7.10 Concrete-Filled Tubes

7.10.1 Scope

This section shall be taken to supersede AASHTO LRFD and AASHTO Seismic requirements for concrete-filled tubes (or pipes). The use of concrete-filled tubes (CFT) and reinforced concrete-filled tubes (RCFT) for bridge foundations requires approval from the WSDOT Bridge Design Engineer. CFT and RCFT shall not be used for bridge columns including extended-pile columns, and they shall not be utilized as the ductile elements of an earthquake resisting system.

CFT and RCFT have been shown to offer strength and stiffness beyond a conventional reinforced concrete (RC) member. And recent research has shown that CFT members can sustain large cyclic drifts with minimal damage. The design methods herein regarding concrete-filled tubes are largely based on study, testing and recommendations compiled by the University of Washington (UW).

The concrete for CFT members tested at the UW was a low-shrinkage, self-consolidating concrete. The nominal concrete strengths were 6 ksi and 10 ksi. This represents structural concrete with a minimum specified strength of 4 ksi, and an expected strength 25 percent to 50 percent larger.

Prior CALTRANS and ARMY research programs studied two types of fully restrained connections for CFT pier to foundation connections. One of those two connections is readily usable as a CFT-to-cap connection. An annular ring is attached to the top of the CFT, and it is partially embedded into the pile cap. This anchored connection resists flexural loading from the pile through strutting action to the bottom of the pile cap (resulting from the portion of tube of the CFT that is in tension) and the top of the pile cap (resulting from the portion of tube of the CFT column that in compression). The tests show this connection is both simple to construct and fully effective in transferring flexure. The current ACI procedure (ACI 318-2011) was recommended by the UW as a conservative approach to design against punching shear in this type of connection.

Transition connections between RC shafts and CFT shafts have not been tested, but considerable analysis has been performed at the UW. Models have been developed to predict the strength of RCFT members, and this RCFT behavior may be used to provide increased strength over a significant length of the pile relative to conventional RC construction. Overstrength factors for capacity protection design of adjacent members and joint shear design at connections were not addressed in the research.

7.10.2 Design Requirements

A. Materials

1. The concrete for CFT and RCFT shall be class 4000P. A reduced compressive design strength of $0.85f'_c$ shall be used for wet placed concrete. Low shrinkage concrete shall be required to ensure the concrete does not shrink relative to the steel tube.

2. Steel tubes shall conform to one of the following:
   i. API 5L Grade X42 or X52 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
   ii. ASTM A 252 Grade 2 or 3 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
   iii. ASTM A 572 or ASTM A 588 for longitudinal seam welded tube
3. For capacity protected members at the extreme event limit state, expected material properties may be used to determine the expected nominal moment capacity. The expected yield strength, F_{ye}, for steel tubes shall be taken as $1.1F_y$.

B. Limit States

For strength limit states, the resistance factors for axial load effects on CFT shall be taken per AASHTO LRFD for tension- and compression- controlled reinforced concrete sections. The resistance factor for shear shall be taken as 0.85. For extreme event limit states, resistance factors shall be taken as 1.0.

C. General Dimensions

The minimum tube wall thickness shall not be taken less than 3/8 inch at the time of installation. To develop the full plastic capacity of CFT or RCFT members, it is necessary to ensure that local buckling does not occur prior to development of the strength of the tube. Therefore the following *D/t* limits are recommended:

1. For members subjected primarily to flexural loading:

$$\frac{D}{t} \leq 0.22\frac{E}{F_y} \qquad (7.10.2\text{-}1)$$

2. For members subjected primarily to axial loading:

$$\frac{D}{t} \leq 0.15\frac{E}{F_y} \qquad (7.10.2\text{-}2)$$

Where D is the outside diameter of the tube (in.), and t is the wall thickness of the tube (in.).

D. Stiffness

The effective stiffness, EI_{eff}, of circular CFT, as defined in Equation 7.10.2-3, shall be used to evaluate deflections, deformations, buckling resistance, and moment magnification. The effective stiffness factor, *C*', is defined in Equation 7.10.2-4.

$$EI_{eff} = E_s I_s + C' E_c I_c \qquad (7.10.2\text{-}3)$$

$$C' = 0.15 + \frac{P}{P_0} + \frac{A_s}{A_s + A_c} \leq 0.9 \qquad (7.10.2\text{-}4)$$

P_0 is the nominal compressive resistance without moment, P is the factored axial load effect, and A_s is the combined area of the steel tube and steel reinforcing.

E. Flexure and Axial Resistance

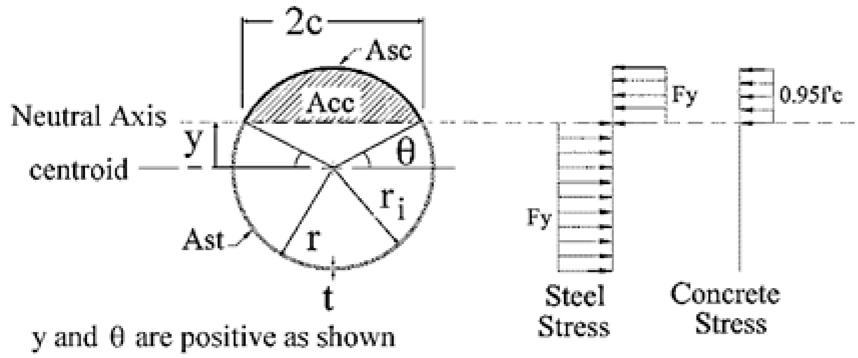
The resistance factor for flexure shall be taken as 0.90 at strength limit states.

The flexural strength of CFT and RCFT members may be determined using the plastic stress distribution method (PSDM). The appropriate limit state stresses and geometry is shown in Figure 7.10.2-1.

Solutions for the interaction diagrams can be developed using parametric equations for $P(y)$ and $M(y)$ where y is the distance from the centroid to the neutral axis. A positive value of P is a net compressive force. M and y are positive with the sign convention shown in Figure 7.10.2-1. The parameter y varies between plus and minus r_i, where r_i is the radius of the concrete core.

Stress is assumed to be plastically developed over the following regions of the section:

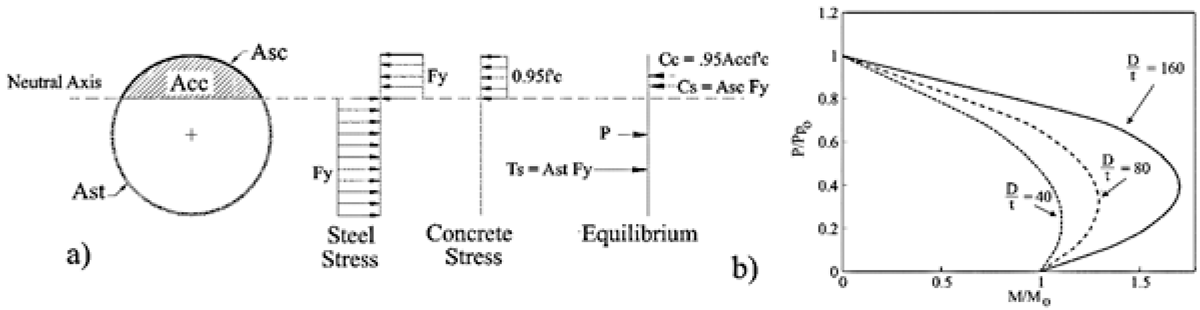
A_{cc} = area of concrete effective in compression
A_{sc} = area of the steel tube in compression
A_{st} = area of the steel tube in tension
A_{bc} = area of the internal steel reinforcing in compression
A_{bt} = area of the internal steel reinforcing in tension



Plastic Stress Distribution Method
Figure 7.10.2-1

Alternatively, a strain-compatibility analysis can be performed with appropriate plastic stress-strain relationships.

1. **CFT Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-2 and Equations 7.10.2-5 through 7.10.2-9. Figure 7.10.2-2b also shows normalized interaction curves for various D/t ratios.



Plastic Stress Distribution for CFT
Figure 7.10.2-2

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^{\,2} - yc\right) * 0.95f'_c - 4\theta t r_m F_y \qquad (7.10.2\text{-}5)$$

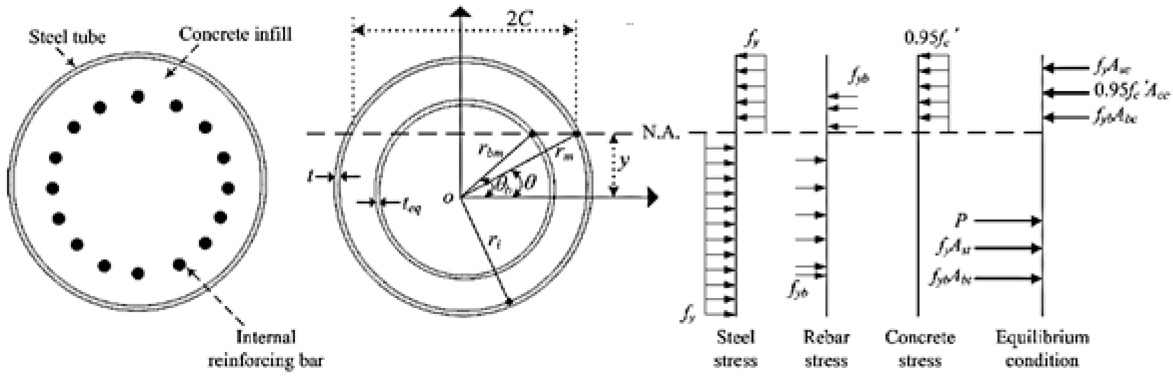
$$M_n(y) = \left(c(r_i^{\,2} - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^{\,2}}{r_i}F_y \qquad (7.10.2\text{-}6)$$

$$c = r_i cos\theta \qquad (7.10.2\text{-}7)$$

$$\theta = sin^{-1}\left(\frac{y}{r_m}\right) \qquad (7.10.2\text{-}8)$$

$$r_m = r - \frac{t}{2} \qquad (7.10.2\text{-}9)$$

2. **RCFT Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-3 and Equations 7.10.2-7 through 7.10.2-14. The internal steel reinforcing is idealized as a thin ring.



Plastic Stress Distribution for RCFT
Figure 7.10.2-3

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

The associated variables are defined as:

r = radius to the outside of the steel tube (in)
r_i = radius to the inside of the steel tube (in)
r_m = radius to the center of the steel tube (in)
r_{bm} = radius to the center of the internal reinforcing bars (in)
t = wall thickness of the tube (in)
t_b = wall thickness of a notional steel ring equivalent to the internal reinforcement (in)
c = one half the chord length of the tube in compression (in)
c_b = one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)
θ = angle used to define c (rad.)
θ_b = angle used to define c_b (rad.) θ_b shall be taken as $\pi/2$ if y/r_b is greater than 1 and θ_b shall be taken as $-\pi/2$ if y/r_b is less than -1.
A_b = area of a typical steel bar comprising the internal reinforcement (in^2)
n = number of internal steel reinforcing bars

The requirements of AASHTO Seismic 8.16.2 for piles with permanent steel casing shall be applied to RCFT. Accordingly, the extent of longitudinal reinforcement may be reduced to only the upper portion of the member as needed to provide the required resistance of the member.

For CFT and RCFT, the area of the steel casing shall be included in the determination of the longitudinal reinforcement ratio. For RCFT, the minimum required longitudinal reinforcement ratio may be reduced to 0.005.

A. Stability Considerations for Unbraced of Partially-braced Members

Piles and shafts are typically assumed to be continually braced by the surrounding soil. Therefore they are not normally subject to P-Δ effects or other secondary effects. However, it is recognized that special circumstances such as scour, soil liquefaction, or other conditions may leave piles and shafts subject to less than full bracing. In these circumstances, it may be necessary to consider stability effects.

B. Shear Resistance

The shear resistance of CFT and RCFT shall be taken as:

$$V_u = \phi V_n = \phi g_4\left[g_1\left(0.6 f_y g_2 A_s\right) + 0.0316 g_3 A_c \sqrt{f_c'}\right] \qquad (7.10.2\text{-}15)$$

Where:

- A_s = *cross-sectional area of the steel tube*
- A_c = *area of concrete within the steel tube*
- g_1 = *coefficient for the shear capacity of the steel tube = 2.0*
- g_2 = *coefficient for the effective shear area of steel tube = 0.5*
- g_3 = *coefficient for the effect on concrete strength in shear due to confinement from the steel tube = 3.0*
- g_4 = *coefficient for bond development between the concrete and steel tube = 1.0*

C. Corrosion

The design wall thickness for tubes shall be reduced for corrosion over a 75-year minimum design life. Minimum corrosion rates are specified below, except that the design thickness loss due to corrosion shall not be taken to be less than 1/16 inch.

Soil embedded zone (undisturbed soil):	0.001 inch per year
Soil embedded zone (fill or disturbed soils):	0.003 inch per year
Immersed Zone (fresh water):	0.002 inch per year
Immersed and Tidal Zone (salt water):	0.004 inch per year
Splash Zone (salt water):	0.006 inch per year
Atmospheric Zone:	0.004 inch per year

The corrosion rates are taken from July 2008 CALTRANS memo to Designers 3-1, FHWA NHI-05-042 Design and Construction of Driven Pile Foundations, and the *Ferries Terminal Design Manual*.

The rates for corrosion in soil above assume that the soil is not highly corrosive. A site-specific assessment should be considered where a corrosive soil environment is suspected or known to exist. The potential for scour shall be considered when choosing a design corrosion rate.

7.10.3 CFT-to-Cap Connections

CFT-to-cap connections shall be designed as fully-restrained connections capable of resisting all load effects. The preferred connection to a concrete cap includes an annular ring at the top of the embedded tube. The connection design involves:

A. Design of the annular ring

B. Determination of the embedment depth

C. A punching shear evaluation in the cap

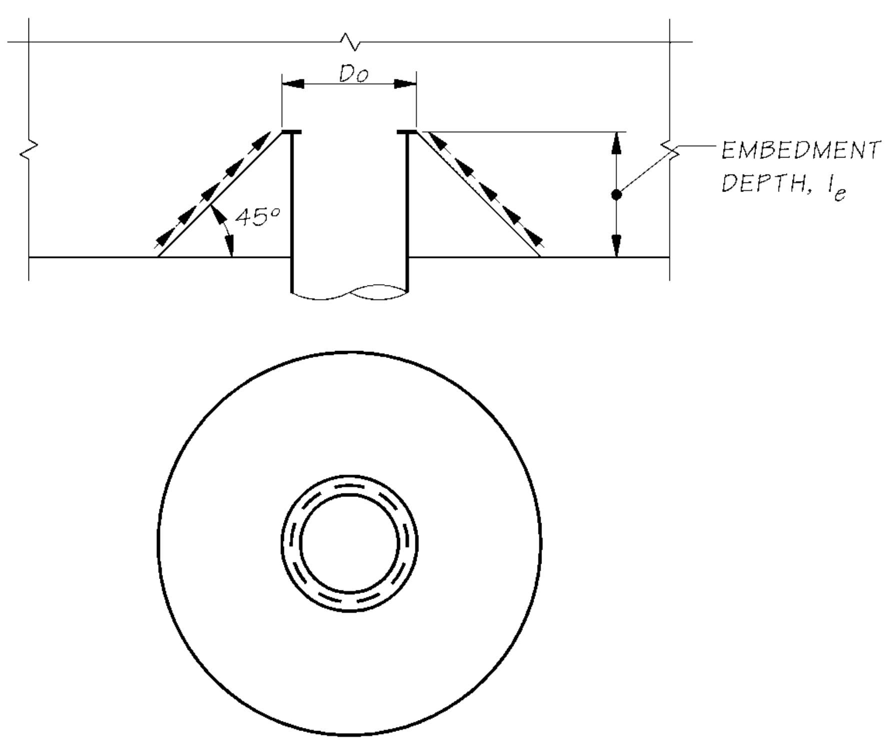
D. General design of the cap for flexure and shear

An alternative to the annular ring connection involves using a conventional reinforcing cage to splice the CFT to the cap.

E. Reinforced concrete connection design

A. Annular Ring

An annular ring shall be welded to the end of the tube to provide anchorage and stress distribution, as shown in Figure 7.10.3-1. The ring shall be made of a steel of the same thickness and grade as the steel tube. The ring shall extend outside and inside the tube a distance of 8*t*, where t is the thickness of the tube.

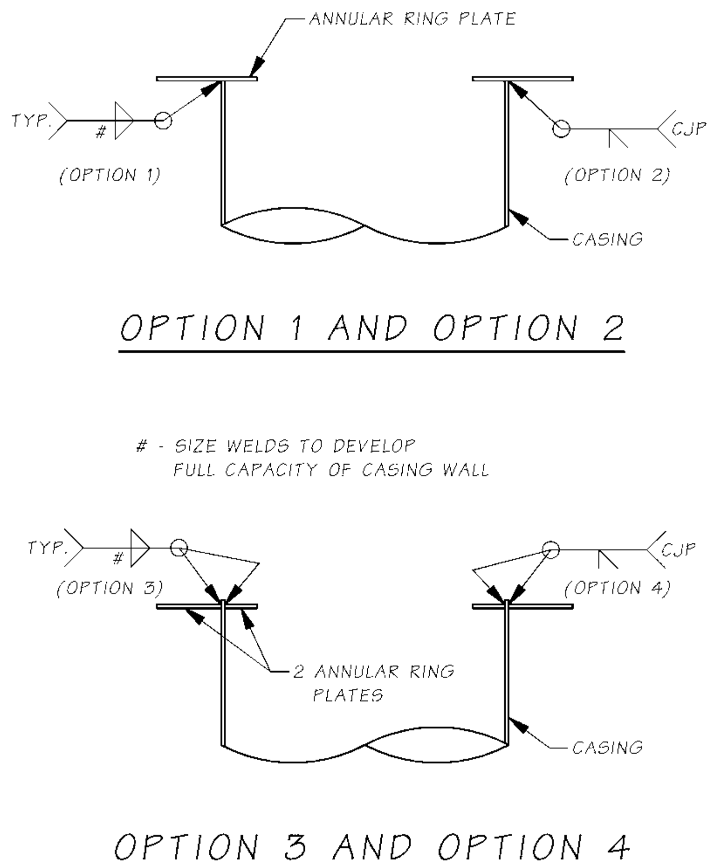


Cone Pullout Mechanism for Cap Connections
Figure 7.10.3-1

The ring shall be welded to the tube with complete joint penetration (CJP) welds or fillet welds on both the inside and outside of the tube. The fillet welds must be capable of developing the full tensile capacity of the tube. For this purpose, the minimum size, w, of the fillet welds shall be taken as:

$$w \geq \frac{1.33 F_u t}{F_{exx}} \tag{7.10.3-1}$$

Where F_u is the specified minimum tensile strength of the tube steel (ksi), and F_{exx} is the classification strength of the weld metal (ksi). Typical CFT weld details are shown in Figure 7.10.3-2.



Annular Ring Weld Detail
Figure 7.10.3-2

B. Embedment

The tube and the annular ring shall be embedded into the pile cap with adequate embedment depth to ensure ductile behavior of the connection. The minimum embedment length, l_e, shall satisfy:

$$l_e \geq \sqrt{\frac{D_o{}^2}{4} + \frac{3.95DtF_y}{\sqrt{f'_{cf}}}} - \frac{D_o}{2} \tag{7.10.3-2}$$

Except that for capacity protected connections, the embedment length shall satisfy:

$$l_e \geq \sqrt{\frac{D_o{}^2}{4} + \frac{5.27DtF_u}{\sqrt{f'_{cf}}}} - \frac{D_o}{2} \tag{7.10.3-3}$$

Where f'_{cf} (ksi) is the specified 28-day compressive strength of the cap, D_o is the outside diameter of the annular ring as shown in Figure 7.10.3-1.

C. Punching Shear

The pile cap shall have adequate concrete depth, h, above the steel tube to preclude punching through the pile cap. The value of h shall be greater than or equal to 32 times the thickness of the steel casing, and shall also satisfy:

$$h \geq \sqrt{\frac{D_0{}^2}{4} + \frac{5.27C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} - l_e \tag{7.10.3-4}$$

Where the total compressive force of the couple, C_{max}, shall be taken as:

$$C_{max} = C_c + C_s \tag{7.10.3-5}$$

C_c and C_s are the compression forces in the concrete and the steel due to the combined bending and axial load as computed by the plastic stress distribution method for the most extreme load effect at the appropriate limit state.

D. Pile Cap Reinforcement

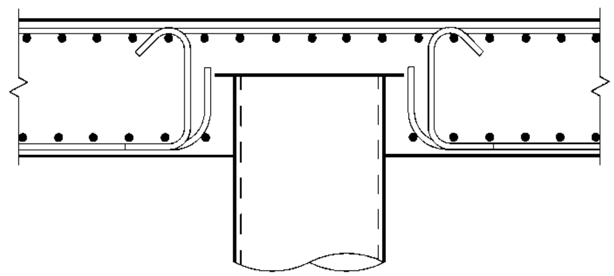
The pile cap should follow conventional design practice and must be adequate to sustain the foundation design loads. However, the concrete cap thickness shall be large enough to preclude punching shear and cone pullout of the CFT piles. The minimum concrete cap thickness, d_f, shall be taken as:

$$d_f \geq h + l_e \tag{7.10.3-6}$$

The edge distance shall be large enough to accommodate concrete struts oriented 60 degrees from the vertical originating at the base of the ring. The minimum edge distance, de, measured from center-of-tube to the edge of the cap shall be taken as:

$$d_e \geq D \tag{7.10.3-7}$$

CFTs shall be adequately spaced to avoid intersecting concrete struts. The cap shall be designed to resist all flexural load effects. The flexural reinforcement in both directions shall be spaced uniformly across the length and width of the cap, but the bottom mat of flexural reinforcement will be interrupted by the concrete tube. The interrupted bars shall be provided, but they shall not be relied on to contribute to the flexural resistance of the cap. Figure 7.10.3-3 shows the configuration of the longitudinal reinforcing where it conflicts with the steel tube. Standard 90° hooks shall be used.



Reinforcement Detail at Cap Connection
Figure 7.10.3-3

The cap shall be designed to resist all shear load effects. Note that the minimum required embedment results in an average shear stress in the critical area surrounding the tube of $6\sqrt{f'_c}$ (psi). Assuming the concrete is capable of resisting a shear stress of approximately $2\sqrt{f'_c}$, vertical reinforcement will be required to resist an average shear stress of approximately $4\sqrt{f'_c}$. Additional requirements for shear demand resulting from other load combinations must also be considered.

Additionally, vertical ties shall be provided within the anchorage regions such that at least two vertical ties intersect the pull-out cone depicted in Figure 7.10.3-1 on each side of the CFT subject to shear. Therefore vertical ties shall be placed in the region within $1.5l_e$ of the outside of the tube, and shall be placed at a maximum spacing s, taken as:

$$s \leq \frac{l_e}{2.5} \qquad (7.10.3\text{-}8)$$

E. Reinforced Concrete Connection

A circular reinforcing cage may be used to connect a CFT member to a concrete cap, where the steel casing is discontinued just below the cap. The reinforcing cage shall satisfy all of the requirements for a reinforced concrete connection, as well as the additional requirements in this section. Transverse reinforcing shall be used.

The minimum embedment length, l_e, of the reinforcing cage into the cap shall satisfy:

$$l_e \geq \frac{\psi_e F_{yb}}{2\sqrt{f'_{cf}}} d_b \tag{7.10.3-9}$$

$$l_e \geq \sqrt{\frac{D^2}{4} + \frac{2F_{yb}A_{st}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \tag{7.10.3-10}$$

Where ψ_e is a coating factor, which shall be taken as 1.0 for uncoated bars, and 1.2 for epoxy-coated bars.

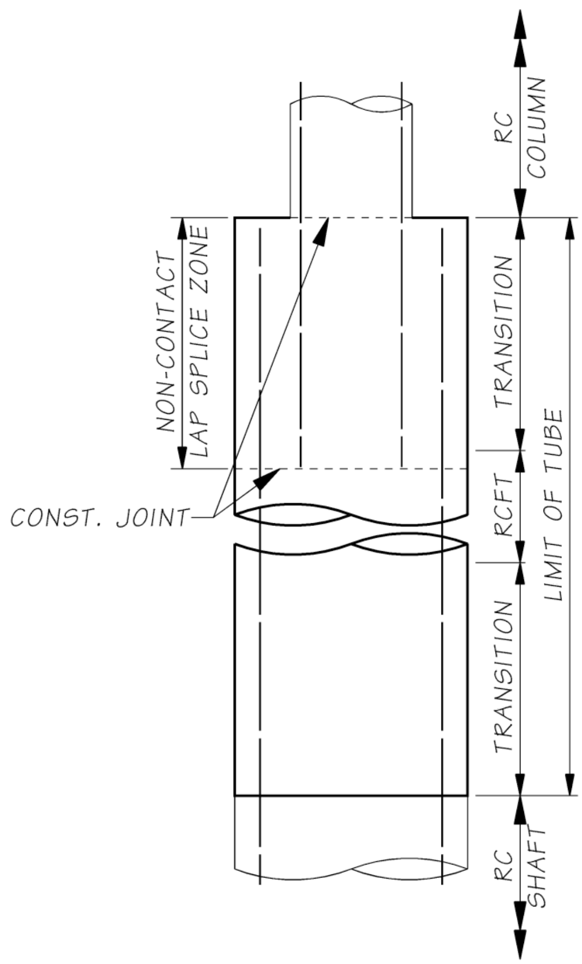
The reinforcing cage shall extend into the CFT at least a distance of $2l_d$ below the top of the steel casing, where l_d is the development length of the longitudinal reinforcing.

The concrete cover above headed longitudinal reinforcing shall exceed $3d_h$, where d_h is the diameter of the head. The concrete side cover adjacent to a head shall exceed d_h.

7.10.4 RCFT-to-Column Connections

Direct RCFT-to-column connections shall be designed as fully-restrained connections capable of resisting all load effects. The recommended RCFT shaft to reinforced concrete column connection is shown in Figure 7.10.4-1.

All column reinforcement shall be extended into the RCFT shaft for a length greater than or equal to the length required for noncontact lap splices between columns and shafts. The contribution of steel casing to the structural resistance of RCFT's varies from zero at the end of the tube to fully composite at the end of the transition zone. The transition zone length may be taken as 1.0D. The use of slip casing in determining the resistance for RFCT shafts is not permitted.



RCFT-to-Column Connection
Figure 7.10.4-1

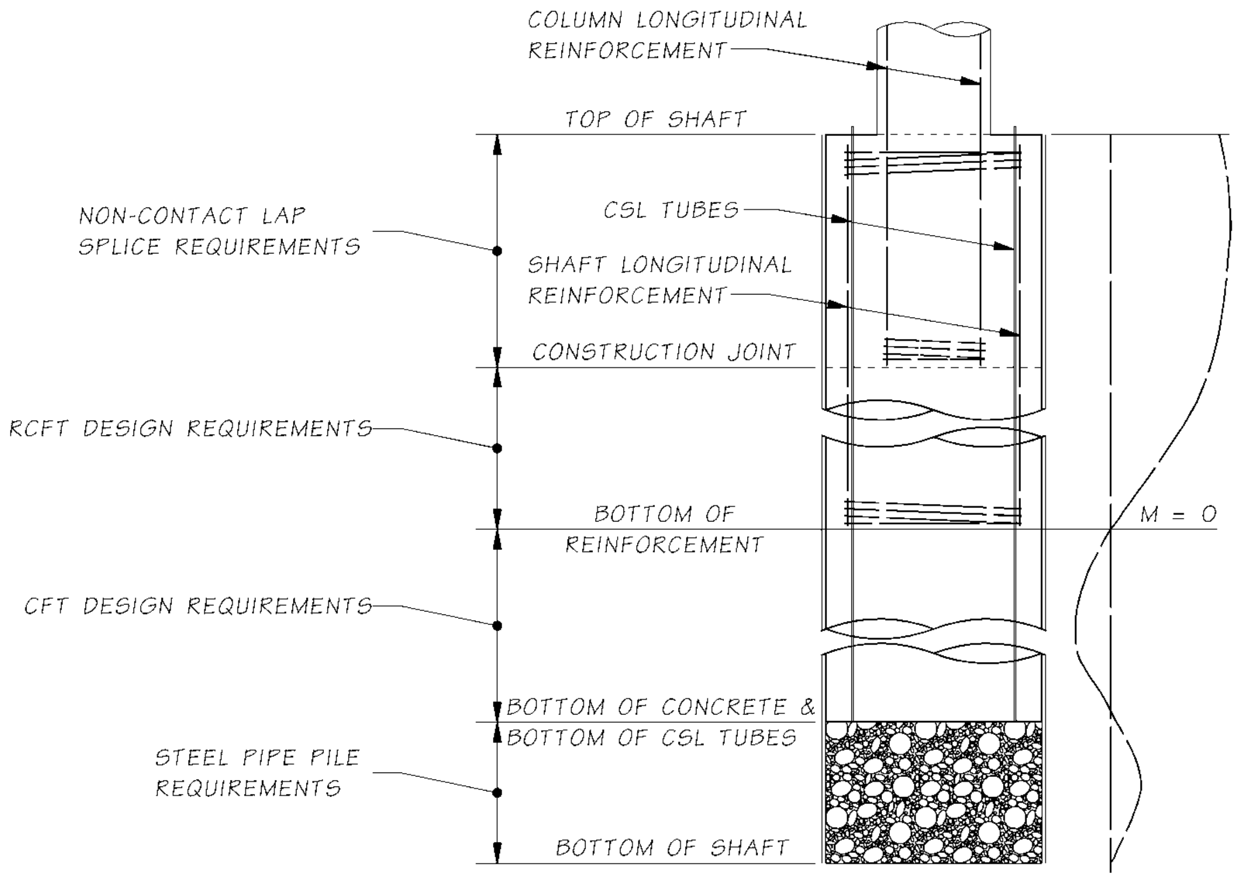
7.10.5 Partially-filled CFT

The use of partially-filled steel tubes for bridge foundations requires the approval of the WSDOT Bridge Design Engineer, and will only be used where conventional CFT members are grossly uneconomical or unconstructible.

Design zones of partially filled steel piles and shafts are shown in Figure 7.10.5-1. Longitudinal and transverse reinforcement shall extend to at least the first point of zero moment along the member under the peak loading condition.

Crosshole sonic log (CSL) testing shall be performed in accordance with *Standard Specifications* Section 6-19.3(9). CSL tubes shall extend to the bottom of concrete.

Corrosion losses shall be considered on each exposed surface of the steel tube.



Partially-filled CFT
Figure 7.10.5-1

7.10.6 Construction Requirements

For CFT with tubes installed open-ended, the insides of the tube shall be cleaned with an appropriate tool to remove all adhering soil and other material.

Welding for ASTM A 252 pipe shall conform to AWS D1.1/D1.1M, latest edition, Structural Welding Code, except that all weld filler metal shall be low hydrogen material selected from Table 4.1 in AASHTO/AWS D1.5M/D1.5:2010 *Bridge Welding Code*. All seams and splices shall be complete penetration welds.

Welding and joint geometry for the seam shall be qualified in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The Contractor may submit documentation of prior qualification to the Engineer to satisfy this requirement.

For the fabrication of helical (spiral) seam submerged-arc welded pipe piles, the maximum radial offset of strip/plate edges shall be 1/8 inch. The offset shall be transitioned with a taper weld and the slope shall not be less than a 1-to-2.5 taper. The weld reinforcement shall not be greater than 3/16 inches and misalignment of weld beads shall not exceed 1/8 inch.

If spirally welded pipe piles are allowed, skelp splices shall be located at least 1′-0″ away from the annular ring.

Nondestructive evaluation (NDE) requirements for field welded splices shall be identified on the plans. The location of splices and NDE requirements shall be divided into 3 possible zones as determined by the Engineer:

1. No splices permitted – highly stressed areas
2. Splices permitted with 100 percent UT and visual inspection – moderately stressed areas
3. Splices permitted with 100 percent visual inspection – low stressed areas

7.10.7 Notation

A_b = area of a single bar for the internal reinforcement (in2)
A_c = net cross-sectional area of the concrete (in2)
A_g = cross-sectional area of the steel tube (in2)
A_s = cross-sectional area of the steel tube and the longitudinal internal steel reinforcement (in2)
c = one half the chord length of the tube in compression (in)
c_b = one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)
D = outside diameter of the tube (in.)
D_o = outside diameter of the annular ring (in.)
d_b = nominal diameter of a reinforcing bar (in)
d_e = minimum edge distance from center of CFT to edge of cap (in)
d_f = depth of cap (in)
E_c = elastic modulus of concrete (ksi)
EI_{eff} = effective composite flexural cross-sectional stiffness of CFT or RCFT (k-in2)
E_s = elastic modulus of steel (ksi)
F_{exx} = classification strength of weld metal (ksi)
F_u = specified minimum tensile strength of steel (ksi)

F_y = specified minimum yield strength of steel (ksi)
F_{yb} = specified minimum yield strength of reinforcing bars used for internal reinforcement (ksi)
f_c' = minimum specified 28-day compressive strength of concrete (ksi)
f'_{cf} = minimum specified 28-day compressive strength of concrete in a cap or footing (ksi)
g_1 = coefficient for the shear capacity of the steel tube
g_2 = coefficient for the effective shear area of steel tube
g_3 = coefficient for the effect on concrete strength in shear due to confinement from the steel tube
g_4 = coefficient for bond development between the concrete and steel tube
h = cap depth above the CFT required to resist punching shear in a cap (in)
I_c = uncracked moment of inertial of the concrete about the centroidal axis (in4)
I_s = moment of inertia of the steel tube and the longitudinal internal steel reinforcement about the centroidal axis (in4)
l_{db} = Basic tension development length of a bar (in)
l_e = Required embedment length for CFT embedded in a concrete cap (in) M(y) = nominal moment resistance as a function of the parameter y (kip-in) Mo = plastic moment resistance of members without axial load (kip-in)
n = number of equally spaced longitudinal internal steel reinforcement
$P(y)$ = nominal compressive resistance as function of the parameter y (kips)
P_u = factored axial load acting on member (kip)
P_o = compressive resistance of a member without consideration of flexure (kips)
r = radius to the outside of the steel tube (in)

7.11 Bridge Standard Drawings

7.12 Appendices

Appendix 7.3-A1	Column Silo Cover
Appendix 7.4-A1	Noncontact Lap Splice Length Column to Shaft Connections
Appendix 7-B1	Linear Spring Calculation Method II (Technique I)
Appendix 7-B2	Pile Footing Matrix Example Method II (Technique I)
Appendix 7-B3	Non-Linear Springs Method III

7.99 References

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