High strength steel composite beams with formed steel deck

High Strength Steel Composite Beams with Formed Steel Deck

Poutres mixtes en acier à haute résistance avec platelage métallique

Hochfeste Verbundträger mit Stahlblechdecke

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1. INTRODUCTION

During the past forty years, formed steel deck has become the most common floor system used in high rise steel frame structures. A natural consequence of this floor system was the development of composite action between the steel beam and the concrete slab by means of shear connectors welded through the deck to the beam flange. However when the corrugations of the deck run perpendicular to the beam, experimental results have shown that a reduction in beam capacity may ensue.

Initial studies of this condition were made on a proprietary basis for specific products in building applications and thus were uncoordinated. Consequently considerable variance among controlled parameters existed, making it difficult to draw any general conclusions. In 1967 a detailed study by Robinson(1) showed that for high, narrow ribs the shear capacity of the connector is a function of the rib geometry and is substantially less than the capacity of connectors embedded in a composite beam with a solid slab. In 1970, Fisher(2) summarized the investigations that had been conducted to date and proposed design criteria. Fisher concluded that composite beams could be modeled as having a haunched slab, equal in thickness to the solid part of the slab above the rib, except that the shear capacity of the connector is reduced. He modeled this reduction in shear capacity by the following formula:

$$Q_{rib} = A \cdot \frac{w}{h} \cdot Q_{sol} \leq Q_{sol} \tag{1}$$

where: Q_{rib} = shear strength of connection in a rib
A = numerical coefficient (0.5 for beam)
w = average rib width
h = height of rib
Q_{sol} = shear strength of a connector in a solid slab

With the many uncontrolled and ill defined variables in these early investigations, there was a need for additional research in this area. Also there was virtually no experimental work done which considered the effect of high strength steel beams and the resulting effect of increased slab force on connector and beam capacity. A research program was initiated at Lehigh University in 1971 involving 17 full scale beam tests, 15 of which utilized high strength steel beams. The work reported herein includes a detailed analysis of these 17 composite beams. Additionally, this analysis is supplemented by an evaluation of 39 other beam tests reported by previous investigators. The work is described in detail in Ref. 3.

This report provides an evaluation of the shear capacity of stud connectors embedded in composite beams utilizing high strength steel with formed steel deck, as well as the flexural capacity of the composite beams themselves. Additionally the stiffness of composite beams with or without formed steel deck is evaluated for service loads.

2. DESCRIPTION OF TESTS

The experimental program at Lehigh consisted of tests on 17 simple span composite beams. The program was designed in accordance with the recommendations suggested in Refs. 2 and 4.

Series A consisted of six beams. It served as the basic series in the program, with average rib width - height ratios of 1.5 and 2. The beams were designed for partial shear connection. Series B consisted of two mild steel beams, as all other beams were high strength steel. Series C consisted of five beams with low degrees of shear connection (below 50%). Series D consisted of four beams with larger rib slopes as their major variable.

The test beams consisted of steel beams on simple spans of 24 or 32 ft. (7.31 to 9.75 m), acting compositely with concrete slabs cast on formed steel deck. All of the steel beams except two were W16 x 40 or W16 x 45 sections with yield strengths between 55 and 70 ksi (379.5 and 483.0 N/mm^2). The two exceptions were both W16 x 58 sections with 36 ksi (248.4 N/mm^2) yield.

The slabs of the beams were made with structural lightweight concrete conforming to the requirements of ASTM C330 (Specification for Lightweight Aggregates for Structural Concrete). The concrete strength and modulus of elasticity where maintained as constants within fabrication tolerances at 4.0 and 220 ksi (27.6 and 151800 N/mm^2) respectively. Minimal reinforcement for all of the beams consisted of 6 x 6 - #10/10 welded wire fabric placed at mid-depth of the slab above the ribs. The thickness of the solid part of the slab was a constant 2-1/2 in. (63.5 mm) for all of the beams. The slab widths were proportioned as 16 times the full thickness of the slab plus the flange width of the steel beam. All slabs were cast without shoring.

The slabs were cast on 20 gauge galvanized steel deck without embossments. The rib heights of the deck were 1-1/2, 2 or 3 in. (38.1, 50.8 or 76.2 mm) for average rib width - height ratios of 1.5 and 2. The slopes of the ribs were a nominal 1 to 12 except for the series D beams which had 1 to 2 and 1 to 3 slopes. The steel deck was fabricated in widths of 24 or 36 in. (609.5 or 914.4 mm) with corresponding rib modules of 6 and 12 in. (152.4 and 304.8 mm).

Composite action between the steel beam and the slab was provided by the placement of 3/4 in. (19 mm) shear connectors. All studs conformed to ASTM A108 specification and were welded through the steel deck to the beam flange in a staggered pattern. All welds were tested by "sounding" the studs with a hammer. Questionable studs were given a 15 degree bend test. Faulty studs were replaced and retested. One or two studs were placed in a rib. The stud spacing was adjusted to accommodate the varying rib geometry but never exceeded 24 in. (609.5 mm). All studs were embedded 1-1/2 in. (38.1 mm) above the rib.

Four point loading was used on all of the beams to provide shear and moment conditions comparable to uniform load conditions. The loads were about equally spaced, but varied slightly so that loads were applied over a rib and not over a void. Figure 1 shows a typical test setup.

The beams were loaded in increments up to their estimated working load, then cycled ten times. After cycling the beams were reloaded in increments to near the ultimate load. Near ultimate, load was applied to produce fixed increments of deflection. Loading was terminated once the plateau of the load-deflection curve was established and deflections became excessive.

The beams were instrumented to measure the deflection at midspan, the slip

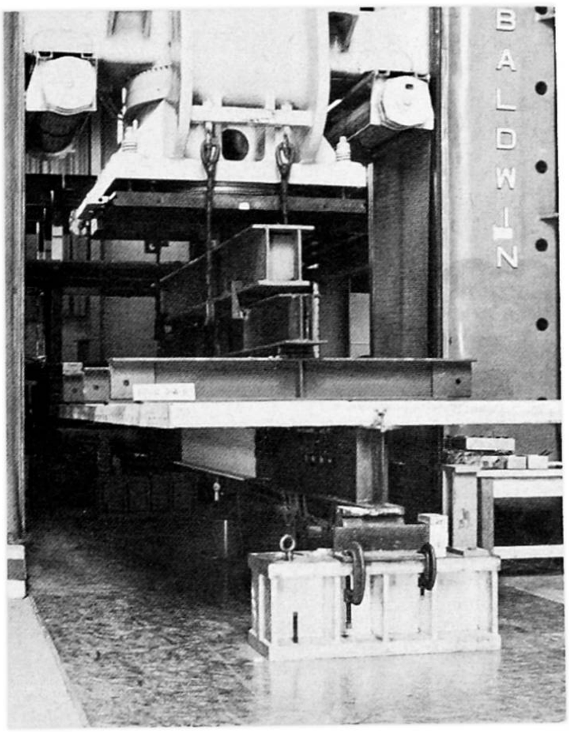

Fig. 1

at various points along the span, and the strain in the steel beam at various points along the span.

3. THEORETICAL CONSIDERATIONS

The flexural capacity of the test beams reported herein was determined essentially from the model suggested by Slutter and Driscoll(4) for composite beams with flat soffit slabs. However, the slab force was assumed to act at the centroid of the solid portion of the slab, above the top of the ribs and not at the center of the concrete stress block.

In many instances, the location of the slab force made little difference in the computed flexural capacity. For beams designed fully composite with the concrete slab governing the shear connection, the center of the stress block coincides with the centroid of the solid portion of the slab. However, for beams with low degrees of partial shear connection and/or high ribs, the location of the stress block has a significant influence on capacity.

For composite beams, with or without formed steel deck, there is loss of interaction or slip between the slab and the steel beam before developing the flexural capacity. This slip has little effect on the shear capcity of the connectors. However, it does effect the location of the slab force. Without any connection at all the compressive stress resultant would lie somewhere in the upper half of the full slab depth. However, with the bottom of the slab constrained by the presence of shear connectors the location of the stress resultant in the slab drops. The assumption that the stress resultant acts at the centroid of the solid portion of the slab seems to more adequately account for all cases involving composite beams with formed steel deck.

Robinson(5) has compared this difference in the assumed location of the slab force for a beam with 3 in. ribs and about 30% partial shear connection. He found that applying the method in Ref. 4 directly, provided an estimated capacity 3% higher than the test data and, that assuming the slab force to act at the center of the solid slab above the ribs, underestimated the capacity. However, he did not include the force on the shear connector directly under the load point, which falls at the edge of the shear span. Had this connector been included, the beam capacity would be overestimated by 9%. With the slab force acting at the center of the solid slab above the rib the capacity would be overestimated by 1%. Strain measurements on this beam confirm the location of the stress resultant in the slab as near the mid-depth of the solid portion of the slab. A similar conclusion was drawn from the Lehigh test beams.

4. BEHAVIOR OF COMPOSITE BEAMS WITH METAL DECK

4.1 Ductility - A significant aspect of these beams is their ductility. This ductility is demonstrated by the large deflections shown in the load-deflection plots in Fig. 2, even for beams with low degrees of partial shear connection. Also shown on the plots are two idealized elastic-plastic load-deflection curves. The elastic portion of the stiffer curve assumes complete interaction between the slab and the beam. The plastic plateau of that curve is the ultimate load for a partial shear connection with a reduced connector capacity defined in Eq. 1. The lower idealized curve is adjusted to account for an effective moment of inertia in the elastic range, which will be discussed later. The plastic plateau for that curve reflects a modified connector capacity as will be discussed later.

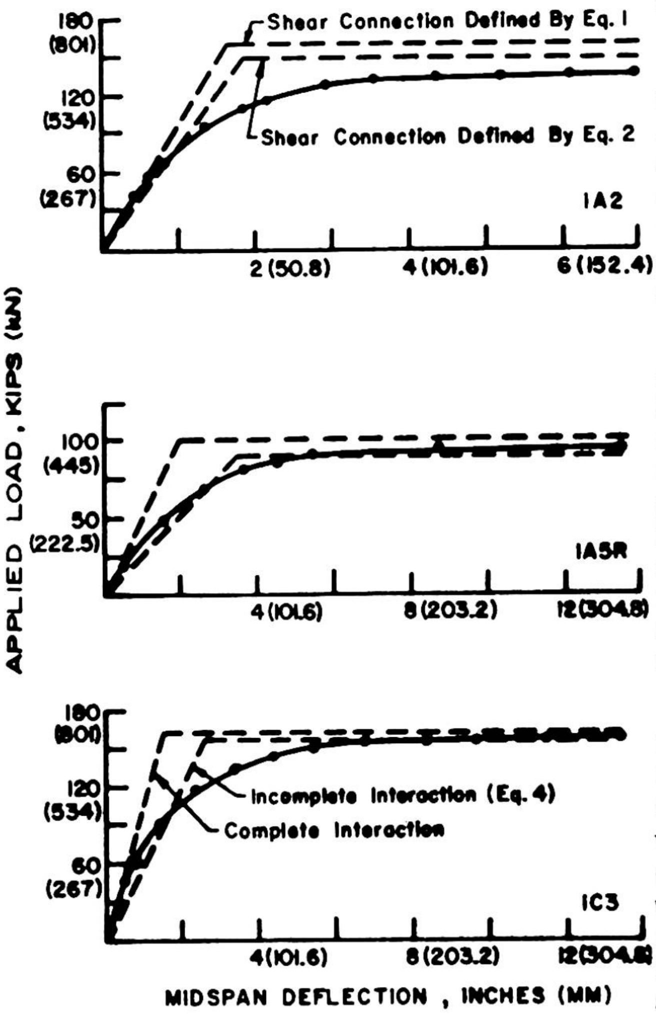

Fig. 2

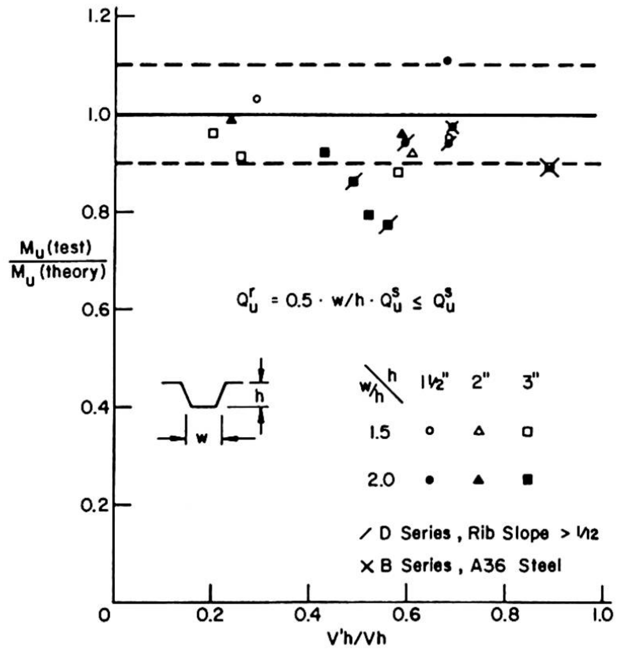

Fig. 3

All of the test beams sustained maximum deflections between 8 and 22 in. (203 to 560 mm). These deflections correspond to more than ten times the deflection at working load in all but two cases. Such large deflections were permitted by the formation of a plastic hinge near the midspan in all of the beams. The formation of these plastic hinges which produced the desired ductility could only have been possible with a ductile shear connection.

Shear connectors were instrumented at selected points along each of the beams. Data on a few of the beams was analyzed and confirms the ductility of the shear connection. All exhibited ductile behavior which permitted the redistribution of the slab force along the span and thus a ductile composite beam. This redistribution of forces permits the prediction of an average connector capacity for the beam, such as suggested in Ref. 2.

The reason for the ductile behavior of the shear connector can be attributed to the relative wide slabs used in the Lehigh test beams. In these tests the slab widths were taken as 16 times the full thickness of the slab, including rib height, plus the width of the steel beam flange. Previous investigators(2,6) have suggested using this slab width for beam tests and for design because it provides an upper limit connector ductility and capacity and more closely simulates the slab-beam interaction in an actual structure. Strain measurements across the slab width have indicated that shear lag is no more severe in a ribbed slab than in a solid slab(3).

<u>4.2 Flexural and Connector Capacity</u> - Unfortunately, the connector model suggested in Ref. 2 (see Eq. 1) for determining the flexural capacity of the composite beams proved unsatisfactory. Figure 3 shows the variation between test moment and theoretical moment using this model for all of the 17 beams. The test moment is nondimensionalized by the predicted moment and plotted against the degree of partial shear connection. Despite the obvious fact that several of the beams fall below their predicted capacity, the plot also shows that several beams with very low degrees of partial shear connection can obtain their predicted capacity. The observation has been made by Robinson(6) as well. Similarly rib slope and yield strength of the steel beam did not appreciably effect the beam capacity, as can be seen in Fig. 3. It is apparent that the connector model must consider other variables.

One such variable was found to be the height of the rib. A reexamination of all available test data, indicated that all of the beams with 3 in. (76.2 mm) deck except one had a stud embedment length greater than 1-1/2 in. (38.1 mm) above the rib. Although not considered as a variable in the Lehigh test program, it was obvious that embedment length is a key parameter in connector capacity. This observation has also been made by Robinson(6).

Thus additional modifications to the connector capacity model proposed in Ref. 2 are required. Besides the average rib width - height ratio, the height of the rib and the embedment of the connector must be taken into account to correctly predict the flexural capacity of composite beams with formed steel deck. To reflect these additional governing parameters, the following revised model was developed:

$$Q_{rib} = 0.6 \cdot \frac{H-h}{h} \cdot \frac{w}{h} \cdot Q_{sol} \leq Q_{sol} \qquad (2)$$

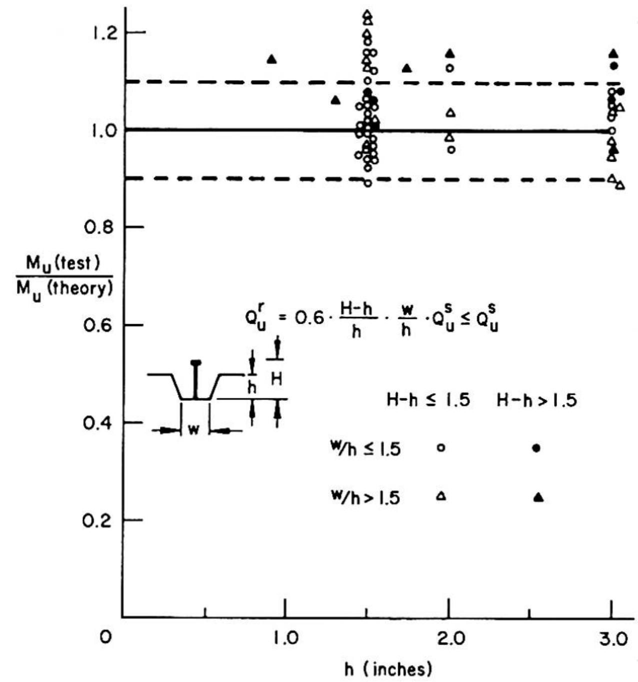

Fig. 4

where: Q_{rib} = strength of stud shear connector in a rib
w = average rib width
h = height of rib
H = heigh of stud shear connector
Q_{sol} = strength of a stud shear connector in a solid slab

Several recent tests on beams having greater connection embedment length were made at the University of Texas(7). These tests have further confirmed the applicability of Eq. 2.

Figure 4 shows all 56 beam test results in terms of test moment nondimensionalized by theoretical moment as a function of rib height. Equation 2 was used in predicting beam capacity. Figure 5 shows the same moment ratio as a function of the degree of partial shear connection, V'h/Vh, but for the 17 Lehigh tests only. The plots indicate that the connector capacity defined by Eq. 2 provides a better estimate of flexural capacity for beams with 3 in. (76.2 mm) deck. About the same flexural capacity is provided for beams with 1-1/2 and 2 in. (38.1 and 50.8 mm) deck. Equation 2 continues to account for the varying width - height ratios as indicated by the relatively even dispersion of the test beams for all rib heights. Further details of this study are given in Ref. 3.

4.3 Stiffness - The load-deflection plots shown in Fig. 2 show that beams with partial shear connection are less stiff than assumed for full composite action. This is due to the loss of interaction accompanying partial shear connection. For the Lehigh test beams with the least amount of shear connection, the stiffness was found to be between 70 and 80% of that calculated for full composite action at the working load level.

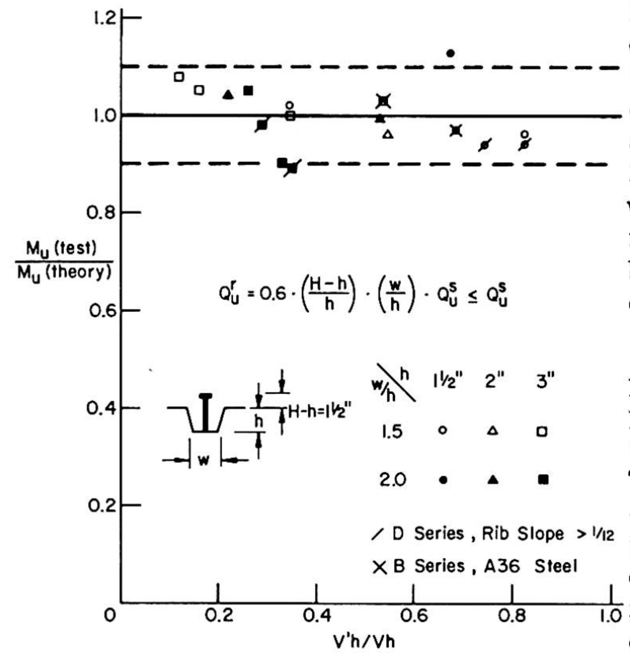

Fig. 5

Early studies at the University of Illinois (8) and more recent studies at the University of Missouri (9) have shown that composite beams with flat soffit slabs designed for full composite action have 85 to 90% of their calculated stiffness at the working load level. This loss in stiffness can be attributed to the fact that the shear connectors are flexible. Thus the connectors permit some slip or loss of interaction between the slab and the steel beam of a composite member, even though they will take all the force required for full composite action.

The shear connectors in a composite beam with formed metal deck behave similarly. Thus one would expect the same sort of difference to exist between actual and assumed stiffness of such beams designed for full composite action. The Lehigh test beams with the lowest degree of partial shear connection exhibited 20 to 30% loss of the stiffness which is about twice as much as experienced for full composite action in flat soffit slabs(8,9). On the other hand, these same beams provided at least twice the stiffness of a non-composite system. Thus a low degree of partial shear connection is very efficient in terms of stiffness.

Because of the complexity of the nonlinear variation of stiffness with the degree of partial shear connection, several emperical relationships have been examined(3). A relationship of the form:

$$I_{eff} + I_s + \frac{V'h}{Vh}^{\alpha} (I_{tr} - I_s) \tag{3}$$

where I_{eff} = effective moment of inertia
I = moment of inertia of the steel section
I_{tr} = moment of inertia of the transformed composite section
α = numerical exponent

was found to provide a reasonable fit to test data when α was taken equal to 1/2 or 1/3 as is demonstrated in Fig. 6. The stiffness provided by the 17 test beams is plotted for comparative purposes. With no shear connection, the stiffness is essentially that of the steel beam alone. A composite beam with full composite action (as provided by 100% shear connection will be assumed to have the stiffness of a transformed section with no loss of interaction between slab and beam. The straight line running from 0 to 1 in Fig. 6 would represent a linear variation of stiffness with α equal to 1. The solid vertical line at V'h/Vh equal to 1.0 shows the possible 15% variation between actual and assumed stiffness for a fully composite member. The plot clearly shows that the variation provided by the exponent α equal to 1/2 is generally conservative yet representative. The maximum deviation occurs as the degree of shear connection approaches unity. In no case is the loss of interaction greater than expected for a full shear connection.

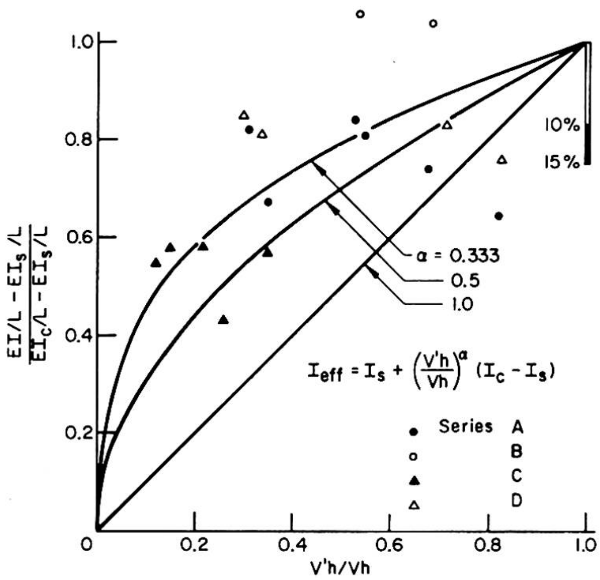

Fig. 6

Studies at the University of Missouri showed comparable behavior for composite beams with flat soffit slabs(9). A comparison of this data indicated general agreement with Eq. 3 when α was taken as 1/2.

5. CONCLUSIONS

The following conclusions may be drawn from the analysis reported herein:

1. The capacity of one or two stud shear connectors in the ribs of composite beams with formed steel deck may be determined from the following emperical expression:

$$Q_{rib} = 0.6 \cdot \frac{H - h}{h} \cdot \frac{w}{h} \cdot Q_{sol} \leq Q_{sol}$$

where H is the height of a stud shear connector in the rib, h is the height of the rib, w is the average rib width and Q_{sol} is the strength of the stud shear connector in a flat soffit slab.

2. The flexural capacity of a composite beam with formed steel deck can be more accurately and conservatively estimated if the slab force is considered to act at the mid-depth of the solid portion of the slab above the ribs, rather than at the centroid of the concrete stress block.

3. The flexural capacity of a composite beam utilizing high strength steel is not adversely affected by the increased slab force and can be predicted provided that the connector capacity is known.

4. The deflection of a composite beam with partial shear connection, with or without formed steel deck, may be estimated with the following expression for an effective moment of inertia:

$$I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}\,(I_{tr} - I_s) \qquad (4)$$

where I_s is the moment of inertia of the steel beam, I_{tr} is the moment of inertia of the transformed composite section, V'h is the total horizontal shear to be resisted by connectors providing partial composite action and Vh is the total horizontal shear to be resisted by connectors under full composite action.

6. ACKNOWLEDGMENTS

The testing described in this report was conducted at Fritz Engineering Laboratory, Lehigh University. The contribution of the Fritz Laboratory staff is gratefully acknowledged.

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SUMMARY

This report presents the results of 17 composite beam tests conducted at Lehigh University incorporating formed steel deck. These results were analyzed in conjunction with 39 additional tests conducted by previous investigators. The purpose of this report was to evaluate shear connector capacity and beam flexural capacity and behaviour, particularly for beams utilizing high strength steel.

RESUME

Ce rapport contient les résultats de 17 essais réalisés à l'Université Lehigh sur des poutres mixtes avec platelage métallique incorporé. Ces résultats ont été analysés conjointement avec 39 autres essais réalisés auparavant par d'autres chercheurs. Le but de ce rapport était d'évaluer la résistance au cisaillement des boulons et la résistance à flexion des poutres, particulièrement pour les poutres en acier à haute résistance.

ZUSAMMENFASSUNG

Dieser Bericht enthält die Ergebnisse von 17 Tests an Verbundträgern, die an der Lehigh Universität durchgeführt wurden, an denen ein Stahlblech eingearbeitet war. Die Resultate wurden anhand 39 zusätzlicher Tests analysiert, die vorher von anderen Forschern ausgeführt wurden. Zweck dieses Berichtes war, die Tragfähigkeit der Verbundmittel und des Trägers selber, sowie das Verhalten, besonders für Tragbalken aus hochfesten Stählen, zu berechnen.