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Behavior of Composite Truss Girders

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Summary

This paper reviews the results of a series of tests that focus on the behavior and strength of composite truss girders. Three laboratory size floors were constructed and loaded to failure. Top chord supported open web trusses were used as secondary members. Three unique methods of generating composite action in truss girders are examined.

1. Introduction

As the use of composite long-span trusses has increased, interest in developing better and more efficient applications of this type of system has also increased. Historically, composite floor design has been applied only to the trusses directly supporting the composite slab. The girders supporting these trusses are usually either non-composite hot-rolled universal beams or non-composite truss girders. As a result, composite truss research has focused only on those trusses which directly support the deck sheeting and concrete.

With the increased use of composite trusses, it is of interest to study the behavior of supporting composite truss girders. A primary consideration in creating a floor system with composite truss girders is the method used to frame the trusses into the truss girders so that the shear studs can be properly attached to the truss girders. Three separate methods of framing were selected for study. Each of these methods is presented and the test results reviewed in this paper.

2. Research Program

A series of three composite floor systems were recently constructed and tested at Virginia Polytechnic Institute and State University. Destructive testing was performed to determine the strength of the girders and observe their behavior within floor systems. These systems included the influence of truss connection behavior on the girders and thus the girder behavior in an actual structure was better approximated.

Each test setup used three full-size girders, spanning 9.1 m and spaced at 2.1 m on center. The setups consisted of an interior truss girder (IG) and two exterior truss girders (EGL, EGR). For the third setup only, a universal beam (EB) was substituted for one of the exterior truss girders to compare the composite action of the two girder types. Secondary trusses, which would generally span from 9 m to 20 m in an actual structure, were shortened for testing to about 2 m and placed between the girders at the third points. Steel sheeting was placed perpendicular to the secondary trusses and parallel to the truss girders. Figure 1 shows the typical framing plan.

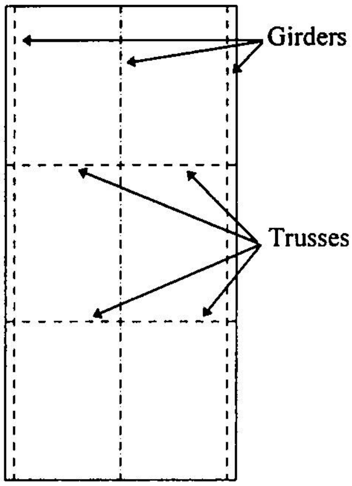

Fig. 1 Test Floor - Plan View

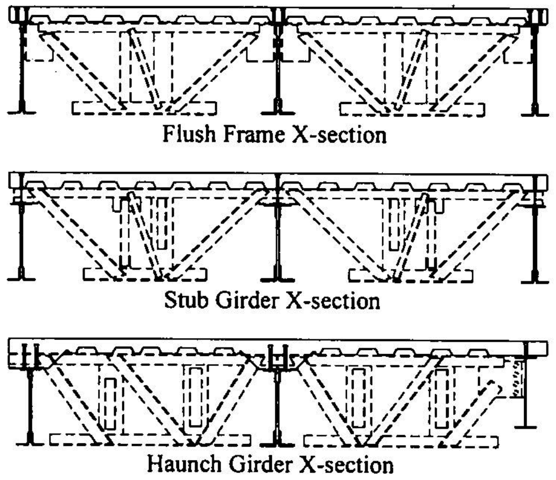

Fig. 2 Specimen Cross-Sections

Headed shear studs were used as the shear connectors; the secondary trusses all had double rows of studs placed in their strong (favorable) positions (Easterling et al 1993). One layer of woven wire fabric was used throughout the entire slab of each setup for temperature and shrinkage effects. Transverse reinforcement was designed separately for each girder in each test setup. The concrete slabs were all constructed of 51 mm steel deck with 76 mm of concrete cover, for a total depth of 127 mm. The girder ends were simply supported by rollers resting on load cells, which were used to measure the girder end reactions.

The systems were loaded with two hydraulic rams which were anchored in load frames and positioned over the secondary trusses. Each ram applied load to a spreader beam which divided the load into two concentrated loads on each secondary truss line. These loads were transmitted by the trusses to the girder third points; the girders then carried the loads to their end supports. The concentrated load points, as well as the rams, could be moved to create different loading situations.

The first setup was a flush framed configuration. The second setup involved the use of stub girders, while the third employed concrete haunches over the girders, as shown in Figure 2. The systems were all tested to observe their composite behavior. Several load applications were performed on each floor. The first application of each test was to load the exterior girders to their predicted service loads and then unload. The load points over the trusses were then repositioned to principally load the interior girder. The second application loaded the interior girder to its predicted service load. The third application loaded the interior girder to 1.67 x service loading, which is the load at which the girder bottom chord would theoretically yield.

The load points were then shifted back to their original positions so that the exterior girders could be loaded to 1.67 x service loading. Further cycles would then be performed with the load points directly over individual girders, as necessary, to load them to failure. During these tests, measurements were taken of strains at various locations along the girders, strains on the surface of the concrete slab, vertical and horizontal displacements of the girders, and slips between the concrete slab and the girder top chords. Detailed descriptions of each of these tests are found in the project reports (Kigudde et al 1996; Showalter et al 1997a, 1997b).

2.1 Flush Framed Tests

For this setup, the secondary trusses were connected to the truss girders with flush-framed, bolted connections. This allowed the top chords of the trusses to be positioned at the same elevation as the top chords of the girders. The steel sheeting could then be placed directly on the top chords of both the girders and the trusses. Shear studs were spaced along the girder lengths to provide sufficient shear resistance to develop the truss girder bottom chord nominal yield stress values. Three-bolt (on EGL) and four-bolt (on EGR) single plate (fin plate) framing connections were used for evaluation purposes.

Eleven different loadings were placed on the system. Symmetric loading patterns were first used to load the girders to their predicted service and first yield strengths. A series of tests were then conducted using unsymmetric loading. Additional symmetrical loads were placed on the system to establish non-linear behavior in the girders, followed by tests to failure of the individual girders.

The exterior girder, EGL, carried a maximum total load (structure dead load + applied load) of 783 kN. The bottom chord was beginning to yield when the girder failed abruptly as a compression web member buckled during a subsequent load application. EGR carried a maximum total load of 776 kN and later failed when a web member buckled. The bottom chord had not reached yield when failure occurred. IG carried a maximum total load of 1535 kN, with definite yielding of the bottom chord. The girder failed completely when compression web members buckled.

2.2 Stub Girder Tests

The second setup utilized a stub girder arrangement, similar to the concept described by Chien and Ritchie (1984), to form the composite truss girders. The secondary truss top chords were seated directly on the girder top chords at the girder span third points. A series of three shallow universal steel beams, or stubs, each approximately 3 m in length, were welded along the length of each girder. The stubs were separated by 200 mm gaps where the secondary truss top chord seats framed in. These stubs filled the 127 mm gaps between the girder top chords and the truss top chords to provide a supporting surface for the steel sheeting. A single row of shear studs was placed directly over the girder centerlines along the girder lengths.

A total of seven loadings were placed on the system. The girders were loaded to their predicted service and first yield strengths, and then the individual girders were tested to failure. Failure in the exterior girder, EGL, occurred in part because of a loss of shear connection at one end of the slab when longitudinal cracking occurred directly along the line of shear studs. Yielding of the bottom chord occurred beneath the third point near the end where the cracks formed. The maximum total load was 670 kN. Similarly, the shear connection on EGR failed when severe longitudinal cracking occurred along the line of studs on the girder at the opposite end of the setup. It carried a maximum total load of 636 kN. The interior girder, IG, carried a maximum load of 1346 kN and failed as a result of concrete crushing beneath the load points directly over the girder as well as by yielding of the bottom chord at the span third points.

Failure was also caused in part by the formation of plastic hinges in the top chords where the cross-sections were reduced because of the gaps between the stub beams. This introduced higher stresses in the bottom chords at the third points than at midspan, where the girders showed no signs of yielding. The rotation of the slab, occurring as the interior girder deflected, caused several interesting results. Transverse reinforcement had been designed over the girders to resist longitudinal shear. This reinforcement was not sufficient, however, to carry transverse tensile forces in the slab as it rotated. Because of this, the slab cracked longitudinally along the row of studs, causing a loss of shear connection. In addition, as the slab rotated, the secondary trusses rotated as well. The truss seats, being held in place on the girders with welds, caused significant local buckling of the top chord horizontal legs, especially on the exterior girders. This effect would almost certainly be magnified by more typical length trusses.

2.3 Haunched Girder Tests

The third setup used concrete haunches over the girders. As a result, the concrete over the girders was twice the thickness of the normal slab and much longer studs were required. These studs were placed in double rows along the lengths of the girders. The haunches were formed to encase the truss seats. This meant that the seats were cast into the haunch concrete and could be considered additional shear connectors.

This setup also included a universal beam in place of one of the exterior truss girders. The two secondary trusses framing into the beam used four-bolt single plate connections to provide a flush frame on that side of the floor, eliminating the need for a haunch over the beam. A single row of studs were welded directly onto the top flange over the beam centerline. The beam was designed to carry a comparable moment to that of the opposite exterior truss girder (*Load* 1993).

The specimen was subjected to a total of six loadings. The girders were loaded to their predicted service and first yield strengths and then the individual girders were loaded to failure. As in the first setup, no problems were encountered with the strengths of the shear studs on the truss girders. Failure of the beam was a result of a loss of shear connection. It carried a maximum load of 617 kN. The truss girders failed due to yielding of the bottom chords at the span third points. EGL carried a maximum load of 796 kN while IG carried 1845 kN.

3. Comparison of Predicted and Experimental Results

The ultimate flexural strength of each girder was calculated for the controlling design limit state of bottom chord yielding using measured cross-section and material properties. The ratio of the experimental ultimate moment to calculated ultimate moment, M_e/M_c, for each test is shown in Table 1 and varied from 0.75 to 1.27.

The values of experimental moment of inertia for the specimens were determined from the slope of load vs. deflection curves for the initial loading up to service load. The calculated values were produced by a method incorporating adjustment factors for slip and for the span/depth ratios of the truss girders. The ratios of experimental moment of inertia to calculated effective moment of inertia indicate that the stiffnesses of the flush framed girders and stub girders were accurately predicted. The haunched girders, on the other hand, were much stiffer than predicted.

SETUP	GIRDER	M_e/M_c	I_{effe}/I_{effc}
Flush Framed Girders	EGL	0.93	0.99
	IG	0.99	1.00
	EGR	0.92	1.02
Stub Girders	EGL	0.89	1.01
	IG	0.82	1.01
	EGR	0.86	1.02
Haunched Girders	EGL	1.04	1.10
	IG	1.27	1.30
	EB	0.75	1.02

Table 1 Summary of Composite Joist Girder Tests

The stub girder configuration involved the least labor in terms of shop fabrication of the trusses and truss girders and field erection, but had the poorest performance in terms of strength. The flush framed setup displayed better performance, but it also had greater fabrication demands due to the flush framed connections. The haunched girder setup had the best strength performance, but the field erection was more labor intensive and the slab the most heavily reinforced.

4. Acknowledgments

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5. Notation

EB = Exterior universal Beam
EGL = Exterior Girder Left (on assigned left of system)
EGR = Exterior Girder Right (on assigned right of system)
IG = Interior Girder of system
I_{effe} = experimental effective moment of inertia
I_{effc} = calculated effective moment of inertia
M_e = experimental moment capacity
M_c = calculated moment capacity
P_{max} = experimental maximum total load on a particular girder

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