Behavior of Free and Connected Double-Tee Flanges
Reinforced with FRP

G. Lucier, A. W. Botros, S. H. Rizkalla and H. Gleich

Introduction

Precast prestressed concrete double-tees are typically used in parking decks and commercial buildings. Traditionally, the flanges of these members are reinforced with conventional steel welded wire reinforcement to carry the load in the transverse direction and to control shrinkage cracking and thermal stress. While traditional steel reinforcement is safe and effective from a structural perspective, it is vulnerable to corrosion.

Recently, carbon fiber-reinforced polymer (CFRP) grid has been used by several precasters to replace steel reinforcement in the flanges of double-tee members. The advantages of he CFRP grid are a high strength to weight ratio, excellent resistance to corrosion, and ease of installation.

The use of CFRP materials in bridge girders was led by the construction of several concrete highway bridges in Canada (Rizkalla et al. 2, 3). Carbon fiber reinforced polymer (CFRP) strands and bars were used as prestressing and shear reinforcement for the bridge girders. Part of the deck slab was also reinforced with CFRP. The long term performance of the bridges was monitored using advanced sensing technologies. Monitoring indicated no significant degradation in the CFRP after 15 years in service. The results indicated that CFRP tendons could be effectively used as prestressed longitudinal reinforcement in precast concrete bridge girders.

The application of CFRP grid as inter-wythe shear reinforcement in precast prestressed concrete sandwich panels was investigated in several studies (Pessiki et al. 4; Frankl et al. 5; Sopal et al. 6). Test results from these studies indicated that a high degree of composite action can be achieved by using CFRP grid as shear connectors.
Naito et al.\textsuperscript{7} conducted an experimental program to investigate the strength and response of fourteen different shear tie connectors used in insulated precast concrete sandwich wall panels. The tie connectors included CFRP and GFRP grids. The strength and stiffness of the connectors were determined in the study. Test results indicated that the shear performance of the ties is highly dependent on the configuration of the tie geometry.

Banthia et al.\textsuperscript{8} examined the behavior of concrete slabs reinforced with FRP grids and conventional slabs reinforced with steel grids under vertical concentrated loads. The study investigated the influence of concrete strength. Test results indicated that the ultimate loads for the FRP reinforced slabs were higher than for the steel reinforced slabs.

Matthys et al.\textsuperscript{9} investigated the behavior of concrete slabs reinforced with FRP grids under concentrated loads. The study included testing steel reinforced control slabs, slabs reinforced with CFRP grid, and finally slabs reinforced with a hybrid type of FRP that included both carbon and glass fibers. Other parameters were considered such as slab depth and reinforcement ratio. Test results documented punching shear failures for most slabs. Punching strength for FRP reinforced slabs was similar to or higher than that the control slabs. A strong interaction between shear and flexural effects was noted for most tested slabs.

El Gamal et al.\textsuperscript{10} tested six full scale deck slabs reinforced with glass FRP and carbon FRP bars under a monotonic single concentrated load applied at the center of the slab. Three deck slabs were reinforced with glass FRP (GFRP) bars, two deck slabs were reinforced with CFRP bars, and a reference steel reinforced slab. Test results indicated that the slabs were capable of resisting loads more than three times the design load specified by the Canadian Highway Bridge Design Code.

Lunn et al.\textsuperscript{11} tested eight double tee beams, 15ft. wide and 10ft. long with a flange thickness of 3.5in. The study was conducted to evaluate the behavior, serviceability, and failure mode of double-tee flanges reinforced with carbon fiber-reinforced polymer (CFRP) grid under uniform applied load. Results of the study indicated that the proposed precast double-tee beams reinforced with CFRP grid were capable of resisting uniform pressures well in excess of the
design loading. The mode of failure of the flanges was governed by the tensile strength of the concrete followed by rupture of the CFRP grid. The research proposed design recommendations for double tee flanges reinforced with CFRP grid.

This paper presents two experimental programs undertaken to examine the serviceability and failure mode of free and connected precast prestressed concrete double-tee flanges reinforced with CFRP grids. The double-tee flanges were tested under uniformly distributed loads and concentrated loads to evaluate the behavior of the flanges under extreme loading conditions.

A. Uniform Load Tests

Two full scale 12 foot wide by 40 foot long double-tee specimens were tested under uniform loading. The first specimen, DT1, was an un-topped member 28 in. deep with a 2 inch thick flange. The second specimen, DT2, was a pre-topped member 29.5 in. deep with a 3.5 in. thick flange. Both specimens were prestressed longitudinally and reinforced for shear at the ends of each stem with steel welded wire reinforcement. A continuous sheet of FRP grid was used as the only transverse flange reinforcement. This grid was comprised of 0.15 inch wide carbon fiber strands spaced at 2.75 in. in the primary (transverse) direction held together by smaller glass fiber strands running in the orthogonal direction. The CFRP strands provided the structural flange reinforcement and the GFRP strands maintained spacing and provided anchorage for the CFRP strands. Specimens were tested using a vacuum chamber, and uniform air pressure was used to apply the load.

The double tees were tested in a simple beam configuration. At one end, the two stems were placed on two load cells, acting as a pin connection. At the other end, the two stems were placed on a 2 inch diameter cylindrical bar to provide a roller support. The specimen, supports, and instrumentation were enclosed in a vacuum chamber constructed around the specimen. The uniform distributed load was applied on the top surface of the double-tee deck by reducing the
pressure inside the chamber using a combination of vacuum equipment. An isometric view of the test setup is shown in Figure 1.

Figure 2 shows a cross section of the test setup and the concept used to apply a uniform pressure loading condition to the top surface of the flange. At the start of the test, pressures P1 and P2 are both equal to atmospheric pressure, and consequently, no load is applied to the top surface of the double-tee deck. With the chamber is sealed, pressure P2 is reduced with vacuum equipment while atmospheric pressure P1 remains constant. With pressure P1 greater than P2, the atmospheric pressure acts evenly inward on the top surface of the double tee, creating downward uniform load on the flange.

Modular formwork panels were used to build the four vertical walls of the chamber around each double tee. Panels were sealed to the pavement surface and wrapped in plastic to minimize air leakage. Chamber walls were also anchored to the pavement for stability and were braced against one another with shoring to resist the horizontal pressures acting on the chamber. Bracing was provided along the length of the chamber at the top and bottom of the sidewalls. The end walls were braced at the bottom against the supporting concrete blocks and were stiffened at their top edges with steel angles. The chamber was constructed with windows in all sides to allow access for instrumentation and for observation of the behavior under the applied load. The chamber under construction and the completed chamber prior to testing are also shown in Figure 2.

The total applied uniform pressure load was monitored by two load cells placed at one end of the stem to measure the reaction. Prior to the start of each test, the self-weight of each specimen was recorded using the two load cells. The performance of the flanges was monitored at selected load levels for each specimen, including the design load. Specimen DT1, the un-topped specimen, was designed for a dead load (D) of 38 psf representing a 2 in. field topping in addition to a 10 psf live load (L). Specimen DT2, the pre-topped specimen, was designed for a live load of 40 psf and a snow load (S) of 20 psf. The controlling load case considered for both specimens was the factored load combination 1.2D+1.6L+0.5S. The factored load was sustained for 1 hour and 24 hours for specimens DT1 and DT2, respectively. Specimens were loaded and unloaded in incremental cycles to failure. The loading sequences followed for testing DT1 and DT2 are summarized in Table 1.
Six standard shop vacuums were used to generate pressures corresponding to lower load steps. A vacuum excavation truck was used in addition to the shop vacuums to increase the applied pressure when needed. A sliding side door was constructed to control the applied load. The door was closed slowly to increase the differential pressure between the atmosphere and the chamber, consequently increasing the uniform load applied to the specimen.

Deflections, strains, and loads were monitored throughout testing. All instruments were connected to an electronic data acquisition system which recorded data at 1 Hz during loading and unloading. Two load cells were used to measure the vertical stem reactions. String potentiometers were used to measure vertical displacements in lines along the width of the flange at the support, quarter-span, and mid-span. Linear potentiometers were used to measure concrete strains on the top surface of the double-tee. Thermocouples were used to measure the temperatures of the concrete surface during one test, and a pressure transducer was used to measure the internal chamber pressure. A U-tube differential manometer was also used for visual measurement of the pressure and to verify the electronically recorded pressures.

Test Results and Discussion

Both specimens, DT1 and DT2 were capable of resisting loads exceeding their factored design loads prior to failure with minimal residual deflections after unloading. Test results for specimens DT1 and DT2 indicated that the flanges were able to resist a maximum applied uniform load equal to 1.3 times and over 1.9 times their full factored design loads, respectively. The measured failure loads, including self-weight, for specimens DT1 and DT2 are given in Table 2.

Specimen DT1, with a 2 in. thick flange, failed at an applied pressure of 90.6 psf. The failure mode was a flexural failure in one of the cantilever flanges of the beam accompanied by rupture of the CFRP grid reinforcement. The failure resulted in complete detachment of the cantilever flange along the entire length of the beam, as shown in Figure 3(a). Failure occurred suddenly after the formation of a longitudinal flexural crack on the top surface of the flange. This crack...
was located 6.5 in. outboard from the center of one stem where the flange tapered into the stem as shown in Figure 3(b). It should be noted that two initial cracks on the top surface of the inner flange were observed in specimen DT1 before testing. The initial longitudinal cracks were located at the web-flange juncture and extended along the entire length of the beam. They were also visible on the bottom face of the flange.

The measured net deflection profile and selected strain measurements for specimen DT1 are shown in Figure 4 and Figure 5, respectively. C1 to C7, in Figure 4, are vertical deflection measurement locations across the cross-section, from the tip of one flange to the tip of the other. Deflection profiles are shown at the service load, factored load, and ultimate load levels. The measured deflections at the tip of the cantilevers at service and factored loads were each less than 0.3 inches. The measured transverse strains, gages 2 and 4, located on the top surface of the cantilever at the flange stem juncture indicated a linear load-strain behavior beyond the factored load and up to failure. This behavior indicates that no cracks occurred at this location up to failure. The behavior also justifies the very small measured deflections at failure. This result indicates that flange failure occurred immediately after cracking of the cross section at the maximum moment location. The load-strain behavior recorded by strain gages 1 and 3 across the the inner flange-stem connection was nonlinear as shown in Figure 5. This result reflects the widening of the pre-existing cracks and straining of the CFRP grid at these locations as the applied uniform pressure was increased.

Specimen DT2, with a 3.5 in. thick flange, did not fail under the applied load. It exhibited excessive flexural deflection at midspan at an applied load of 203 psf and the test was terminated due to the inability of the test setup to increase the applied load above this level. This result indicated that the flange was capable of carrying an applied load greater than 203 psf which is 1.9 times the full factored design load. It should be noted that the applied uniform pressure of 203 psf was maintained for approximately 10 minutes, during which time the double-tee continued to deflect under constant load, indicating global yielding of the longitudinal strands. Specimen DT2 after termination of the test is shown in Figure 6.
At the conclusion of the test of DT2, the specimen was visually inspected for cracks. No cracks were observed in the top or bottom surface of the flanges or on either side of both stems. Specimen DT2 was entirely intact and visibly undamaged in any way at the conclusion of testing. It is assumed that residual prestressing was sufficient to close flexural cracks in the stems that certainly would have developed.

The measured net deflection profile and measured concrete strains for specimen DT2 are shown in Figure 7 and Figure 8, respectively. The measured load-deflection behavior indicates very small deflections at the service and factored load levels and the flange remained nearly flat, even at factored load. As the load increased beyond the factored level, the deflection at the mid span increased in comparison to the end of the cantilever. The deflection profile for specimen DT2 indicates global deformation of the flange beyond the factored load. The measured concrete strains, shown in Figure 8, remained below 500 microstrain and the load-strain behavior was relatively linear up to the maximum applied pressure with no evidence of cracks.

Comparing the behavior of the two specimens indicates the significant effect of the flange thickness on both the load carrying capacity and the overall and local deflection of the double-tees.

B. Concentrated Load Tests

According to the PCI Handbook\(^1\), edition 7, precast concrete double-tee flanges should be designed to resist concentrated loads at various locations on the surface. Since Specimen DT2 remained entirely intact after uniform loading, additional concentrated load tests were conducted to evaluate the behavior of the flange at various locations on the surface of the tee. Concentrated flange loads would commonly be caused in a parking structure by a vehicle jack and can control the flange design. The concentrated loads were applied through 4.5 inch by 4.5 inch steel plates bearing directly on the flange surface. Tests were conducted at the edge, mid-width of the flange at the end and mid span sections of the beam using a simple test setup. The typical test setup and failure patterns at various tested locations are shown in Figure 9. Measured failure loads and
observed failure modes for the concentrated load tests are summarized in Table 3 by location of loading. The numbers shown in Figure 9 correspond to the test numbers given in Table 3.

It should be noted that tests conducted on isolated edges of the flange do not simulate the typical conditions in the field. Typically, double tee flanges are welded together at discrete points along the span. Accordingly, concentrated loads applied on the flange are resisted by the connected flanges of the adjacent double tees. The fact that a diaphragm action is developed in connected double tees to resist the applied concentrated loads may result in higher failure loads and different failure surfaces. This action requires testing of the connected double tees to determine the structural mechanism and the failure surfaces under the effect of concentrated loads.

To study the influence of connected tee flanges, it was decided to perform additional tests in the yard of a precast concrete facility on full-scale 12’ wide by 60’ long double tees. This field testing program was designed to evaluate the behavior of adjacent, connected, CFRP-reinforced double-tee flanges as well as free double tee edges subjected to concentrated loads. The double tee members were connected by welding embedded flange connections located every 6 feet along the length of the beam. The two types of flange connections used in the field testing program (one straight and one twisted) are shown in Figure 10.

All prestressed concrete double tees used in the field testing portion of this program were 12 feet wide and 60 feet long. The total depth for each beam was 29.5 in. including a 3.5 in. thick flange. The specified nominal compressive strength of the concrete was 6000 psi, and each beam was prestressed longitudinally by ten ½-S in. diameter strands. The double tees flanges were reinforced with a continuous sheet of CFRP grid with a 3” spacing. The grid was the only transverse flange reinforcement with the exception of two #3 steel reinforcing bars placed at each end and a welded chord detail consisting of two #5 bars placed also at one end only. The steel bars and chord detail are typical of all specimens, as shown in Figure 11. A total of six double tees were tested in connected pairs while only two double tees were tested individually. A total of forty-eight tests were performed.
Tests were conducted by applying concentrated loads at selected locations on the top surface of the flange using the hydraulic jack and self-reacting frame shown in Figure 12. The jack was secured to the reaction beam, and the entire frame able to be moved along the length of the specimens. The double tees were supported on masticord bearing pads at each end and were elevated off the ground by concrete blocks.

Specimens were tested individually or in connected pairs. Table 4 summarizes the configurations of the 48 tests. Specimens tested in pairs were welded together at their discrete flange connections and at the chord connection. The location of the chord connection with respect to each test is designated by a dotted line on the sketches shown in Table 4. After conducting tests on the pair at the connected edges, the double tees were separated, rotated, and their remaining intact edges welded together to conduct the additional tests shown in Table 4. Locations of the applied concentrated loads are shown as open circles in the Table, and each test is designated by a number and a letter. Letters A and B indicate similar locations on the adjacent flanges. For each critical location, at least four data points were measured to provide adequate statistical data. The shaded areas on each sketch represent the locations of pre-existing failures from prior tests in previous configurations.

For the connected double tees, two types of loading were considered at the joint. In the first, the concentrated load was applied with the bearing plate entirely on one side of the gap and was referred to as “side of gap” loading. In the second, the bearing plate was placed to span the gap and was referred to as “spans gap”. This difference is illustrated in Figure 13.

For all tests, the applied load was measured by an electronic load cell. The load was applied by a hydraulic jack through a spherical bearing connection to a 4.5 inch square by 1 inch thick steel plate bearing on a 4.5 inches square by 0.5 inch thick neoprene pad. For selected specimens, the vertical deflection of the flange was measured during each test with string potentiometers placed directly under the applied load.
Test Results and Discussion

The failure modes observed for the flange of the double tee were either a flexure failure or a punching shear failure. All failures occurred in a sudden fashion. Test results of all tests are summarized in Table 5 by location of loading. Photos for the observed failure modes for selected tests, listed in Table 5, are shown in Figure 14.

Test results of connected flanges loaded at internal corner locations with the load beside of the gap (see Figure 14a), failed due to formation of an inclined flexure crack on the top surface of the loaded flange. This inclined crack extended from the web flange junction to the connector plate joining the two flanges. It should be noted that a portion of the concentrated load was also carried by the adjacent flange since it is connected to the loaded flange, however, failure took place in the loaded flange only and the adjacent flange remained intact after the test.

Loading the two flanges simultaneously at an internal corner in the configuration referred to as “spans gap” resulted in a flexure failure in the two flanges. Failure occurred after the formation of inclined flexure cracks on the top surfaces of the two connected flanges. These cracks extended from the web flange juncture to the connector plate for each flange as shown in Figure 14 (b).

Test results for the case of a free DT loaded at the corner indicated failure due to formation of an inclined flexure crack on the top surface of the flange. This crack extended from the web flange juncture to the edge of the flange with an inclination of about 60 degrees. The failure mode for a corner test of a free DT is shown in Figure 14 (c).

Comparing similar pairs of corner tests shown in Table 5 indicated that presence of the welded chord steel at the corners of the double tees increased the ultimate capacity by 43% in some cases. For the connected double tees, the location of the load with respect to the joint (on one side of gap or spanning the gap) had a significant effect on the ultimate capacity of the flange. In some cases, the failure load for the configuration with the load spanning the gap was 58% higher than the case where the load was placed on one side of the gap. Comparing the corner test results
of connected DTs and free DTs indicated that the concentrated load carrying capacity of internal
corners of connected DTs was higher than that of a free DT. This behavior is attributed to the
fact that the concentrated load is resisted by the two flanges in the case of connected DTs which
consequently results in higher load carrying capacity.

Test results of connected flanges loaded at the midspan location for the case of the load spanning
the gap (Figure 14d), indicated a flexure failure in the two flanges. The flexural cracks that
constituted failure occurred immediately after reaching the cracking capacity of the flanges at the
critical section. The cracks took place on the top surface of the flanges close to the web-flange
juncture. Under the applied load the cracks widened and propagated longitudinally and
transversely towards the gap between the two flanges. For each of the connected flanges, the
flexure crack intersected the edge of the flange beside the gap at two points almost 25 feet apart.
This behavior resulted in complete detachment of segments of the two connected flanges upon
rupture of the CFRP grid as shown in Figure 14 (d). In general, the failure was brittle and
occurred immediately after the formation of the flexure cracks.

Test results of connected flanges loaded at the midspan location for the case of the load placed at
one side of the gap exhibited a different failure mode. The failure mode for midspan “side of
gap” tests is also shown in Figure 14 (e), (f) and (g). As shown in Figure 14, the failure observed
in this case was a local punching shear failure in the flange at the location of the applied load.
The failure took place in a sudden fashion. In some tests, a flexure crack occurred on the top
surface of the flange close to the web flange juncture prior to the punching failure of the flange.

Test results for the case of loading the edge of a free DT at midspan, shown in Figure 14 (h),
indicated a flexural failure in the flange. Failure occurred after the formation of a longitudinal
flexural crack on the top surface of the flange. This crack initiated close to the web flange
juncture and progressed longitudinally and transversely towards the edge of the flange. The
failure resulted in complete detachment of a segment of the flange upon rupture of the CFRP
grid.
Results of tests conducted directly over connectors indicated a flexural failure in the two connected flanges. Failure occurred immediately after the formation of flexural cracks on the top surface of the two flanges. The cracks initiated close to the web flange juncture and propagated in the longitudinal and transverse directions towards the gap between the two flanges as shown in Figure 14 (I).

In general, the load carrying capacity of the flange at midspan for connected double tees was markedly higher than that of the free double tee. For example, the load carrying capacity of the connected flanges at the midspan location (test 19 in Table 5) was almost twice that of a free flange at same location, test 26. The case of connected double tees more closely match the realistic conditions in parking structures where double tees are connected together using welded joints.

For a connected pair of double tees, the corner locations are the most critical locations with the least load carrying capacity as compared to the midspan locations. Test results indicated that the ultimate capacity of the flange at a midspan location exceeded the capacity at an internal corner location by up to five times in some cases. Results of tests conducted directly over the connectors indicated that the two types of connectors considered in this study provided almost equivalent strengths at all tested locations.

In general, concentrated load test results indicated that the failure load and mode of failure were highly influenced by the test configuration, load location and presence of chord steel. Figure 15 (a) and (b) show the measured load versus the vertical displacement under the load for selected tests at the corner and midspan locations, respectively.

Idealized Failure Surfaces and Design Methodology

Results of the concentrated loads tests conducted on the double tees in the field testing program were used to develop idealized failure surfaces at various locations of the DT flange.
Figure 16 shows the idealized failure surfaces for a corner location and a location at any point along the edge of a free flange sufficiently far from an end or other discontinuity. Figure 17 shows the idealized failure surfaces and mode of failure for the internal corner and midspan locations of connected DTs for the cases of the load placed at one side of the gap and spanning the gap.

The nominal moment capacity of the flange $M_n$ was determined using the method recommended by ACI 440.1R-06. In Eq. 1, $A_f$ is the area of FRP reinforcement, $b$ is the width of the flange and $d$ is the effective depth of the reinforcement. In Eq. 2, $E_f$ is the modulus of elasticity of the FRP, $\varepsilon_{cu}$ is the concrete crushing strain, $f'_c$ concrete compressive strength at 28 days, $f_{fu}$ is the rupture stress of the FRP, and $\beta_1$ is a factor relating the depth of the equivalent rectangular compressive stress block to the neutral axis depth as specified by ACI 318-14. According to ACI 440.1R-06, if the FRP reinforcement ratio, $\rho_f$ calculated using Eq. 1 is less than the balanced ratio, $\rho_{fb}$ from Eq. 2, failure is likely due to rupture of the FRP before crushing of the concrete. In this case, a simplified and conservative method recommended by ACI 440.1R-06 was used to determine the nominal moment capacity, $M_n$ as given by Eq. 3, where $\varepsilon_{fu}$ is the rupture strain of the FRP and $c_b$ is the distance from the extreme compression fiber to the neutral axis at the balanced strain condition as determined by Eq. 4.

$$\rho_f = \frac{A_f}{b \ d} \quad \text{Eq.1}$$

$$\rho_{fb} = 0.85 \beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \quad \text{Eq.2}$$

$$M_n = A_f f_{fu} (d - \beta_1 c_b/2) \quad \text{Eq.3}$$

$$c_b = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}}\right) d \quad \text{Eq.4}$$

The experimental results of this study indicated that for most of the tests, ultimate failure was controlled by the flexural cracking capacity of the flange rather than the FRP reinforcement ratio. This was also indicated by calculations since the cracking moment $M_{cr}$, exceeds the nominal flexural moment corresponding to the tensile rupture strength of the FRP $M_n$. The cracking
moment $M_{cr}$ can be determined using Eq. 5, where $t$ is the thickness of the flange and $f_r$ is the modulus of rupture, which is assumed to be $7.5 \sqrt{f'c}$ as specified by ACI 318-14\textsuperscript{13}.

$$M_{cr} = f_r bt^2/6 \quad \text{Eq.5}$$

The observed idealized failure surfaces and the cracking capacity of the flange $M_{cr}$ were used to determine the nominal concentrated load carrying capacity of the flange, $P_n$, at various locations of the DT flange as illustrated in the design example below. Due to the brittle nature of the failure, it is recommended that the section be designed such that $0.75 P_n$ exceeds $P_u$, where $P_u$ is the factored design concentrated load, as given in Eq. 6.

$$0.75 P_n > P_u \quad \text{Eq.6}$$

<subhead 2>

**Design Example**

The following design example illustrates the procedure proposed for predicting the nominal concentrated load carrying capacity of the flange, $P_n$, at various locations for free and connected DT flanges. For CFRP grid with 3 inch spacing; a tensile strength, $f_{fu}$, of 120 ksi; a rupture strain, $\varepsilon_{fu}$, of 0.014; and a cross-sectional area, $A_f$, of 0.0216 in$^2$/ft, the rupture strength of FRP is 2.6 k/ft. Using a clear concrete cover of $\frac{3}{4}$ inch and effective depth, $d$, of 2.75 inches, the reinforcement ratio, $\rho_f$, can be determined using Eq.1 as:

$$\rho_f = A_f / b d$$

$$= (0.0216) / (12 \times 2.75)$$

$$= 0.00065$$

Using the measured concrete strength at 28 days, $f'c$, as 8400 psi, elastic modulus of the CFRP grid, $E_f$, of 8570 ksi and concrete crushing strain, $\varepsilon_{cu}$ of 0.003, the balanced reinforcement ratio can be determined using Eq. 2:
\[ \rho_{fb} = 0.85 \beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \]

\[ = 0.85 \times 0.65 \frac{8.4}{120} \frac{8570 + 0.003}{(8570 + 0.003 + 120)} \]

\[ = 0.0068 \]

Since the FRP reinforcement ratio, \( \rho_f \) is less than the balance ratio \( \rho_{fb} \), the nominal moment capacity of the flange, \( M_n \) can be determined based on the conservative estimation of depth of the compression zone for the balanced section \( c_b \) using Eq. 3 and Eq. 4 as follows:

\[ c_b = \left( \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) d \]

\[ = \left( \frac{0.003}{0.003 + 0.014} \right) 2.75 \]

\[ = 0.485 \text{ in} \]

\[ M_n = A_{ffu} (d - \beta_1 c_b/2) \]

\[ = 0.0216 \times 120 (2.75 - 0.65 \times 0.485/2) \]

\[ = 6.7 \text{ kip-in/ft} \]

The cracking moment capacity of the flange, \( M_{cr} \), can be determined using Eq. 5

\[ M_{cr} = f_c b t^2 / 6 \]

\[ = 7.5 \times \sqrt{8400} (12) (3.5^2) / (6 \times 1000) \]

\[ = 16.8 \text{ kip-in/ft} \]

The analysis indicated that the cracking moment capacity \( M_{cr} \) exceeds the nominal flexural moment \( M_n \). Therefore, the flexural capacity of the flange is controlled by the cracking moment cracking and should be used to predict the concentrated load carrying capacity of the flange, \( P_n \), at the proposed failure surfaces for the free and connected DT flanges.
Case 1: Load spanning the gap at the corner of connected flanges

The proposed failure mode for this case is flexure along the proposed failure surface.

Assume that the load is shared equally by the two flanges; therefore the load acting on each flange is $P_n/2$.

The length of the failure surface, $L$, can be determined based on the 30 inches cantilever length of the flange as:

$$ L = \frac{30''}{\cos 45} = 42.4'' $$

The lever arm, $a$, can be determined as:

$$ a = 30'' \times \sin 45 = 21.2'' $$

Using the cracking moment capacity, $M_{cr}$ along the length, $L$, the nominal concentrated load, $P_n$ can be determined as:

$$ M_{cr} \times L = \frac{P_n}{2} \times a $$

$$ 16.8 \times 42.4''/12 = \frac{P_n}{2} \times 21.2'' $$

$$ P_n = 5.5 \text{ kips} $$

This value compares well with the average load of 6 kips measured for five tests.
Case 2: Load spanning the gap at mid span of connected flange

The proposed failure mode for this case is flexure along the failure surface.

Similarly, assuming that the load is equally shared by the two flanges, thus the load acting on each flange is $P_n/2$.

The dimensions of the failure surface:

- $t =$ thickness of flange  
  - $= 3.5"$
  - $L = 8t/\sin 20$
  - $= 81.8"$

The lever arm can be determined based on the proposed failure surface:

- $a = 8t \times \cos 20$
  - $= 26.3"$

The nominal load can be determined as follows:

\[
\frac{P_n}{4} \times a = M_{cr} \times L
\]

\[
\frac{P_n}{4} \times 26.3" = 16.8 \times 81.8"/12
\]

$P_n = 17.3$ kips

This value compares well with the average load of 17.5 kips measured for two tests.
Case 3: Load at one side of the flange at mid-span of the connectors

The proposed failure mode for this case is a punching shear failure along the failure surface, using concrete shear strength $f'_c$, the nominal load $P_n$ can be determined based on flange thickness, $t$ and length of the failure surface, $L$ as follows:

$t = 3.5''$

$L = 2t / \sin 20$

$= 20.4''$

$P_n = \sqrt{f'_c} \times 2L \times t$

$P_n = \sqrt{8400} \times 2 \times 20.4'' \times 3.5'' / 1000$

$= 13.0 \text{kips}$

The predicted value also compares well to the average load of 14.6 kips measured for seven tests.

Case 4: load at the corner of free flange

The proposed failure mode for this case is flexure along the proposed failure surface

The length of the failure surface can be determined as:

$L = 30'' / \cos 45$

$= 42.4''$

$a = 30'' \times \sin 45$

$= 21.2''$

$P_n \times a = M_{cr} \times L$

$P_n \times 21.2'' = 16.8 \times 42.4'' / 12$

$P_n = 2.7 \text{kips}$

The predicted value also compares well to the average load of 2.8 kips measured for eight tests.
Case 5: Load at mid-span of free flange

The proposed failure mode for this case is flexure along the proposed failure surface.

From the geometry of the failure surface:

\( t = \text{thickness of flange} \)

\( = 3.5'' \)

\( L = 8t/\sin 20 \)

\( = 81.8'' \)

\( a = 8t \times \cos 20 \)

\( = 26.3'' \)

The load carried by each segment of the failure surface = \( P_n/2 \)

\( \frac{P_n}{2} \times a = M_{cr} \times L \)

\( \frac{P_n}{2} \times 26.3'' = 16.8 \times 81.8''/12 \)

\( P_n = 8.7 \text{kips} \)

The predicted value compare well to the average load of 9.3 kips measured for four tests.
Conclusions

Based on this study, the following conclusions and recommendations may be drawn:

1. CFRP grid can be used effectively as transverse flange reinforcement for prestressed precast concrete double-tees.

2. The ultimate flange bending capacity is controlled by the flexural cracking capacity of the flange.

3. Failures of flanges reinforced with CFRP grid are brittle, however, the load carrying capacity is substantially higher than the specified factored load.

4. The concentrated load carrying capacity and failure surface of flanges reinforced with CFRP grid is highly dependent on the location of the applied concentrated load.

5. The concentrated load carrying capacity for a connected pair of double tees is higher than that for a free double tee.

6. For connected double tees, the location of the concentrated load with respect to the joint had a significant effect on the ultimate capacity of the flange.

7. Idealized failure surfaces at various tested locations were developed for the free and connected flanges and can be used to determine the concentrated load carrying capacity of the flange.
<subhead 1>

Acknowledgments
The authors would like to thank Altus-Group for supporting this research effort. They would also like to thank Metromont Corporation for supporting the field trial at their Richmond, VA facility. In addition, the authors are grateful to the staff and students at the Constructed Facilities Laboratory at North Carolina State University for their help throughout the experimental program. In particular, Mr. Johnathan McEntire was instrumental in the field testing program.

<subhead 1>

References
12. ACI Committee 440. 2006. “Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars (ACI 440.1R-06).” American Concrete Institute, Farmington Hills, Michigan
13. ACI Committee 318. 2014. “Building Code Requirements for Structural Concrete (ACI 318-14)” American Concrete Institute, Farmington Hills, Michigan

<subhead 1>

<subhead 2>Notation</subhead 2>

\( \rho_f \): FRP reinforcement ratio
\( A_f \): Area of FRP reinforcement
\( b \): Width of the flange
\( d \): Effective depth of the reinforcement
\( \rho_{fb} \): Balanced FRP reinforcement ratio
\( \beta_1 \): Factor relating the depth of the equivalent rectangular compressive stress block to the neutral axis depth
\( f'_c \): Concrete compressive strength at 28 days
\( f_{fu} \): Rupture stress of the FRP
\( E_f \): Modulus of elasticity of the FRP
\( \varepsilon_{cu} \): Concrete crushing strain
$\varepsilon_{fu}$: Rupture strain of the FRP

$M_n$: Nominal moment capacity of the flange

$c_b$: Distance from the extreme compression fiber to the neutral axis at the balanced strain condition

$M_{cr}$: Cracking moment of the flange

$t$: Thickness of the flange

$P_n$: Nominal concentrated load carrying capacity of the flange

$P_d$: Factored design concentrated load

$L$: Length of failure plane

$a$: Lever arm from the applied concentrated load to the failure plane
About the authors

Gregory W. Lucier, is a Research Assistant Professor in the Civil, Construction and Environmental Engineering Department and Manager of the Constructed Facilities Laboratory at North Carolina State University.

Amir W. Botros, is a Ph.D. Candidate and graduate research and teaching assistant at the Civil, Construction and Environmental Engineering Department, North Carolina State University, Raleigh, NC. He obtained his B.Sc. and M.Sc. from Ain Shams University in Egypt.

Sami H. Rizkalla, PhD, FPCI, FACI, FASCE, FIIFC, FEIC, FCSCE, is a Distinguished Professor of Civil Engineering and Construction, Director of the Constructed Facilities Laboratory, and Director of the NSF Center on Integration of composite into infrastructural at North Carolina State University.

Harry Gleich, P.E., FACI, FPCI is Vice President of Engineering for Metromont Corporation, serves on many PCI committees including TAC and is the immediate past chair of R&D Council for PCI and past chair of sandwich panels at PCI. At ACI he also serves on numerous committees including former chair of ACI533 Precast Panels and is Chair of ACI550 Precast Structures.
<subhead 1>

Abstract
This paper presents an experimental program carried out to evaluate the performance of precast concrete double-tee flanges reinforced with CFRP grid under various types of loading. The experimental program is comprised of two different studies with a total of ten full scale precast prestressed double tee beams subjected to uniform and concentrated loads applied to the top surface of their flanges. The first study included testing two double tee beams with 2 and 3.5 in. thick flanges to evaluate the behavior, including the flange bending behavior, under uniformly distributed loads using an enclosed vacuum chamber. In the second study, a total of eight double-tees were tested either individually or in connected pairs to determine the load carrying capacity of the flanges when subjected to concentrated loads applied at various points on the flange. Test results from the first study indicated that the flanges were capable of resisting a maximum applied load significantly higher than their factored design loads. Results of the second study indicated that the concentrated load carrying capacity of the flange is highly dependent on the location of the applied load. Based on this investigation, an idealized failure surface and the corresponding mode of failure were identified for the free and connected flange of prestressed concrete double tee subjected to concentrated loads.

<subhead 1>

Keywords
Double-tee, CFRP grid, flange bending, flange connections, uniform loading, concentrated loading
Figures

Figure 1: Test Setup for Double Tee Specimens
Figure 2: Sketch of chamber cross section (top), the chamber under construction (left), and the chamber prior to testing (Right)
Figure 3: (a) Overview of DT1 failed flange, (b) CFRP Grid rupture
Figure 4: Deflection profile for specimen DT1
Figure 5: Measured strains for specimen DT1
Figure 6: Specimen DT2 after failure and removal of chamber
Figure 7: Vertical deflection profile for specimen DT2
Figure 8: Measured strains for specimen DT2
Figure 9: Test setup and cracking patterns of concentrated load tests for specimen DT2
Figure 10: The two types of flange connectors used in the program
(Straight connector, left; and twisted connector, right)
Figure 11: Typical steel bars (both ends of beam) and chord detail (one end of beam)
Figure 12: Concentrated load test setup (two tees welded together)
Figure 13: Loading plate at one side of the gap (left), and loading plate spanning the gap for connected double tees (Right)
Figure 14: Failure modes for selected load tests given in Table 4
Figure 15: Load deflection for selected tests (a) Corner tests for connected DTs and free DT (b) Mid span tests of connected DTs “Side of Gap” configuration
Figure 16: Idealized failure surfaces for free flange
Figure 17: Idealized failure surfaces for connected flanges

●: Applied Concentrated Load
t: Flange thickness
Table 1: Loading Sequence for Specimens DT1 and DT2

<table>
<thead>
<tr>
<th>Spec</th>
<th>Load Step</th>
<th>Description</th>
<th>Applied Pressure (psf)</th>
<th>Applied Stem Reaction (lbs)</th>
<th>Total Stem Reaction (lbs)</th>
<th>Unload after Step</th>
</tr>
</thead>
<tbody>
<tr>
<td>DT1</td>
<td>0</td>
<td>Self-weight</td>
<td>0</td>
<td>0</td>
<td>5,600</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Dead topping</td>
<td>38</td>
<td>4,560</td>
<td>10,160</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Dead topping + live</td>
<td>48</td>
<td>5,760</td>
<td>11,360</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.2D+1.6L</td>
<td>66.6</td>
<td>7,990</td>
<td>13,590</td>
<td>1 hr</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Increase to failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DT2</td>
<td>0</td>
<td>self-weight</td>
<td>0</td>
<td>0</td>
<td>8,400</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>applied snow</td>
<td>20</td>
<td>2,400</td>
<td>10,800</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>live (service)</td>
<td>40</td>
<td>4,800</td>
<td>13,200</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2 x flange weight</td>
<td>43.75</td>
<td>5,250</td>
<td>13,650</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>live + snow</td>
<td>60</td>
<td>7,200</td>
<td>15,600</td>
<td>hold then continue</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.2D+1.6L+0.5S</td>
<td>82.75</td>
<td>9,930</td>
<td>18,330</td>
<td>hold 24-hrs, unload</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>recovery</td>
<td>0</td>
<td>0</td>
<td>8,400</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>90 psf</td>
<td>90</td>
<td>10,800</td>
<td>19,200</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>100 psf</td>
<td>100</td>
<td>12,000</td>
<td>20,400</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>110 psf</td>
<td>110</td>
<td>13,200</td>
<td>21,600</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>120 psf</td>
<td>120</td>
<td>14,400</td>
<td>22,800</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>Continue incremental loading to failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2: Ultimate Loads for Specimens DT1 and DT2

<table>
<thead>
<tr>
<th>Load Level</th>
<th>Total Load Resisted by Flange (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DT1</td>
</tr>
<tr>
<td>Self Weight of Flange</td>
<td>25</td>
</tr>
<tr>
<td>Service Load (D+L)</td>
<td>73</td>
</tr>
<tr>
<td>Factored Load (1.2D+1.6L+0.5S)</td>
<td>91.6</td>
</tr>
<tr>
<td>Failure*</td>
<td>115.6</td>
</tr>
<tr>
<td>Failure Load / Service Load</td>
<td>1.6</td>
</tr>
<tr>
<td>Failure Load / Factored Load</td>
<td>1.3</td>
</tr>
</tbody>
</table>

* Flange of DT2 did not fail. The test of DT2 was terminated with a significantly nonlinear load-deflection behavior at mid span.
Table 3: Concentrated Load Results for Specimen DT2

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Failure Load (lbs.)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) End of DT, Corner of Flange</td>
<td>2,530</td>
<td>Flexure</td>
</tr>
<tr>
<td>(2) End of DT, Center of Flange</td>
<td>11,300</td>
<td>Punching</td>
</tr>
<tr>
<td>(3) Midspan of DT, Edges of Flange</td>
<td>8,170</td>
<td>Flexure</td>
</tr>
<tr>
<td>(4) Midspan of DT, Center of Flange</td>
<td>22,190</td>
<td>Flexure</td>
</tr>
</tbody>
</table>
**Table 4: Summary of Concentrated Load Tests for Connected Flanges**

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Plan View</th>
<th>Connection</th>
<th>Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Setup 1</td>
<td><img src="image1" alt="Diagram" /></td>
<td>Type I</td>
<td>1A, 1B, 2A, 2B, 3A, 3B</td>
</tr>
<tr>
<td>DT001.1 and DT001.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 2</td>
<td><img src="image2" alt="Diagram" /></td>
<td>Type II</td>
<td>4A, 4B, 5A, 5B, 6A, 6B</td>
</tr>
<tr>
<td>DT003.1 and DT003.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 3</td>
<td><img src="image3" alt="Diagram" /></td>
<td>Type I</td>
<td>7A, 7B, 8, 9A, 9B</td>
</tr>
<tr>
<td>Reuse</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DT001.1 and DT001.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 4</td>
<td><img src="image4" alt="Diagram" /></td>
<td>Type II</td>
<td>10A, 10B, 11, 12A, 12B, 13, 14, 15, 16</td>
</tr>
<tr>
<td>Reuse</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DT003.1 and DT003.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 5</td>
<td><img src="image5" alt="Diagram" /></td>
<td>Type I</td>
<td>17, 18, 19, 20, 21</td>
</tr>
<tr>
<td>DT001.3 and DT001.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 6</td>
<td><img src="image6" alt="Diagram" /></td>
<td>Type I</td>
<td>34, 35, 36, 37, 38</td>
</tr>
<tr>
<td>Reuse</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DT001.3 and DT001.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 7</td>
<td><img src="image7" alt="Diagram" /></td>
<td>None</td>
<td>22, 24, 26, 29, 31, 32</td>
</tr>
<tr>
<td>DT002.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Setup 8</td>
<td><img src="image8" alt="Diagram" /></td>
<td>None</td>
<td>23, 25, 27, 28, 30, 33</td>
</tr>
<tr>
<td>DT002.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test ID</td>
<td>Test Description</td>
<td>Test Configuration</td>
<td>Connectors</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------</td>
<td>-------------------</td>
<td>------------</td>
</tr>
<tr>
<td>5A</td>
<td>Tests of Corners Without Chord Steel</td>
<td>Connected DTs</td>
<td>Type II</td>
</tr>
<tr>
<td>5B</td>
<td></td>
<td></td>
<td>Type I</td>
</tr>
<tr>
<td>10A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>Free DTs</td>
<td>None</td>
</tr>
<tr>
<td>34</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>31</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1A</td>
<td>Tests of Corners with Chord Steel</td>
<td>Connected DTs</td>
<td>Type I</td>
</tr>
<tr>
<td>1B</td>
<td></td>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td>4A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>Free DTs</td>
<td>None</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2A</td>
<td>Tests Between Connectors at Mid span</td>
<td>Connected DTs</td>
<td>Type I</td>
</tr>
<tr>
<td>7A</td>
<td></td>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td>7B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2B</td>
<td></td>
<td></td>
<td>Type I</td>
</tr>
<tr>
<td>6A</td>
<td></td>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td>6B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12A</td>
<td></td>
<td></td>
<td>Type I</td>
</tr>
<tr>
<td>12B</td>
<td></td>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td>Type I</td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>Tests of Free Edges at Mid span</td>
<td>Free DTs</td>
<td>None</td>
</tr>
<tr>
<td>29</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Tests at Connector</td>
<td>Connected DTs</td>
<td>Type II</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>2nd Conn.</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td>2nd Conn.</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37</td>
<td></td>
<td></td>
<td>Type I</td>
</tr>
<tr>
<td>38</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>