Lightweight Concrete-on-Steel Composite Beams

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CURRENT DESIGN specifications[1] for composite beams include no provisions for beams with lightweight concrete. Conceptually, the use of lightweight concrete for composite beams can lead to greater economy, since a substantial reduction in dead load can be obtained. However, there have been some questions regarding the use of lightweight concrete in composite construction. These include:

1. Effectiveness of the lightweight concrete slab,
2. Strength of shear connectors in lightweight concrete,
3. Value of modular ratio to be used in elastic flexural calculations.
4. Effects of creep and shrinkage, and
5. Effects of partial shear connection.

With these questions in mind and with the primary objective of establishing design criteria for lightweight concrete composite beams, a joint research effort between the University of Missouri-Columbia and Lehigh University was undertaken. The Lehigh portion of the study was aimed at determining shear connector strength from pushout tests.[4] The work done at Missouri was comprised of an experimental and theoretical study of full scale composite beams to determine the overall strength and behavior of the beams. This paper presents the work done at the University of Missouri-Columbia.

SCOPE OF WORK AT MISSOURI

Six composite beams, five companion pushout specimens, and the necessary control specimens were tested. The major variables were the type of lightweight concrete, the type of loading (short term or sustained), and the degree of shear connection. The results were evaluated and compared with a previously developed theoretical analysis.[2]

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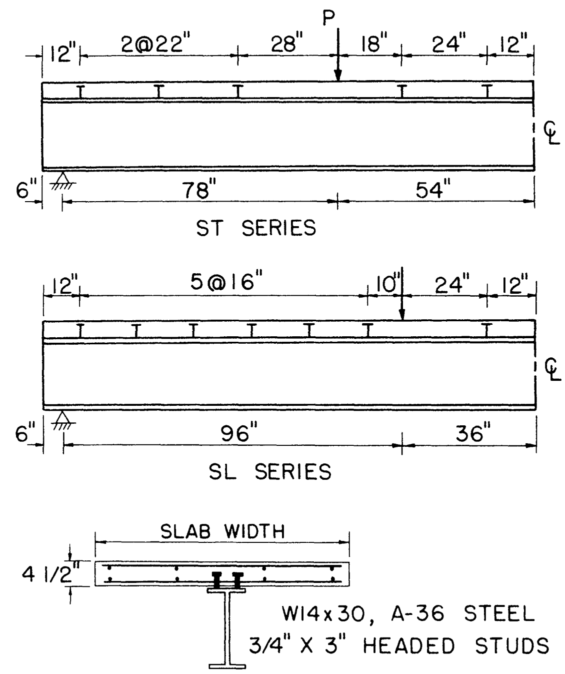


Fig. 1. Beam geometry

The theoretical analysis was used to investigate a wide range of beams that would have been too expensive to study by testing. The variables considered included:

1. The degree of shear connection
2. The effects of unshored construction
3. The modular ratio of the concrete

The results from the theoretical study were compared with current design procedures.

TEST SPECIMENS AND PROCEDURES

The geometry and material properties of the beams tested are given in Table 1 and Fig. 1. The ST series beams were designed with about a 50 percent shear connection to insure a shear connector failure, whereas the SL series beams were designed with an adequate shear connection.

Table 1. Beam Geometry and Material Properties

Designation	Series[a]	Concrete: Type, weight (pcf)	Slab Width (in.)	f_c' (ksi)	f_{sp}' (ksi)	E_c (10^3 ksi)		Static Yield Point of Beam (ksi)
						Measured	Calculated	
B1	SL	E, 112	36	6.03	0.46	2.6	3.06	...
B2	ST	E, 112	36	5.56	0.42	2.2	2.89	36.6
B3	ST	E, 112	72	5.62	0.43	2.2	2.88	35.7
B4	ST	C, 85	72	4.90	0.27	1.7	1.81	35.4
B5	ST	C, 85	36	4.90	0.27	1.7	1.81	36.0
B6	SL	C, 112	36	4.45	0.40	2.2	2.60	36.2

[a] SL = Sustained load test.
ST = Short term test

Two different lightweight concretes were used in this study. Concrete C was an all lightweight mix composed of an expanded shale, rotary kiln produced aggregate, which was brown in color and rounded in shape, with a maximum size of ½-in. Concrete E was a sanded lightweight mix composed of natural sand and a lightweight coarse aggregate, rotary kiln produced, gray to black in color, angular shaped, with a maximum size of ¾-in. The aggregates and mixes were the same as those used in the Lehigh phase.

The test beams were instrumented to measure deflection, slip, and strain, as shown in Fig. 2. The strains were read with an electronic strain meter[3] which made use of the spherical gage points shown in Fig. 2.

The ST series beams were tested as shown in Fig. 3. The loads were applied to the composite beam with four independently controlled hydraulic ram systems. Equal quarter-point deflections were used for control instead of the applied load.

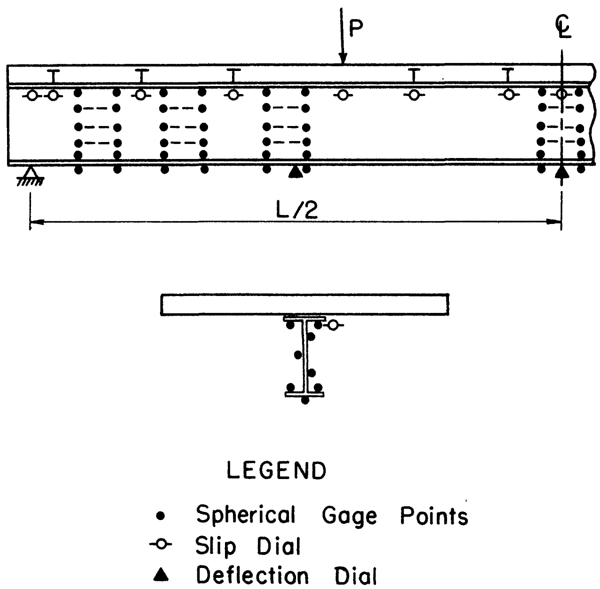


Fig. 2. Instrumentation

The SL series beams were loaded to their working load (14,500 lbs at each load point) in the frame shown in Fig. 4. The load was maintained at a constant value throughout the sustained test period by an automatic hydraulic pumping system. Creep cylinders and free shrinkage cylinders were tested along with each sustained load beam.

The design of the companion pushout specimens was identical to that used in the Lehigh phase.

TEST BEAM BEHAVIOR

Series ST: All beams in this series exhibited a consistent and uniform pattern of behavior. Slips, strains, and deflections were linear to first yield of the steel beam, at which time flexural cracking was observed in the slab in the maximum moment region. As the beams approached failure, a significant amount of ductility was observed with midspan deflections of 12 to 14 in. and end slips of 0.3 to 0.4 in. All beams failed in the shear connection, with two general modes of connector failure being observed: "Shear-off" and "Pull-out." The "Shear-off" failure consisted of shearing the stud through or just above the weld, while the "Pull-out" failure resulted in tearing of the steel beam flange.

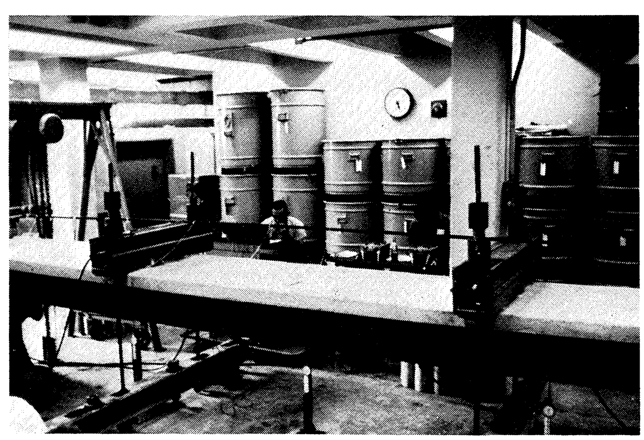


Fig. 3. Series ST

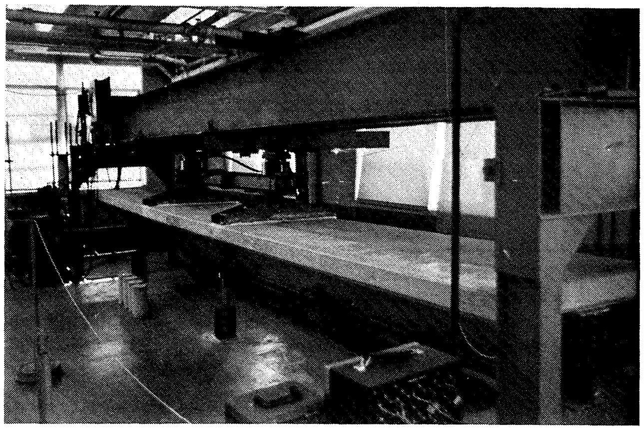
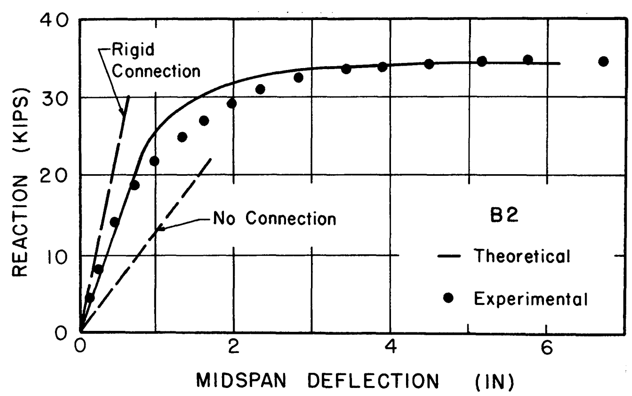


Fig. 4. Series SL

Series SL: Both series SL beams showed a similar behavior pattern under sustained load. Deflections increased quite rapidly for the first two or three weeks and then increased at a slower rate for the next three or four months. At the termination of the tests, the deflections were gradually stabilizing. The slip along the beam interface remained essentially constant for two or three days after loading and then increased quite rapidly for the next week. This was followed by a gradual stabilization of the slip values. It is felt that the initial constant slip values were due to existing bond and friction, which was overcome after two or three days under load.

When the beams were unloaded, cracks across and through the depth of the slab were observed. These cracks were due to the shrinkage of the concrete slab while in the deflected position. Beam B6 was tested to failure after unloading. The beam failed by slab crushing, which was not surprising, since an adequate shear connection was provided.



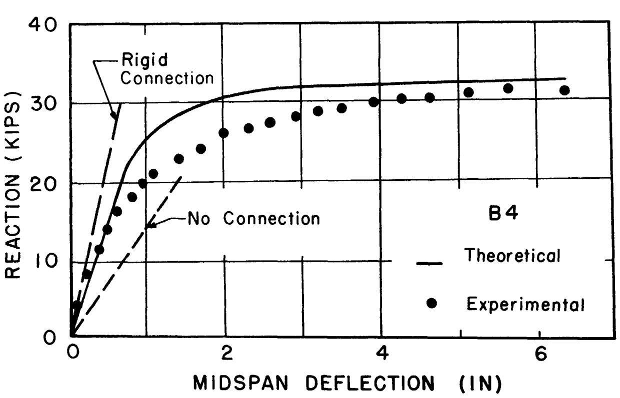


Fig. 5. Reaction vs. midspan deflection—ST

Table 2. Summary of Results

(a) Pushout Specimens

Designation[a]	Ultimate Load per Stud (kips)	Slip at Ultimate Load (in.)
P2	20	0.285
P3	23	0.215
P4	19	0.300
P5	21	0.255
P6	17	0.275

[a] (P2 was companion specimen to B2, P3 to B3, etc.)

(b) Short Term Beam Tests

Designation	Ultimate Moment (in.-kips)		
	Experimental	Predicted[b]	Predicted[c]
B2	2730	2680	2730
B3	2670	2660	2680
B4	2500	2610	2620
B5	2540	2580	2630
B6	3170	2910	2930

[b] Theoretical analysis used in this report; Reference 2.
[c] Ultimate moment analysis; Reference 6.

DISCUSSION AND COMPARISON OF RESULTS

The results for the pushout specimens and beams tested to failure are summarized in Table 2.

Series ST: Illustrated in Fig. 5 are the experimental and theoretically predicted end reaction-midspan deflection curves for B2 and B4 (B3 and B5 were similar). Good agreement between the experimental and theoretical values is observed in the elastic and ultimate regions of the plots. It is felt that the discrepancy in results in the "knee" portion of the curves is due to residual stresses which existed in the steel beam but were not accounted for in the theoretical analysis.

Given in Fig. 6 is a comparison of the measured and predicted slip distribution for B2 and B4 (B3 and B5 were similar). The slip distributions for a load in the

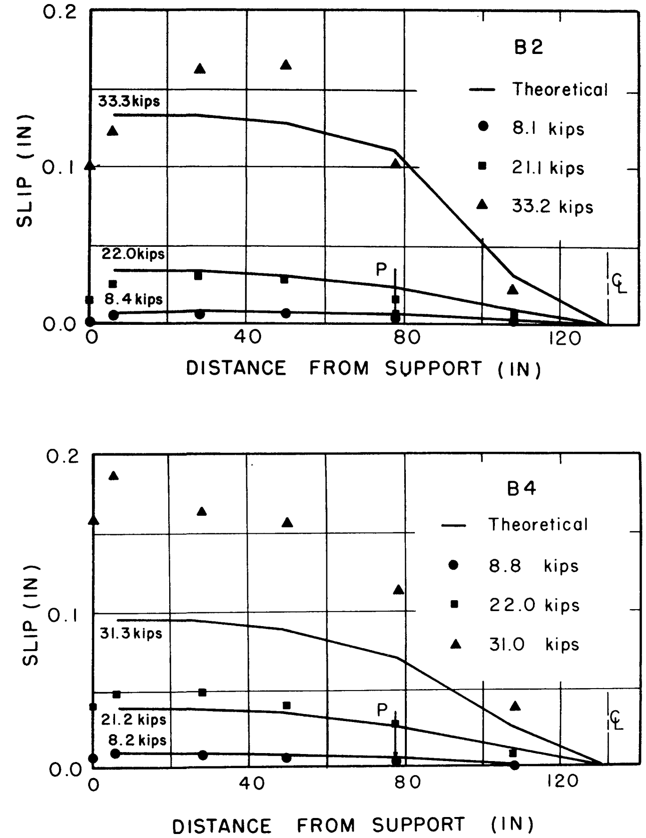


Fig. 6. Slip distribution—ST

elastic range, a load shortly after first yield, and a load near failure are presented on this plot. The agreement between experimental and predicted values at the two lower loads is quite reasonable. The difference in results at the load near failure may be attributed to the extreme ductility of the beam and severe slab cracking, which made accurate slip measurements difficult.

After the initial planning stages of the project and fabrication of the test specimens, a procedure was developed for determining the load-slip characteristics of study shear connectors directly from beam tests. Such characteristics are compared to those obtained from pushout tests in Fig. 7. The experimental beam data were obtained by very accurately measuring the strain profiles between connector groups with an electronic strain meter.[3] These profiles were then converted to stress distributions, which were integrated to determine the net force in the steel beam. Assuming that all the shear is transmitted by the shear connectors, the force in the shear connector group must then be equal to the difference in beam force on either side of the connector group.

The data obtained from B2 indicate an initial connector stiffness essentially the same as the pushout, and a shear connector strength for the exterior and central connector group about the same as the pushout. The connector group nearest the load point behaved differently in that connector forces 50 percent greater than the pushout ultimate were observed at relatively low slip values. This may have been due to the increase in slab compression or the influence of the concentrated load. Beam B3 showed a similar pattern of shear connector behavior.

The data obtained from B4 (B5 was similar) indicate a somewhat different behavior in that the initial stiffness of the exterior and central connector group is lower than that of the pushout, and the ultimate strength of the exterior studs are only 60 percent of the pushout strength. This shows some correlation with the strength and slip characteristics (Figs. 5 and 6) for B4, since the ultimate load was somewhat lower and the slips greater than theoretically predicted.

From the results presented in this section, it may be concluded that composite beams with lightweight concrete slabs are just as effective as those with normal weight concrete, and their behavior can be predicted with reasonable accuracy by the analysis used in this report.

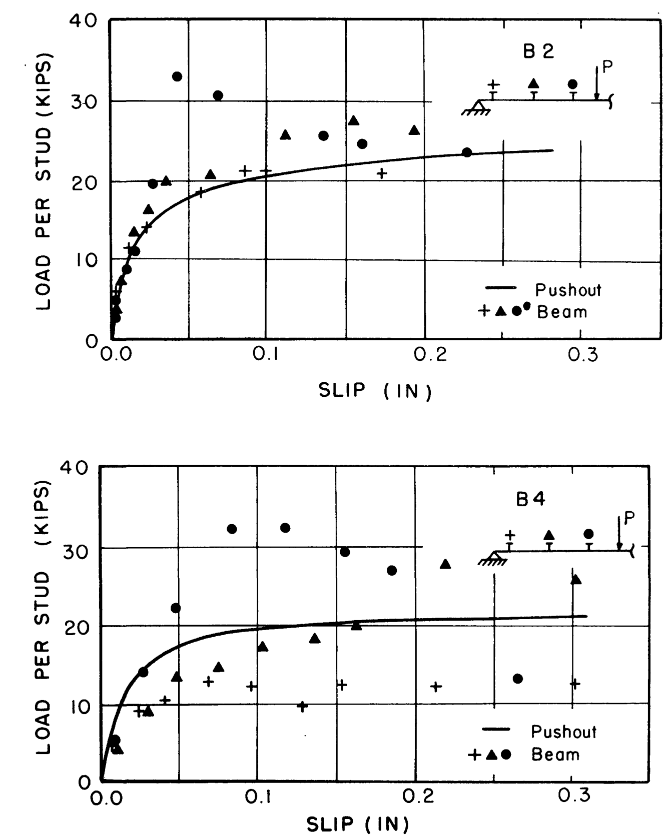


Fig. 7. Load vs. slip—ST

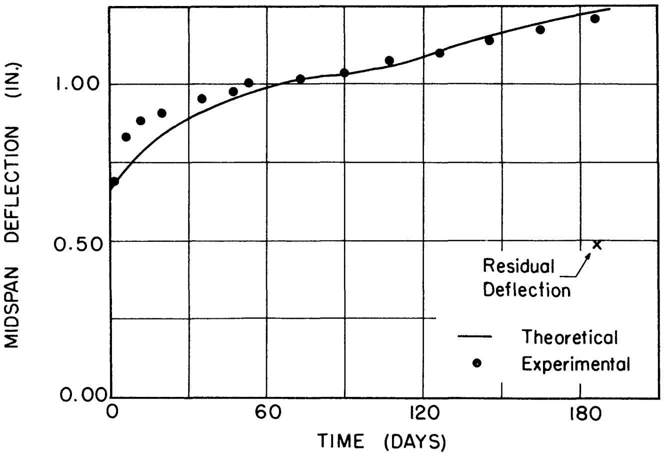


Fig. 8. Time deflection—B6

Series SL: Given in Fig. 8 are the measured and theoretically predicted time-deflection characteristics for B6 (B1 was similar). The measured creep and shrinkage properties of the concrete were included as input in the modified theoretical analysis.[7] About 70 percent of the time-dependent deflection was due to shrinkage of the concrete. The fluctuations in the results presented are due to changes in temperature and humidity in the laboratory, which affect the creep and shrinkage characteristics of the concrete.

Presented in Fig. 9 are the experimental and theoretically predicted end reaction-deflection data for B6 when tested to failure. The lesser than theoretically predicted initial stiffness exhibited by the beam in the elastic range is felt to be due to the effects of creep and shrinkage while under sustained load. Agreement between experimental and predicted ultimate strength is good and the residual stress effect is again illustrated in the "knee" portion of the curve.

The sustained load tests and additional theoretical analyses have shown that the effects of creep and shrinkage increase deflections substantially, and that for a given shrinkage strain the time dependent deflection for beams with lightweight concrete slabs is essentially the same as with normal weight concrete. For design purposes, with creep and shrinkage strains in the range of 500–800 μin. the time dependent deflection may be taken as equal to the instantaneous deflection.

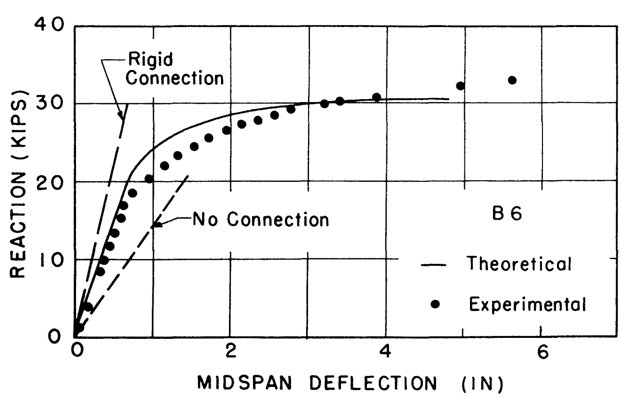


Fig. 9. Reaction vs. midspan deflection—B6

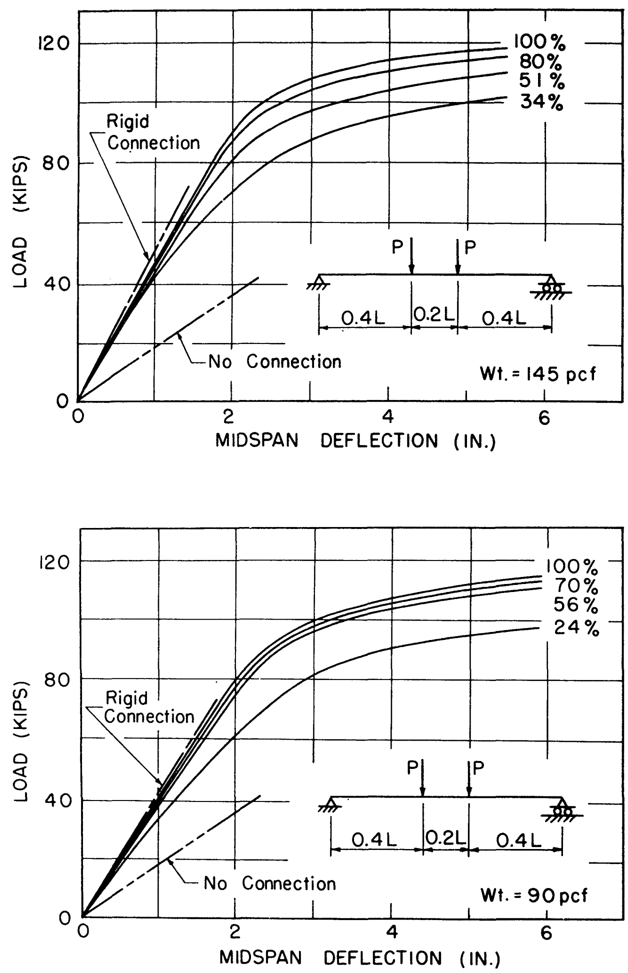


Fig. 10. Load vs. midspan deflection—W36×150

THEORETICAL STUDY

The results reported herein and in previous research[2, 5] have verified the theoretical composite beam analysis. Thus, the analysis may be used confidently to study other parameters.

Partial Shear Connection—Two composite beam sections representing typical extremes found in practice were chosen for this study. The details of these beams are given in Table 3. The 100 percent or adequate connection for the beams with normal weight concrete was designed according to the current AISC specifications.[1] The number of shear connectors for 100 percent shear connection for the beams with lightweight slabs was determined from

Table 3. Partial Shear Connection Beam Geometry

Steel Beam	No. of Shear Connectors	% Shear Connection	Concrete Weight (pcf)	Slab Width (in.)	Slab Thickness (in.)	Span Length (ft)
W12×36	18 14 10 6 4 2	100.0 77.8 55.5 33.3 22.2 11.1	145 ($f_c' = 3$ ksi)	72	4	20
W12×36	26 18 14 10 6 4 2	100.0 69.2 53.9 38.4 23.1 15.4 7.7	90 ($f_c' = 3$ ksi)	72	4	20
W36×150	70 56 36 24 8	100.0 80.0 51.5 34.5 11.4	145 ($f_c' = 3$ ksi)	128	8	60
W36×150	100 70 56 24 8	100.0 70.0 56.0 24.0 8.0	90 ($f_c' = 3$ ksi)	128	8	60

$$N = \frac{q_{ult}\text{ (normal weight)}}{q_{ult}\text{ (lightweight)}} \times \text{current AISC allowable} \quad (1)$$

where

N = number of studs

$$q_{ult} = 0.5A_s\sqrt{f_c'E_c}\text{ (strength of shear connector)} \quad (2)$$

Equation (2) was developed at Lehigh[4] in the companion part of this research. Equal numbers of shear connectors were placed in each connector group for any given beam and a constant group spacing was used.

Illustrated in Fig. 10 are the load-deflection results for the W36×150 beam with both normal and lightweight concrete for various degrees of shear connection. The results for the W12×36 were similar. As is evident from the two plots a reduction in the degree of shear connection results in a decrease in initial beam stiffness and ultimate strength.

Shown in Fig. 11 is the variation in ultimate moment with the degree of shear connection. Concrete weight makes no difference in the ultimate moment if the shear connection is properly designed. As illustrated on the plot, a reduction in shear connection of 50 percent results in a decrease in ultimate moment of only 10–15 percent.

Given in Figs. 12 and 13 is the variation in effective section modulus and beam stiffness with the degree of shear connection. As illustrated, the AISC Specification overestimates the section modulus at 100 percent shear connection, but is conservative at shear connections of 80 percent and less. Furthermore, the beam stiffness at 100 percent shear connection is about 85 percent of the stiffness determined by usual transformed section calculations.

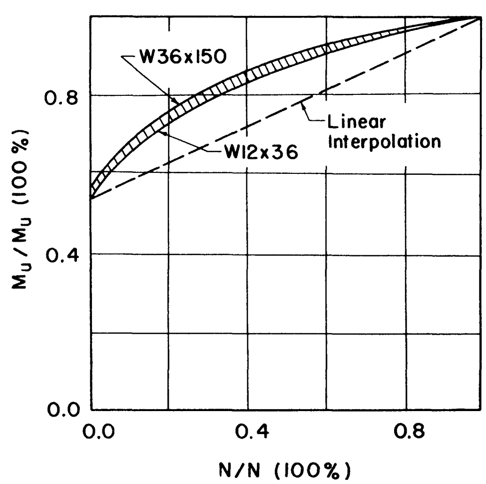


Fig. 11. Ultimate moment vs. degree of shear connection

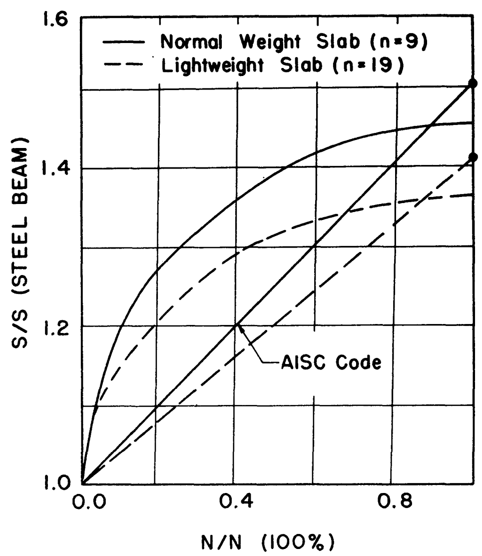


Fig. 12. Effective section modulus vs. degree of shear connection

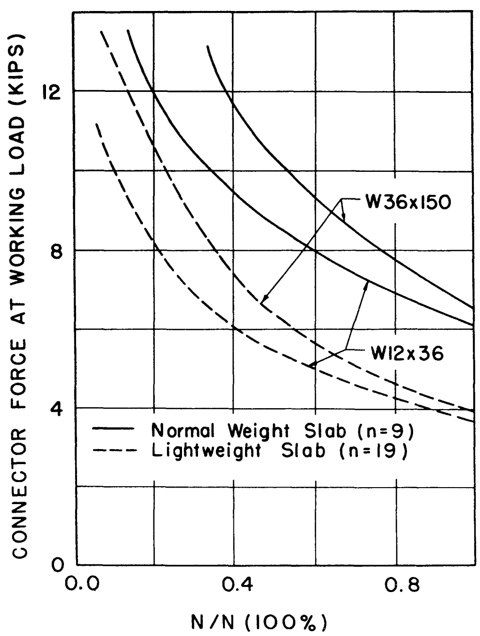


Fig. 14. Shear connector force vs. degree of shear connection

Figures 14 and 15 show the variation in shear connector force and slip at service loads with the degree of shear connection. It is apparent that the connector force and slip increases with decreasing degree of interaction. The rate of increase is particularly large for shear connections of 50 percent and less. Hence, shear connections of less than 50 percent effectiveness should not be permitted.

Modular Ratio, n—The results presented in the preceding section have shown that an increase in modular ratio results in some decrease in section modulus and stiffness, but no change in ultimate moment. Additional studies,[7] using usual transformed section design calculations and covering a wide range of beam geometries and modular ratios, have shown that when the steel beam stress is the controlling factor an increase in n from 8 to 20 results in a 2 to 5 percent increase in steel stress. When the concrete stress is critical an increase in n reduces the concrete stress.

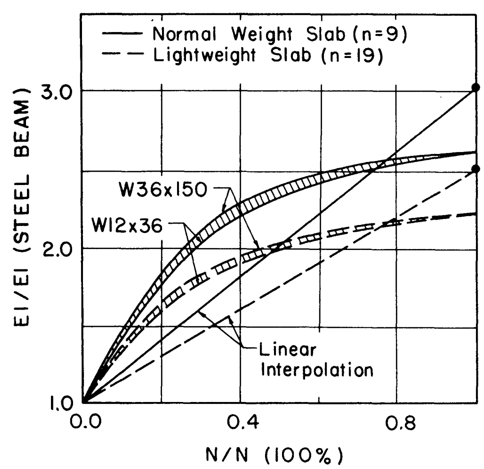


Fig. 13. Stiffness vs. degree of shear connection

From these results it may be concluded that, when using the transformed section method for calculating stresses in beams of normal proportions, the effects of the modular ratio are negligible. Deflection calculations, however, should be based on the actual modular ratio for the lightweight concrete.

Unshored Construction—When unshored construction has been used for composite beams, some uncertainties have existed concerning the effects of the additional steel beam stresses due to dead load. To study this problem the beams with 100 percent shear connection given in Table 2 were investigated with the modified theoretical analysis.[7] Presented in Fig. 16 are the linear portions of the moment-bottom flange steel strain curves for the beams studied. In each case the plot for unshored construction starts at the origin and increases linearly to a point representing the dead load moment and strain. After the concrete has hardened the beam behaves elastically under increasing load to first yield as does the shored case.

Formula (1.11-2) of the AISC[1] Specification,

$$S_{tr} = \left(1.35 + 0.35 \frac{M_L}{M_D}\right) S_s \qquad (3)$$

allows a maximum steel stress, when the live load moment is zero, of 1.35 times the usual allowable bending stress, which for A36 steel is 32.4 ksi for compact sections and 29.7 ksi for non-compact sections. All

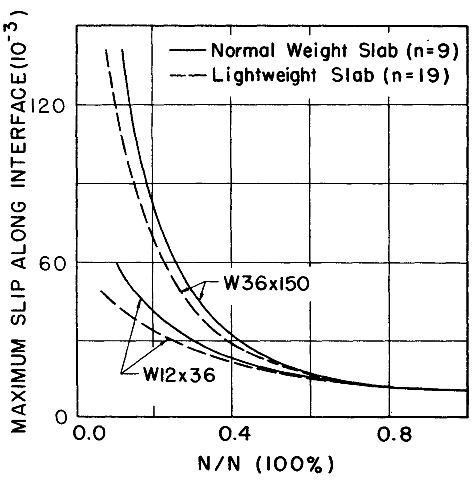


Fig. 15. Shear connector slip vs. degree of shear connection

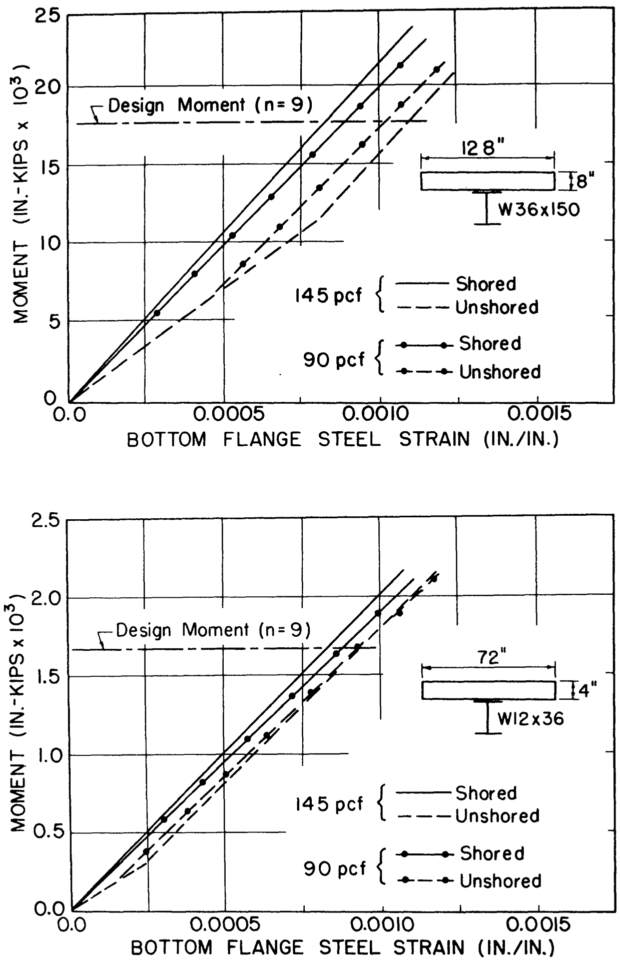


Fig. 16. Moment vs. bottom flange steel strain

beams studied met the requirements of Eq. (3), with the W36×150 beam with normal weight concrete giving an actual stress of 31.6 ksi. This stress, as such, is not excessive, but when the effects of creep and shrinkage[7] and residual stresses are included, the bottom flange steel strain may be well past the yield point, which may affect the serviceability of the beam. For example, a shrinkage strain of 400 microinches results in an increase in bottom flange steel stress of 2.6 ksi for the W36×150 beam.

From Fig. 16 it can also be seen that the effects of unshored construction are not as great when lightweight concrete is used. This is not surprising because of the significant reduction in dead load.

From the results presented, it appears that in some cases AISC Formula (1.11-2) may not provide enough reserve to avoid yielding under the combined effects of full design loads, creep, shrinkage, and residual stresses. However, any criteria that is adequate for normal weight concrete composite beams would be satisfactory for lightweight concrete beams.

SUMMARY AND CONCLUSIONS

Six composite beams with lightweight concrete slabs were tested and the results compared with a previously developed theoretical analysis. The theoretical analysis was then used to study the effects of partial shear connection, modular ratio, and unshored construction on the behavior of composite beams.

Based on the results of this research project the following conclusions may be drawn:

1. Composite beams with lightweight concrete can be just as effective as those with normal weight concrete, and their behavior can be predicted with satisfactory accuracy with the analysis used in this report.

2. The effects of creep and shrinkage increase the deflections substantially. For design purposes (with time dependent strains in the range of 500–800 microinches), the time dependent deflection may be taken equal to the instantaneous deflection for both lightweight and normal weight composite beams.

3. Shear connection of less than 50 percent should not be permitted.

4. In selecting the beam cross section, the design methods used currently for normal weight concrete may be used also for lightweight concrete. All other calculations should be based on the actual modular ratio for the lightweight concrete.

5. The current AISC Specification overestimates the beam stiffness and section modulus at 100 percent connection by 15 and 5 percent, respectively, but underestimates the section modulus at 80 percent connection and less.

6. In some cases, the current AISC Formula (1.11-2) may not provide enough reserve to avoid yielding under the combined effects of full design loads, creep, shrinkage, and residual stresses.

ACKNOWLEDGMENT

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NOMENCLATURE

A_s = cross-sectional area of stud shear connector
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of steel
EI = stiffness of composite beam
f_c' = concrete compressive strength
f_{sp}' = concrete splitting tensile strength
M_D = dead load moment
M_L = live load moment
M_U = ultimate moment of composite beam
N = number of shear connectors
n = modular ratio of concrete (E_s/E_c)
q_{ult} = ultimate shear connector strength
S = section modulus

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