A NEW GENERATION OF COMPOSITE PILES: BEHAVIOUR, DESIGN CONSIDERATIONS AND FIELD APPLICATIONS

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ABSTRACT: Piles used in bridges, structures and marine environments are typically produced using concrete, steel, and timber. High repair and replacement costs of these piles have led North American highway agencies and port authorities to investigate the feasibility of using durable materials such as fiber-reinforced polymers (FRP) in pile applications. This paper presents a new generation of composite piles composed of concrete-filled FRP circular tubes. The tube provides permanent formwork and non-corrosive reinforcement at the same time, leading to a simpler construction process. It also protects the concrete from severe environmental conditions and most importantly confines the concrete core, which increases its strength and ductility. The laminate structure of the FRP tubes can be engineered to provide the required strength and stiffness in the longitudinal and circumferential directions by controlling the proportions of fibers oriented in both directions. The structural performance of this type of piles under bending, axial loads and combined loading have been investigated experimentally, using large scale test specimens. Theoretical models and charts have been developed for analysis and design purposes. Field application of the composite pile in the first bridge ever built utilizing these piles, the Route 40 Bridge in Virginia, is introduced. Numerous field applications of this new pile as fender piles in marine environments in the United States are also introduced.

Keywords: Pile, FRP, Tube, Composite, Fender, Concrete-filled, Confinement
INTRODUCTION

Concrete-filled fiber reinforced polymer (FRP) tubes (CFFT) provide a new and attractive use of composite materials in several applications including piles. Traditional pile materials such as steel, concrete, and timber have limited service life and high maintenance costs if they are used in harsh marine environments\(^1\). It has been estimated that repair and replacement of piling systems costs the U.S. over $1 billion annually\(^2\). High repair and replacement costs have led North American highway agencies and researchers to investigate the feasibility of using composite materials for civil engineering infrastructures including bridge pile foundations. FRP tubes provide a permanent, non-corrosive, lightweight formwork for the concrete and reinforcement element at the same time. The laminate structure of the composite tube can be engineered to provide different proportions of strength and stiffness in the longitudinal and transverse directions, depending on the application and nature of loading. Under axial loads, the FRP tube confines the concrete by reducing its lateral expansion, therefore, increases its ultimate strain and strength. Several researchers have studied the structural behavior of CFFT under concentric axial loads\(^3,4,5\). Flexural behavior of these members was also studied for glass-FRP (GFRP) tubes\(^6\) and for carbon-FRP (CFRP) tubes\(^7\). Recently, a field application of CFFT as piles for bridge applications was also completed in the Route 40 Bridge in Virginia\(^8\).

BEHAVIOR OF CFFT UNDER DIFFERENT LOADING CONDITIONS

An experimental program has been carried out in order to test CFFT under concentric and eccentric axial loads using column specimens as well as under pure bending using beam specimens. Two different laminate structures were used for the GFRP tubes. Table 1 provides details of test specimens including the type of loading (bending, axial compression and combined bending and axial compression), the type of GFRP tube, the eccentricity of the axial load, \(e\) (for the eccentrically loaded columns), the spans of the beams (L), the heights of the concentrically and eccentrically loaded columns (H), the average compressive strength \(f'_c\) of the concrete used to fill the tubes based on standard cylinder tests, the measured axial load \(P_n\) and moment \(M_n\) at failure.

The GFRP tubes had 51 percent fiber volume fraction and were fabricated using the filament winding method. Table 2 provides details of the two types of GFRP tubes including the diameter, wall thickness, stacking sequence of different layers including the angle of the fibers and fiber/matrix types. Table 2 also provides the effective mechanical properties of the laminates based on the classical lamination theory including the effect of progressive laminate failure. Type I tubes have almost equal fiber percentages, oriented at 3 and 88 degrees with the longitudinal direction, while type II tubes have 70 percent of the fibers oriented at \(\pm 34\) degrees and 30 percent at 80 degrees with the axial direction.

**Beam Tests**

Table 1 provides details of B1-I and B1-II beam specimens using FRP tubes Types I and II. The specimens were tested using four-point bending as shown in Fig. 1. Two identical specimens were tested for B1-I and B1-II. The span of the beams was 5.5 m while the distance between the two applied loads was 1.5 m. Specimens were
instrumented within the constant moment region to measure the longitudinal and circumferential strains along the depth. Mid-span deflection and applied load were also measured. The beams were tested to failure to determine the flexural capacity $M_n$. Fig. 2 shows the load-deflection behavior of the two beams. The behavior indicates that the cracking load is quite low in comparison to the ultimate load and the behavior is almost linear, after cracking, up to failure.

Table 1. Details of test specimens

<table>
<thead>
<tr>
<th>ID</th>
<th>Type</th>
<th>Tube</th>
<th>Eccentricity (mm)</th>
<th>Span / Height (m)</th>
<th>Concrete strength $f'_c$ (MPa)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Ultimate moment $M_n$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1-I</td>
<td>B *</td>
<td>Type I</td>
<td>$\infty$</td>
<td>5.6 (L)</td>
<td>0</td>
<td>163</td>
<td></td>
</tr>
<tr>
<td>BC1-I</td>
<td>BC *</td>
<td>Type I</td>
<td>839</td>
<td>342</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
<tr>
<td>BC2-I</td>
<td>BC *</td>
<td>Type I</td>
<td>750</td>
<td>320</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
<tr>
<td>BC3-I</td>
<td>BC *</td>
<td>Type I</td>
<td>750</td>
<td>320</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
<tr>
<td>BC4-I</td>
<td>BC *</td>
<td>Type I</td>
<td>750</td>
<td>320</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
<tr>
<td>BC5-I</td>
<td>BC *</td>
<td>Type I</td>
<td>750</td>
<td>320</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
<tr>
<td>C1-I</td>
<td>C *</td>
<td>Type I</td>
<td>750</td>
<td>320</td>
<td>145</td>
<td>55</td>
<td>0</td>
</tr>
</tbody>
</table>

*B = Bending using beam test
*BC = Combined bending and axial load using eccentrically loaded column
*C = Axial load using concentrically loaded column

Table 2. Details of GFRP tubes

<table>
<thead>
<tr>
<th>Properties</th>
<th>Type I</th>
<th>Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer diameter (mm)</td>
<td>326</td>
<td>320</td>
</tr>
<tr>
<td>Structural wall thickness (mm)</td>
<td>6.4</td>
<td>5.86</td>
</tr>
<tr>
<td>Stacking sequence (degrees)</td>
<td>$-88/3/-88/3/-88/3/-88/3$</td>
<td>$-34/-34$</td>
</tr>
<tr>
<td>Fiber / matrix type</td>
<td>E-glass / epoxy</td>
<td>E-glass / polyester</td>
</tr>
<tr>
<td>Elastic modulus – Axial (GPa)</td>
<td>21.7-17.6 *</td>
<td>16.2-12.0 *</td>
</tr>
<tr>
<td>Elastic modulus – Hoop (GPa)</td>
<td>23.5-19.7</td>
<td>16.9-12.0</td>
</tr>
<tr>
<td>Axial tensile strength (MPa)</td>
<td>346</td>
<td>146</td>
</tr>
<tr>
<td>Axial compressive strength (MPa)</td>
<td>343</td>
<td>238</td>
</tr>
<tr>
<td>hoop tensile strength (MPa)</td>
<td>402</td>
<td>131</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.09 (0.11)**</td>
<td>0.24 (0.31)**</td>
</tr>
</tbody>
</table>

* Initial and final stiffness based on the method of progressive laminate failure
** Experimental

Fig. 1. Beam test setup.

Fig. 2. Load-deflection behavior test beams.

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Concentrically Loaded Column Tests

Table 1 provides details of C1-I and C1-II column specimens using FRP tubes, Types I and II. Both specimens were tested under concentric axial load to failure, as shown in Fig. 3, to provide the axial strength \( P_n \). Two identical specimens were tested for C1-I. Axial and circumferential strains were measured using both displacement and strain gauges along the perimeter of the tube at mid-height. Fig. 4 shows the axial load-axial strain behavior of the columns. The unconfined concrete had a relatively high compressive strength \( f'_c \), and therefore, the concrete lacks the post-peak strain softening behavior typically observed in unconfined low strength concrete. As a result of this brittle nature of the concrete, the load reached a peak value, corresponds approximately to \( f'_c \) and dropped slightly, in C1-I specimens, after development of few major internal cracks. Beyond this stage, the behavior of C1-I specimens showed plastic behavior and the peak load was gradually recovered until the tube was fractured and the columns failed. For C1-II specimen, larger drop in the load was observed without noticeable recovery, mainly due to the laminate structure of Type II tubes, which has low stiffness in the hoop direction and relatively high Poisson’s ratio value, leading to a limited confinement effect. In a typical situation, where low or normal strength concrete is confined, internal cracks are very uniform and well distributed within the concrete mass, resulting in a bi-linear load-strain behavior, if the FRP tubes have adequate stiffness. In this test, few internal cracks occurred due to the brittle nature of the relatively high strength concrete used. The measured axial load capacity of C1-II was slightly higher than C1-I in spite of the better confinement of Type I tubes in comparison to Type II tubes. This is attributed to \( f'_c \) value of the concrete filling, which was slightly higher in C1-II than in C1-I.

Fig. 3. Column test setup

Fig. 4. Axial load – strain behavior of columns

Eccentrically Loaded Column Tests

Table 1 provides details of the eccentrically loaded column specimens (BC1-I to BC5-I) of Type I tubes as well as (BC1-II to BC5-II) of Type II tubes. Rigid steel caps were installed at the top and bottom of the specimens, as shown in Fig. 5, to allow for variation of the eccentricity of the applied axial load in order to provide different combinations of axial loads \( P_n \) and bending moments \( M_n \) at failure. The total maximum moment \( M_n \) at
mid-height, which is reported in Table 1 for all eccentrically loaded column specimens, is composed of the primary moment, based on the initial eccentricity, and the secondary moment due to the lateral deflection at failure at mid-height. Table 1 also presents the eccentricity based on the final \( M_n \) and \( P_n \), which ranged from 55 to 839 mm for Type I tubes and from 11 to 329 mm for Type II tubes. These values are equivalent to eccentricity-to-outer diameter \((e/D_o)\) ratios of 0.169 to 2.574 for Type I and 0.034 to 1.028 for Type II tubes. Fig. 6 shows the axial load-bending moment interaction diagrams for Types I and II specimens. The behavior reflects very well the transition from tension to compression failure through the balanced point. The interaction diagrams are discussed in more details in the analytical modeling section.

Fig. 5. Test setup for combined bending and axial loads

![Fig. 5. Test setup for combined bending and axial loads](image)

Fig. 6. Interaction diagrams of test specimens

**Failure Modes**

Fig. 7 shows the different failure modes of some of the test specimens. Beam specimens B1-I and B1-II failed by rupture of the fibers in the tension side within the constant moment region in a similar fashion to the eccentrically loaded column specimens which failed in tension as shown in Fig. 7. C1-I and C1-II column specimens failed by fracture of the tube under a state of bi-axial stresses including axial compressive and hoop tensile stresses as shown in Fig. 7. Eccentrically loaded column specimens failed either in tension or compression as shown in Fig. 7, depending on the eccentricity of the applied load.
ANALYTICAL MODELING AND DESIGN TOOLS

The objective of the proposed analytical model is to establish the axial load-bending moment interaction diagram of concrete-filled FRP tubes. The model is based on the equilibrium and strain compatibility approach using the layer-by-layer method for the integration process of the stresses over the cross-section. The classical lamination theory is used to establish the effective stress-strain curves of the laminate of FRP tubes in the axial and hoop directions, utilizing the progressive failure approach of different layers of the laminate. Failure of the laminate is determined by the Tsai-Wu failure criteria. For the stress-strain curve of concrete in the compression zone, three different confinement mechanisms are examined, including an upper bound representing the full confinement model, which represent fixed level of confinement (independent of the eccentricity of the applied load), a lower bound unconfined concrete model, and a partial confinement model, which is variable and dependent on the eccentricity of the load.

Full Confinement Mechanism (Upper Bound)

In this case, the stress-strain curve of the confined concrete can be determined using any of the confinement models available in the literature for the case of pure axial loading condition as shown in Fig. 8(a) (full confinement). The analysis in this case utilizes the same full confinement stress-strain curve in the compression zone, for the entire range of eccentricity, to establish the interaction diagram. The confinement model by Fam and Rizkalla\(^4\) has been adopted in this analysis. The model is well suited for this case because it accounts for the bi-axial state of stress developed in the tube, including the axial compressive stresses, which result from the composite action, and the hoop tensile stresses resulting from confinement. The model adopts Tsai-Wu failure criteria to target the failure point of the tube. The ultimate confined strength and corresponding strain are defined as \(f_{ce}^c\) and \(e_{ce}^c\), respectively.

Unconfined Concrete Model (Lower Bound)

In this case, an unconfined stress-strain concrete model, shown in Fig. 8(a), is adopted in the compression zone for the full range of eccentricity to establish the interaction diagram. In this analysis the model by Popovics\(^4\) is used due to its capability of simulating
the strain softening behavior. In this model, the stress $f_c$ at a given axial strain $\varepsilon_c$ is given as a function of the unconfined strength $f'_c$ and the corresponding strain $\varepsilon'_c$ as follows:

$$f_c = \frac{f'_c x r}{r - 1 + x'}$$

(1)

Fig. 8. Confinement mechanisms of concrete and the bi-axial state of stress in FRP tube

Where $x = \varepsilon_c / \varepsilon'_c$ and $r = E_{co} / (E_{co} - E_{sec})$. $E_{co}$ is the tangent elastic modulus of unconfined concrete, and can be estimated as $5000\sqrt{f_c}$ (MPa). $E_c$ is the secant modulus of unconfined concrete and can be estimated as $f'_c / \varepsilon'_c$.

The curve could be terminated at 0.003 strain, which is the ultimate strain, specified by ACI 318-02, or could be extended to strain $\varepsilon_{cco}$. Fam and Rizkalla\(^6\) have shown that for the case of pure bending the effect of confinement on concrete strength is insignificant, however, the ductility and strain of concrete are increased significantly beyond 0.003. It is also well established that failure of the system is normally governed by fracture of the FRP tube before complete failure of the concrete inside. Therefore, $\varepsilon_{cco}$ is assumed equal to the ultimate compressive strain of the FRP tube in the axial direction.

**Variable Confinement Mechanism**

The proposed variable confinement model assumes that the confinement level of concrete is gradually reduced as the eccentricity of the axial load increases. This mechanism is very representative to observed behavior. Test results indicated that increasing the eccentricity results in a strain gradient that subject large part of the cross-section to tensile strains, which would significantly reduce the level of confinement. Fig. 8(b) shows the variable stress-strain curve of concrete, which ranges from the upper bound of the fully confined concrete (zero eccentricity) to the lower bound of the unconfined stress-strain curve with extended ductility for the case of infinite eccentricity (pure bending). The proposed model assumes that the initial ascending part of the curve is similar for both unconfined and confined. The part of the curve beyond the peak point of unconfined strength $f'_c$, is variable and dependent on the eccentricity of the applied load.

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For a given general eccentricity $e$, the ultimate strength of concrete $f_{cc}$, is calculated as a function of the fully confined strength $f_{cc}'$ and the unconfined stress $f_{cco}$ (corresponding to $e_{cco}$), from the following proposed equation:

$$f_{cc} = (f_{cc}' - f_{cco}) \left[ \frac{D_o}{D_o + e} \right] + f_{cco} \quad (2)$$

Where $D_o$ is the outer diameter. This expression satisfies the upper and lower bounds. For a case of pure axial load ($e = 0$), $f_{cc} = f_{cc}'$ and for the case of pure bending ($e = \infty$), $f_{cc} = f_{cco}$.

Fam and Rizkalla have shown that for the case of pure axial load ($e = 0$), a bi-axial state of stress is developed in the FRP tube and therefore, a bi-axial strength failure criteria such as Tsai-Wu should be used to detect failure. Fig. 8(c) shows the stress path (point 0 to 1) during the loading history under pure axial load. By increasing the eccentricity, less confinement is generated, and therefore, less hoop tensile stresses are developed and the stress path would gradually shifts from (point 0 to 1) to (point 0 to 2). As the eccentricity approaches infinity (pure bending), the stress path would be from point 0 to 3. Therefore, the path between points 1 and 3 on Fig. 8(b) corresponds to the path between points 1 and 3 on the failure envelope in Fig. 8(c). Accordingly, the locus of failure points between $f_{cc}$ and $f_{cco}$ in Fig. 8(b) is analogous to Tsai-Wu failure envelope and is approximated as elliptical. The strain $\varepsilon_{cc}$, which corresponds to the strength $f_{cc}$, ranges from $\varepsilon_{cco}$ to $\varepsilon_{cc}$, and is calculated from the following elliptical equation:

$$\varepsilon_{cc} = \left( \varepsilon_{cco} - \varepsilon_{cc} \right) \sqrt{1 - \left( \frac{f_{cc}' - f_{cco}}{f_{cc} - f_{cco}} \right)^2} + \varepsilon_{cco} \quad (3)$$

The shape of the concrete stress-strain curve between $f_{c}'$ and $f_{cc}$ ranges from approximately linear at $e = 0$ to the non-linear function of Popovics at $e = \infty$, which is given in Equation 1, depending on the value of $f_{cc}$. In order to allow for the gradual and smooth transition between the upper and lower bounds, a modified expression of Equation 1 is used, as shown in Equation 4, where a shape parameter $\alpha$ has been introduced.

$$\frac{f_{cc}}{f_{c}'} = \frac{\bar{\varepsilon}}{(ar) - 1 + \bar{\varepsilon}^{-1}} \quad (4)$$

Where $\bar{\varepsilon} = \varepsilon_{cc} / \varepsilon_{c}'$. Knowing $f_{cc}$ and $\varepsilon_{cc}$, Equation 4 can be solved for $\alpha$. The full curve between $f_{c}'$ and $f_{cc}$ can then be established using the same equation, Equation 4, to get different points (stresses and corresponding strains) using the obtained value of $\alpha$. A trial and error procedure would be used to solve Equation 4 for $\alpha$, due to its complexity.
The analysis procedure, using the variable confinement model, can be summarized as follows: 1) Use any available confinement model, such as the one by Fam and Rizkalla to establish the stress-strain curve of confined concrete under pure axial compression as shown in Fig. 8-a. 2) Use the model by Popovics, given in Equation 1, to establish the unconfined stress-strain curve of unconfined concrete as shown in Fig. 8(a) and the curve is terminated at axial strain $\varepsilon_{uc}$ equals to the ultimate axial compressive strain of the FRP tube, obtained from classical lamination theory. 3) At each eccentricity $e$, Equations 2 and 3 are used to calculate the ultimate strength $f_{ce}$ and corresponding strain $\varepsilon_{ce}$. 4) Using $f_{ce}$ and $\varepsilon_{ce}$, Equation 4 can be used to calculate the shape factor $\alpha$. 5) Using $\alpha$, Equation 4 can be used again to establish the full stress-strain curve ($f_{ce}-\varepsilon_{ce}$) between $f_c$ and $f_{ce}$ as shown in Fig. 8-b. The part of the curve before $f_c$, is assumed similar to the unconfined curve. 6) The obtained stress-strain curve of concrete along with the stress-strain behavior of FRP tube obtained from classical lamination theory are used in the cracked section analysis (for that particular eccentricity) to obtain one point on the interaction diagram.

The model has been applied to the test specimens, in order to predict the interaction diagrams, using the procedure described above. Fig. 6-a and 6-b shows the experimental results as well as the predicted interaction diagrams using different confinement mechanisms, for specimens of Types I and II respectively. Fig. 6-a shows that the variable confinement mechanism provides the best prediction. It however over-estimated the moment at the balanced point by 1.6 percent and under-estimated the axial load by 17.8 percent. It is also noted that the full confinement mechanism provides reasonable prediction of the interaction diagram, however, it overestimates the bending capacity under low axial loads or under pure bending. The unconfined concrete model significantly underestimated the interaction diagram. However, under pure bending, the unconfined concrete model with extended strain softening predicts the bending capacity very well. Fig. 6 also shows the contribution of the plain concrete core (without the effect of the tube) to the interaction diagrams. Fig. 6-b shows that the full confinement model provides good agreement with the test results, including the case of pure bending, mainly due to the very low level of confinement of this type of tube in the first place (in comparison to Type I). This is evident by the fact that both the confined and unconfined models provided very similar predictions at high axial load level for Type II specimens. For this reason, there was no attempt to use the variable confinement model in Type II specimens, as the predicted interaction curve would have been very close to that predicted using the full confinement model. The full confinement model underestimated the moment at the balanced point by 4.6 percent and over estimated the axial load by 7.4 percent for Type II specimens.

By examining the interaction diagrams, four distinct zones can be recognized as shown in Fig. 6-a and 6-b. Zone 1 represents the contribution of plain concrete core alone. Zone 2 reflects the contribution of the FRP tube, provided that the confinement effect on the concrete core is ignored. In this case, it can be noticed that the contribution of the tube (zone 2) to the bending moment is much more significant than it is to the axial load, since it provides the flexural tension reinforcement element to the system. It should also be noted that the sizes of zones 1 and 2 are very similar for both Type I and II specimens. Zone 3 reflects the effect of the confinement mechanism imposed by the FRP tube, which
is much more significant in case of Type I specimens than it is in Type II, due to the better confinement of the tube. The confinement effect is insignificant under pure bending, however, the contribution of zone 3 becomes more significant as the axial load increases, due to the larger portion of the section becoming under compression, as the neutral axis shifts. Zone 4 reflects the effect of the extended strain softening (ductility) for concrete beyond the 0.003 axial strain.

**Design Charts**

For design purposes, the procedure stated above is quite elaborate, therefore, a computer program has been developed to produce design charts for some of the available standard products such as the Lancaster Composite concrete-filled FRP tubular piles. Fig. 9 shows sample interaction curves of four standard sizes of the Lancaster Composite piles that can be used for design purposes. For a given design moment and axial load, the charts will assist the designer in selecting the appropriate size of the pile. The charts also show the locus of the balanced failure points on the curves. Fig. 10 shows the moment-curvature charts of the same piles, which can be used for design of fender piles subjected to pure bending. Fig. 11 shows the axial load-strain behavior of the same standard size piles, which can be used to design axially loaded piles. Also shown in Fig. 10 and Fig. 11, are the cut-off lines representing different levels of factor of safety (FOS) of 1.5, 2 and 3. For a given design moment or axial load, the most suitable size can be determined based on the desired safety factor.

![Fig. 9. Design interaction charts of the Lancaster Composite CFFT piles](image)

![Fig. 10. Moment-curvature design charts of the Lancaster Composite CFFT piles](image)
FIELD APPLICATIONS

CFFT has been successfully used in a number of field applications including marine piles and bridges. In the following sections, examples of the two types of field application are presented.

CFFT for Bridge Piers

In 2000, the Virginia Department of Transportation (VDOT) employed CFFT composite piles for an entire bent of the new Route 40 Bridge over the Nottoway River in Sussex County, Virginia. The FRP tube used for this pile is a Glass-FRP tube, type CP40, supplied by Lancaster Composite Inc., of Lancaster, Pennsylvania. The 13.1 m long tube was fabricated using a filament-winding technique, where E-Glass continuous fiber roving were impregnated with polyester resin and wound over a rotating steel mandrel, following a predetermined winding pattern. The fiber volume fraction of the FRP tube was 51.2 percent. The FRP tube has an average outer diameter of 625 mm and a structural wall thickness of 5.67 mm. The wall structure of the tube consists of five layers. The two inner and outer layers contain fibers oriented at \( \theta = \pm 35 \) degrees, where \( \theta \) is the inclination of the fibers with respect to the longitudinal axis of the tube. The middle layer contains fibers oriented at \( \theta = 85 \) degrees. The FRP tube was filled with a 41 MPa concrete mix that includes an expansive additive to reduce the effect of shrinkage, and consequently, enhance the bond between the concrete and the FRP tube. Wooden end plugs were used to seal the tubes and concrete was pumped from one end as shown in Fig. 12. The CFFT piles were driven using a hydraulic impact hammer with a ram weight of 71.2 kN. A 190 mm thick plywood pile cushion was used during pile driving. Fig. 13 shows the piles during and after driving.

After the piles were driven, and prior to casting the RC cap beams, special preparation of the pile heads were necessary in order to facilitate connecting the piles to the cap beams. Eight, 25.4 mm diameter holes were drilled through the top flat surface of each pile, using a regular rock drill. The holes were 457 mm deep, parallel to the longitudinal direction of the piles. The holes were equally spaced in a 447 mm diameter circular pattern. Eight, 1219 mm long No. 7 steel rebar were inserted in the holes and epoxy resin was used to
secure the bars inside the concrete. The formwork arrangement of the cap beams were placed such that the bottom surface of the cap beam is 152 mm below the upper surfaces of the piles to allow for embedment of the piles inside the cap beam as shown in Fig. 14. Fig. 15 shows a general view of the new completed bridge pier, which is supported entirely by CFFT composite piles.

Fig. 12. Filling of FRP tubes with Concrete. Fig. 13. Piles during and after driving.

Fig. 14. Connection of pile to cap beam. Fig. 15. Completed bridge pier.

**CFFT for Marine Piles**

The CFFT composite pile can be used in marine environments in a variety of applications as shown in Fig. 16. The most common applications include:

(a) **Fender Piling**: In front of marine structures as a buffer to absorb and dissipate the impact energy of a ship during berthing and prevent vessels from going underneath the pier.

(b) **Dauphins**: Dauphins are groups of piles placed near piers and wharves to guide vessels into their moorings, to keep them away from structures or to serve as mooring points.

(c) **Light Structural Piling**: To support the loads of light-duty piers and wharves. Bracing between piles is normally used to increase the strength and stiffness of the foundation of the structure.

(d) **Bridge Pier Protection**: Piles can also be used to create protective structures for bridge piers and to guide vessels into the channel and away from bridge supports.

The following sections describe three selected field applications of the CFFT composite pile in marine environments. The unique features of each project are highlighted.
Naval Station, Ingleside, Texas: US Naval Station Ingleside in the Gulf of Mexico houses the Navy’s mine sweeper class vessels that come for routine degaussing (a process that removes electronic fuzz and static from the very sensitive detection equipment on board). At such facilities, steel and iron are not allowed to be used in construction of the pier. CFFT piles were the most suitable for such requirements. Gee & Jenson as well as Whitney, Bailey, Cox and Magnani consultants were responsible for the design of the piers and piles. Southern Division NAVFAC coordinated the design efforts and the project was built by Orion Construction. The project utilized the composite piles in different applications including fender piles, structural battered (inclined) and vertical piles, and dauphin clusters as shown in Fig. 17. A total of 180 CFFT composite piles of 367 mm diameter and 22 m long were used. The project was completed in 1998.

Port Hadlock, Indian Island, Washington: The Navy is in the process of renovating the ammunition pier at Port Hadlock, on Indian Island, Washington. Deep draft transport ships utilize this facility, and accordingly, long fender piles are necessary to protect the pier. A total of 138 CFFT piles of 418 mm diameter and 25.9 m long were used as shown in Fig. 18. The piles were transported to the job site by way of barge to minimize costs. The piles have been fitted out with a 38 mm thick HDPE sleeve to protect from abrasion abuse. Blaylock Engineering was the consultants for the Navy and Oceaneering International was the general contractor organizing the project. The project was completed in 2002.

Brooklyn Navy Yard, New York City, New York: The New York and New Jersey waterfront is contaminated with marine borers capable of causing severe damage, even to the heavily treated wood piles. The State and local environmental agencies have banned the use of toxic wood treatments. In the Brooklyn Navy Yard, many waterfront portions of the facility are in need of serious repair. CFFT piles have been selected for this project. A total of 38 CFFT piles of 323 mm diameter and 25 m long were used to support a wharf.
extension as shown in Fig. 19. McLaren Engineers Nyack, NY designed this project. The project was completed in 1999.

CONCLUSIONS

(1) CFFT have great potential for marine pile applications. Advantages over conventional piles include non-corrosive characteristics, elimination of internal steel reinforcement and utilizing the FRP tube as reinforcement and permanent formwork simultaneously.

(2) The variable confinement model provides best prediction of interaction curves of CFFT. The full confinement model also provides reasonable prediction, however, for tubes with adequate confinement stiffness it overestimates bending strength at low axial loads. The unconfined concrete model underestimates the interaction diagrams for tubes with adequate confinement stiffness.

(3) Laminate structure of FRP tubes significantly affects the interaction diagram.

(4) In CFFT, both tension and compression failures are governed by the FRP tube.

(5) For a given laminate structure of FRP tube, increasing the wall thickness of the tube increases the axial and bending strengths. It should be noted that there are several combinations of wall thickness and laminate structure that satisfies a particular combination of bending moment and axial load.

(6) For both thin and thick tubes, increasing the ratio of fibers in axial direction significantly increases the flexural strength. However, increasing the ratio of fibers in the hoop direction would increase the axial strength of CFFT with thin tubes only.

(7) The proposed analytical model is used to develop simple design charts for specific CFFT products. Efforts are underway for development of design guidelines through the American Concrete Institute (ACI) Committee 440-J (FRP Stay-In-Place Formwork).

(8) A number of field applications in bridges and marine piles using CFFT has been successfully completed.

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