Washington State Department of Transportation
Bridge and Structures Office

DESIGN MEMORANDUM

TO: All Design Staff
FROM: Bijan Khaleghi
DATE: October 7, 2012
SUBJECT: Structural Design Recommendations of CFT and RCFT for Bridge Foundation

This design memorandum allows steel casing to be considered in the structural capacity of piles, shafts, and connections of pile-to-pile cap and column-to-shaft Foundation. This memorandum provides guidance for structural design of concrete filled steel tubes (CFT) and reinforced concrete filled steel tubes (RCFT) for bridges and other structures foundations. Use of CFT and RCFT requires approval from the WSDOT Bridge Design Engineer. The design approach is as follows:

1. Establish the demands using an appropriate model of the bridge including soil-structure interaction.
2. Establish the material properties for the concrete and steel tube.
3. Determine an initial size of tube to meet constructability, geometric limitations (D/t ratio) and a target axial stress ratio.
4. Design the shaft/pile for combined flexure and axial loading using a P-M interaction curve by means of the plastic stress distribution method that incorporates buckling.
5. For column connections, determine the required development length of the reinforcement into the RCFT shaft. For pile/shaft cap connections, determine the required embedment of the CFT into the cap.

The step-by-step design procedure for structural design of CFT and RCFT for bridge foundation applications is as follows:

Step 1: Establish Material Properties

CFT piles, RCFT shaft, and their connections require three types of materials, including the concrete, the steel tube, and the reinforcing steel. The reinforcing steel used in the foundation or cap-beam element shall be in accordance with WSDOT STD Specification Section 9-07. The steel casing shall be in accordance with WSDOT STD Specifications Section 9-36.1(1). Yield strength of the steel casing shall be specified in the Contract Plans. The yield strength beyond 36 ksi may be used if advantageous in design. The concrete for CFT and RCFT shall be class 4000P. The compressive strength reduction factor of $0.85f'_c$ shall be considered for wet placed concretes.

Step 2: Establish Initial Tube Geometry

There are three different aspects of CFT/RCFT that should be considered when selecting the initial cross section of the tube. First, minimum thickness values may be required for constructability such as driving or corrosion. The minimum thickness shall not be taken less than 3/8 inch at the time of installation. Second, it may be desirable to limit the axial stress on the member during seismic loading. Commonly axial stresses under earthquake loading are limited to 10-20% of the gross capacity. And finally, tubes with very high D/t ratio may compromise their capacity, where as tubes with low D/t ratios may not develop their strength. Therefore the recommended D/t limit is encouraged. 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. This is accomplished by providing an upper limit on the D/t ratio. As a result, the recommended slenderness limit for:

- CFT or RCFT subjected primarily to flexural loading is:

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

- CFT and RCFT subjected primarily to axial loading is:

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

Note: it is recommended that these D/t ratios are satisfied even though design of CFT and RCFT elements will still typically be limited to elastic capacities.

Step 3: Stiffness Models For CFT And RCFT

The effective stiffness, EI_{eff}, of circular CFT/RCFT as shown in Eq. 3a is used to evaluate deflections, deformations, buckling capacity, and moment magnification. This effective stiffness factor is termed *C'* shown in Eq. 3b.

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

$$C' = 0.15 + \frac{P}{P_0} + \frac{A_S}{A_S + A_C} < 0.9 \qquad (3b)$$

P_0, the ultimate axial crushing load, is defined in Eq. 6c, and A_s is the combined area of the steel tube and steel reinforcing.

Step 4: Flexural Strength and Material Interaction Curve

The flexural strength of CFT and RCFT members is determined using the plastic stress distribution method (PSDM), shown in Fig. 1. This method can be used to generate a full P-M interaction curve, as shown in Figure 1b. For each neutral axis depth, pairs of axial and bending resistances can be determined. Figure 1b shows the resulting material interaction curve where the values on the moment (x) axis and axial load (y) axis are normalized to the flexural strength without axial load (M_0) and the axial crush load without moment (P_0) of the member, respectively. Smaller D/t values result in larger resistance, because the area of steel is larger.

Larger D/t ratios result in significantly increased bending moment for modest compressive loads, because of the increased contribution of concrete fill as shown in Fig. 1b.

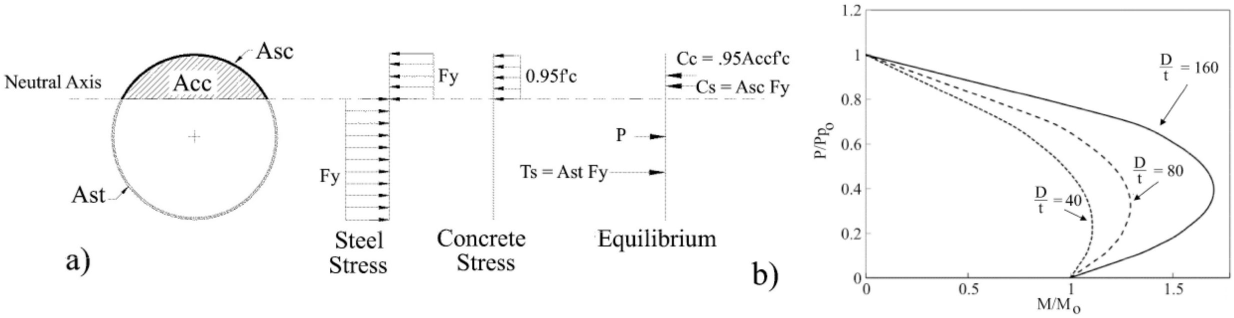

Fig 1. Strength determination for CFT; a) Plastic stress distribution method, b) Material-based interaction curves (no buckling)

A closed-form solution for the interaction curve for CFT piles has been derived based upon the geometry described in Fig. 2 and is expressed in Equations 4.

$$r_m = r - \frac{t}{2} \tag{4a}$$

$$\theta = \sin^{-1}\left(\frac{y}{r_m}\right) \tag{4b}$$

$$c = r_i \cos\theta \tag{4c}$$

$$P = F_y t r_m\left\{(\pi - 2\theta) - (\pi + 2\theta)\right\} + \frac{0.95 f'_{ct}}{2}\left\{(\pi - 2\theta) r_i^2 - 2yc\right\} \tag{4d}$$

$$M = 0.95 f'_{ct} c\left\{\left(r_i^2 - y^2\right) - \frac{c^2}{3}\right\} + 4F_y tc\frac{r_m^2}{r_i} \tag{4e}$$

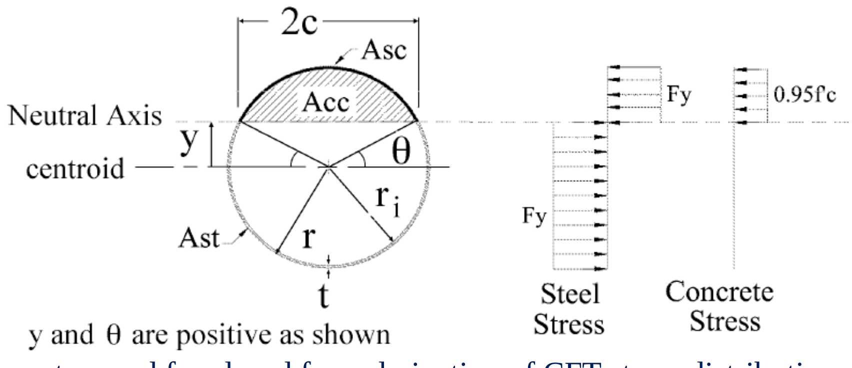

Fig. 2. Geometry used for closed form derivation of CFT stress distribution prediction

For RCFT members, a similar material interaction curve is expressed by Eqs. 5 with slightly revised geometry as illustrated in Fig. 3.

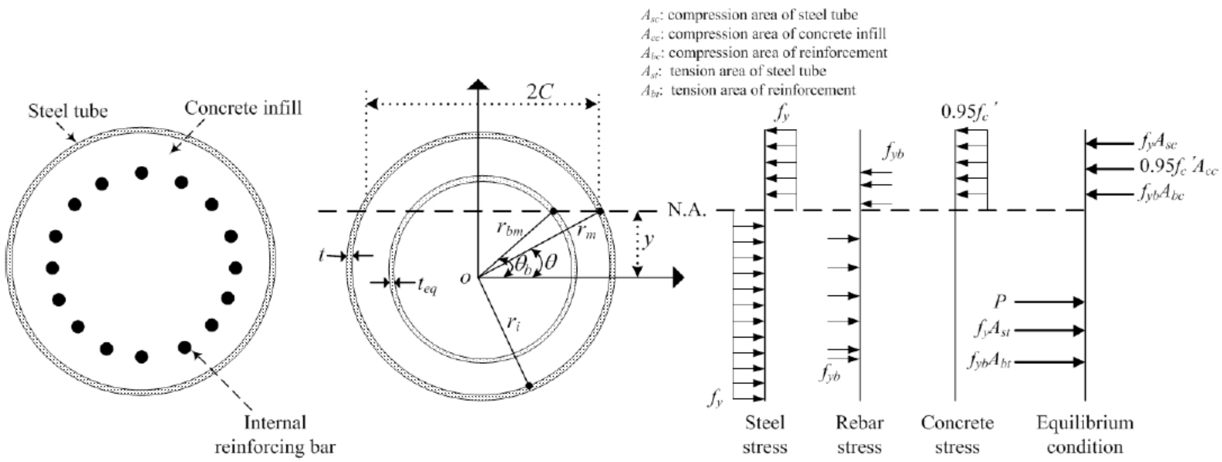

Fig. 3. Geometry for derivation of plastic stress distribution method for RCFT

$$r_m = r - \frac{t}{2} \tag{5a}$$

$$\theta_s = \sin^{-1}\left(\frac{y}{r_m}\right) \quad \text{and} \quad \theta_b = \sin^{-1}\left(\frac{y}{r_b}\right) \tag{5b}$$

$$c = r_i \cos\theta_s \ \text{ and } \ c_b = r_b \cos\theta_b \tag{5c}$$

$$t_b = \frac{n A_b}{2\pi r_b} \tag{5d}$$

$$P = F_{ys} t\, r_m \left\{(\pi - 2\theta_s) - (\pi + 2\theta_s)\right\} + t_b r_b \left\{F_{yb}(\pi - 2\theta_b) - \left(F_{yb} - .95 f_c'\right)(\pi + 2\theta_b)\right\} + \frac{0.95 f_c' c}{2}\left\{(\pi - 2\theta_s) r_i^2 - 2yc\right\} \tag{5e}$$

$$M = 0.95 f_c' c \left\{\left(r_i^2 - y^2\right) - \frac{c^2}{3}\right\} + 4 F_{ys} t c \frac{r_m^2}{r_i} + 4 F_{yb} t_b c_b r_b \tag{5f}$$

A positive value of P implies a compressive force, and y and M are positive with the sign convention shown in the figures. The variable y varies between plus and minus r_i. The P-M interaction curve is generated by solving the equations for selected points in this range and connection of those points. The above equations for nominal capacities could be used for both strength and extreme limit states with corresponding resistance factors per LRFD specifications.

Step 5: Impact of Stability on P-M Interaction Curve (Option For Non-restrained CFT and RCFT)

Piles and shafts are typically assumed to be continually braced by the surrounding soil. Hence, piles and shafts are not normally affected by P-Δ effects or other secondary effects. As a result, this is not needed for most applications of CFT or RCFT as deep foundation elements. However, it is recognized that special circumstances such as scour, soil liquefaction, or other design issues may leave piles and shafts subject to stability and P- Δ effects or other secondary effects. In these circumstances, it is necessary to adjust the material interaction curve shown in Fig. 1b to account for stability and slenderness effects. To do this, it is necessary to employ the flexural stiffness, EI_{eff}, of the pier, provided in Eq. 3a.

Global column buckling is determined by the equations provided in AISC and repeated here for clarity:

$$P_{cr} = 0.658^{\frac{P_o}{Pe}} P_o \quad (6a)$$

$$P_{cr} = 0.877 P_e \quad (6b)$$

$$P_o = 0.95 f_c^{'} A_c + F_y A_s \quad (6c)$$

where P_e is the elastic buckling load by the Euler equation, and A_c and A_s are areas of the concrete and steel, respectively.

The interaction curve, including stability effects, is a modified version of the material interaction curve (Fig. 1b), which forms its basis. A series of points are jointed to form the curve. The points are as defined as follows:

- Points *A* and *B* are the axial and flexural capacity by the PSDM.
- Point *C* corresponds to the location on the PSDM interaction curve that results in the same moment capacity as point *B* but with axial load.
- Points *A'* and *C'* are obtained by multiplying the axial load associated with points *A* and *C* by the ratio, P_{cr}/P_o.
- Point *D* is located on the PSDM interaction curve and corresponds to one half of axial load which is determined for point *C'*.
- Point *A''* is the intersection of PSDM *P-M* interaction and a line parallel with the *x*-axis through the point *A'*.

The resulting curve accounts for global buckling of the tube and an example is shown in Fig. 4. The axial strength is limited to point *A'* (or the AISC buckling capacity). The stability based *P-M* interaction curve is then constructed by connecting points *A'*, *A''*, *D*, and *B*, as shown in the figure. This interaction curve should then be used for strength design of the member for all load conditions. Seismic design requires that less ductile elements be designed for the expected maximum plastic capacity of the ductile members. Given a specified axial load, the expected maximum bending moment will be 125% of the moment obtained from the interaction curve and this is the demand that should be used to design any less-ductile connecting elements.

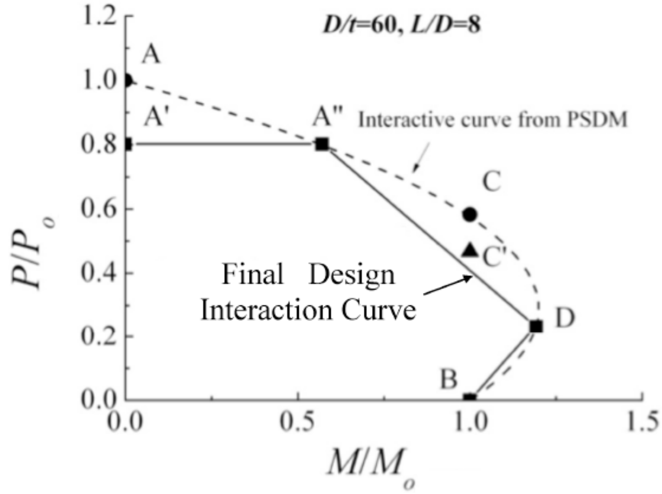

Fig. 4. Construction of the Stability-Based Interaction Curve for CFT and RCFT

Step 6: Determine the Shear Strength of CFT and RCFT Members

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

$$V_u = \phi V_n = \phi\,(V_s + 0.5{*}V_c) \qquad (7)$$

where:

V_s = Shear resistance of steel casing = $0.6F_y\ 0.5\ \pi tD$

V_c = Shear resistance of concrete = $0.0316\ \beta\ \text{SQRT}(f_c') A_c$. if P_u is compressive

ϕ = 0.85 LRFD resistance factor for shear

A_c = area of concrete within the steel casing

f'_c = compressive strength of concrete

Step 7: Pile or Shaft Connection Design

Two types of connection are considered for piles or shafts. The full strength connection, illustrated in Fig. 5, is proposed for connecting piles to pile caps. The partial strength connection, illustrated in Fig. 7, is proposed for connecting to pier columns.

Full Strength Connection

The foundation connection design for the CFT must consider several different factors. A central part of this research study is the design and detailing of the connecting CFT to the foundation or cap-beam. The connection design should include:

1. Detailing/sizing of the annular ring
2. Determination of the embedment depth

3. Punching shear evaluation
4. General design (flexure and shear) of the connecting (foundation or cap-beam) element to sustain the CFT column demands.

Detailing of Annular Ring and Embedment Depth

An annular ring is welded to the end of the tube to provide anchorage and stress distribution, as illustrated in Fig. 5. The ring is made of steel of the same thickness and yield stress as the steel tube. The ring extends outside the tube 16 times the thickness of the tube and projects inside the tube 8 times the thickness of the tube. This gives a width of the ring of 25 times the thickness of the tube, as shown figure 5.

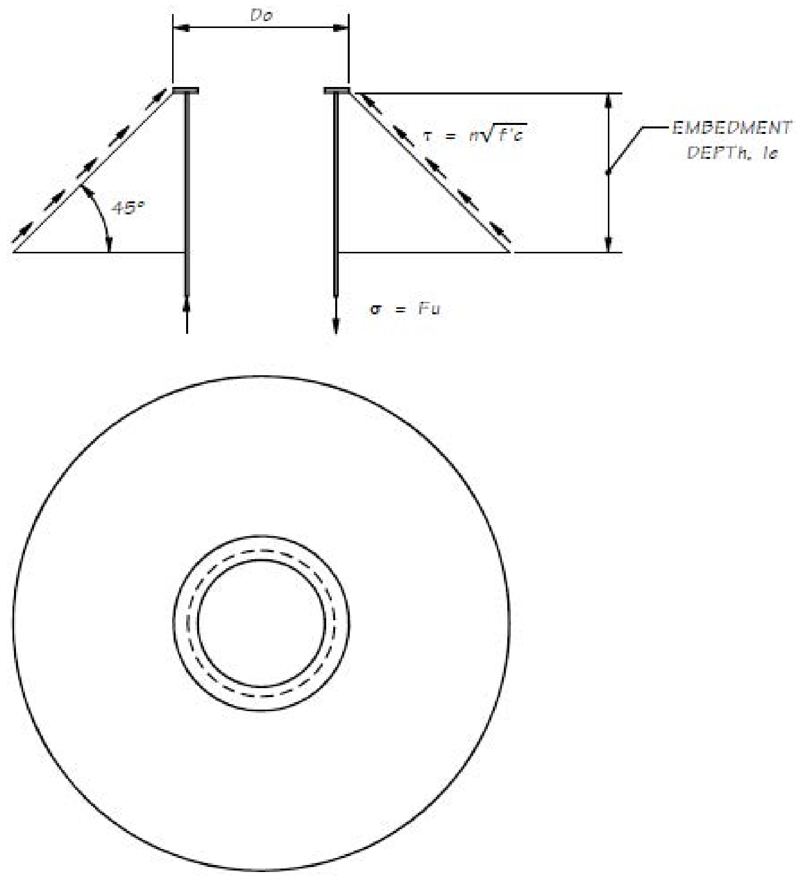

Fig. 5. Cone pullout requirements for the full strength pile cap connection

The ring is welded to the tube with complete joint penetration (CJP) welds of matching metal 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, and for this purpose the minimum weld size, w, of the fillets can be defined by Eq. 8.

$$w \geq \frac{1.47 F_u t}{F_{EXX}} \qquad (8)$$

where F_{EXX} and F_u are the minimum tensile strength of the weld metal and tube steel, respectively. If spirally welded pipe piles are allowed, skelp splices should be located at least 1’-0” away from the annular ring. The CFT weld detail is shown in Figure 6.

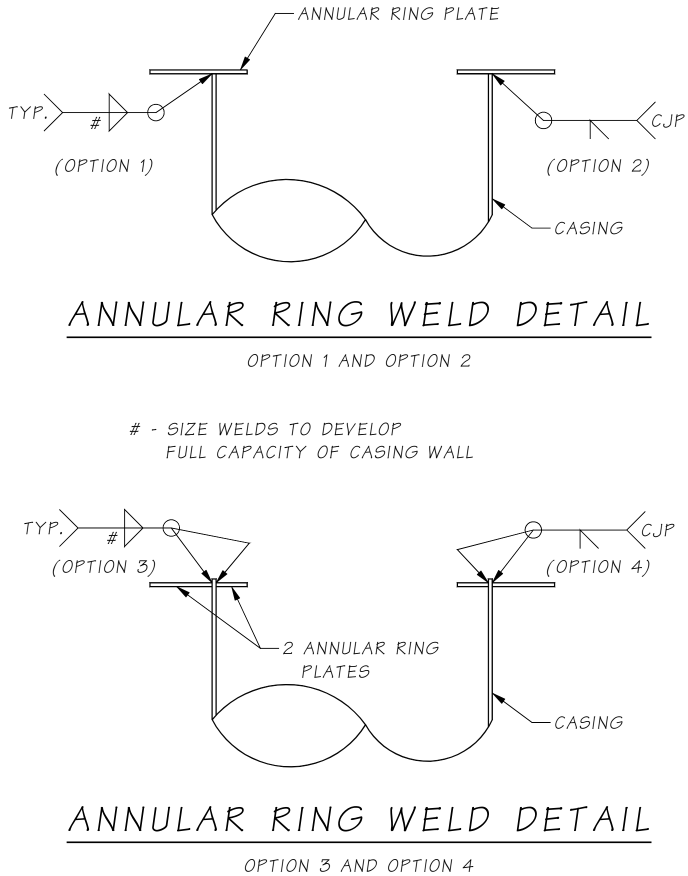

Figure 6. CFT Weld Detail

The tube and the annular ring are embedded into the pile cap with an embedment depth, l_e, needed to assure ductile behavior of the connection as depicted in Fig. 7.

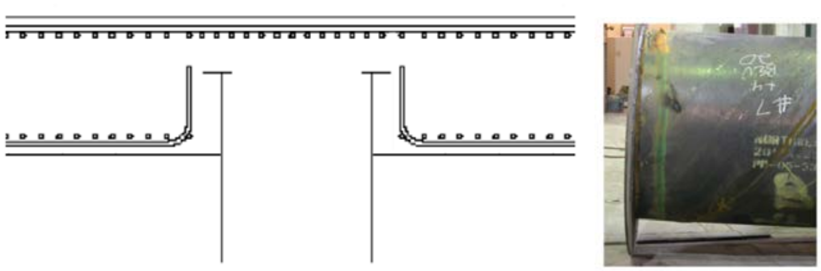

Fig. 7. Detailing of longitudinal reinforcement adjacent to the tube

This minimum embedment length is defined as:

$$l_e \geq \sqrt{\frac{D_o^2}{4} + \frac{DtF_u}{6\sqrt{f'_{cf}}}} - \frac{D_o}{2} \qquad (9)$$

The concrete in the pile cap will have a minimum compressive strength, f'_{cf}, (psi). The variable D_o is the outside diameter of the annular ring for the embedded connection as shown in Figure 5, and F_u is the minimum specified tensile strength of the steel (psi).

The pile cap must have adequate concrete depth, h, above the concrete filled tube to avoid punching through the pile cap. This is similar to the force transfer mechanism for a reinforced concrete column.

$$C_{max} = C_s + C_c \qquad (10a)$$

$$d_f = \sqrt{\frac{D^2}{4} + \frac{250C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \qquad (10b)$$

Where 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 PSDM for the most extreme load effect.

Pile Cap Reinforcement

The pile cap must follow conventional design practice and must be adequate to sustain the foundation design loads. As a result, the concrete, reinforcement and pile cap thickness usually will be identical to that required by normal pile cap design. However, the total concrete pile cap thickness, d_f, also must be large enough to control punching shear and cone pullout of the CFT piles, as expressed in Eq. 11a. The width and length of the pile cap, b_f, must be large enough to accommodate the concrete struts of 60 degrees from the vertical originating at the base of the ring, as indicated in Eq. 11b. Piles shall be adequately spaced avoiding intersecting concrete struts.

$$d_f \geq h + l_e \qquad (11a)$$

$$b_f \geq D_o + 3.5 l_e \qquad (11b)$$

The shear and flexural reinforcement in the pile cap must be designed for the normal shear and flexural loadings based upon the bridge loads, the soil conditions, and applicable overstrength loads. The flexural reinforcement in both directions should be spaced uniformly across the length and width of the footing, but the bottom layer of flexural reinforcement will be interrupted by the concrete tube. The longitudinal bars that are not interrupted by the tube must be designed with adequate capacity to develop the required foundation resistance. The interrupted bars are needed, but these bars do not contribute to the flexural strength of the footing. Fig. 7 shows the configuration of the longitudinal reinforcing bars that do not penetrate the tube but are placed within the tube diameter. The hooked length is equal to $12d_b$, where d_b is the diameter of the longitudinal bar. Standard 90^o rebar hooks shall be used in order to develop the full yield strength.

The shear reinforcement in the footing must be designed to meet the shear demand. The vertical reinforcement used to resist the shear must meet an additional constraint within the anchorage region of the embedded tube, such that at least two (2) vertical bars intersect the cone depicted in Fig. 5. Therefore vertical ties spaced no greater than *s* in the region within $1.5l_e$ of the outside of the tube, as defined by Eq. 12.

$$s \leq \frac{l_e}{2.5} \qquad (12)$$

In addition, it is noted that the required embedment results in a shear stress in the critical area surrounding the tube (Figure 5) of $6\sqrt{f'_c}$ (psi). Assuming the concrete is capable of resisting a shear demand of $2\sqrt{f'_c}$ (psi), the vertical reinforcement required by Eq. 12 should be designed to resist $4\sqrt{f'_c}$ (psi). Additional requirements for the shear demand resulting from other load combinations must also be considered.

RCFT Shaft-to-RC Pier Column and RC Shaft Connections

The recommended RCFT shaft to RC pier column connection is made as illustrated in Fig. 8. This detailing will develop the fully ultimate capacity of the RC pier at the top of the RCFT shaft. The connection extends all column reinforcement into the RCFT shaft for a length greater than or equal to the WSDOT requirement for noncontact lap splice reinforcement. The contribution of steel casing to the structural capacity of RCFT varies from zero at top of the shaft to fully composite at the end of transition zone. Transition zone is taken as1.0D for RCFT.

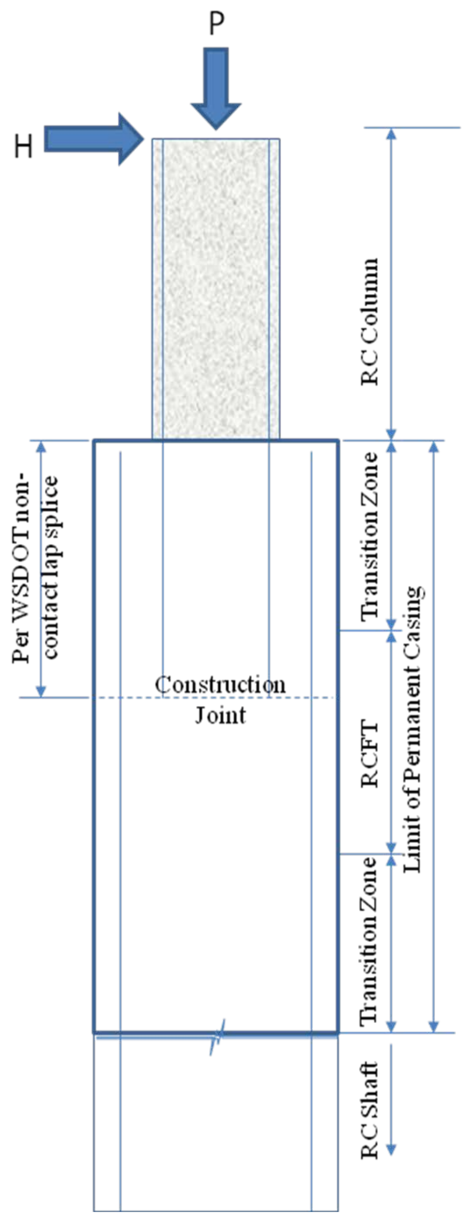

Figure 8. RC to RCFT connection

The requirement of AASHTO SGS Section 8.16.2 for piles with permanent steel casing is applicable. Per this section of the SGS, the extent of longitudinal reinforcement may be reduced to only the upper portion of the pile required to develop ultimate tension and compression capacities of the permanent steel cased pile.

Steel casing shall be left as permanent for RCFTs Slip casing shall not be permitted for RCFT foundation.

The cross-section for CFT and RCFT shall be adjusted for corrosion rates as specified below but not less than 1/16 inch at the end of design life (75 years minimum) after corrosion:

Soil embedded zone (undisturbed soil):	0.001 inch per year
Soil embedded zone (fill or disturbed natural soils)	0.003 inch per year
Immersed Zone (fresh water):	0.002 inch per year
Immersed Zone (salt water):	0.004 inch per year
Scour Zone (salt water):	0.005 inch per year

The area of the steel casing shall be included in determining the percentage of reinforcement, ρ for both CFT and RCFT. In case of RCFT the minimum longitudinal reinforcement could be reduced to 0.5% of the gross shaft cross-section area.

Notations

A_b = area of a single bar for the internal reinforcement (in^2)
A_c = net cross-sectional area of the concrete (in^2)
A_s = cross-sectional area of the steel tube and the longitudinal internal steel reinforcement ($in.^2$)
b_f = width and length of pile cap (in)
c = one half the chord length for a given stress state as shown in Figures 7.10.2, Figure 7.10.3 and Eq. 7.10.5c (in)
c_b = one half the chord length for a given stress state of a fictional tube modeling the internal reinforcement (in)
CFT = concrete filled tube.
D = outside diameter of the tube (in.)
D_o = outside diameter of the annular ring as shown in Figure 7.10.5
d_b = nominal diameter of a reinforcing bar (in)
d_f = depth of pile cap (in)
E_c = elastic modulus of concrete (ksi)
EI_{eff} = effective composite flexural cross-sectional stiffness of the CFT or RCFT element
E_s = modulus of elasticity 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 stress of reinforcing bars used for internal reinforcement (ksi)
F_{ys} = specified minimum yield stress of the steel tube (ksi)
f'_c = minimum specified 28-day compressive strength of concrete (ksi)
f'_{cf} = minimum compressive strength of the concrete in the footing (psi)
h = depth required for punching shear of footing (in)
I_c = uncracked moment of inertial of the concrete about centroidal axis (in^4)
I_s = moment of inertial of the steel tube and the longitudinal internal steel reinforcement about centroidal axis (in^4)
K = effective length factor as specified in Article 4.6.2.5 of the AASHTO LRFD Bridge Design Specifications.
l = unbraced length in the plane of buckling (in)

l_e	=	embedment length for cone pullout of full-strength CFT foundation connection (in)
M	=	nominal moment resistance as function of nominal axial load, P, for a given stress state (kip-inches) (Eqs.7.10.4e and 7.10.5f)
M_o	=	composite plastic moment resistance of the CFT and RCFT members without axial load (kip-in)
n	=	number of equally spaced longitudinal internal steel reinforcement
P	=	applied axial dead load (kips) (Eq. 7.10.3b), nominal compressive load capacity of the member as function of nominal bending moment for a given stress state (kips) (Eqs. 7.10.4d and 7.10.5e)
P_{cr}	=	maximum strength of an axially loaded compression member (kip)
P_e	=	elastic critical buckling resistance (kips) $P_e = \frac{\pi^2 EI_{eff}}{(Kl)^2}$
PSDM	=	plastic stress distribution method.
P_u	=	factored axial load acting on member (kip)
P_o	=	maximum compressive load capacity of the column without consideration of buckling (kips)
RC	=	reinforced concrete.
RCFT	=	concrete filled tube with internal reinforcement.
r	=	radius to the outside of the steel tube as shown in Figure 7.10.2 (in)
r_b	=	radius to the center of the internal reinforcing bars (Eq. 7.10.5c) (in)
r_i	=	radius to the inside of the steel tube as shown in Figure 7.10.3 (in)
r_m	=	radius to the center of the steel tube as shown in Figure 7.10.3 (in)
r_{bm}	=	radius to the center of the steel tube as shown in Figure 7.10.3 (in)
s	=	maximum spacing of shear reinforcing in cone pullout region (in)
t_b,t_{eq}	=	thickness of a fictional steel tube used to model the contribution of the internal reinforcement as shown in Figure 7.10.3 (in)
t	=	wall thickness of the tube (in)
V_u	=	shear due to the factored loads (kip)
V_n	=	nominal shear resistance (kip)
w	=	size of fillet weld (in)
y	=	distance from the center of the tube to neutral axis for a given stress state as shown in Figures 7.10.2 and 7.10.3 (in)
ϕ	=	resistance factor for shear
ρ	=	ratio of area of steel to area of gross concrete area
θ_b	=	angle used to define c_b in the reinforcing bars for a given stress state as shown in Figure 7.10.3 (rad.)
θ	=	angle used to define c in the steel tube for a given stress state as shown in Figures 7.10.2 and 7.10.3 (rad.)

Background:

CFTs have engineering properties that offer strength and stiffness beyond a conventional reinforced concrete (RC) member. The research reported herein was undertaken to develop engineering expressions to fully use CFT piles and shafts in bridge construction, which include the steel shell. The research has shown that (1) CFT elements can sustain multiple cyclic drifts to large levels with minimal damage and (2) the expected strength and stiffness of RCFT is approximately that of CFT components and can be estimated using similar tools. However, RCFT offers very little advantage beyond that achieved with CFT, while adding considerable cost and complexity. As a result, RCFT is recommended only as a transition between CFT and RC elements.

The prior CALTRANS and ARMY research programs studied two types of fully restrained connections for CFT pier-to-foundation connections. One of those two connections are readily usable as CFT pile-to-pile cap connections. This annular ring is attached to the top of the CFT pile, and it is then 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 column 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 the flexural strength of the CFT column.

The effective stiffness, EI_{eff}, of circular CFT is important, because it is used to evaluate deflections, deformations, buckling capacity, and moment magnification. A new expression that accounts for the impact of the axial load and effective reinforcing area on the concrete stiffness was developed; this effective stiffness factor is termed *C'*. Eq. 3b gives the resulting expression.

Transition connections between RC concrete caissons and CFT caissons have not been tested, but considerable analysis has been performed. Models have been developed to predict the strength of the RCFT elements, and this RCFT behavior may be used to provide increased strength over a significant length of the pile relative to conventional RC construction. Figure 7 shows a proposed connection between a CFT or RCFT pile and an RC pile segment.

Although CFT columns have been studied by prior researchers, the tubes were typically small (6 inches or less in diameter) and few researchers have studied connections for bridge construction. Therefore an experimental study is critical to the implementation of CFT in bridge construction. The CFT column-to-foundation research has demonstrated this importance. While CFT components are permitted by codes, there is little community consensus on their design.

Overstrength factor for capacity design of adjacent members and joint shear design issue at column-crossbeam joint are not addressed.

The concrete fill for the CFT member used in the testing was a low-shrinkage, self consolidating concrete. In the testing, the concrete strength nominal strengths were 6 ksi and 10 ksi. This is a structural concrete, and the minimum strength is 4 ksi, with an expected strength 25 to 50% larger. Low shrinkage concrete is required to ensure the concrete does not shrink relative to the steel tube. Conventional concrete results in an amount of shrinkage that eliminates composite action, thereby comprising the stiffness of the component.

Two different steel tubes have been tested. The steel tubes may either be straight seam or spiral welded tubes. Spiral welded tubes offer reduced cost, greater versatility and more rapid fabrication with large diameter tubes, since they can be formed to any diameter from a more limited inventory of materials. Spiral welded tubes are formed from coil steel, which has different material designations (AISI designations) than commonly used in AASHTO. It is recommended that that a low-carbon, low-alloy steel with a minimum yield stress of 50 ksi and minimum elongation at break of 15% be employed.

Prior research has not evaluated the shear strength of RCFT. This was not studied as part of this research. Such experiments are clearly warranted, but the shear resistance of the composite section clearly cannot be less than the shear resistance of the steel or concrete acting alone. The shear resistance of the steel will invariably be larger than the shear resistance of the concrete unless the *D/t* ratio of the tube is extremely large (approaching 200).

Several methods may be used for punching shear evaluation, but the current ACI procedure for single shear (ACI 318 2011) is recommended as a conservative approach. In compression, the column carries the axial force (P_u) and the compression force from the moment couple from the same load case. However, unlike the tension case, the data show that a portion of the compressive force is distributed to the foundation through bond. This is similar to the force transfer mechanism for a reinforced concrete column. In compression, the column carries the axial force (P_u) and the compression force from the moment couple from the same load case. However, unlike the tension case, the data show that a portion of the compressive force is distributed to the foundation through bond.

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

Contract specifications for steel casing will be developed, and WSDOT BDM will be revised for welding details.

If you have any questions regarding these issues, please contact Anthony Mizumori at 360-705-7228 (MizumoA@wsdot.wa.gov), Geoff Swett at 360-705- 7157 (SwettG@wsdot.wa.gov), Chyuan-Shen Lee at 360-705-7441 (LeeCH@wsdot.wa.gov), or Bijan Khaleghi at 360-705-7181 (KhalegB@wsdot.wa.gov).

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BDM Revisions:

7.10 Structural Design Recommendations of CFT and RCFT for Bridge Foundation

7.10.1 Introduction

Concrete filled tubes (CFT) have engineering properties that offer strength and stiffness beyond a conventional reinforced concrete (RC) member. The research reported herein was undertaken to develop engineering expressions to fully use CFT piles and shafts in bridge construction, which include the steel shell.

The research has shown that:

1. CFT elements can sustain multiple cyclic drifts to large levels with minimal damage

2. The expected strength and stiffness of RCFT is approximately that of CFT components and can be estimated using similar tools.

However, RCFT offers very little advantage beyond that achieved with CFT, while adding considerable cost and complexity. As a result, RCFT is recommended only as a transition between CFT and RC elements.

The 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 CFT pile-to-pile cap connections. This annular ring is attached to the top of the CFT pile, and it is then 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 column 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 the flexural strength of the CFT column.

The effective stiffness, EI_{eff}, of circular CFT is important, because it is used to evaluate deflections, deformations, buckling capacity, and moment magnification. A new expression that accounts for the impact of the axial load and effective reinforcing area on the concrete stiffness was developed; this effective stiffness factor is termed *C'*.

Transition connections between RC concrete caissons and CFT caissons have not been tested, but considerable analysis has been performed. Models have been developed to predict the strength of the RCFT elements, and this RCFT behavior may be used to provide increased strength over a significant length of the pile relative to conventional RC construction.

Overstrength factor for capacity design of adjacent members and joint shear design issue at column-crossbeam joint are not addressed.

The concrete fill for the CFT member used in the testing was a low-shrinkage, self consolidating concrete. In the testing, the concrete strength nominal strengths were 6 ksi and 10 ksi. This is a structural concrete, and the minimum strength is 4 ksi, with an expected strength 25 to 50% larger. Low shrinkage concrete shall be required to ensure the concrete does not shrink relative to the steel tube. Conventional concrete results in an amount of shrinkage that eliminates composite action, thereby comprising the stiffness of the component and shall not be used.

Two different steel tubes have been tested. The steel tubes may either be straight seam or spiral welded tubes. Spiral welded tubes offer reduced cost, greater versatility and more rapid fabrication with large diameter tubes, since they can be formed to any diameter from a more limited inventory of materials. Spiral welded tubes are formed from coil steel, which has different material designations (AISI designations) than commonly used in AASHTO. It is recommended that that a low-carbon, low-alloy steel with a minimum yield stress of 50 ksi and minimum elongation at break of 15% be employed.

Prior research has not evaluated the shear strength of RCFT. This was not studied as part of this research. Such experiments are clearly warranted, but the shear resistance of the composite section clearly cannot be less than the shear resistance of the steel or concrete acting alone. The shear resistance of the steel will invariably be larger than the shear resistance of the concrete unless the *D/t* ratio of the tube is extremely large (approaching 200).

Several methods may be used for punching shear evaluation, but the current ACI procedure for single shear (ACI 318-2011) is recommended as a conservative approach. In compression, the column carries the axial force (P_u) and the compression force from the moment couple from the same load case. However, unlike the tension case, the data show that a portion of the compressive force is distributed to the foundation through bond. This is similar to the force transfer mechanism for a reinforced concrete column. In compression, the column carries the axial force (P_u) and the compression force from the moment couple from the same load case. However, unlike the tension case, the data show that a portion of the compressive force is distributed to the foundation through bond.

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

7.10.2 Implementation

Steel casings could be considered in the structural capacity of piles, shafts, and connections of pile-to-pile cap and column-to-shaft Foundation. This section provides guidance for structural design of concrete filled steel tubes (CFT) and reinforced concrete filled steel tubes (RCFT) for bridges and other structures foundations. Use of CFT and RCFT requires approval from the WSDOT Bridge Design Engineer.

The design approach is as follows:

1. Establish the demands using an appropriate model of the bridge including soil-structure interaction.
2. Establish the material properties for the concrete and steel tube.
3. Determine an initial size of tube to meet constructability, geometric limitations (D/t ratio) and a target axial stress ratio.
4. Design the shaft/pile for combined flexure and axial loading using a P-M interaction curve by means of the plastic stress distribution method that incorporates buckling.
5. For column connections, determine the required development length of the reinforcement into the RCFT shaft. For pile/shaft cap connections, determine the required embedment of the CFT into the cap.

The step-by-step design procedure for structural design of CFT and RCFT for bridge foundation applications is as follows:

Step 1: Establish Material Properties

CFT piles and their connections require three types of materials, including the concrete, the steel tube, and the reinforcing steel. The reinforcing steel used in the foundation or cap-beam element shall be in accordance with WSDOT STD Specification Section 9-07. The steel casing shall be in accordance with WSDOT STD Specifications Section 9-36.1(1). Yield strength of the steel casing shall be specified in the Contract Plans. The yield strength beyond 36 ksi may be used if advantageous in design. The concrete for CFT and RCFT shall be class 4000P. The compressive strength reduction factor of $0.85f'_c$ shall be considered for wet placed concretes.

Step 2: Establish Initial Tube Geometry

There are three different aspects of CFT/RCFT that should be considered when selecting the initial cross section of the tube. First, minimum thickness values may be required for constructability such as driving or corrosion. The minimum thickness shall not be taken less than 3/8 inch at the time of installation. Second, it may be desirable to limit the axial stress on the member during seismic loading. Commonly axial stresses under earthquake loading are limited to 10-20% of the gross capacity. And finally, tubes with very high D/t ratio may compromise their capacity, where as tubes with low D/t ratios may not develop their strength. Therefore the recommended D/t limit is encouraged. 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. This is accomplished by providing an upper limit on the D/t ratio.

This is accomplished by providing an upper limit on the D/t ratio. As a result, the recommended slenderness limit for:

- CFT or RCFT subjected primarily to flexural loading is:

$$\frac{D}{t} \leq 0.22\frac{E}{F_y} \qquad (7.10.1)$$

- CFT and RCFT subjected primarily to axial loading is:

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

Note: it is recommended that these D/t ratios are satisfied even though design of CFT and RCFT elements will still typically be limited to elastic capacities.

Step 3: Stiffness Models For CFT And RCFT

The effective stiffness, EI_{eff}, of circular CFT as shown in Eq. 7.10.3a is used to evaluate deflections, deformations, buckling capacity, and moment magnification. This effective stiffness factor is termed *C'* shown in Eq. 7.10.3b.

$$EI_{eff} = E_s I_s + C' E_C I_C \qquad (7.10.3a)$$

$$C' = 0.15 + \frac{P}{P_0} + \frac{A_S}{A_S + A_C} < 0.9 \qquad (7.10.3b)$$

Where P_0, the ultimate axial crushing load, is defined in Eq. 7.10.6c, and A_s is the combined area of the steel tube and steel reinforcing.

Step 4: Flexural Strength and Material Interaction Curve

The flexural strength of CFT and RCFT members is determined using the plastic stress distribution method (PSDM), shown in Fig. 7.10.1. This method can be used to generate a full P-M interaction curve, as shown in Figure 7.10.1b. For each neutral axis depth, pairs of axial and bending resistances can be determined. Figure 7.10.1b shows the resulting material interaction curve where the values on the moment (x) axis and axial load (y) axis are normalized to the flexural strength without axial load (M_0) and the axial crush load without moment (P_0) of the member, respectively. Smaller D/t values result in larger resistance, because the area of steel is larger. Larger D/t ratios result in significantly increased bending moment for modest compressive loads, because of the increased contribution of concrete fill as shown in Fig. 7.10.1b.

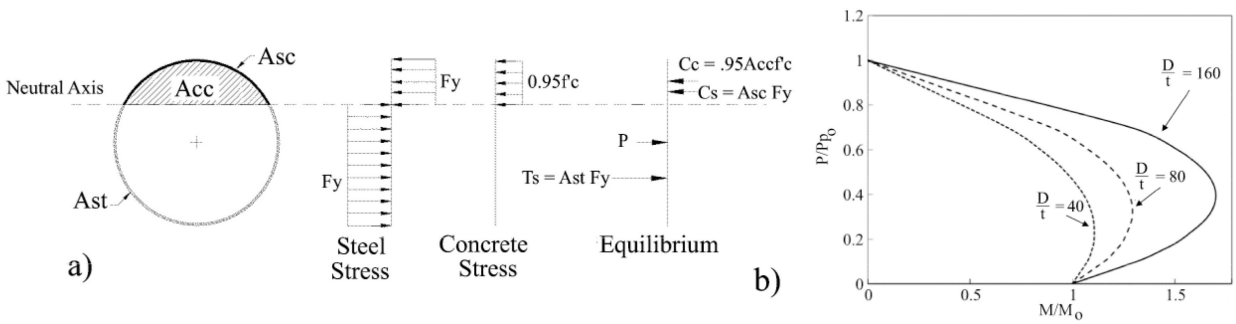

Fig 7.10.1 Strength determination for CFT; a) Plastic stress distribution method, b) Material-based interaction curves (no buckling)

A closed-form solution for the interaction curve for CFT piles has been derived based upon the geometry described in Fig. 7.10.2 and is expressed in Equations 7.10.4.

$$r_m = r - \frac{t}{2} \qquad (7.10.4a)$$

$$\theta = \sin^{-1}\left(\frac{y}{r_m}\right) \qquad (7.10.4b)$$

$$c = r_i \cos\theta \qquad (7.10.4c)$$

$$P = F_y t r_m \left\{ (\pi - 2\theta) - (\pi + 2\theta) \right\} + \frac{0.95 f'_{ct}}{2} \left\{ (\pi - 2\theta) r_i^2 - 2yc \right\} \qquad (7.10.4d)$$

$$M = 0.95 f'_{ct} c \left\{ \left(r_i^2 - y^2 \right) - \frac{c^2}{3} \right\} + 4 F_y t c \frac{r_m^2}{r_i} \qquad (7.10.4e)$$

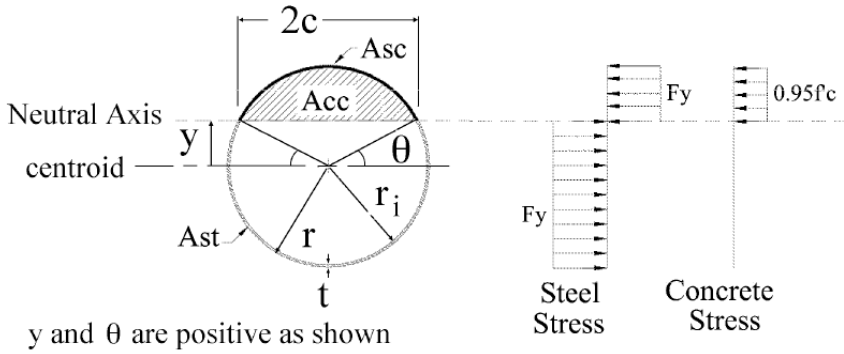

Fig. 7.10.2 Geometry used for closed form derivation of CFT stress distribution prediction

For RCFT members, a similar material interaction curve is expressed by Eqs. 7.10.5 with slightly revised geometry as illustrated in Fig. 7.10.3.

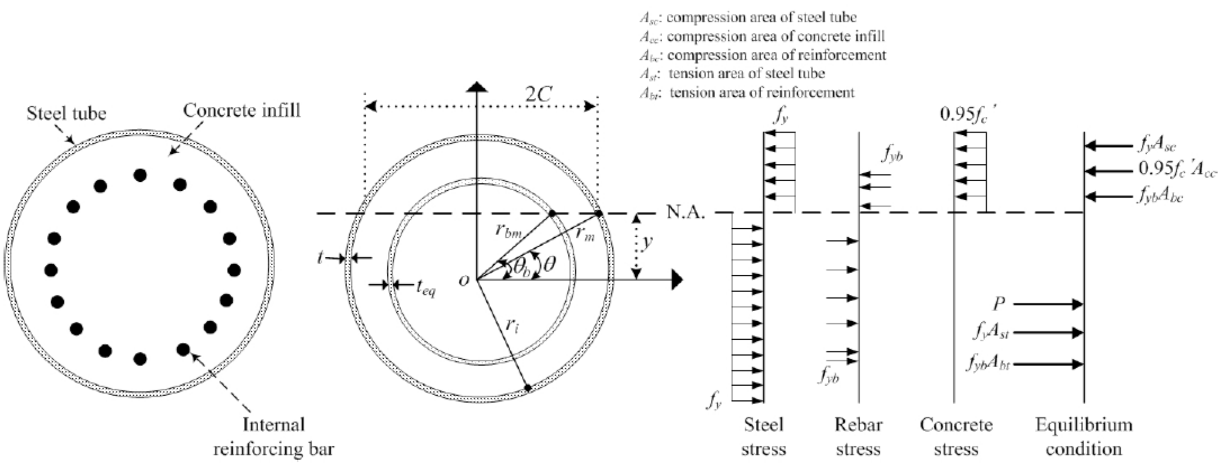

Fig. 7.10.3. Geometry for derivation of plastic stress distribution method for RCFT

$$r_m = r - \frac{t}{2} \qquad (7.10.5a)$$

$$\theta_s = \sin^{-1}\left(\frac{y}{r_m} \right) \quad \text{and} \quad \theta_b = \sin^{-1}\left(\frac{y}{r_b} \right) \qquad (7.10.5b)$$

$$c = r_i \cos\theta_s \ \text{ and } \ c_b = r_b \cos\theta_b \qquad (7.10.5c)$$

$$t_b = \frac{n A_b}{2\pi r_b} \qquad (7.10.5d)$$

$$P = F_{ys} t r_m \left\{ (\pi - 2\theta_s) - (\pi + 2\theta_s) \right\} + t_b r_b \left\{ F_{yb}(\pi - 2\theta_b) - \left(F_{yb} - .95 f_c^{'}\right)(\pi + 2\theta_b) \right\}$$

$$+ \frac{0.95 f_c^{'} c}{2} \left\{ (\pi - 2\theta_s) r_i^2 - 2yc \right\} \qquad (7.10.5e)$$

$$M = 0.95 f_c^{'} c \left\{ \left(r_i^2 - y^2\right) - \frac{c^2}{3} \right\} + 4 F_{ys} t c \frac{r_m^2}{r_i} + 4 F_{yb} t_b c_b r_b \qquad (7.10.5f)$$

A positive value of P implies a compressive force, and y and M are positive with the sign convention shown in the figures. The variable y varies between plus and minus r_i. The P-M interaction curve is generated by solving the equations for selected points in this range and connection of those points. The above equations for nominal capacities could be used for both strength and extreme limit states with corresponding resistance factors per LRFD specifications.

Step 5: Impact of Stability on P-M Interaction Curve (Option For Non-restrained CFT and RCFT)

Piles and shafts are typically assumed to be continually braced by the surrounding soil. Hence, piles and shafts are not normally affected by P-Δ effects or other secondary effects. As a result, this is not needed for most applications of CFT or RCFT as deep foundation elements. However, it is recognized that special circumstances such as scour, soil liquefaction, or other design issues may leave piles and shafts subject to stability and P- Δ effects or other secondary effects. In these circumstances, it is necessary to adjust the material interaction curve shown in Fig. 7.10.1b to account for stability and slenderness effects. To do this, it is necessary to employ the flexural stiffness, EI_{eff}, of the pier, provided in Eq. 7.10.3a.

Global column buckling is determined by the equations provided in AISC and repeated here for clarity:

$$P_{cr} = 0.658^{\frac{P_o}{Pe}} P_o \qquad (7.10.\ 6a)$$

$$P_{cr} = 0.877 P_e \qquad (7.10.6b)$$

$$P_o = 0.95 f_c^{'} A_c + F_y A_s \qquad (7.10.6c)$$

where P_e is the elastic buckling load by the Euler equation, and A_c and A_s are areas of the concrete and steel, respectively.

The interaction curve including stability effects is a modified version of the material interaction curve (Fig. 7.10.1b), which forms its basis. A series of points are jointed to form the curve. The points are as defined as follows:

- Points *A* and *B* are the axial and flexural capacity by the PSDM.
- Point *C* corresponds to the location on the PSDM interaction curve that results in the same moment capacity as point *B* but with axial load.

- Points *A'* and *C'* are obtained by multiplying the axial load associated with points *A* and *C* by the ratio, P_{cr}/P_o.
- Point *D* is located on the PSDM interaction curve and corresponds to one half of axial load which is determined for point *C'*.
- Point *A''* is the intersection of PSDM *P-M* interaction and a line parallel with the *x*-axis through the point *A'*.

The resulting curve accounts for global buckling of the tube and an example is shown in Fig. 7.10.4. The axial strength is limited to point *A'* (or the AISC buckling capacity). The stability based *P-M* interaction curve is then constructed by connecting points *A'*, *A''*, *D*, and *B*, as shown in the figure. This interaction curve should then be used for strength design of the member for all load conditions. Seismic design requires that less ductile elements be designed for the expected maximum plastic capacity of the ductile members. Given a specified axial load, the expected maximum bending moment will be 125% of the moment obtained from the interaction curve and this is the demand that should be used to design any less-ductile connecting elements.

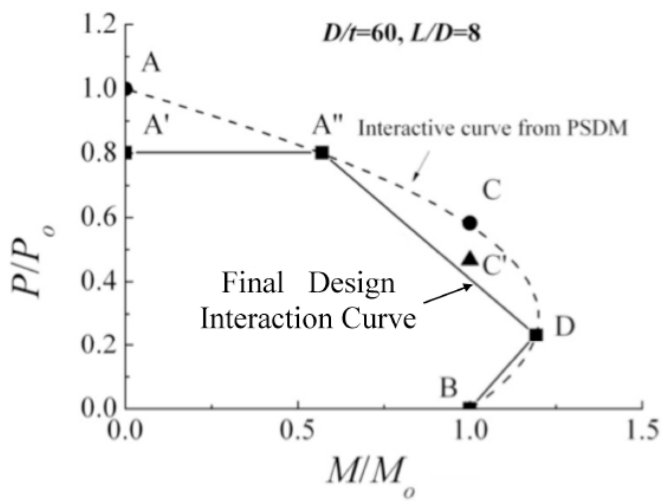

Fig. 7.10.4. Construction of the Stability-Based Interaction Curve for CFT and RCFT

Step 6: Determine the Shear Strength of CFT and RCFT

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

$$V_u = \phi V_n = \phi\,(V_s + 0.5{*}V_c) \qquad (7.10.7)$$

where:

V_s = Shear resistance of steel casing = $0.6F_y\, 0.5\, \pi t D$

V_c = Shear resistance of concrete = $0.0316\, \beta\, \mathrm{SQRT}(f_c')\, A_c$. if P_u is compressive

ϕ = 0.85 LRFD resistance factor for shear

A_c = area of concrete within the steel casing

f'_c = compressive strength of concrete

Step 7: Pile or Shaft Connection Design

Two types of connection are considered for piles or shafts. The full strength connection, illustrated in Fig. 7.10.5, is proposed for connecting piles to pile caps. The partial strength connection, illustrated in Fig. 7.10.7, is proposed for connecting to pier columns.

7.10.3 Full Strength Connection

The foundation connection design for the CFT must consider several different factors. A central part of this research study is the design and detailing of the connecting CFT to the foundation or cap-beam. The connection design should include:

1. Detailing/sizing of the annular ring
2. Determination of the embedment depth
3. Punching shear evaluation
4. General design (flexure and shear) of the connecting (foundation or cap-beam) element to sustain the CFT column demands.

7.10.4 Detailing of Annular Ring and Embedment Depth

An annular ring is welded to the end of the tube to provide anchorage and stress distribution, as illustrated in Fig. 7.10.5. The ring is made of steel of the same thickness and yield stress as the steel tube. The ring extends outside the tube 16 times the thickness of the tube and projects inside the tube 8 times the thickness of the tube. This gives a width of the ring of 25 times the thickness of the tube, as shown in the figure.

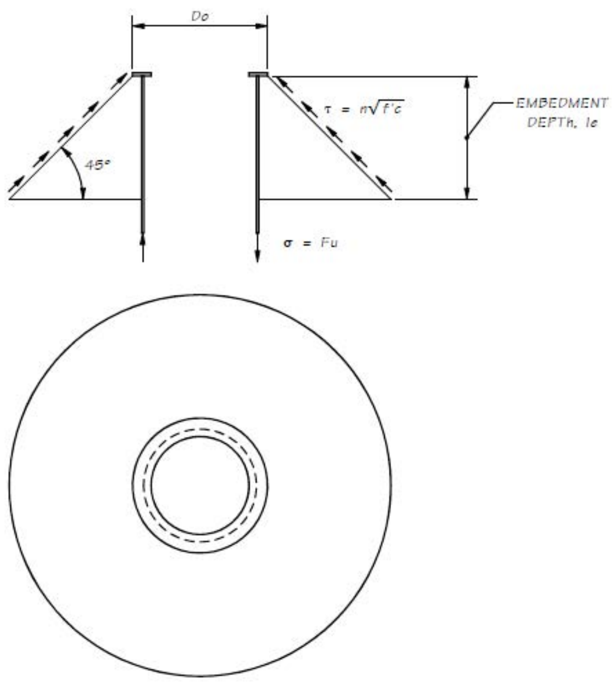

Fig. 7.10.5. Cone pullout requirements for the full strength pile cap connection

The ring is welded to the tube with complete joint penetration (CJP) welds of matching metal 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, and for this purpose the minimum weld size, w, of the fillets can be defined by Eq. 7.10.8.

$$w \geq \frac{1.47 F_u t}{F_{EXX}} \qquad (7.10.8)$$

where F_{EXX} and F_u are the minimum tensile strength of the weld metal and tube steel, respectively. If spirally welded pipe piles are allowed, skelp splices should be located at least 1'-0" away from the annular ring. The CFT weld detail is shown in Figure 7.10.6.

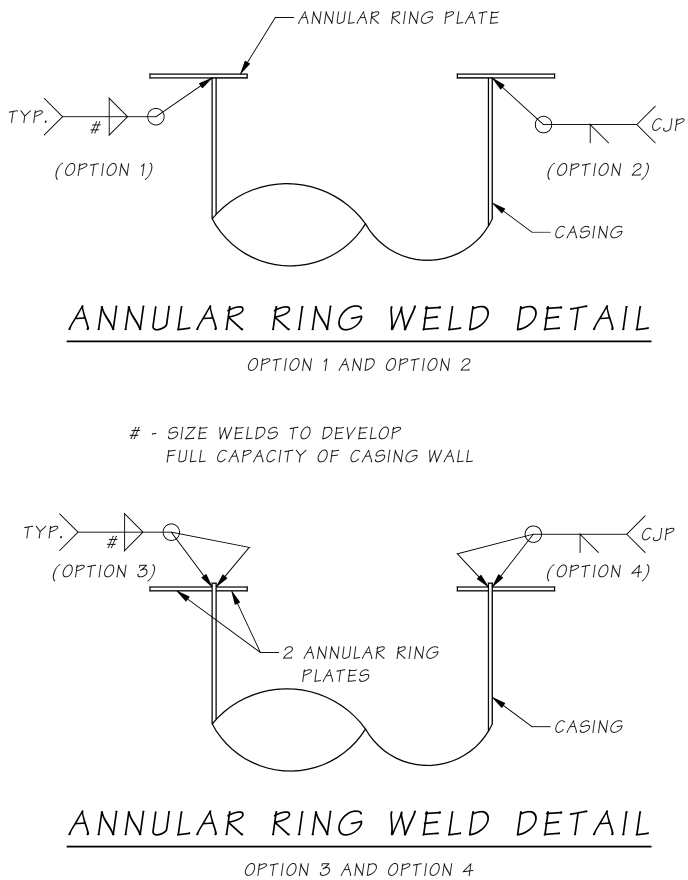

Figure 7.10.6. CFT Weld Detail

The tube and the annular ring are embedded into the pile cap with an embedment depth, l_e, needed to assure ductile behavior of the connection as depicted in Fig. 7.10.6. This minimum embedment length is defined as:

$$l_e \geq \sqrt{\frac{D_o^2}{4} + \frac{DtF_u}{6\sqrt{f'_{cf}}}} - \frac{D_o}{2} \tag{7.10.9}$$

The concrete in the pile cap will have a minimum compressive strength, f'_{cf}, (psi). The variable D_o is the outside diameter of the annular ring for the embedded connection as shown in Figure 7.10.5, and F_u is the minimum specified tensile strength of the steel (psi).

The pile cap must have adequate concrete depth, *h*, above the concrete filled tube to avoid punching through the pile cap. This is similar to the force transfer mechanism for a reinforced concrete column.

$$C_{max} = C_s + C_c \tag{7.10.10a}$$

$$d_f = \sqrt{\frac{D^2}{4} + \frac{250C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \tag{7.10.10b}$$

where 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 PSDM for the most extreme load effect.

7.10.5 Pile Cap Reinforcement

The pile cap must follow conventional design practice and must be adequate to sustain the foundation design loads. As a result, the concrete, reinforcement and pile cap thickness usually will be identical to that required by normal pile cap design. However, the total concrete pile cap thickness, d_f, also must be large enough to control punching shear and cone pullout of the CFT piles, as expressed in Eq. 7.1011a. The width and length of the pile cap, b_f, must be large enough to accommodate the concrete struts of 60 degrees from the vertical originating at the base of the ring, as indicated in Eq. 7.1011b. Piles shall be adequately spaced avoiding intersecting concrete struts.

$$d_f \geq h + l_e \tag{7.10. 11a}$$

$$b_f \geq D_o + 3.5l_e \tag{7.10. 11b}$$

The shear and flexural reinforcement in the pile cap must be designed for the normal shear and flexural loadings based upon the bridge loads, the soil conditions, and applicable overstrength loads. The flexural reinforcement in both directions should be spaced uniformly across the length and width of the footing, but the bottom layer of flexural reinforcement will be interrupted by the concrete tube. The longitudinal bars that are not interrupted by the tube must be designed with adequate capacity to develop the required foundation resistance. The interrupted bars are needed, but these bars do not contribute to the flexural strength of the footing. Fig. 7.10.7 shows the configuration of the longitudinal reinforcing bars that do not penetrate the tube but are placed within the tube diameter. The hooked length is equal to $12d_b$, where d_b is the diameter of the longitudinal bar. Standard 90^o rebar hooks shall be used in order to develop the full yield strength.

The shear reinforcement in the footing must be designed to meet the shear demand. The vertical reinforcement used to resist the shear must meet an additional constraint within the anchorage region of the embedded tube, such that at least two (2) vertical bars intersect the cone depicted in Fig. 7.10.5. Therefore vertical ties spaced no greater than *s* in the region within $1.5l_e$ of the outside of the tube, as defined by Eq. 7.10.12.

$$s \leq \frac{l_e}{2.5} \qquad (7.10.12)$$

In addition, it is noted that the required embedment results in a shear stress in the critical area surrounding the tube (Figure 7.10.5) of $6\sqrt{f'_c}$ (psi). Assuming the concrete is capable of resisting a shear demand of $2\sqrt{f'_c}$ (psi), the vertical reinforcement required by Eq. 7.10.12 should be designed to resist $4\sqrt{f'_c}$ (psi). Additional requirements for the shear demand resulting from other load combinations must also be considered.

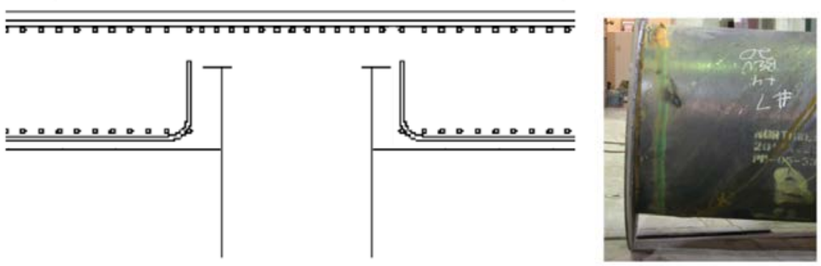

Fig. 7.10. 7. Detailing of longitudinal reinforcement adjacent to the tube

7.10.6 RCFT Shaft-to-RC Pier Column and RC Shaft Connections

The recommended RCFT shaft to RC pier column connection is made as illustrated in Fig. 7.10.8. This detailing will develop the fully ultimate capacity of the RC pier at the top of the RCFT shaft. The connection extends all column reinforcement into the RCFT shaft for a length greater than or equal to the WSDOT requirement for noncontact lap splice reinforcement. The contribution of steel casing to the structural capacity of RCFT varies from zero at top of the shaft to fully composite at the end of transition zone. Transition zone is taken as1.0D for RCFT.

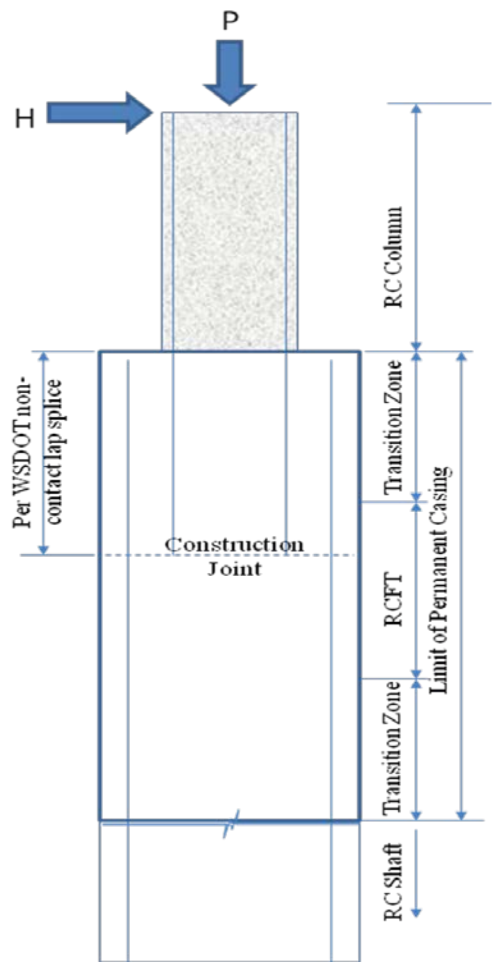

Figure 7.10.8. RC to RCFT connection

The requirement of AASHTO SGS Section 8.16.2 for piles with permanent steel casing is applicable. Per this section of the SGS, the extent of longitudinal reinforcement may be reduced to only the upper portion of the pile required to develop ultimate tension and compression capacities of the permanent steel cased pile.

Steel casing shall be left as permanent for RCFTs installed either by rotary drilling or oscillators. Slip casing is not permitted for RCFT foundation.

The cross-section for CFT and RCFT shall be adjusted for corrosion rates as specified below but not less than 1/16 inch at the end of design life (75 years minimum) after corrosion:

Soil embedded zone (undisturbed soil):	0.001 inch per year
Soil embedded zone (fill or disturbed natural soils)	0.003 inch per year
Immersed Zone (fresh water):	0.002 inch per year
Immersed Zone (salt water):	0.004 inch per year
Scour Zone (salt water):	0.005 inch per year

The area of the steel casing shall be included in determining the percentage of reinforcement, ρ for both CFT and RCFT. In case of RCFT the minimum longitudinal reinforcement could be reduced to 0.5% of the gross shaft cross-section area.

7.10.7 Notations

A_b	=	area of a single bar for the internal reinforcement (in^2)
A_c	=	net cross-sectional area of the concrete (in^2)
A_s	=	cross-sectional area of the steel tube and the longitudinal internal steel reinforcement ($in.^2$)
b_f	=	width and length of pile cap (in)
c	=	one half the chord length for a given stress state as shown in Figures 7.10.2, Figure 7.10.3 and Eq. 7.10.5c (in)
c_b	=	one half the chord length for a given stress state of a fictional tube modeling the internal reinforcement (in)
CFT	=	concrete filled tube.
D	=	outside diameter of the tube (in.)
D_o	=	outside diameter of the annular ring as shown in Figure 7.10.5
d_b	=	nominal diameter of a reinforcing bar (in)
d_f	=	depth of pile cap (in)
E_c	=	elastic modulus of concrete (ksi)
EI_{eff}	=	effective composite flexural cross-sectional stiffness of the CFT or RCFT element
E_s	=	modulus of elasticity 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 stress of reinforcing bars used for internal reinforcement (ksi)
F_{ys}	=	specified minimum yield stress of the steel tube (ksi)
f_c	=	minimum specified 28-day compressive strength of concrete (ksi)
f'_{cf}	=	minimum compressive strength of the concrete in the footing (psi)
h	=	depth required for punching shear of footing (in)
I_c	=	uncracked moment of inertial of the concrete about centroidal axis (in^4)
I_s	=	moment of inertial of the steel tube and the longitudinal internal steel reinforcement about centroidal axis (in^4)
K	=	effective length factor as specified in Article 4.6.2.5 of the AASHTO LRFD Bridge Design Specifications.
l	=	unbraced length in the plane of buckling (in)
l_e	=	embedment length for cone pullout of full-strength CFT foundation connection (in)
M	=	nominal moment resistance as function of nominal axial load, P, for a given stress state (kip-inches) (Eqs.7.10.4e and 7.10.5f)
M_o	=	composite plastic moment resistance of the CFT and RCFT members without axial load (kip-in)

n = number of equally spaced longitudinal internal steel reinforcement

P = applied axial dead load (kips) (Eq. 7.10.3b), nominal compressive load capacity of the member as function of nominal bending moment for a given stress state (kips) (Eqs. 7.10.4d and 7.10.5e)

P_{cr} = maximum strength of an axially loaded compression member (kip)

P_e = elastic critical buckling resistance (kips)

$$P_e = \frac{\pi^2 EI_{eff}}{(Kl)^2}$$

$PSDM$ = plastic stress distribution method.

P_u = factored axial load acting on member (kip)

P_o = maximum compressive load capacity of the column without consideration of buckling (kips)

RC = reinforced concrete.

$RCFT$ = concrete filled tube with internal reinforcement.

r = radius to the outside of the steel tube as shown in Figure 7.10.2 (in)

r_b = radius to the center of the internal reinforcing bars (Eq. 7.10.5c) (in)

r_i = radius to the inside of the steel tube as shown in Figure 7.10.3 (in)

r_m = radius to the center of the steel tube as shown in Figure 7.10.3 (in)

r_{bm} = radius to the center of the steel tube as shown in Figure 7.10.3 (in)

s = maximum spacing of shear reinforcing in cone pullout region (in)

t_b, t_{eq} = thickness of a fictional steel tube used to model the contribution of the internal reinforcement as shown in Figure 7.10.3 (in)

t = wall thickness of the tube (in)

V_u = shear due to the factored loads (kip)

V_n = nominal shear resistance (kip)

w = size of fillet weld (in)

y = distance from the center of the tube to neutral axis for a given stress state as shown in Figures 7.10.2 and 7.10.3 (in)

ϕ = resistance factor for shear

ρ = ratio of area of steel to area of gross concrete area

θ_b = angle used to define c_b in the reinforcing bars for a given stress state as shown in Figure 7.10.3 (rad.)

θ = angle used to define c in the steel tube for a given stress state as shown in Figures 7.10.2 and 7.10.3 (rad.)

7.10.8 References

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