Foam-void precast concrete double-tee members

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- The weight of precast concrete double-tee members can be reduced by placing a foam board in the stem to create a void. The void reduces the volume of concrete and results in a foam-void precast concrete double tee.
- Reducing the weight of a double-tee member could allow for efficiency in transportation, and this research aims to reduce double-tee weight, thereby extending the range of situations where two double-tee members can be transported in the same load.
- Four single-tee specimens cut from two double-tee members were tested to study the flexural and shear behavior of foam-void precast concrete members. This paper presents results of the structural tests.

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Double-tee members are a staple of the precast concrete industry. Millions of square feet of double-tee members are fabricated annually in the United States. These members offer flexibility in design and construction and are an ideal choice for projects that require long uninterrupted spans with high load-carrying capability and quick erection times, such as parking structures. Small improvements in the efficiency of double-tee members, because of their widespread use, can have a significant effect on the overall environmental footprint and economic competitiveness of the precast concrete industry.

The gross vehicular weight limit for U.S. highways—80 kip (355 kN) in most states and circumstances—can sometimes limit the economical use of double-tee members. Due to their self-weight, two untopped 12 ft (3.65 m) wide × 28 in. (711 mm) deep double-tee (12DT28) members with normal-weight concrete cannot be legally transported on one truck if they are over 40 ft (12.2 m) long. The current research is motivated by a desire for two-at-a-time transport, which would improve both economic and environmental efficiency. Two-at-a-time shipping has the potential to reduce both costs and emissions from trucking.

Placing foam boards in the stems of a double-tee members creates a foam-void double-tee member (**Fig. 1**) that can have up to 8% less self-weight than that of comparable double-tee members without foam voids. Although this degree of weight reduction may appear marginal, even small reductions in weight can enable two-at-a-time shipping in

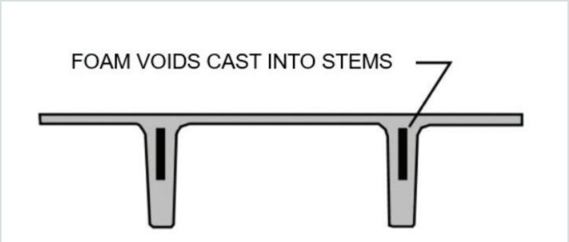

Figure 1. Concept of foam-void double tee.

select circumstances. Percentage of weight loss corresponds to an equivalent potential percentage increase in member length. If two foam-void double-tee members are shipped on the same truck in lieu of 12DT28 members, then the foam-void double tees can be up to 43 ft (13.1 m) before the gross vehicle weight limit is exceeded. When used in combination with lightweight or reduced-weight concrete, the possibility of two-at-a-time shipping expands even further.

This paper describes an experimental program that was conducted to evaluate the structural suitability of foam-void double-tee members. Flexural and shear loads were considered in the test program. Variables in the test program included the unit weight of concrete and the size of the foam void. Practical issues surrounding the detailing and construction of foam-void double-tee members are also discussed.

Background

Precast, pretensioned concrete double tees were first built in 1951. The history of these members in the precast concrete industry has been documented by Nasser et al.,[1] Wilden,[2] and Edwards.[3] The form of double-tee members is well suited for precast concrete construction. Standardized cross sections lead to fabrication efficiency and the cross-section shape provides structural stability for storage, shipping, erection, and service. The original double-tee cross section shown in Edwards[3] has evolved over the years to suit different needs and opportunities. The cross section has been modified to account for changes in steel and concrete materials and to suit different loading conditions. Double tees have been used as floor, roof, and wall structures of buildings and have also been used in industrial applications and in bridges. The northeast extreme tee (NEXT) beam for highway bridges is one of the most recently developed double-tee members.

Double-tee members are either field topped or factory topped. Factory-topped members are cast completely at a precasting plant. In comparison, field-topped members have thinner precast concrete flanges and have cast-in-place concrete topping (commonly 2 in. [50.8 mm] thick) placed on them after erection in the field. The topping acts compositely with the precast concrete double tee to carry vertical and diaphragm loads. Field-topped members are commonly used in regions with high seismic loads because the diaphragm reinforcement can be cast into the topping and placed continuous across joints between the precast concrete members. In contrast, diaphragm forces in factory-topped members are carried through connections between adjacent double-tee members. The experimental program presented in this paper used untopped foam-void double-tee members. If the experimental specimens were to be used in a building, they would have received a field-cast topping.

Reducing the self-weight of double-tee members has been the subject of previous research. Barney et al.,[4] Savage et al.,[5] and Saleh et al.[6] studied double-tee members with transverse web openings. In these studies, concrete was eliminated from locations in the web that do not contribute significantly to stiffness or flexural strength. Special reinforcement was detailed around the web openings to carry shear forces. The locations of openings and reinforcement details around the openings were variables in an experimental program. Throughout the test observations, the behavior of the test specimens with web openings was similar to that of a Vierendeel truss. That is, segments between the voids carried bending moments and axial forces. The test specimens demonstrated satisfactory strength or serviceability. To achieve adequate structural performance for this type of double-tee member, reinforcement must be provided adjacent to openings and the openings must be placed away from the end regions. A more recent study by Classen and Dressen[7] also investigated web openings in precast concrete members and arrived at similar conclusions. Although the previous studies focused on the use of transverse voids through the stems to achieve lighter members, the research in this paper studied the possibility of removing concrete from within the cross section of double-tee stems.

One proprietary system already in use reduces structure self-weight by placing voids where concrete is not needed for structural capacity. The current research takes a similar approach. Foam is used to displace concrete (and thus reduce self-weight) at locations where the concrete does not contribute significantly to structural capacity. Development of foam-void double-tee members aims to enhance the precast concrete industry's ability to produce components that are structurally, environmentally, and economically efficient.

Experimental program

The experimental program was conducted to study the flexural and shear capacities of members that have foam voids. For efficiency in fabrication and testing, each specimen in the study was a single-tee member. Four total specimens were fabricated by cutting two foam-void double-tee members lengthwise. Because the cross section of the foam-void double-tee members was symmetric about a vertical line (Fig. 1), the stiffness and capacity of each single-tee test specimen is reasonably assumed to be one-half of a full foam-void double tee. Four-point bending tests were conducted on the specimens to evaluate flexural and shear performance in different load stages from 50% of service load to ultimate load.

Specimen details and construction

Specimens were created from two 35 ft (10 m) long 12DT28 members. One of the double-tee members was cast with normalweight concrete with a unit weight of 145 lb/ft^3 (22.8 kN/m^3) and the other with semilightweight concrete with a unit weight of 126 lb/ft^3 (19.8 kN/m^3). One stem of each double-tee member was cast with a 1 in. (25.4 mm) thick foam board, and the second stem of each double-tee member was cast with a 2 in. (50.8 mm) thick foam board. The cross section of the specimens with prestressing and reinforcement details is shown in **Fig. 2**. The foam void started 5 ft (1.5 m) from the ends. The cutoff location for the foam was approximately equal to two times the member depth. This distance was estimated to be sufficient for facilitating transfer and distribution of the prestressing force and to prevent shear failures near the supports. The foam boards were 12 in. (305 mm) deep in all four specimens. Foam used for the voids had relatively low weight and high *R*-value and is commonly used as insulation in precast concrete sandwich panels. Extruded polystyrene foam was used for the voids. Extruded polystyrene is slightly more expensive than the alternative expanded polystyrene foam; however, extruded polystyrene is generally more robust and hence was used in this project.

The test specimens were fabricated in the same bed as production members for a building project, and the strand pattern was based on the production members. By casting on the same bed as production members, the interruptions associated with fabricating the test specimens were minimized. Because the test specimens had a shorter span than the production members, stresses in the specimens were controlled by debonding the topmost strand. For safety purposes, a 5 ft (1.5 m) segment of the topmost strand was left bonded at midspan.

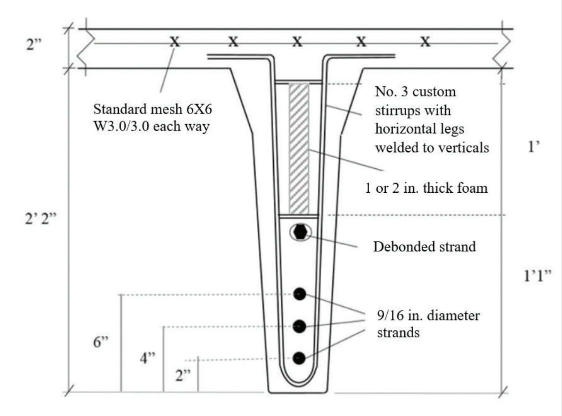

Figure 2. Specimen cross section. Note: no. 3 = 10M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

Transverse reinforcement in the specimens was custom-made no. 3 (10M) stirrups that included a gap for holding the foam board. The custom stirrups were placed at 12 in. (305 mm) spacing along the length of the double tee. An additional stirrup was placed near each specimen end and at the foam board start and stop locations. To prevent the foam from floating up during casting, it was anchored down by the stirrups, which were in turn anchored down by the strands. Concrete and reinforcement material properties are listed in **Table 1**. The members were manufactured at Tindall Corp.'s plant in Spartanburg, S.C., in November 2015. **Figure 3** shows the foam boards and reinforcement prior to casting.

Each specimen was given a unique two-character label. For the first character, N stands for normalweight concrete and L

Figure 3. Foam-void double tee prior to casting.

Table 1. Material properties of concrete and reinforcement

Material	Properties		
Semilightweight concrete* (L)	Unit weight, lb/ft³		126
	Compressive strength, psi	28 days	7810
		401 days	11,310
		441 days	10,360
Normalweight concrete† (N)	Unit weight, lb/ft³		145
	Compressive strength, psi	28 days	7270
		464 days	9610
		576 days	10,790
Reinforcing bars‡	Size		no. 3
	Grade		ASTM 615M-14 Grade 60
	Yield strength, ksi		77.4
	Tensile strength, ksi		107
Strands	Diameter, in.		9/16
	Grade, ksi		Grade 270 low-relaxation strand

* Concrete used for all lightweight concrete beams. Load tests were conducted between days 401 and 441.

† Concrete used for all normalweight concrete beams. Load tests were conducted between days 464 and 576.

‡ Properties based on reinforcing bar supplier documentation.

Note: no. 3 = 10M; 1 in. = 25.4 mm; 1 lb/ft³ = 0.157 kN/m³; 1 psi = 6.895 kPa; 1 ksi = 6.685 MPa; Grade 60 = 414 MPa; Grade 270 = 1860 MPa.

stands for semilightweight concrete. For the second character, 1 or 2 indicates the width of the foam board in inches. For example, specimen L1 is a semilightweight concrete member with 1 in. (25.4 mm) foam in the stem. Similarly, specimen N2 is a normalweight concrete member with 2 in. (50.8 mm) foam board. In total, there were four single-tee specimens and they were labeled L1, L2, N1, and N2.

Test setup and procedures

Specimens were loaded in four-point bending (**Fig. 4**). Steel saddles provided stability to the single-tee specimens at each support (**Fig. 5**). Load was applied quasi-statically using a hydraulic jack system. A steel I-beam was used to spread load from the jack to the specimen. Rubber bearing pads were used at all supports and load points.

Specimens were tested in seven different stages. For the first six stages, the boundary and load were designed such that the shear forces and flexural-tension stresses in the specimens mimicked those of a typical 60 ft (18.3 m) span parking structure double-tee member. At an experimental load P of approximately 28 kip (124.5 kN) (total for both load points), the flexural-tension stress in the specimens was equal to the service-level stress in a parking structure double tee. At a load of 28 kip, shear force in the specimens was also approximately the same as the service-level shear force in a parking structure double-tee. The service conditions were based on a 60 ft (18.3 m) span, 40 lb/ft² (1.915 kN/m²) live load, and 80 lb/ft² (3.83 kN/m²) dead load (including self-weight and 5 lb/ft² [0.24 kN/m²] superimposed dead load).

Displacement, strain, and force were monitored and logged using a computer data-acquisition system. Strain gauge locations are labeled in Fig 4. Six strain gauges monitored the concrete strain. Two were placed at the edges of the foam voids, two at the bottom of the member below the load points, and two on top of the flange at midspan. Four string potentiometers measured vertical displacement at midspan. Two were attached to the stem, and two were attached to the flange midspan of the specimen. For the shear tests (load stage seven), the shear span was reduced by moving the supports to the edge of the foam board location. This was done to evaluate the shear capacity of the foam-void cross section.

Specimens were loaded in seven different stages, in the following order:

1. Loading to 50% of flexural service load

2. 100 cycles between 20% and 50% of flexural service load

3. Loading to 100% of service load

4. 100 cycles between 20% and 100% of flexural service load

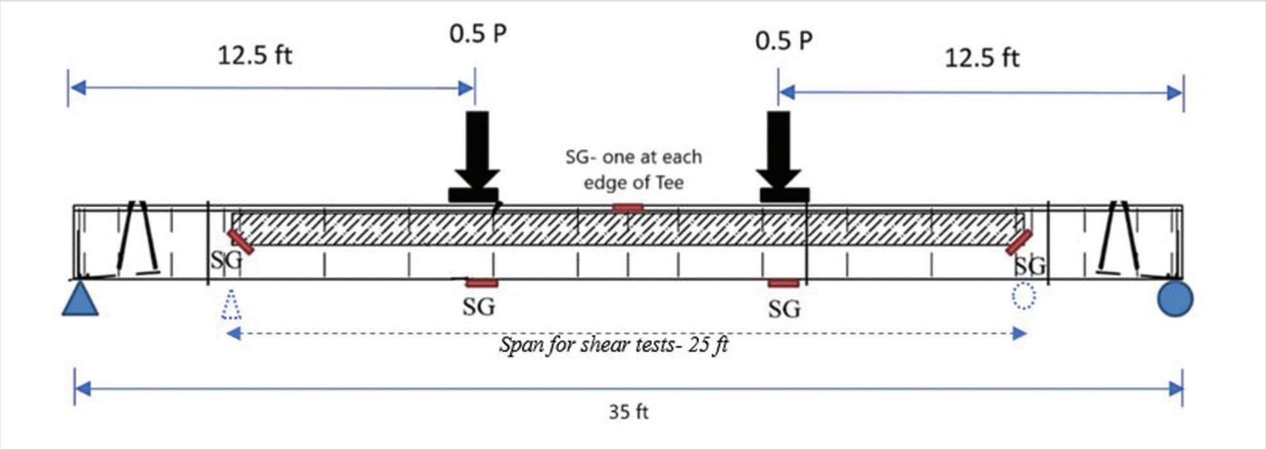

Figure 4. Four-point bending test setup shown with strain gauge locations. Note: P = applied load; SG = strain gauge. 1 ft = 0.305 m.

5. 24-hour sustained load test (specimen L2 only)

6. Loading to ultimate flexural capacity

7. Shear load test (different boundary conditions)

This paper focuses on the results of load stages 6 and 7, quasi-static loading to evaluate flexural and shear capacity. Other than flexural cracking, the specimens did not experience any damage during loading stages 1 to 5. The sustained-load test was only conducted on L2 because it was assumed to be the weakest specimen in the program. Sustained-load tests were omitted from the other specimens due to scheduling constraints at the laboratory. A complete discussion of the seven stages of loading is available in the detailed report on this project by Sreedhara.[8]

Results and discussion

Flexural testing

Load-displacement behavior during ultimate flexural tests is shown in **Fig. 6**. Moment in the figure is calculated based on the total applied load from the hydraulic jack plus the self-weight moment. Displacement is the midspan displacement due to applied loads only and is the average of all string potentiometers placed at midspan. The figure also shows the moments associated with specific stress conditions and nominal flexural capacity. Comparisons with flexural capacity are discussed in the next section.

The load-displacement behavior was similar for all specimens during the ultimate flexural tests. Response was initially linear elastic. Stiffness decreased as flexural cracking opened at a load of approximately 19 kip (84.5 kN) (120 kip-ft [162.7 kN-m] moment). Note that these cracks had already formed during service load testing, so opening of the cracks corresponded to decompression of the prestress.

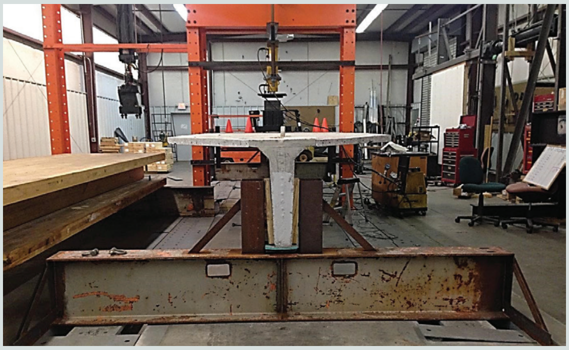

Figure 5. Specimen braced by steel saddle at each support.

Cracking of specimen L1 is representative and is shown in **Fig. 7**. New cracks formed and existing cracks extended as load was increased beyond the previous peak of 28 kip (124.5 kN) (175 kip-ft [237.25 kN-m] moment) from the service load tests. As the force approached 50 kip (222.4 kN) (313 kip-ft [424.35 kN-m] moment), stiffness was effectively gone and the imposed displacement did not result in significant increase in load. Testing continued until the jack reached its maximum stroke length. The maximum displacement achieved during testing was different for each specimen. This was due to changes in the spacers and I-beams placed between the jack and specimen. These changes were made because of concerns that the I-beam would yield; however, they had no impact on the test results other than changing the amount of stroke length of the jack that could be applied to the specimens.

Figure 8 shows the crack pattern near midspan for each of the specimens during ultimate flexural tests. Crushing of the top flange was not observed in any of the specimens during flexural testing. It is likely that the specimens could have supported additional displacement prior to crushing of the flange; however, it is not likely that the peak load would not have in-

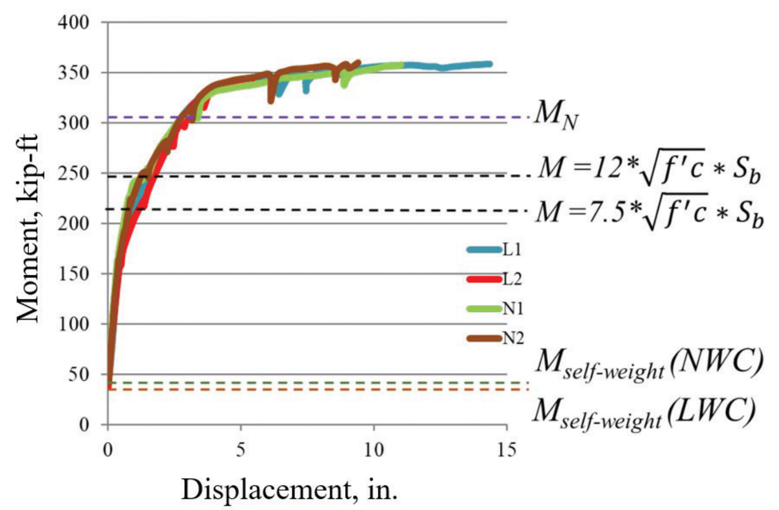

Figure 6. Moment-displacement response during flexural tests. Note: f'_c = concrete strength; L1 = semilightweight concrete specimen with 1 in. foam; L2 = semilightweight concrete specimen with 2 in. foam; LWC = semilightweight concrete; M = moment corresponding to bottom fiber stress; M_N = nominal flexural capacity of 35 ft single-tee specimen; $M_{self\text{-}weight}$ = unfactored moment corresponding to self-weight of a 35 ft single-tee specimen; N1 = normalweight concrete specimen with 1 in. foam; N2 = normalweight concrete specimen with 2 in. foam; NWC = normalweight concrete; S_b = section modulus about bottom fiber. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip-ft = 1.356 kN-m.

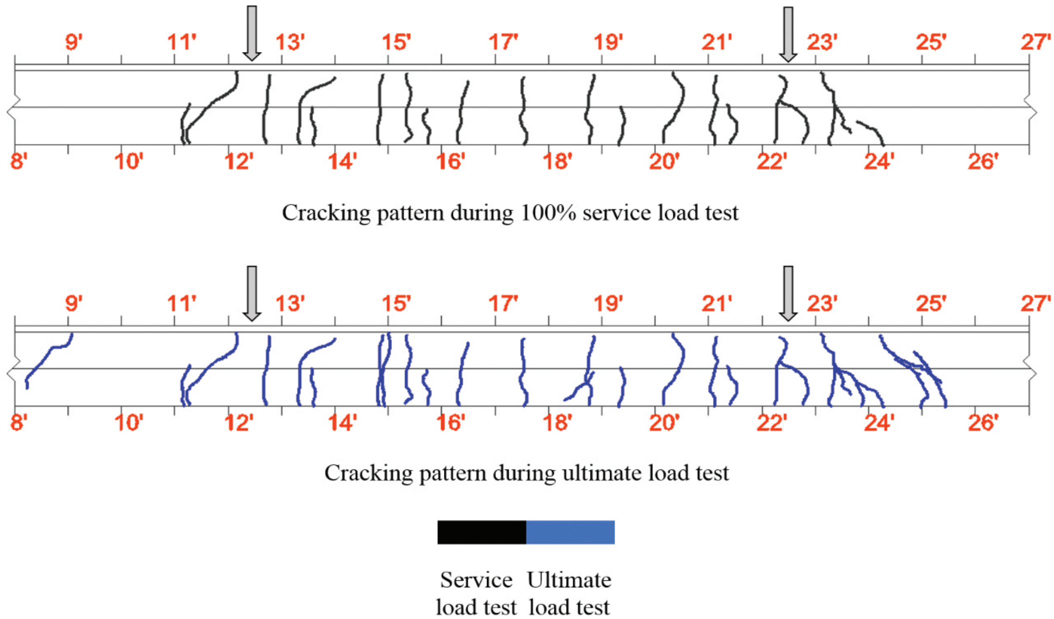

Figure 7. Crack locations of specimen L1. Note: L1 = semilightweight concrete specimen with 1 in. foam. 1 in. = 25.4 mm; 1' = 1 ft = 0.305 mm.

creased significantly. Residual displacement of approximately 4 to 9 in. (101.6 to 228.6 mm) was observed in the specimens after the load was removed.

Each specimen's behavior was ductile at loads near the peak experimental load. As mentioned, testing was terminated when the hydraulic jack reached the maximum stroke, thus the apparent differences in ductility shown in Fig. 6 are a function of testing limitations and not a function of the specimens.

Strain gauges SG1 and SG4 were oriented at an angle of 45 degrees from horizontal and were placed on the surface of

Figure 8. Midspan crack patterns of all specimens during flexural load testing. Note: L1 = semilightweight concrete specimen with 1 in. foam; L2 = semilightweight concrete specimen with 2 in. foam; N1 = normalweight concrete specimen with 1 in. foam; N2 = normalweight concrete specimen with 2 in. foam. 1 in. = 25.4 mm.

the stem near the foam ends (**Fig. 9**) to monitor cracking. This location is of interest because of the abrupt change in cross section due to the end of the foam.

The load-strain response of these gauges was effectively linear elastic throughout the ultimate flexural tests, suggesting that cracks did not form near this location. Visual inspection during testing also confirmed that cracks did not form in the concrete adjacent to the ends of the foam, thus it is considered unlikely that cracks would form at this location under service conditions in foam-void double-tee members that have detailing and material properties similar to the test specimens.

Shear testing

For the shear tests, the supports were moved inward (Fig. 4) so that the entire span consisted of the foam-void segment. Displacement data were collected using string potentiometers; however, these data are not as insightful as the specimens already had significant cracking and residual displacement from the flexural testing. Thus the primary goal of the shear tests was to determine the experimental shear capacity through the foam void segment of the specimens.

Specimens were loaded by imposing displacements using the hydraulic loading system. Loading continued until failure or the hydraulic jack reached its maximum stroke distance. Failure behavior was distinct for different specimens. Specimen L1 failed in flexure due to a strand rupture, which occurred below one of the load points. Specimen L2 did not completely fail because it was not possible to load it further after the stroke length of the jack reached its maximum limit. Specimens N1 and N2 failed in flexural shear. In the case of N1 (**Fig. 10**), the crack crossed one stirrup before reaching the flange. For specimen N2, failure occurred without any inclined cracks that would have intersected a stirrup. The critical cracks (those associated with shear failure) in N1 and N2 had occurred previously during flexural testing. For N2, the failure occurred due to shear failure of the compression zone. For N1, the failure was a typical flexural-shear mechanism. The peak applied shear force was 46.7, 58.9, 47.5, and

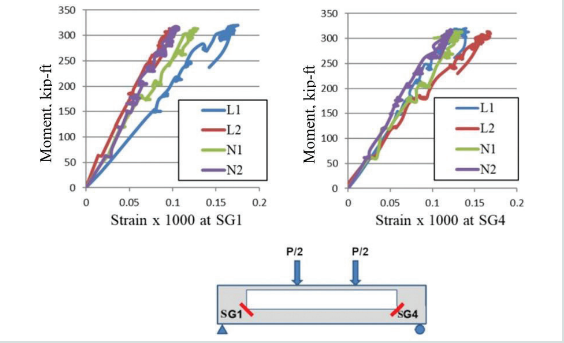

Figure 9. Load-strain response at ends of the foam during flexural testing. Note: L1 = semilightweight concrete specimen with 1 in. foam; L2 = semilightweight concrete specimen with 2 in. foam; N1 = normalweight concrete specimen with 1 in. foam; N2 = normalweight concrete specimen with 2 in. foam; P = applied load; SG1 = strain gauge at end of foam void; SG4 = strain gauge at end of foam void. 1 in. = 25.4 mm; 1 kip-ft = 1.356 kN-m.

Table 2. Comparison of experimental and nominal moments

Specimen	Maximum moment due to self-weight, kip-ft	Maximum moment due to applied load, kip-ft	Experimental flexural capacity M_E, kip-ft	Nominal flexural capacity M_N, kip-ft	Strength ratio M_E/M_N
Semilightweight concrete with 1 in. foam void (L1)	38.3	320	358.3	305.5	1.17
Semilightweight concrete with 2 in. foam void (L2)	36.7	314.4	351.1	305.5	1.15
Normalweight concrete with 1 in. foam void (N1)	44.1	314.4	358.5	305.3	1.17
Normalweight concrete with 2 in. foam void (N2)	42.3	319.4	361.6	305.3	1.18

Note: 1 in. = 25.4 mm; 1 kip-ft = 1.356 kN-m.

59.1 kip (208, 262, 211, and 263 kN) for specimens L1, L2, N1, and N2, respectively.

Although it would have been desirable to have virgin specimens for determining shear capacity, the results are still considered meaningful. Prior to the shear tests, the specimens were subjected to flexural cracking from the earlier stages. It is reasonable to assume that the shear strength of damaged (preloaded) beams should be the same or lower than that of a comparable undamaged virgin beam. It is considered unlikely that the shear strength of an undamaged beam would be less than the shear strength of the damaged test specimens.

Comparison of experimental and theoretical capacities

Comparison of flexural capacities

Flexural capacity was calculated using the strain compatibility approach. Results are summarized in **Table 2**. Calculations used the constitutive model for strands from the *PCI Design Handbook: Precast and Prestressed Concrete*.[9] Concrete compressive strength was taken to be 9380 psi (64,675 kPa) for normalweight concrete and 9880 psi (68,123 kPa) for semilightweight concrete based on the results of concrete cylinders tested at the time of experimental investigations. The presence of foam did not affect the calculations because the theoretical compression block was within the flange at nominal capacity. The calculated nominal capacity was 305.3 kip-ft (413.9 kN-m) for the normalweight concrete specimens and 305.5 kip-ft (414.2 kN-m) for the semilightweight concrete specimens. In each case, the maximum experimental moment exceeded the calculated nominal flexural capacity (Fig. 6). The experimental-to-nominal moment ratios were 1.17, 1.15, 1.17, and 1.18 for specimens L1, L2, N1, and N2, respectively.

Figure 10. Flexural-shear failure of specimen N1. Note: N1 = normalweight concrete with 1 in. foam. 1 in. = 25.4 mm.

Comparison of shear capacities

To provide context for evaluating the shear test results, the experimental shear capacities of the specimens are compared to two baseline values. The first baseline is the theoretical contribution of the stirrups to shear capacity V_s of the test specimens. The shear contribution of the stirrups was calculated to be 26.4 kip (117.4 kN) using ACI 318-14 Eq. (22.5.10.5.3). The concrete contribution V_c was not considered in the baseline values because the concrete shear area was interrupted by the foam voids. As such, it was considered conservative to ignore the concrete contribution to shear capacity. The second baseline value is the factored shear force V_u from the same 60 ft (18.3 m) parking structure 12DT28 discussed earlier in the paper. The factored shear force was calculated to be approximately 28 kip (124.5 kN). To facilitate comparison with the experimental results, the calculated values are based on a single stem.

Comparisons of shear forces are made in **Fig. 11**. Two points are made regarding this figure. First, the experimental shear

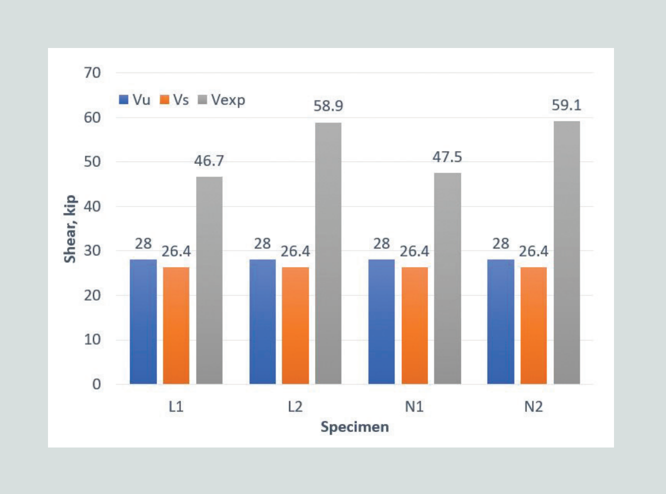

Figure 11. Comparison of experimental and theoretical shear forces. Note: L1 = semilightweight concrete specimen with 1 in. foam; L2 = semilightweight concrete specimen with 2 in. foam; N1 = normalweight concrete specimen with 1 in. foam; N2 = normalweight concrete specimen with 2 in. foam; V_{exp} = experimental shear force; V_s = shear strength provided by steel in specimen; V_u = factored shear force. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

forces were significantly (at least 66 %) more than the factored shear demand for the parking structure member. This suggests that foam-void double tees are likely suitable for carrying shear forces in typical parking structures. Second, the experimental capacity was significantly more than the steel contribution to nominal shear capacity. From this result it is concluded that the concrete, despite the foam void, was a significant contributor to shear capacity. This may be attributed to the relatively high concrete strength, which was approximately 10 ksi (69 MPa) at the time of testing. Lower concrete strength should be considered in any future tests.

The test specimens did not have a concrete topping slab that would be present in production members. The presence of a topping would have increased the shear capacity of the specimens; however, the specimens exceeded the baseline shear demand even without the topping. This further confirms that foam-void double-tee members can have sufficient strength for carrying shear loads associated with typical applications.

Conclusion

This paper reports the results of flexural and shear testing on four foam-void precast, pretensioned concrete tee beams. The tests were part of a larger experimental program focusing on the use of foam voids to reduce the self-weight of precast concrete double-tee members. The motivation for the research was to reduce the self-weight of double-tee members in order to expand the number of situations where two double-tee members can be shipped in one truckload. The following key observations were made in this study:

- The foam-void test specimens demonstrated ductile behavior at near-ultimate flexural loads.
- The specimens supported experimental moments that exceeded theoretical nominal capacity by 15% to 18%.
- During the flexural testing, cracking was not observed at the ends of the foam voids at near-ultimate load levels. Thus cracking at the foam ends would not be expected in service conditions for similar foam-void double-tee members used in buildings.
- The experimental shear capacities through the foam-void segments of the test beams were significant. The experimental shear forces on the foam-void segments were at least 66% greater than the factored shear demand on a comparison parking structure double-tee member.

These observations are specific to the test specimens and depend on the concrete strength, transverse reinforcement, and other structural details. The compressive strength for the test specimens approached 10 ksi (69 MPa). Transverse reinforcement consisted of double-leg no. 3 (10M) stirrups spaced at 12 in. (304.8 mm). Compared to solid double-tee members, the percentage of weight reduction was 4% for specimens with 1 in. (25.4 mm) thick foam board and 8% for specimens with 2 in. (50.8 mm) thick foam board.

Design recommendations

Based on experimental tests and the resulting conclusions, the following recommendations are suggested for the design of double-tee members with foam voids:

- Foam boards with 1 or 2 in. (25.4 or 50.8 mm) thickness can be used in the stems of precast concrete double-tee members while still maintaining significant structural capacity. The 28 in. (711.2 mm) deep untopped specimens in the test program supported shear loads in excess of the ultimate loads on a typical 60 ft (18.3 m) span parking structure double tees. The use of wider foam boards would lead to larger weight reductions; however, additional testing is recommended prior to using boards wider than 2 in. in production members.
- The nominal flexural capacity of foam-void double tees can be calculated using traditional flexural theory.
- Any strands or other flexural reinforcement should have the same cover requirements from foam board as they would have from a free concrete surface.
- Solid segments without foam voids are suggested near the supports over a distance equal to at least twice the member depth. This distance corresponds to the solid spans used in the test program.
- Stirrups, ties, or other shear reinforcement around the foam boards is considered critical. The spacing of such reinforcement may be designed so that the reinforcement contribution to shear capacity exceeds the factored shear

force. The concrete contribution should be neglected at sections with a foam void. Additional shear reinforcement should be placed near the start and end of each foam board segment.

- The shear reinforcement should be detailed to keep the foam board in position and prevent it from floating up due to buoyancy inside of wet concrete during placing. To keep the shear reinforcement in position, it can be anchored to the prestressing strands.

- Extruded polystyrene foam boards were suitable for the rigors of handling during fabrication and casting of foam-void double-tee members.

- Consideration should be given to rounding the top and bottom corners at the ends of the foam. This was not done in the test program but would nevertheless help to mitigate the possibility of stress concentrations in the concrete adjacent to the foam ends.

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Notation

f_c' = concrete strength

M = moment corresponding to bottom fiber stress

M_E = experimental flexural capacity of specimen

M_N = nominal flexural capacity of 35 ft single-tee specimen

$M_{self\text{-}weight}$ = unfactored moment corresponding to self-weight of a 35 ft (10.7 m) single-tee specimen

P = applied load

R = thermal resistance

S_b = section modulus about bottom fiber

V_c = shear strength provided by concrete in specimen

V_{exp} = experimental shear force

V_s = shear strength provided by steel in specimen

V_u = factored shear force

About the authors

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Abstract

This paper presents an experimental study of precast concrete double-tee members with foam voids cast in their stems. Extruded polystyrene foam boards were placed in the stems of the double-tee members to produce foam-void double tees. The foam-void double-tee specimens in the test program had 4% to 8% less self-weight than comparable solid double-tee members. The motivation for this study was to reduce the self-weight and thereby increase the number of situations where two precast concrete double-tee members can be shipped on a single truck. The experiential program used four 35 ft (10.7 m) long specimens with either 1 or 2 in. (25.4 or 50.8 mm) thick foam voids. Two of the specimens were built using normalweight concrete and the other two using semilightweight concrete. Specimens were subjected to flexural and shear loading. The results demonstrated that when properly detailed and fabricated, foam-void double-tee members can provide sufficient structural capacity for common service and ultimate loading.

Keywords

Double tee, efficient double tee, extruded polystyrene foam, foam-void double-tee stem, lightweight concrete, parking structure, shipping and handling.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

Please address any reader comments to *PCI Journal* editor-in-chief Tom Klemens at tklemens@pci.org or Precast/Prestressed Concrete Institute, c/o *PCI Journal*, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.