buckling of a CFT column is significantly reduced from that of an unfilled steel tube column, but the chance of local buckling is not eliminated completely. Local buckling is dependent upon the ratio of the outside diameter of the steel tube to its thickness (D/t ratio). Concrete filling increases the buckling threshold as much as 70% [4]. The CCFT’s concrete filling will quickly provide lateral bracing for the steel column as stiffness is attained much more quickly than comparative strength. The concrete provides continuous lateral support to the column, and therefore, the column bending and shear capacity is of main concern, not its buckling capacity.

**Temporary Conditions on CFT Columns**

In the case of ABC projects, reducing curing time through use of concrete accelerant ad-mixtures could be disadvantageous due to the resulting reduction in concrete toughness. The alternative is to address the performance of CFT columns under gravitational and seismic loading before the concrete reaches its design strength, and consider the seismic hazard risk reduction due to this temporary condition. Temporary conditions are used in nuclear facilities, but there is ambiguity about what constitutes a temporary condition [5]. By contrast, temporary conditions for CFT within ABC are well-defined, having an upper limit for the temporary condition designation: after the concrete reaches its design strength, the temporary condition is discarded.

Amin, et al. [6] [7] evaluated seismic loading for temporary conditions in nuclear plants. They used annual seismic hazard curves to determine acceleration levels corresponding to temporary loads in which the corresponding acceleration is dependent upon the duration of the temporary loading. It is directly used to create a new plot in which the new curve plotted takes the same shape of the previous, but the probability is reduced by a linear proportion where the
shorter the temporary condition duration, the more the probability of exceedance may be reduced. Amin, et al. also propose a methodology of calculating and applying reduced seismic loads (RSLs) for evaluation of temporary conditions using design basis allowable loads [8]. The design basis seismic event recurrence interval for a temporary condition of concrete strength is specified, through use of a reduced seismic load, such that the probability of failure is the same as in traditional design of permanent structures.

Research Description

Design Basis Bridge
The highway bridge considered in this research is one of the bridges designed by Caltrans in California [9]. The selected bridge (Figure 1) consists of five straight spans. The deck is post-tensioned cast-in-situ 39 ft. wide 6 ft. deep concrete box girders to allow two 12 ft. lanes for traffic, a 4 ft. left shoulder, an 8 ft. right shoulder, and traffic barriers at the perimeter. The single column bents are 4 ft. diameter reinforced concrete (RC) columns 22 ft. tall. The basis of this research is replacement of one of the single mid-span RC columns with a CCFT column.

![Design basis bridge: a) bridge elevation b) column elevation c) RC and CCFT column sections](image)

Experimental CCFT Data
There are extensive experimental databases for CCFT [10][11]. However, cyclic loading experimental data of normal strength CCFT of appropriate dimensions and boundary conditions for highway bridges are scarce. A set of four columns tested by Marson and Bruneau [12][13] was originally selected due to its goal of approximating the various D/t ratios of highway bridges, its concentration on providing a fixed base condition, constant axial load of appropriate $P/P_y$ values, and quasi-static cyclic loading protocol.

Proposed CCFT Column
A CCFT column is designed to match the moment capacity of the original RC column. The CCFT column has a 39 in. outside diameter and 0.61 in. thick steel tube. Steel strength is chosen
using AISC’s recent adoption of ASTM A1085-13 steel specification for HSS [14]. The data for the proposed CCFT and two of the experimental columns with similar elastic stability coefficients is shown in Table 1.

<table>
<thead>
<tr>
<th>Column Properties</th>
<th>CCFT 28</th>
<th>14</th>
<th>7</th>
<th>3</th>
<th>CFST 64</th>
<th>CFST 42</th>
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</thead>
<tbody>
<tr>
<td>ratio of tube diameter to thickness D/t</td>
<td>73.9</td>
<td></td>
<td></td>
<td></td>
<td>73.9</td>
<td>42.8</td>
</tr>
<tr>
<td>Outside tube diameter, D (in)</td>
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<td>16.0</td>
<td>16.0</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Column height, h (in)</td>
<td>264</td>
<td>86.6</td>
<td>86.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c$ (ksi)</td>
<td>5.2</td>
<td>4.7</td>
<td>3.4</td>
<td>2.1</td>
<td>5.4</td>
<td>5.1</td>
</tr>
<tr>
<td>$F_y$ (ksi)</td>
<td>50.0</td>
<td>64.1</td>
<td>73.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$EI_{eff}$ (AASHTO) (k-in$^2$)</td>
<td>6.00E+08</td>
<td>5.92E+08</td>
<td>5.69E+08</td>
<td>5.39E+08</td>
<td>1.50E+07</td>
<td>2.11E+07</td>
</tr>
<tr>
<td>Reduction of $EI_{eff}$ used for model</td>
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<td>80%</td>
<td>80%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_s/A_c$</td>
<td>6.2%</td>
<td>5.6%</td>
<td>10%</td>
<td></td>
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</tr>
<tr>
<td>$P/P_y$</td>
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<td>0.20</td>
<td>0.24</td>
<td>0.30</td>
<td>0.14</td>
<td>0.00</td>
</tr>
<tr>
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<td>6039</td>
<td>5949</td>
<td>5677</td>
<td>4990</td>
<td>450</td>
<td>722</td>
</tr>
<tr>
<td>$M_c$ ($M_c/M_y=1.3$) (k-ft)</td>
<td>7851</td>
<td>7734</td>
<td>7380</td>
<td>6487</td>
<td>585</td>
<td>939</td>
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<tr>
<td>$\theta$, Stability coeff. ($P\Delta/Vh$)</td>
<td>0.069</td>
<td>0.070</td>
<td>0.073</td>
<td>0.077</td>
<td>0.047</td>
<td>0.061</td>
</tr>
</tbody>
</table>

**Effect of Time on Column’s Strength**

To determine the strength of the proposed CCFT column, an interaction P-M diagram was created using the AISC recommended method for CCFT [15]. Figure 2 shows the P-M interaction diagrams of the bridge RC column at full design strength and the proposed CCFT as a function of concrete curing time. The RC column has the greater resistance axially, due in part to its larger cross-sectional area, but the CCFT column has a larger moment resistance because of the steel tube outer perimeter location.

![Figure 2. CCFT and RC column interaction diagrams as a function of concrete curing time.](image)

To evaluate the effect of time on the strength of the proposed CCFT, full bond strength is assumed regardless of concrete curing time. The gain in concrete strength as a function of time is shown in Figure 3a. Figure 3b uses the P-M diagram of Figure 2 to show the relative capacity of the CCFT column as a function of concrete curing time as a percent of the design $f_c'$ (at 28 days).
The results are presented for the conditions of pure axial load, pure bending moment, and maximum moment capacity. As observed, the moment capacity of the column is less dependent on time than the axial capacity because the largest contribution to moment capacity is provided by the steel tube.

Figure 3.  
(a) Concrete strength as a function of concrete curing time.  
(b) Ratio of capacities of proposed CCFT to same capacities at 28 days.

**Hysteretic Deterioration Models**

A peak-oriented deteriorating hysteretic model [16] is used in the study. Figure 4a shows the backbone curve of this model, which consists of an elastic stiffness $K_e$, a strain hardening interval capped at a maximum strength $F_c$, and a negative tangent stiffness $\alpha_c K_e$ (post-capping stiffness). The hysteretic model includes four modes of cyclic deterioration based on energy dissipation: strength deterioration, post-capping strength deterioration, as well as unloading and reloading stiffness deterioration. The amount of deterioration depends on the parameter $\beta_i$, which may be different for each cyclic deterioration mode. The model includes a lumped mass, and the effect of $P-\Delta$ is directly included in the analysis. For instance, the unloading stiffness in the $i^{th}$ excursion ($K_{u,i}$) is deteriorated as:

$$K_{u,i} = (1 - \beta_{k,i})K_{u,i-1}$$  \hspace{1cm} (1)

where $\beta_{k,i}$ is the deterioration parameter for unloading stiffness in the $i^{th}$ excursion. In its general form, $\beta_i$ is expressed as:

$$\beta_i = \left( \frac{E_i}{E_i - \sum_{j=1}^{\Sigma E_j}} \right)^c$$  \hspace{1cm} (2)

where $E_i$ is hysteretic energy dissipated in excursion $i$, $\Sigma E_j$ is hysteretic energy dissipated in previous positive and negative excursions, $E_i = \gamma F_y \delta_y$ is the reference hysteretic energy dissipation capacity of component in the original Ibarra-Krawinkler model [17], $E_i = \lambda F_y \delta_p = \Lambda F_y$ is the reference hysteretic energy dissipation capacity of component in the modified model [18]. The parameter $c$ is 1 for this study, implying a constant rate of deterioration. $\delta_y = F_y/K_e$ is the yield deformation, $\delta_c$ is cap deformation (deformation
associated with $F_c$ for monotonic loading, used in the original model), and $\delta_p$ is plastic deformation capacity (used in the modified model).

A modified version of the deteriorating hysteretic model developed by Ibarra et al. [19] was used in the study. This model [20] is implemented in OpenSees and emphasizes the rotational capacity, as one of the main parameters in nonlinear seismic evaluation. The central difference is in the parameters $\lambda$ or $\Lambda$ used instead of $\gamma$, to account for the underlying difference between $\delta_p$ and $\delta_y$, where $\delta_p = \delta_c - \delta_y$. This modified peak-oriented hysteretic model (Figure 4b) is used to model the equivalent stiffness as a spring in the concentrated plasticity model within OpenSees.

![Monotonic Load-Deformation Model](image)

![Chord Rotation $\theta$](image)

**Figure 4.** a. Parameters of backbone curve for modified Ibarra-Krawinkler model [21].

b. Parameters for modified IMK model with peak-oriented hysteretic response [22].

**Analytical Model Results**

Deteriorating hysteretic parameters were determined from the experimental data of Marson and Bruneau [23]. These parameters are in large part a function of $D/t$ and $A_s/A_c$. Figure 5. presents the analytical cyclic and monotonic models, which account for deterioration due to both material degradation and P-Δ effects. The main parameters used in curve-fitting the analytical model to the experimental columns included a plastic rotation capacity $\theta_p = 0.08$, a post-capping rotational capacity $\theta_{pc} = 0.10$, basic strength deterioration $\Lambda_c = 4.0$, and accelerated reloading stiffness deterioration $\Lambda_a = 1.6$. These results are used to select the backbone curve properties and cyclic deterioration parameters for the proposed CCFT column model.

![Analytical Model Results](image)

Figure 6 shows the effects of gain in concrete compressive strength on the hysteretic response of the deteriorating CCFT column, again including both material degradation and P-Δ effects. The results indicate that the CCFT column behaves favorably cyclically as soon as the concrete has cured 3 days, and by 14 days the behavior is virtually identical with that expected of full design-strength concrete.
Incremental dynamic analyses (IDAs) were performed using FEMA P695 set of 44 far-field ground motion records [24], using a modified OpenSees script [25][26]. The records were scaled at the 5-percent damped spectral acceleration of the fundamental period of the system (Figure 7a).

Figure 5. Experimental and predicted analytical hysteretic behavior (under static cyclic loading) and monotonic backbone curve.

Figure 6. Proposed CCFT: monotonic and cyclic behavior as a function of concrete curing time.

(a) Incremental dynamic analysis for proposed CCFT at 28 days
(b) Median incremental dynamic analyses as a function of concrete curing time.

Figure 7. a) Incremental dynamic analysis for proposed CCFT at 28 days
b) Median incremental dynamic analyses as a function of concrete curing time.
The comparative results of the median IDA values, in Figure 7b, show that the dynamic behavior of the column at 14 days is practically the same as expected for the full-capacity column at 28 days. Furthermore, the column’s capacity after 3 days of concrete curing time is already about 80 percent of the full-capacity column.

Effect of Concrete Compressive Strength on Temporary Conditions

The time required for the concrete’s column to meet the design compressive strength creates a temporal condition, if the bridge is open to traffic before 28 days of concrete curing. One of the goals of this study is to estimate the seismic probability of failure for this temporary condition. The alternatives for developing seismic criteria for temporary conditions include i) reduction of seismic loads with a standard design criteria, or ii) the use of standard seismic loads with a relaxed acceptance criteria [27]. Reduced seismic loads (RSLs), the most common approach, can be achieved by several approaches that are controversial due to the illogical implications for temporary conditions that can be obtained from arbitrarily discretizing a permanent condition, as well as the inability to provide constant failure or fatality frequencies. In the case of CCFT columns, the temporal conditions are clearly predefined (i.e., the days the concrete needs to reach the design compressive strength), and arbitrary discretizations of time can be excluded. Thus, RSLs are obtained following the approach presented by [28][29], in which:

\[
RSL = k \cdot DBSL
\]

Where \(DBSL\) is the design basis seismic load, and \(k\) is a reduction factor that depends on the duration of the temporary conditions. In this study, it is conservatively assumed that \(k = 0.083\) (representing about one month) for the three temporary conditions at 3, 7, and 14 days. Figure 8a shows the mean hazard curve (\(\lambda_{Sa}\)) assuming the bridge is located in Salt Lake City, Utah, as well as the reduced hazard curve for evaluation of temporary conditions.

Regarding the system’s capacity, for the CCFT at 28 days (Figure 7a), the median collapse capacity is \(\tilde{m}_{Sa} = 0.88\) g, and the standard deviation of the log of the collapse capacity \(\beta = \sigma_{lnSa} = 0.43\). This dispersion on collapse capacity due to record to record (RTR) variability is practically the same for CCFT columns with concrete curing of 7 and 14 days. For CCFT at 3 days, the dispersion is slightly lower (\(\beta = 0.415\)) because the system is less ductile [30]. Figure 8b presents the fragility curves of the CCFT columns, \(F_{C, Sa,c}(x)\), at the collapse capacity limit state that was used to obtain the probability of failure. The mean annual frequency of collapse (\(\lambda_{CC}\)) can be expressed as the mean annual frequency of the strong motion intensity (\(S_a\)) being larger than the collapse capacity (i.e., \(\lambda_{Sa}\), multiplied by the probability of having such a strong motion intensity (i.e., \(F_{C, Sa,c}(x)\)),

\[
\lambda_{CC} = \int_{0}^{\infty} F_{C, Sa,c}(x) \cdot d\lambda_{Sa} \tag{4}
\]

The above equation was solved by numerical integration for the four CCFT columns. For the CCFT column at 28 days and the original hazard curve, the computed probability of failure was \(P_f = 1.32 \times 10^{-4}\). The numerical integration of the RSL hazard curve and the CCFT columns at 3, 7, and 14 days, resulted in \(P_f = 1.67 \times 10^{-5}, 1.25 \times 10^{-5}, \) and \(1.11 \times 10^{-5}\), respectively. These
probabilities of failure are significantly lower than that of the design CCFT column \( P_f = 1.32 \times 10^{-4} \), indicating that the capacity reduction of CCFT columns at 3, 7, and 14 days is compensated by assuming a temporary condition. The hazard curve reduction factor could have been assumed as \( k = \frac{1}{2} \) (i.e., a temporary condition of six months), and the critical CCFT at 3 days would render a \( P_f = 1.0 \times 10^{-4} \), still lower than the probability of failure for the base case. This conservative calculation implies that the temporary condition could last for six months, instead of three days, and the probability of failure would not exceed that of the base case.

![Figure 8](image)

Figure 8. a) Hazard Curve for Salt Lake City, UT for \( T_1=1.40 \) s. for DBSL and RSL conditions, and b) Fragility curves for the four CCFT evaluated conditions.

**Conclusions**

The study assesses the effect of partial concrete compressive strength on the seismic performance of CCFT columns prior to full curing time. If the bridge needs to be open to traffic after a few days, the time required for the columns’ concrete to meet the design compressive strength creates a temporary condition. The study evaluates whether this temporary condition increases the seismic probability of failure.

The first phase of the study addresses the design of CCFT columns. These components may require a third less cross-sectional area than the original reinforced concrete columns to achieve similar capacity under combined axial and bending forces. Because of the highly localized failure mode of CCFT columns, concentrated plasticity models can reliably predict the nonlinear performance of these components up to the collapse limit state. This study calibrates for first time the deteriorating nonlinear parameters required for these numerical simulations. It is observed that the gain in concrete strength is not as critical for CCFT columns as in reinforced concrete columns. Concrete only reaches about 40% of its design strength after 3 days, where the CCFT column achieves 92% and 68% of its full pure moment capacity and full pure axial capacity, respectively, on day 3. Thus, the steel tube is largely responsible for initial capacity of the CCFT, as long as the concrete provides lateral constraint preventing buckling failure.

Based on the IDA study, the collapse capacity of CCFT columns at 3, 7, and 14 days correspond to 80, 93, and 98% of the CCFT collapse capacity at 28 days, when the concrete reaches its design compressive strength. If a conservative temporary condition of one month is assumed for CCFT columns with less than one month of curing, the probability of failure for CCFT columns at 3, 7, and 14 days is about one order of magnitude smaller than that for the CCFT column over the lifespan of 75 years.
Acknowledgments

The authors are grateful to the University of Utah and to the Mountain-Plains Consortium for the funding provided for this research.

References

17. Ibid.
19. See [16].
20. See [18].
21. See [18].
23. See [13].
27. See [8].
28. See [8].
29. See [6].