Gold, W.J., G.J. Blaszak, M. Mettemeyer, A. Nanni, and M.D. Wuerthele, "Strengthening Dapped Ends of Precast Double Tees with Externally Bonded FRP Reinforcement," ASCE Structures Congress 2000, Philadelphia, PA, M. Elgaaly, Ed., May 8-10, CD version, #40492-045-003, 9 pp.

Strengthening Dapped Ends of Precast Double Tees with Externally Bonded FRP Reinforcement

William J. Gold, Gregg J. Blaszak, Matt Mettemeyer, Antonio Nanni, and Michael D. Wuerthele

Abstract

A 90,000 square meter, three-story, precast concrete parking garage in Pittsburgh, PA was strengthened in order to address concerns regarding distress in the double tee beams supporting each elevated floor. Initially, the distress was manifest in inclined cracks forming at the reentrant corner of the dapped ends. During an independent load test of sample tees, additional cracks at approximately 1.5 m from the dapped end formed and became so severe that, at roughly 75% of the design load, the tees were deemed to have failed. This failure condition resulted in an effort to strengthen the ends of the tees in the garage for shear and flexure. In assessing various strengthening alternatives, externally bonded carbon fiber reinforced polymer (FRP) reinforcement was determined to be the most cost effective, the least disruptive to the operation of the parking garage, and virtually unnoticeable once the installation was complete. A load test of several tees after the installation of the FRP reinforcement demonstrated that the retrofitted tees could support loads over 100% of the design load. This paper presents the details of the design, specification, installation, and testing of the FRP strengthening system for this large retrofit project.

Introduction and Background Information

The short term parking garage at Pittsburgh International Airport is a 3-story precast concrete structure. The two elevated levels (Levels 2 and 3) consist of 2.8-m wide by 18.5-m long precast/prestressed double tees. The double tee units are generally 760-mm in total depth and are dapped at the ends to bear on ledger beams.

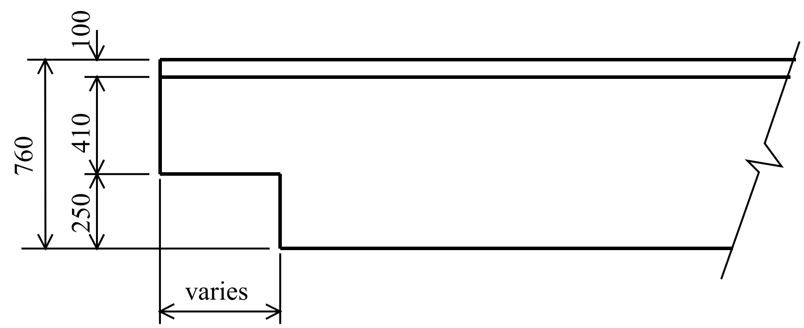

Figure 1: Typical Double Tee Dap Geometry

The garage was originally designed to receive asphalt overlays on Level 2 (250-mm overlay) and Level 3 (400-mm overlay). However, almost 5 years after the garage's original construction, these overlays had yet to be placed. Arising concerns for the structural adequacy of the precast double tee beams further delayed placement of the overlays. Cracking at the re-entrant corners of the dapped ends of the double tees was observed and initially gave cause for concern. Cracking was observed in nearly every double tee in the garage, and though the crack angle varied widely, most cracks were inclined between 0° (horizontal) and 45°.

Due, in part, to concerns arising from these cracks, an independent engineering firm was retained to perform a comprehensive condition survey of the garage. As part this condition survey, four of the double tee beams were selected to be load tested in order to assess the structural adequacy of these beams. During load testing, flexural-shear cracking developed at a location approximately 1.5-m from the dapped ends. As loading increased, the flexural-shear cracking became so severe that the beams were deemed to have failed at roughly 75% of their design capacity.

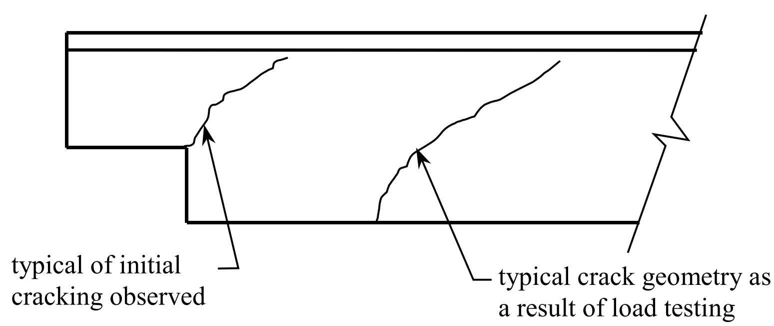

Figure 2: Crack Patterns Observed

At the recommendation of the engineering firm conducting the condition survey, the double tees required supplemental strengthening. The strengthening work had to be completed before any additional loads (such as the asphalt overlays) could be placed on the structure.

Strengthening System Selection

Several methods for strengthening the double tees were investigated including external post-tensioning, steel plate and steel angle bonding, and externally bonded FRP reinforcement. The primary considerations were economics, constructibility, durability, and aesthetics (Wuerthele, 1999).

From an economic perspective, the cost of installing various strengthening systems had to be carefully weighed. Factoring in to the economics was the speed of installation. It was important to choose a retrofit that could be installed within a short schedule so as not to disrupt the on-going parking operations. Long term limited access to parking meant inconveniencing the public and significant loss of revenue for the owner of the garage.

From a constructibility standpoint, the primary consideration was limiting the penetrations into the tee stems. The existing tee stems are prestressed with strands spaced 50-cm on center vertically throughout the depth of the stem. Any penetrations made into the stem could potentially damage the strands and further reduce the capacity of the tees. Therefore, drilling and through-bolting into the stem was not acceptable.

From an aesthetic standpoint, the retrofit had to be simple enough so as to not alarm the public both during and after installation. Strengthening methods that, when installed, would closely match the original appearance of the concrete surface were considered highly advantageous.

Since externally bonded FRP reinforcement could be installed relatively quickly without penetrating the tee stem and with minimal aesthetic impact, this retrofit method was chosen as the most suitable.

Engineering the FRP Reinforcement

The overall gravity loads that the double tees were designed to resist are shown in Table 1. The total demand on the Level 3 double tees is greater due to a thicker asphalt overlay and exposure to snow loading. Based on the independent condition survey, the deficiencies in the double tees that needed to be addressed where both flexural and shear deficiencies at the ends of each tee.

Table 1: Parking Deck Load Requirements

	Level 2	Level 3
Dead Loads		
• Self-weight	3800-Pa	3800-Pa
• Overlay	625-Pa	865-Pa
Live Loads		
• Live Load	2400-Pa	2400-Pa
• Snow Load	N/A	1450-Pa

Table 2 shows the factored shear deficiencies in the tees. The shear values listed are the magnitudes that the strengthening system alone had to resist in an ultimate strength condition.

Table 2: Factored Shear Deficiencies

Location	Level 2	Level 3
At Dapped End	49.5-kN	94.5-kN
At 8 ft from Beam End	36.0-kN	63.0-kN

In designing externally bonded FRP reinforcement to increase the shear capacity of a beam element, it is common to consider the FRP reinforcement in a similar manner to steel stirrups. The overall contribution of the FRP reinforcement to the shear capacity of a section can be computed using Equation (1) which is analogous to an equation typically used to compute the shear contribution of steel stirrups.

$$V_f = \frac{A_{fv} f_{fe} (\sin\beta + \cos\beta) d_f}{s_f} \tag{1}$$

where,

$$A_{fv} = 2nt_f w_f$$

The difficulty in applying this equation is in determining the level of stress that can be developed in the FRP reinforcement at the ultimate condition. Typically, with steel stirrups, the stirrups will yield at ultimate, and the stress in the steel can be taken as the yield stress. However, with FRP reinforcement it has been observed that the ultimate condition rarely results in the full strength of the material being utilized. The stress in the FRP material depends on the failure mode. There are two common ultimate limit states that govern the level of stress that can be developed in the FRP reinforcement.

At a high levels of strain in the FRP shear reinforcement, shear cracks can become so wide as to reduce or eliminate the shear contribution of the concrete itself. This is due to a loss in aggregate interlock along the internal shear crack plane. For this reason, the maximum strain in the FRP is typically limited to a value in the range of 0.4% to 0.6%.

Furthermore, when reinforcing double tee beams (as well as most other concrete beams) it is not possible to wrap around the entire section with “closed” reinforcement because of the presence of the top flange. An FRP “U” wrap is, therefore, used that is continuous along the two sides and bottom of the section but terminates at the connection between the stem and top flange. In this configuration, there is a second limit state due to the potential for the FRP to debond from the side of the beam. The level of stress in the FRP that will cause debonding is generally dependent on the concrete strength and the stiffness of the FRP sheet.

In order to accommodate both potential limit states, the approach taken for this project was to determine the level of stress in the FRP at ultimate based on a reduction factor, R. The reduction factor is a factor less than 1.0 that is applied to the tensile strength of the FRP material to result in the effective stress level in the FRP at ultimate.

$$f_{fe} = Rf_{fu} \tag{2}$$

The reduction factor used was determined from Equation 3 (Khalifa, et al., 1998). In this equation, the FRP is limited to a maximum strain level of 0.4% to maintain shear integrity of the concrete. However, the debonding failure mode is also considered by taking into account the stiffness of the FRP material ($n\,t_f E_f$) and the strength of the concrete (f'_c).

$$R = \frac{k_1 k_2 L_e}{11900\varepsilon_{fu}} \le \frac{0.004}{\varepsilon_{fu}} \tag{3}$$

where,

$$k_1 = \left(\frac{f'_c}{27.6}\right)^{2/3}$$

$$k_2 = \frac{L_e - d_f}{d_f}$$

$$L_e = \frac{415}{\left(nt_f E_f\right)^{0.58}}$$

For this project, the precast double tees have a nominal concrete strength of 31-MPa. The FRP material used for this project was MBrace™ CF 130 supplied by Master Builders, Inc. This material utilizes a high strength carbon fiber fabric and has a thickness, t_f = 0.165-mm; an elastic modulus, E_f = 227-GPa; and an elongation at rupture, ε_{fu} = 0.017-mm/mm. The ends of the double tees were to be reinforced continuously with the FRP reinforcement (as opposed to reinforcing with strips of reinforcement spaced apart). Therefore, the width of the FRP reinforcing, w_f, and the spacing of the reinforcing, s_f, were set equal to one another.

The depth of the FRP reinforcement, d_f, varied based on variations in the effective depth of the section, d_p. Some tendons closest to the bottom of the section were allowed to slip during casting of the beams. Depending on where along the span the tendons were engaged, the effective depth varied along the span.

Table 3 shows a summary of the design results. The strength reduction factor, ϕ, used for the FRP reinforcement was 0.70. This factor is more conservative than the factor used for steel stirrups to account for the novelty of this strengthening method.

Table 3: Summary of Design Results

Location	d_f	n	R	V_f	ϕV_f
Level 2					
• At dapped end	420-mm	1	23.5%	126-kN	85-kN
• At 8-ft from beam end	430-mm	1	23.5%	130-kN	90-kN
Level 3					
• At dapped end	420-mm	2	16.9%	175-kN	121-kN
• At 8-ft from beam end	430-mm	1	23.5%	130-kN	90-kN

In order to accommodate any horizontal component of force and to provide additional anchorage for the vertical FRP reinforcement, horizontal plies of FRP reinforcement were added to the sides of the beams as well. The final layout of the reinforcement is shown in Figure 3.

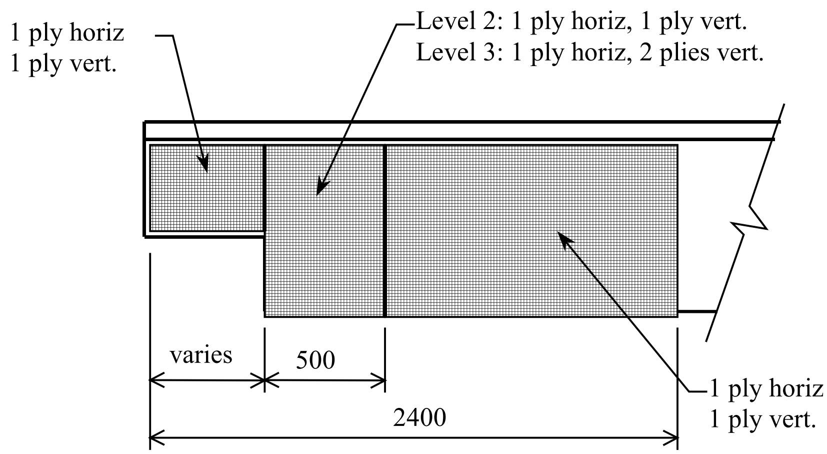

Figure 3: Layout of the FRP Reinforcement

In addition to the FRP shear reinforcement, two 100-mm wide strips of MBrace CF 130 were placed longitudinally on the soffit of the double tee stems. This reinforcement was used to limit the flexural strain level on the stem soffit to 0.005 mm/mm under a positive bending moment of 493-kN·m.

Installation

The installation for this project involved little in the way of concrete repair work since the double tees were in relatively good condition with no signs of corrosion. However, all of the existing cracks wider that 0.50-mm in the areas were the FRP was to be installed required epoxy injection. The surface to which the FRP was being installed also had to be profiled by water blasting to a minimum ICRI CSP 3.

Load Test Verification

In order to verify the addition of shear strength to the double tees, a load testing program was carried out as part of the scope of work.

A pilot test was conducted to prove the design of the FRP reinforcement. The pilot test was conducted on the same two double tees that were previously failed by load testing during the condition survey (one on Level 2 and one on Level 3). The failed tees were repaired by epoxy injecting the cracks that had formed and installing the FRP reinforcing system. These tees were then load tested using hydraulic jacks to apply the load. The load testing setup is shown in Figure 4.

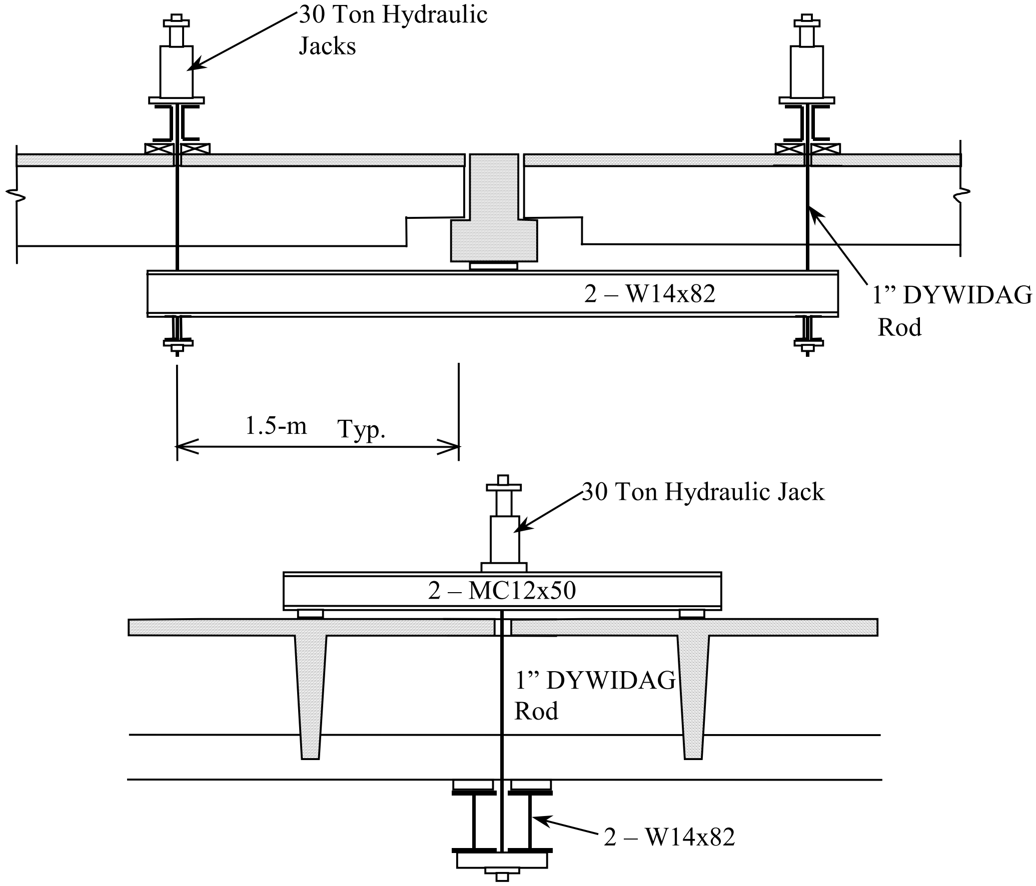

Figure 4: Load Testing Setup

In order to prove the FRP system, the test loads used had to generate at least 85% of the total design factored shear force at the support. During testing, deflections and strain levels in the beam were monitored to insure that the beam was performing in the elastic range. Table 4 summarizes the actual shear levels that were induced in the beams during the load test.

Table 4: Load Test Results

Level	Test Load Applied	Shear Applied at support	Design Factored Shear at support	Percent of Factored Shear at support
2	160-kN	143-kN	139-kN	102.6%
3	211-kN	152-kN	175-kN	87.0%

In order to assure that the quality of the installation was consistent throughout execution of the project, additional load tests were performed on randomly selected double tees. In all, 16 load tests were carried out and consistently revealed adequate shear capacity in the double tees.

References

Khalifa, A.; Gold, W.; Nanni, A.; and Abel-Aziz, M.I. "Contribution of Externally Bonded FRP to the Shear Capacity of RC Flexural Members*." J. of Composites in Construction*, ASCE, Vol. 2, No. 4, Nov 1998.

Wuerthele, M.D. "CFRP Strengthening at Pittsburgh International Airport's Short Term Parking Garage" *Proceedings of the Fifth ASCE Materials Engineering Congress*, Cincinnati, OH, May 10-12, 1999.

Corresponding Author

For further information, please contact: William J. Gold, Engineering Consultant, Structural Preservation Systems, Inc., 7455-T New Ridge Road, Hanover, MD 21076. Phone: (410) 850-7000. Fax: (410) 850-4111. e-mail: wgold@structural.net