Brittle Failures in Precast Parking Structures

Collapses of double-tee flanges are attributed to design and fabrication flaws

by William L. Gamble, Gordon H. Reigstad, and Jason Reigstad

"The measured failure pressure for all tested specimens corresponds to the cracking moment resistance of the flange…[and]…was not sensitive to the type of grid used... Immediate failure occurred after cracking and prior to development of the full strength of the CFRP [carbon-fiber-reinforced polymer] grid."

…"Due to the brittle nature of the failure, it is recommended to use a strength-reduction factor ϕ of 0.75, similar to the factor used for shear failure of concrete structures. This recommendation differs from the strength-reduction factor of 0.55 provided by ACI 440.1R-06[1] for sections reinforced with FRP [fiber-reinforced polymer] bars controlled by FRP rupture. The conservative value recommended by ACI 440.1R-06 is intended to prevent global failure in the event of rupture of the FRP bars. However, CFRP grid reinforcement is believed to have a uniform distribution of the reinforcement and the failure of one strand will not result in global failure."

The statements quoted in the textbox are from a paper published in 2015,[2] about 5 months after flanges on precast, prestressed double-tee (DT) members failed in a parking structure in North Carolina. That paper described tests of DTs constructed with a polymer-coated carbon-fiber grid product as the primary reinforcement for the flanges in test specimens—the DT flanges in the damaged parking structure were similarly reinforced (Fig. 1 and 2). Readers are asked to keep the statements in mind as they read this article, which includes information we discovered as the result of field observations and study of related peer-reviewed papers, marketing literature, design calculations, and construction documents.

Flange Reinforcement

The grid product that served as flange reinforcement comprised an array of flattened, polymer-coated carbon-fiber tows bonded to an orthogonal array of flattened, polymer-coated glass-fiber tows. The grid product was manufactured by a single supplier, and it was used by multiple precast concrete producers in the production of DTs for parking structures.

Construction documents and inspections of the failed flanges show that the failed areas contain only concrete, steel embedments placed along the flange edges (used as connections to adjacent flanges after installation of the DTs in the structure), and the grid product. As installed, the carbon-fiber tows in the grid are oriented transverse to the stems of the DTs (Fig. 1). We understand that this product was used in this way in more than 100 parking structures constructed from 2006 to 2015. To date, we know of three structures that exhibited brittle failures in DT flanges containing this product. This article is our attempt to set down some of the facts concerning parking structures containing the grid product.

Reported Failures

The failures have been reported by *ENR* (in References 3 and 4). They will be summarized herein, followed by discussions of the flexural resistance of the as-built members,

Editor's note: This article discusses garage structures that have been or are the subjects of litigation precipitated by the failure of flanges on precast, prestressed double-tee members. The first two failures occurred on February 19, 2015, and April 15, 2016, in parking structures in North Carolina. The authors have provided forensic legal support for the claimants in disputes with the contractors who built the structures. The second and third authors provided professional services to the owner of the structures, and these services included evaluating and designing repairs. All three authors are engaged in an ongoing study of the design and construction of similar structures. The views expressed are solely those of the authors and do not necessarily represent the opinion of ACI.

the required resistance, and accepted practices for ensuring ductile failure modes. Readers are reminded that the basic requirement of any structural design is to ensure that reliable resistance is equal to or exceeds the maximum probable applied load. Readers are also reminded that a basic requirement of any reinforced concrete design is to achieve a structure capable of ductile failure and thereby provide warning of imminent collapse. Based on the tests reported in Reference 2 and on the analyses provided in this article, the as-designed system provides no apparent ductility.

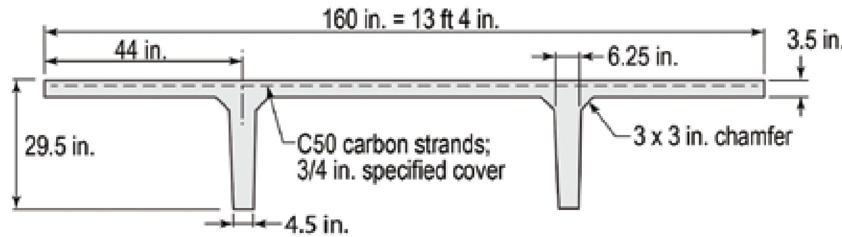

Fig. 1: Cross section of a double-tee (DT) member in which an epoxy-coated interlaid carbon-fiber mesh was provided as the flange reinforcement (based on the authors' forensic investigations) (Note: 1 in. = 25 mm; 1 ft = 0.3 m)

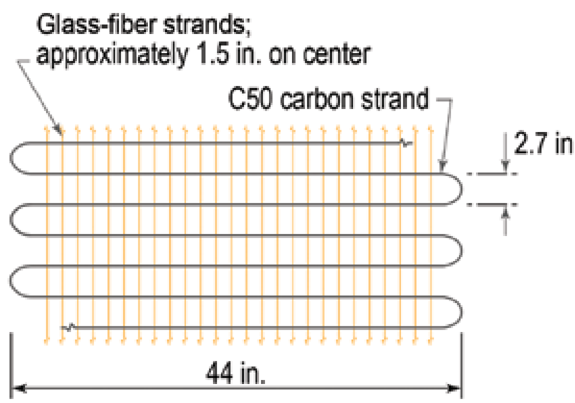

Fig. 2: Schematic showing the arrangement of the C50 and glass-fiber strands in the epoxy-coated fiber grid (based on the authors' forensic investigations) (Note: 1 in. = 25 mm)

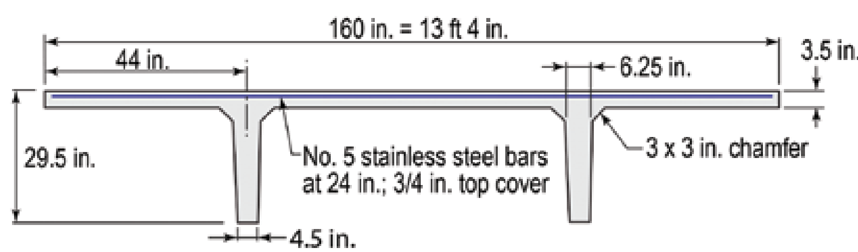

Fig. 3: Schematic of the repair completed in Structure A, with 3.5 in. thick flanges (Note: 1 in. = 25 mm; 1 ft = 0.3 m)

Incidents

Each of the subject structures was about 6 years old when one or more DT flanges fractured, and large pieces of concrete fell to the level below. The initial failure occurred on February 19, 2015, in a 1200-car parking structure at the Harrah's Cherokee Resort in Cherokee, NC (Structure A). The grid product in Structure A had carbon-fiber tows on 2.7 in. (69 mm) centers. The DTs were of normalweight concrete, with 3.5 in. (89 mm) thick, 13 ft 4 in. (4.06 m) wide flanges.

The failure occurred when the rear axle of a tow truck (also called a recovery vehicle) passed over a joint between two DT units. Both flanges fractured, large sections of the flanges fell to the level below, and the rear wheels of the truck dropped into the resulting gap. The rear-axle loading, including the load from the vehicle being towed, was later calculated by a consultant to be 13,620 lb (60.6 kN).

The second failure occurred on April 15, 2016, in a 2400-car parking structure, also at the Harrah's Cherokee Resort in Cherokee, NC (Structure B). This failure occurred as a Jeep Wrangler was exiting the parking structure and passed over a joint between adjacent DTs. As with Structure A, portions of the flanges fell to the level below. The grid product in Structure B had carbon-fiber tows on 2.7 in. centers. However, in contrast to the concrete in Structure A, the DTs in Structure B comprised semi-lightweight concrete (specified unit weight = 120 lb/ft^3 [density = 1920 kg/m^3]), with 4.75 in. (120 mm) thick, 13 ft 4 in. wide flanges. Per information supplied by the precast producer, the material and thickness selections for the DTs in Structure B were dictated by a fire-resistance requirement.

The third failure occurred on August 26, 2016, in the 650-car Vulcan parking structure on the campus of California University of Pennsylvania in California, PA (Structure C). This failure occurred as a single minivan passed over a joint between two DT units. Again, large portions of the flanges fell to the level below. We understand that the grid product in this structure had carbon-fiber tows on 4 in. (101 mm) centers. The DTs were of normalweight concrete, with 3.5 in. thick, 12 ft (3.66 m) wide flanges.

Initial actions

In August 2015, following a 6-month evaluation by the second and third authors, the DTs in Structure A were repaired by adding stainless steel deformed bars to reinforce the flanges (Fig. 3). A similar program began

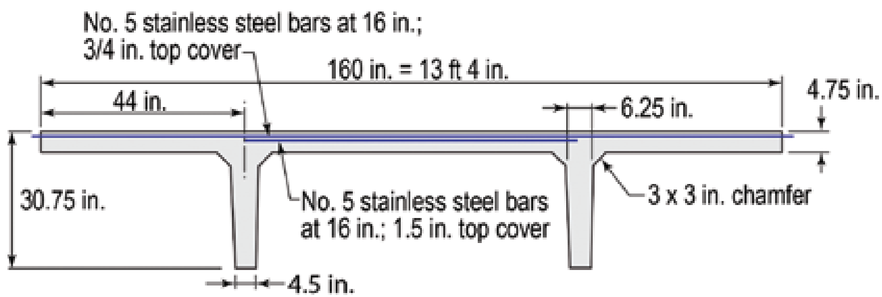

Fig. 4: Schematic of the repair completed in Structure B, with 4.75 in. flanges (Note: 1 in. = 25 mm; 1 ft = 0.3 m)

immediately on Structure B after the failure in that structure (Fig. 4). On February 9, 2016, the owner of Structures A and B filed suit against the contractor, precast manufacturer, and others.

As required by the construction contract, the owner's demands for restitution of damages resulting from the failures went to arbitration. The proceedings regarding Structure A concluded in April 2018. We will not report on the agreed terms related to Structure A or the lawsuits for Structures B and C, as they are not relevant to our central concerns.

We do not know if the DTs on Structure C have been repaired. However, Structure C remains closed, and the owner is pursuing litigation against the contractor.

Dimensions and properties

In a guide specification provided to designers by a consortium comprising many precast concrete producers and the grid producer, the grid is termed an "epoxy-coated interlaid carbon-fiber mesh."[5] According to information published by the grid supplier, each strand in the carbon-fiber portion of the grid contains about 50,000 carbon-fiber filaments, and the strand is termed C50 strand. Per Reference 2, the filaments total about 0.00286 in.^{2} (1.85 mm^2) in cross-sectional area. We have observed that the epoxy coating results in a very smooth surface on a somewhat irregularly shaped band that is about 1/16 in. (1.6 mm) thick by 1/8 in. (3.2 mm) wide.

In Structures A and B, the C50 strands were spaced at 2.7 in. on center, giving a unit area of carbon filament of 0.0127 in.^{2}/ft (26.8 mm^2/m). The design documents show that the grid product was to be placed in the flange with a nominal cover of 0.75 in. (19 mm) from the top surface of the concrete.

The grid arrangement consisted of continuously looped C50 strand bonded to orthogonal polymer-coated glass-fiber strands on approximately 1.5 in. (38 mm) centers (Fig. 2). The polymer coating bonded the intersections of the C50 and glass-fiber strands and thus set the spacing of the strands in each direction. Because the grid sheets are not as wide as the DT flanges, the producers installed at least three sheets across a DT flange. The sheets overlapped, and loops in the C50 strands would be expected to provide positive anchorage.

The Resistance Side of the Equation

The following discussion is focused on Structure A. As previously noted, the DT flanges in Structure A were 3.5 in. thick. The cantilever length on a typical DT flange was 37-3/8 in. (949 mm) (the length is from Reference 2—the dimension may vary slightly in the structures). The contract documents indicate that the 1/16 in. thick grid was to be placed with a nominal 0.75 in. cover, giving a design effective depth d of 2.7 in. Normalweight concrete was specified. A concrete strength f_c' of 6000 lb/in.^{2} (41 MPa) is assumed in the following calculations.

Strand force and resistance factor

The moment capacity is computed as strand force times lever arm. Calculations provided in Reference 2 use a mean strength of a C50 strand (1218 lb [5.42 kN]) as the strand force.

While the mean strength may be useful when trying to understand test results, we do not believe it is suitable for design. Based on product documents published contemporaneously with the DT fabrication for Structure A, the grid producer reported a tensile strength of 830 lb (3.69 kN) per strand, reportedly based on mean test strength μ minus 2 times the standadard deviation σ. In contrast, ACI 440.1R-06, Section 3.2.1,[1] recommends that "Manufacturers should report a guaranteed tensile strength" of $\mu - 3\sigma$. Application of the recommendations in References 1 and 6 to the design of the structure would have resulted in a tensile strength of 683 lb (3.04 kN) per strand. Further, as noted in Reference 2 and quoted in the textbox at the beginning of this article, the DT producer made the decision to use a strength reduction factor ϕ of 0.75 for flexure, rather than ϕ of 0.55 as recommended by ACI Committee 440[1] and supported by reliability analyses provided in Reference 7.

Lever arm and capacity

The simplest approach for finding the lever arm is to adopt the usual equation for nominal moment M_n

$$M_n = A_f f_f (d - a/2) \tag{1}$$

where A_f is the fiber area in the strand; f_f is the stress in the fiber; and a is given by $a = A_f f_f/(b(0.85f_c'))$. This introduces a strain compatibility problem: when the carbon-fiber strand fractures, the extreme fiber strain in the concrete will be well below the usually assumed compression strain of 0.003. Even so, we believe it gives a reasonably good estimate of the lever arm and moment capacity.

An alternative approach is to calculate the moment based on an elastic cracked section, which can be stated as:

$M_n = A_f f_f (d - kd/3)$ (2)

where $k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f$; ρ_f is given by A_f/bd; and n_f is the modular ratio E_f/E_c. The nominal moment found using Eq. (2) will be slightly smaller than that from Eq. (1). As a point of interest, recommendations provided in Reference 1 result in even smaller moments than found using Eq. (2). For either calculation, the capacity is extremely sensitive to the position of the grid. If the cover is 1 in. (25 mm) rather than 0.75 in., for example, the nominal values would be lowered by 8%. Measurements made after the failures showed that the product was not held in position within the flange, as the cover and thus the effective depth varied widely (Fig. 5).

The moment capacities from three different assumptions about the strand capacity are given in Table 1. Values are computed using Eq. (1), with d = 2.72 in. and d = 2.50 in. These are stated in the usual terms for slabs, in kip·in./ft.

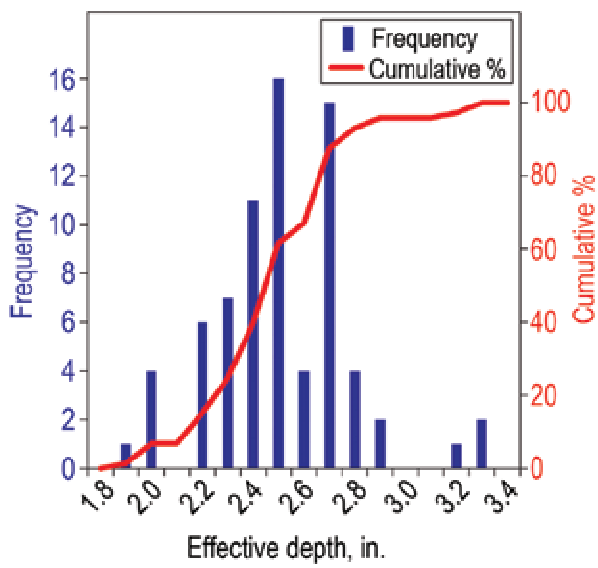

Fig. 5: Effective depth *d* measured on DTs installed on Level 4 of Structure A. Note that more than half of the measurements indicate that *d* is at or below 2.5 in. (63 mm)

The Load Side of the Equation

The design loads for parking structures are given in the International Building Code (IBC)[8] and ASCE 7-10.[9] For structures designed for vehicles carrying no more than nine passengers, the design loads are either:

- Dead load plus 40 lb/ft² (1.9 kN/m²) live load plus snow load; or
- Dead load plus a single concentrated load of 3000 lb (13 kN).

The load combination producing the larger forces governs, and the resultant forces are to be computed using the appropriate load factors.

For Structure A, the 3.5 in. flange dead load is 43.75 lb/ft² (2.1 kN/m²), and the specified snow load is 20 lb/ft² (1 kN/m²), leading to w_u = 1.2(43.75) + 1.6(40) + 0.5(20) = 126.5 lb/ft² (6.1 kN/m²). For a cantilever length of 37-3/8 in., m_u = 7.36 kip·in./ft (2.73 kN·m/m).

References 8 and 9 instruct the designer that the 3000 lb concentrated load represents the force from a jack, and it is to be applied to an area that is 4.5 in. (114 mm) square. The moment M is simply force P times cantilever length l. We would reasonably assume that a load applied at a joint between flanges will spread out at a 45-degree angle, leading to the moment being resisted by a width of $2l$. Thus, the moment becomes $m = Pl/2l = P/2$, where this represents a moment per unit width, such as kip·ft/ft or kip·in./in. As noted in Reference 10, this agrees quite well with the maximum local moment obtained from an elastic plate theory solution of $m = 0.509P$.

At the time Structures A, B, and C were designed, the sixth edition of the Prestressed Concrete Institute (PCI) Design Handbook[11] was widely used in the industry, and it provides an example illustrating the calculation of the moment due to the concentrated load in flanges of prestressed DT members. The example indicates that the moment is to be resisted by a width of $2l$ + 8 in., where the 8 in. (203 mm) is the assumed width of a tire. Using the PCI Handbook recommendations, the tributary width for the DTs in Structure A is 6.896 ft (2.1 m). The 3000 lb concentrated load is assumed to be shared evenly between two adjacent flanges, and the factored moment from this component of the load can be calculated as

m_u = 1.5 kip (1.6 × 37.375 in./6.896 ft) = 13.01 kip·in./ft (4.82 kN·m/m)

Table 1:

Summary of moment capacities with different strand force assumptions based on a 3.5 in. thick flange and effective depths of 2.72 and 2.50 in. Moment values for the latter effective depth are provided in parentheses ( )

Case	Strand force, lb	Strand stress, kip/in.²	ϕ	m_n, kip·in./ft	ϕm_n, kip·in./ft
1	1218 Value listed in Reference 2	426	1.0	14.49 (13.29)	14.49 (13.29)
2	830 = Avg. – 2σ Based on data published by the grid producer	290	0.75	9.92 (9.11)	7.44 (6.83)
3	683 = Avg. – 3σ Based on ACI 440.1R[1] recommendations	239	0.55	8.18 (7.51)	4.50 (4.13)

Note: 1 lbf = 4.4 N; 1 kip/in.² = 0.007 kN/mm²; 1 kip·in./ft = 0.37 kN·m/m

To this must be added the factored moment due to the dead load of the flange, 3.06 kip·in./ft (1.13 kN·m/m), for a total m_u = 16.07 kip·in./ft (5.96 kN·m/m). The total factored moment is much larger than the moment due to the factored distributed loads. It is also much greater than the capacities calculated as recommended by the grid producer or by ACI Committee 440[1] (refer to Table 1).

Splitting the load between two adjacent flanges is predicated on the assumption that there are adequate, intact, DT flange connectors. These connectors are steel shapes that are embedded in the DT flanges. The connectors are distributed along the edge of a DT flange so that connectors in adjacent DTs can be joined via a steel "slug" or "jumper plate" that is welded to each connector in an aligned pair (refer to Reference 12).

If multiple flange connectors are broken at a joint between DTs, however, the moment due to the 3000 lb load may be nearly doubled. Inspections showed that Structure A, with 3.5 in. flanges, had numerous failed flange connectors at the time of the flange failure. Inspections also showed that Structure B, with 4.75 in. flanges, had only a few broken flange connectors.

Reference 10 states that the two codes that provide live loads, References 8 and 9, require significant revisions to make the loads more representative of actual conditions in parking structures. It is our opinion that these documents could be more specific about how the concentrated load is to be considered, or they could replace the single load with a more realistic loading, such as the one proposed by Malik.[13] Gamble[10] provided additional discussion of the design problem, with numerical examples. The concept of a nonredundant flexural structural system that exhibits linear-brittle structural behavior is contrary to all concepts of behavior taught in structural design courses. The welded flange connectors cannot be considered a mechanism to provide redundancy.

Ductility Demands

Readers are again referred to the statements contained in the textbox at the beginning of this article. One of the general requirements for the design of reinforced concrete slab elements is that the nominal moment should exceed the cracking moment. FRP-reinforced members should satisfy this requirement, so ACI Committee 440 reports have requirements that parallel those in ACI 318 Building Codes. Quoting from ACI 440.1R-03, Section 8.1.1[14]: "Experimental results (Nanni 1993b; Jaeger, Mufti, and Tadros 1997; GangRao and Vija 1997a; Teriault and Benmokrane 1998) indicated that when FRP reinforcing bars ruptured in tension, the failure was sudden and led to the collapse of the member." Quoting from Section 8.2.4 in ACI 440.1R-03 and ACI 440.1R-06: "If a member is designed to fail by FRP rupture, $\rho_f < \rho_{fb}$, a minimum amount of reinforcement should be provided to prevent failure upon concrete cracking (that is, $\phi M_n \geq M_{cr}$ where M_{cr} is the cracking moment)."

Per Eq. (8-3) in that document

$$\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \quad (3)$$

Using the strand tensile force of 1218 lb, a strand modulus of elasticity of 34×10^6 psi (2.3×10^5 MPa), and strand area of 0.00286 in.^{2}, as reported in Reference 2; and using an assumed concrete strength of 6000 psi, $\rho_{fb} = 0.00174$. Thus, ρ_{fb} is 4.5 times the value of ρ_f calculated for the flange in Structure A.

The cracking moment is calculated as $m_{cr} = f_r S$, where $f_r = 7.5\sqrt{f_c'}$ = modulus of rupture and $S = bh^2/6$ = section modulus, for concrete with a compressive strength of 6000 lb/in.^{2}, f_r = 581 lb/in.^{2} (4 MPa), and m_{cr} = 14.23 kip·in./ft (5.28 kN·m/m). This is about the same as m_n from Case 1 in Table 1, meaning the margin is very small, even for the case of highest possible value of the strand force.

Reference 2 reports a much higher modulus of rupture, about 795 lb/in.^{2} (5.5 MPa), and an average compressive strength of 7800 psi (54 MPa). This leads to m_{cr} = 19.48 kip·in./ft (7.22 kN·m/m), much higher than the largest nominal failure moment from Table 1. Reference 2 appears to indicate that the cracking moment was used for design. We do not recommend this approach, as almost all nonprestressed reinforced concrete members can be expected to crack from the accumulated effects of handling and shipping stresses, fatigue, cyclic freezing and thawing, wetting and drying, restrained shrinkage, and impact.

Resilience of Standard Designs

As stated previously, the DT flange failure in Structure A occurred when a recovery vehicle imposed an axle loading reportedly totaling 13,620 lb. While this loading exceeds current code requirements as stated in Reference 9, an Internet search for "low-clearance towing" will show that such vehicles are widely available. This raises the question: Why aren't we seeing reports of failures in DT flanges with conventional reinforcement? The following subsections discuss the resilience of existing parking structures reinforced with conventional reinforcement.

The resistance side of resilience

Consider a DT flange with the same span as the flanges in Structure A (37-3/8 in.) but with a 4 in. thickness as required to provide adequate cover for steel reinforcement. Assume that welded wire reinforcement (WWR) is located at middepth of the flange, and the WWR is comprised of Grade 65 D4 wires compliant with ASTM A1064/A1064M, "Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete." Further, assume the D4 wires are spaced at 3 in. (75 mm). Therefore, A_s = 0.16 in.^{2}/ft, f_y = 65 kip/in.^{2}, and d = 2 in.; m_n = 19.92 kip·in./ft; and ϕm_n = 17.93 kip·in./ft.

The load side of resilience

Consider a 3 kip point load per Reference 9 and distribute the load per Reference 11 (45 degrees relative to the cantilever span). For this loading, m_u = 16.50 kip·in./ft and $\phi m_n/m_u$ = 17.93/16.50 = 1.087. Thus, the selected reinforcement is adequate.

Further, consider a 13.6 kip axle loading with dual tires spaced at 5 ft 8 in. on center. This is an overload condition, so apply ϕ = 1.0 and compare the nominal capacity to the unfactored load. If the load is distributed per Reference 11 but tire width is conservatively ignored, the loading width at the support will be 11 ft 11 in. For this loading, m = 24.27 kip·in./ft and m_n/m = 19.92/24.27 = 0.82. The section is not adequate.

Reference 10 discusses a 60-degree distribution of the load relative to the cantilever span. While this distribution is not conservative for design, it does seem appropriate for analyzing the effect of a single loading condition. Using a 60-degree distribution but again ignoring tire width, the loading width at the support will be 16 ft 5 in. Thus, m = 18.35 k-in./ft and m_n/m = 19.92/18.35 = 1.09. The section is therefore capable of resisting the overload condition.

Lastly, note that for f_c' = 6 kip/in.^{2}, m_{cr} = 18.59 kip·in./ft, which is only slightly greater than the moment due to the loading. It is therefore essential that the flange reinforcement ensures ductility. Steel reinforcement provides this necessary behavior, as strain-hardening will add about 15% to the capacity with WWR, and the WWR will have a fracture strain of at least 10 to 15 times the yield strain.

Required Approvals for New Systems

The ACI 318 Building Code is accepted throughout the United States as the standard to be used for the design of reinforced concrete structures. However, polymer-coated carbon fiber is not covered by the ACI 318 Building Code. While ACI Committee 440 reports provide design recommendations, they are not written in mandatory language. Therefore, there is no code covering the use of this relatively new material as flexural reinforcement.

The ACI Code has long had a procedure for the approval of new structural systems. In ACI 318-14, this procedure is covered in Section 1.10—Approval of special systems of design, construction, or alternative construction materials (refer to Reference 15). Similar provisions were in effect at the time Structures A, B, and C were designed and constructed. The procedure in ACI 318 involves submitting the design to the "Building Official" for approval, with appropriate documentation and test results. The building official has considerable discretion in this process, but one of the options is the appointment of an expert committee to evaluate the design and make recommendations to the building official. We understand that no such submittal was made for these structures, although the State of North Carolina Building Code[16] in effect in 2006 required submission of new structural systems for approval.

Call to Action

We have been told that no parking structures have been constructed using the polymer-coated carbon-fiber grid product since 2015. Without data showing that the structures containing this grid product have adequate quantities and placement of the product, the authors call on the precast producers to notify their customers of potential design issues and initiate appropriate remediation to ensure ductile behavior.

Further, the authors call on other organizations to act on the issues raised herein. Specifically, the authors recommend that:

- ACI Committee 318, Structural Concrete Building Code, updates
  - Sections 7.6.1 and 8.6.1 in ACI 318-14 and ACI 318-19[17] to require that the reinforcement in a slab is sufficient to ensure that the nominal moment capacity exceeds the cracking capacity; and
  - Section 1.10.1 in ACI 318-14 and ACI 318-19 to mandate that "Sponsors of any system of design, construction, or alternative construction materials… shall ~~have the right to~~ present the data on which their design is based to the building official or to a board of examiners appointed by the building official";
- ACI Committee 440, Fiber-Reinforced Polymer Reinforcement, completes a structural design standard for fiber-reinforced polymer bars; and
- ASCE/SEI Committee 7-22, Minimum Design Loads and Associated Criteria for Buildings, updates the loadings in ASCE/SEI 7 to include the effects of recovery vehicles. Alternatively, this information could be added to the commentary of ASCE/SEI 7.

In our opinion, it is improper to use provisions in ACI 318-14, Section 27.4—Strength evaluation by load test, to evaluate flanges reinforced with the polymer-coated fiber grid discussed in this article. Because the grid reinforcement has been shown to provide no increase in bending capacity over the cracking moment, flanges containing the grid are essentially plain concrete members. While Chapter 14 of ACI 318-14 allows plain concrete members in building structures, Section 14.1.3 forbids plain concrete to be used in bending members that are not continuously supported.

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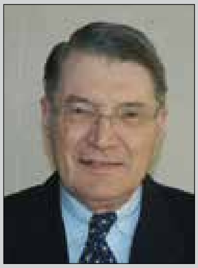
17. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)," American Concrete Institute, Farmington Hills, MI, 2019.

Note: Additional information on the ASTM standard discussed in this article can be found at **www.astm.org**.

Received and reviewed under Institute publication policies.



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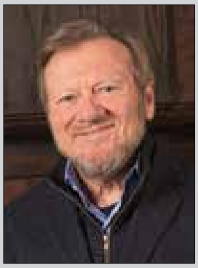

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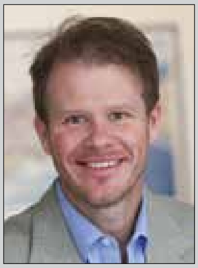

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