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Reprinted from

Advances in Structural Engineering

Volume 15 No. 4 2012

MULTI-SCIENCE PUBLISHING CO. LTD.
5 Wates Way, Brentwood, Essex CM15 9TB, United Kingdom
Innovative Use of FRP for the Precast Concrete Industry

S. Rizkalla¹, G. Lucier¹ and M. Dawood¹,²,*
¹Department of Civil, Construction & Environmental Engineering, North Carolina State University, Raleigh, NC, USA
²Department of Civil and Environmental Engineering, University of Houston, Houston, TX, USA

Abstract: This paper presents several advancements in the use of fiber reinforced polymer (FRP) materials for the precast concrete industry. Precast concrete members are commonly selected for reasons such as the high level of quality control used in their production, the durability of the finished structure, reduced labor costs and shorter construction schedules, and the economics of scale achieved with mass-production of components. The environmental durability, high strength to weight ratio, and ease of installation of FRP reinforcements an attractive alternative material for the precast industry. This paper presents several advancements in the use of FRP grid as flange reinforcement for precast double-tee members, as a shear transfer mechanism for thermally efficient composite and partially-composite load bearing wall panels, as reinforcement for precast architectural cladding panels. Each of these applications highlights the advantages of using FRP materials to achieve significant enhancement of the structural, thermal and architectural performance. The innovative use of the FRP materials and the unique construction techniques described have resulted in the development of safe and functional structures, as demonstrated by the research conducted by the authors and others in collaboration with the precast industry.

Key words: precast concrete, double-tee, wall-panels, composite action, architectural cladding, bridge girders, CFRP grid.

1. INTRODUCTION

Researchers have been studying new and innovative uses for composite materials in civil engineering infrastructure for the past 20 years. The progress of the research has led to the extensive use of composites for repair and rehabilitation of existing buildings and bridges. The relatively high initial costs of FRP materials have historically proven to be a barrier to the adoption of this technology for new construction, despite the potential life-cycle cost savings that have been reported.

The use of fibers and plastics for prestressing concrete members was first suggested by Freyssinet in 1938. In 1949, Freyssinet foresaw the practical challenges in implementing FRP reinforcement, many of which are still currently faced by engineers, including the variability of the material properties, the brittleness of the material, durability, bond, and cost (Leonhardt 1964). Therefore, the use of FRP materials in reinforced and prestressed concrete is not a new challenge, but one that is decades old. Recently, the weight savings, environmental durability, and reduced labor costs associated with the use of fiber reinforced polymer (FRP) composite materials have encouraged the use of these materials by the North American precast concrete industry. The research undertaken has helped to overcome many of the practical, technical, and economic challenges associated with implementing FRP materials in several types of precast concrete members.

*Corresponding author. Email address: mmdawood@uh.edu; Fax: (713) 743-4260; Tel: (713) 743-2983.
The use of carbon FRP (CFRP) materials in the fabrication of precast concrete members in North America was led by the construction of several bridges in Canada (Rizkalla and Tadros 1998; Rizkalla et al. 1994) and in the United States (Grace et al. 2002). These applications demonstrated the effective use of CFRP materials as prestressed longitudinal reinforcement for various types of prestressed girders including AASHTO, bulb-tee and double-tee type precast members. The use of CFRP transverse reinforcing in the girders of the Taylor Bridge in Canada (Rizkalla et al. 1994) and the Bridge Street Bridge in Michigan (Grace et al. 2002) demonstrated that CFRP materials could be used as the only internal reinforcement in precast concrete bridge girders. The use of FRP materials in these applications was complemented by the installation of advanced sensing technologies to monitor the long-term performance of the structures. After 15 years in service, monitoring of the Beddington Trail Bridge in Canada indicated no significant degradation of the CFRP reinforced bridge girders.

Other researchers have demonstrated that the environmental durability of FRP materials make them suitable for use as stay-in-place formwork and as external reinforcement for precast concrete elements. Concrete-filled FRP tubes (CFFT) have recently been used as precast piles in the construction of one bent of the Route 40 bridge in Virginia (Fam et al. 2003). The behavior of CFFT precast piles can be optimized by installing internal prestressing strands (Fam and Mandal 2006), or by incorporating a central void in the pile (Qasrawi and Fam 2008) to reduce the weight and to optimize the utilization of the concrete material properties. Another innovative use of CFFT members is in the construction of modular precast CFFT bridge piers (Zhu et al. 2006). This study indicated that the use of precast CFFT members can eliminate the need for internal steel reinforcement except at the joint locations to maintain ductility.

Until recently, the use of FRP materials in the precast industry has focused on bridge applications. This paper presents the research work related to the use of FRP materials in the precast concrete industry specifically for building applications. The paper focuses on the use of FRP materials in several selected precast products including double-tee beams, insulated precast load-bearing wall panels, and architectural cladding.

2. DOUBLE-TEE BEAMS
Precast double-tee beams are commonly used in the construction of parking decks and roof structures. In these applications, the top surface of the flange is often subjected to severe environmental exposure including rain, snow, and de-icing salts. Since precast members are optimized to provide the most structurally efficient and cost effective use of materials, the flanges of these members are relatively thin, typically in the range of 50 to 100 mm. These thin flanges are traditionally reinforced with steel welded wire fabric to control shrinkage and thermal cracks, and to transmit the load from the flange to the stems of the double-tees in one-way slab-bending action. Over time, under harsh exposure conditions, the flanges are subjected to penetration of moisture and chlorides. This typically leads to corrosion of the reinforcement, cracking and spalling of the cover concrete, and discoloration of the precast member. The non-corrosive properties and high strength-to-weight ratio of CFRP grids make them ideally suited to replace conventional steel welded wire fabric as reinforcement for the flanges of double-tees as shown in Figure 1(a). Further, the grid spacing and size of individual CFRP ligaments can be optimized to meet the structural requirements and construction limitations of various applications. Typically grid spacing ranges between 38 mm to 102 mm and grid strengths range between 27 kN/m to 109 kN/m.

In addition to enhancing the durability of precast concrete members, lightweight CFRP grids are commercially available in coils which can be installed using an automated process. A specially designed trolley, shown in Figure 1(b), is mounted on rails above the formwork for the double-tee member. Coils of CFRP grid are mounted on the trolley, and the CFRP...
grid is dispensed and pressed directly into the fresh concrete using a mechanism designed to ensure that the CFRP is placed at any depth specified by the designer. Since the CFRP grid is non-corrosive, smaller concrete cover thicknesses, typically within the range of 19 mm can be specified. This technique accelerates the construction process while reducing the associated labor costs and improving the quality control.

To examine the serviceability and failure mode of precast double-tee flanges reinforced with CFRP grids, two full-scale 3.6 m wide by 12.2 m long double-tees were tested. The double-tees were subjected to uniformly distributed load tests and concentrated load tests to evaluate the behavior of the flanges under extreme loading conditions (Lucier et al. 2010b).

### 2.1 Uniformly Distributed Load Tests

A unique experimental program was undertaken which allowed the application of a uniformly distributed load to the double-tees by constructing a pressure chamber around the double-tee members and reducing the pressure within the chamber using high-powered vacuums, as shown in Figure 2. The external uniform load was applied by the atmospheric pressure, as shown in the figure, which was higher than the pressure inside the chamber.

![Figure 2. Chamber cross-section (sketch), photograph of specimen in chamber under load (inset), and photograph of specimen with chamber under construction (below) (Lucier et al. 2010b)](image)

The two tested double-tees were similar except for the thickness of the top flanges. Beam DT1 was an untopped member with a flange thickness of 50 mm. As such, DT1 was designed to resist the loads during construction of a 50 mm thick cast-in-place concrete topping with uniformly distributed live load, L, dead load, D, and snow load, S, of 0.48 kPa and 1.82 kPa and 0 kPa respectively. Beam DT2 was a pre-topped member with a top flange thickness of 90 mm. In addition to the dead load, D, due to self-weight, the member was designed to resist a live load, L, and snow load, S, of 1.9 kPa and 0.96 kPa respectively. Members DT1 and DT2 were subjected to sustained factored loads for 1 hour and 24 hours respectively and subsequently loaded to failure. The limiting load combination that was considered was $1.2D + 1.6L + 0.5S$ in both tests. Therefore, the sustained factored loads applied to DT1 and DT2 (in addition to the self-weight) were 3.20 kPa and 3.97 kPa respectively. The members were instrumented to measure the strains and deflections at various locations along the top flange.

Both of the tested double-tees were capable to resist the applied sustained load while exhibiting minimal residual deflections upon recovery. The subsequent ultimate load tests indicated that flanges of members DT1 and DT2 were able to sustain a maximum load equal to 1.3 and 1.9 times the full factored load, respectively. DT1, which had a 50 mm thick flange, failed along the entire span of the double-tee due to the nature of the uniform loading condition which was applied. The failure occurred due to formation of a large longitudinal crack in the overhanging flange near the outer edge of one of the stems which was accompanied by rupture of the CFRP grid. DT2, which had a 90 mm thick top flange did not fail, and the test was halted when the applied load was equal to 1.9 times the maximum factored load. This load level corresponded to the maximum capacity of the test chamber. As expected, test results indicated that the flange behavior of the double-tees was highly dependent on the flange thickness. For the two tested double-tees, increasing the flange thickness from 50 mm to 90 mm increased the flexural rigidity of the overhanging flange by more than five times. This resulted in a significant increase of the cracking load, a reduction of the measured flange deflections, and more effectively controlled crack widths, even at the factored design load.

The measured net deflection profiles of the flanges at the mid-spans of the double-tee members are shown in Figures 3(a) and (b). Inspection of the figures indicates that, under service and factored loading conditions, the tips of the flange exhibited minimal deflections. Measurements of the transverse strain at the top of the
flange on either side of the flange-to-web juncture indicated that the load-strain behavior remained linear beyond the service load levels for both of the tested members. This indicates that the flange remained uncracked leading to the small measured deflections.

At the ultimate loading conditions, however, the measured strains exhibited highly non-linear behavior which indicates significant cracking and possible crushing of the concrete. Figure 3 indicates that the tips of the overhanging flanges of both members deflected considerably under the ultimate loading conditions. Comparison of the figures indicates that the thickness of the top flange had a significant effect on the deflection profile. The thinner flange of member DT1 was relatively flexible compared to the stems of the double-tee. Due to this relative flexibility, the tips of the flanges exhibited large downward deflections. In contrast, the flange of member DT2 was relatively thick and rigid. Consequently, the deflection profile across the width of the flange was uniform and equal to the deflection of the stems up to the factored load level as shown in the figure. At higher load levels, approaching ultimate, the formation of longitudinal cracks caused the tops of the stems of DT 2 to rotate towards each other slightly. This induced large downward deflections at the center of the flange and large upward deflections of the flange overhangs as shown in the figure.

2.2. Concentrated Load Tests

Since member DT2 remained intact after the application of the uniformly distributed load, additional testing was conducted to evaluate the resistance of the top flange to concentrated loads, as required by Precast/Prestressed Concrete Institute (PCI) design guidelines. This loading condition simulates the load applied by jacking up a passenger vehicle in a parking garage, the concentrated load case which typically governs the design. The typical test setup is shown in Figure 4. The concentrated load was applied using a 114 mm × 114mm steel bearing plate set on top of a thin rubber sheet at several locations along the top of the flange, as shown in the figure. Figure 4 also shows the observed failure pattern at each of the test locations. The measured failure load for each test location is given in Table 1. The numbered test locations in the table correspond to the numbered locations shown in Figure 4. It is important
**Table 1. Double-tee concentrated load test results**

<table>
<thead>
<tr>
<th>Test location</th>
<th>Description</th>
<th>Maximum applied load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Corner of flange</td>
<td>11*</td>
</tr>
<tr>
<td>2</td>
<td>End of flange (between stems)</td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>Edge of flange overhang</td>
<td>36</td>
</tr>
<tr>
<td>4</td>
<td>Center of flange</td>
<td>99</td>
</tr>
</tbody>
</table>

*Capacity of single corner. In practice, welded steel plates would be used to connect the top flanges of adjacent DT’s. Thus, load would be shared between 2 DT’s and nominal capacity would be expected to be doubled to 22 kN.

To note that in field applications, loads applied near the edge of a member would normally be supported by the connected flanges of two adjacent double-tees. To share loads and develop diaphragm action, double-tee flanges are welded together at discrete points along the span, and are also welded to the spandrel beams at their ends. Therefore, it can be seen that the measured ultimate capacity at all load locations exceeded the design value of 13 kN specified in the Precast/Prestressed Concrete Institute Design Handbook (PCI 2004).

**3. PRECAST CONCRETE LOAD-BEARING WALL PANELS**

The use of reinforced and prestressed concrete sandwich wall panels has been in practice for approximately 50 years. These panels consist of two concrete wythes separated by a rigid foam core. These highly efficient precast members can serve multiple purposes in a structure. The wall panels can act as the primary load carrying system to transfer gravity loads and lateral loads, due to wind or seismic events, to the foundation. Further, the presence of the foam core increases the overall insulation properties of the panel, thereby improving the overall thermal characteristics of a structure. Finally, the exterior face of the precast panels can be constructed with an architectural finish to contribute to the overall aesthetics of a structure. Panels can be designed to act in fully composite action, partial-composite action, or non-composite action depending on the intended purpose of the panel. The research reported in this paper deals with full and partial-composite action of two concrete wythes separated by a rigid foam core. Composite action between the two concrete wythes is achieved by the same CFRP grid used for the double-tee members.

For conventional precast concrete fully composite structural wall panels, the shear connection between wythes is typically provided by casting concrete solid zones within the core of the panels, or by using through-thickness steel ties or trusses. A series of tests was conducted to investigate the behavior of panels in which the shear transfer was provided by a steel truss assembly (Benayoune et al. 2007). The research findings indicated that the panels behaved as nearly fully composite up to failure. While the structural efficiency of traditional wythe connections is clearly demonstrated, other research indicates that traditional connections may be thermally inefficient due to thermal bridging. For example, one study showed that solid concrete zones with an area of 1 percent of the panel surface area, or steel pin connectors with an area equal to 0.1 percent of the panel surface area, can reduce the insulation properties (R-value) of a wall panel by up to 40 percent (Eina et al. 1991). The locations of the shear connectors form “hot spots” on the exterior of the structure due to thermal bridging between the two wythes. In these cases heat is transmitted through the shear connectors resulting in a thermally inefficient structure with increased heating and cooling costs and greater energy consumption.

This paper describes the research undertaken to use the CFRP grid described above to provide a shear transfer mechanism to achieve fully-composite action between the two concrete wythes of a concrete sandwich wall panels. The effective use of CFRP grid as a shear transfer mechanism would result in a structurally and thermally-efficient wall. The proposed shear-transfer mechanism involves cutting the square CFRP grid at a 45° angle, as shown in Figure 5(a). The grid is then embedded through the foam core to project approximately 3/4” beyond each face of the foam. One wythe of the panel is cast in a horizontal orientation. The foam core is placed onto the fresh concrete and the 3/4” CFRP projection is embedded into the wythe. The second wythe is subsequently cast on top of the core and encases the 3/4” of CFRP grid protruding from the other face of the foam. The assembly is then allowed to cure. In this configuration, the FRP reinforcement is oriented to resist shear stresses through a truss mechanism. The cross-section of a typical panel is shown in Figure 5(c). In some cases, pilasters are used to transmit vertical loads from the roof system to the foundation while providing additional rigidity to the overall panel.

The experimental research program that was undertaken to evaluate the behavior of sandwich panels constructed with the proposed CFRP shear connectors included six 6.1 m tall wall panels and two 12.8 m wall panels. The panels were fabricated using different types of foam cores and different configurations of CFRP grid shear connectors (Frankl et al. 2011; Lucier et al. 2010a). The 6.1 m tall panels were tested under the combined effect of simulated vertical gravity loads and lateral cyclic loads to simulate wind loading over the lifespan of a...
designed to transmit both ‘pushing’ and ‘pulling’ forces to the panel (not shown). The measured deflections and strains indicated that the CFRP shear grid provided adequate shear connection to develop nearly complete composite action at service load levels for all of the tested panels. The testing also indicated that the behavior of the panels and the degree of composite action was dependent on the type of foam used in the panel core.

Figure 5. FRP shear connectors for precast concrete sandwich panels: (a) CFRP ‘shear truss’ connectors; (b) Typical wall panel cross-section; (c) Testing of 6.1 m tall panel; (d) Testing of 12.8 m tall panel in horizontal orientation (Lucier et al. 2010a); (e) Typical crack pattern for 12.8 m tall panel (Lucier et al. 2010a); (f) Proposed guideline for design of CFRP shear connection for precast concrete wall panels (Hassan and Rizkalla 2009)

typical structure. The 12.8 m tall panels were subjected to transverse cyclic loading only. Typical testing of a 6.1 m tall panel is shown in Figure 5(d). The sustained vertical load was applied to the panel using hydraulic jacks and high-strength steel cables which were attached to the corbels on the inner wythes of the panels. The lateral cyclic loads were applied by a servo-controlled hydraulic actuator through a steel loading mechanism which was designed to transmit both ‘pushing’ and ‘pulling’ forces to the panel (not shown). The measured deflections and strains indicated that the CFRP shear grid provided adequate shear connection to develop nearly complete composite action at service load levels for all of the tested panels. The testing also indicated that the behavior of the panels and the degree of composite action was dependent on the type of foam used in the panel core.
Two of the tested 6 m tall panels were fabricated with expanded polystyrene (EPS) foam cores and contained a total length of 27 m of embedded CFRP shear grid. Transverse cyclic loads were applied to simulate the effect of wind speeds up to 241 km/hr. These panels exhibited stable behavior under the effect of the applied cyclic loading conditions and axial load without any indication of panel failure. A similar panel manufactured with extruded polystyrene foam (XPS) and the same configuration of CFRP shear grid exhibited unstable behavior under the effect of a simulated wind speed of 193 km/hr. This unstable behavior led to premature failure of the panel due to debonding of the concrete wythes and failure of the CFRP shear grid. Another panel constructed using XPS foam reinforced with 33 percent more shear grid (37 m) exhibited stable behavior at a simulated wind speed of 193 km/hr. Under ultimate loading conditions, the panels made with EPS foam exhibited nearly full composite action while the panels made with XPS foam exhibited partial composite action.

The two 12.8 m tall panels one with an EPS foam core and one with an XPS foam core, were tested in a horizontal configuration as shown in Figure 5(d). Transverse loading cycles were applied using hydraulic actuators to simulate the effect of wind loading conditions. Both of the tested panels exhibited highly ductile behavior with significant flexural cracking observed on both concrete wythes, as shown in Figure 5(e). The measured strain behavior indicated that after the completion of the cyclic load tests, the panels did not exhibit any indication of a degradation of the shear transfer mechanism. Further testing to higher load levels indicated that the panels were able to safely withstand applied loads equal to the factored load level. The measured strains indicated that the panels exhibited a high degree of composite action, even under these severe loading conditions.

Design of these composite load bearing panels is based on a proposed analytical model which implements partial interaction theory, and was developed to predict the degree of composite action of different panel configurations (Hassan and Rizkalla 2009). The total moment resisted by the panel, $M_T$, is carried by three components:

$$M_T = M_i + M_o + FZ$$  \hspace{1cm} (1)

where $M_i$ and $M_o$ are the moments carried by the inner and outer wythes respectively, $F$ is the shear force transmitted by the core of the panel and $Z$ is the distance between the centroids of the two concrete wythes. The proposed procedure is based on an iterative solution approach in which the shear force is iterated until equilibrium and compatibility are satisfied. The moment-curvature response of each of the two wythes under the effect of the applied loads is considered in the analysis. The degree of composite action, $K$, is defined as the ratio between calculated shear force, $F$, and the shear force required to achieve fully composite behavior, $F_C$.

The analytical study confirmed that at the service load level, the panels exhibited nearly fully-composite action; while near the ultimate load level, the degree of composite action was reduced. The analysis indicated that the shear flow capacity of the panels fabricated using EPS foam was twice the capacity of the panels fabricated using XPS foam. This finding has been attributed to the relatively smooth surface of the XPS foam compared to the EPS foam. The rough surface of the EPS foam appears to promote a strong bond with the concrete wythes (Kim et al. 2009). Therefore, it was concluded that for panels constructed with XPS foam, the shear flow capacity was provided mainly by the CFRP grid with minimal contribution from the XPS foam. In contrast, for panels manufactured using EPS foam, the foam and the grid both contributed significantly to the total shear flow capacity of the system.

Based on the experimental and analytical studies, a design guide was established, as shown in Figure 5(f) (Hassan and Rizkalla 2009). The guide allows a designer to use a curve, developed from an iterative procedure for a particular panel cross-section, to determine the quantity of CFRP grid required for a chosen degree of composite action, $K$, and the required moment capacity. The quantity of shear grid is directly related to the required shear force at the interface between the two wythes shown on the curve. Three experimental data points are plotted on top of the design curve, and show excellent correlation with the proposed method. An alternative simplified design procedure, which is based on the principle of virtual work has also been proposed which can be used to predict the overall deflection of the panel due to flexure of the panel and shear deflection due to the effect of incomplete composite action (Kim et al. 2009).

4. PROPOSED INNOVATIVE CONCEPT FOR ARCHITECTURAL CLADDING

This new concept proposes the use of FRP materials reinforcement for flexural as well as shear mechanisms for architectural cladding. For architectural products, uniformity of color, crack mitigation, and thermal efficiency are primary considerations, in addition to the structural demands. One of the proposed construction techniques of an architectural cladding panel is shown in...
Figure 6(a). The panels consist of a thin exterior concrete diaphragm face, typically 50 to 100 mm thick. Since corrosion of steel reinforcement could lead to cracking or discoloration of the architectural façade, the façade is reinforced with the same CFRP grid reinforcement introduced in this paper. The outer façade is supported by a concrete Vierendeel frame which is reinforced with conventional steel reinforcing bars. This frame typically consists of triangular or trapezoidal shaped intermediate members. The two components of the panel are separated by an insulating foam core layer and connected by a 45° CFRP shear grid, similar to that used for composite wall panels, as shown in Figure 6(a). The Vierendeel frame, which supports the self-weight of the panel, is then attached to the primary load carrying frame of the structure. The CFRP reinforced panels can weigh as little as 30 percent of the weight of similar conventional steel reinforced precast architectural cladding panels. The primary weight savings are due to the reduction of the required concrete cover thickness to protect the secondary reinforcement from corrosion for conventional steel-reinforced cladding.

The second design system proposed for architectural cladding is shown in schematically in Figure 7. The panel consists of two thin concrete wythes and an inner rigid foam core as shown in the figure. Each of the concrete wythes is reinforced with the same CFRP grid for flexural, shrinkage and temperature. The same CFRP is grid cut at a 45° angle to provide the shear mechanism for the composite action of the two concrete wythes to resist the wind pressure. To investigate the behavior of the proposed panel, a large-scale chamber was constructed to apply uniformly distributed pressure and suction to simulate wind loads, as shown schematically in Figure 7. A full-scale architectural panel, 9.1 m long by 2.4 m high was tested.

The architectural panel was attached to a steel supporting frame at discrete locations, as indicated in Figure 7. The test setup includes additional rows of 2.4m high fiberglass panels to simulate the presence of windows above and below the architectural test panel. In a real structure, windows or curtain walls transfer horizontal loads to the architectural wall panel without enhancing its load carrying capacity. The simulated window units were attached along one edge to the architectural panel and along the opposite edge to the steel frame. All of the connections were designed to simulate typical connections in field applications. The completed test setup is shown in Figure 8.
Pressure and suction loads were produced by a large turbine fan and a specially designed control system. Positive and negative pressure cycles were conducted to simulate the effects of wind loading during 50 years of service life of the structure. The panel was subjected to over 5000 pressure/suction cycles at a load level corresponding to 75% of the service load and was subsequently loaded to the full factored design load condition. The panel was able to resist the applied loading in each direction (pressure and suction), without any indication of failure. The measured pressure-deflection relationship at the midspan of one of the typical test panel is given in Figure 9. Data from the 5000 lower-level load cycles are not shown. The figure indicates that under the effect of cyclic loading the test panels exhibited relatively narrow, but stable hysteresis loops. This behavior is expected when considering the relatively slender nature of the concrete wythes.

5. CONCLUSIONS

This paper summarizes recent and ongoing research investigating the use of FRP materials as reinforcement for sustainable, durable, and structurally efficient precast concrete members for building applications. The technologies discussed in this paper represent three selected advancements in the precast industry. The three products demonstrate the distinct advantages of using FRP materials in precast concrete products, and illustrate how the use of such materials can result in improved structural, thermal, and architectural characteristics. Typically, the use of FRP materials can result in long-term economic benefits when the life-cycle costs are considered. The advancements described in this paper represent the growing acceptance of FRP in the engineering community, and symbolize what can be achieved when new technologies are combined with ingenuity. These advancements further paint the picture of a bright future for composite materials in the precast industry, and, more generally, in civil engineering.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the contributions of AltusGroup, Gate Precast, and Metromont Corporation for providing support to many of the experimental programs presented in this paper.

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