PRESTRESSING THE NEGATIVE MOMENT REGION
OF COMPOSITE BEAMS

by
F. W. Sarnes
J. Hartley Daniels

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Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

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ABSTRACT

Two simply supported steel-concrete beams with pre-stressed slabs designated PSC-1S and PSC-2S were tested under negative moment (slab in tension). These beams were designed to simulate the prestressed negative moment region of a two-span continuous composite beam. Each of the simple span beams had a span length of 15'-0". The cross section of each beam consisted of a 6-in. x 60-in. reinforced concrete slab connected to a W21x62 rolled steel beam by 4-in. high by 3/4 in. diameter head steel studs. The stud connectors were attached to the steel beams in two patterns; concentrated groups and equally spaced.

A prestress force was applied to the slab of each beam by means of four 7/8-in. diameter STRESSTEEL bars placed in four thin steel tubes which were cast into the slab. In one beam the prestress force was applied prior to making the shear connection while in the other the prestress force was applied after the shear connection was made.

The purpose of this investigation was two fold:

(1) To study the effect of prestressing on the behavior of composite beams under negative moment.

(2) To compare the effectiveness of concentrated groups to stud connectors with equally spaced connectors.
The results of these tests are reported herein and the following conclusions were made:

(1) Composite action of the full transformed section can be obtained in the negative moment region by placing shear connectors throughout this region and applying a suitable prestress force to the slab.

(2) Prestressing can be used to effectively control slab cracking in the negative moment region. Cracks can be eliminated up to certain load levels depending upon the amount of prestress force which is applied.

(3) When the same magnitude of prestressing force is used, slab cracking is more effectively controlled when the slab is prestressed before the shear connection is made.

(4) Concentrated groups of shear connectors provide an adequate shear connection when the number of connectors used is the same as the number that would be used if the design called for the connectors to be equally spaced.

(5) Satisfactory composite behavior can be obtained when a low shrinkage grout is used to make the concrete slab composite with the steel beam.
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1. INTRODUCTION

Two developments which have recently appeared in the design of bridge superstructures are the frequent use of continuity and the increasing use of composite action between the concrete deck and the steel beams. Both of these developments have resulted from the necessity for a more efficient and economical use of materials.

Until now, attempts to combine both continuity and composite construction have been frustrated by the lack of suitable methods of analysis and specific design recommendations and specifications. Previous designs could lead to premature fatigue failure of shear connectors near the dead load points of inflection and undesirable cracking of the deck in the negative moment (slab in tension) regions of continuous composite bridge beams.

Recent research at Fritz Engineering Laboratory has concentrated on providing suitable analytical and design recommendations which would result in satisfactory designs based on static and fatigue criteria and minimize slab cracking.\(^1,2,3,4,5\) Although full-scale tests indicate that these recommendations will result in improved structural behavior of the superstructure, several problems still remain. Some transverse cracking of the concrete deck in the negative moment regions will always occur whether or not connectors are placed continuously throughout
those regions. Fatigue criteria will likely reduce the allowable stress in the tension flange of the steel beam if connectors are placed continuously throughout the negative moment regions. In addition, the full slab in the negative moment regions is not effective in resisting live loads as it is in the positive moment regions.

At an early stage in the research it was realized that all three of these problems might be overcome by longitudinal prestressing of the negative moment regions. Transverse cracking of the slab would be eliminated over a certain range of live loads depending upon the level of prestress. With prestressing, the full slab would become effective in resisting live loads. This has two beneficial effects. First, real continuity is achieved when connectors are placed continuously throughout the beam, resulting in more efficient use of materials. Secondly, the position of the neutral axis in the negative moment regions can be controlled by the designer thus making it possible to eliminate the reduction of the allowable stress in the tension flange of the steel beam due to fatigue considerations.

With prestressing of the negative moment regions, however, additional questions are encountered which require further consideration before this procedure can be recommended for design. For instance, there are several ways to prestress the concrete slab depending upon the method and stage of construction. One construction method might be to cast the
slab compositely with the steel beams in the usual way and to prestress after the slab has reached its desired strength. A second construction method might be to cast the slab non-compositely with the steel beams, prestress the slab, and then make the slab composite with the steel beams.

There are several ways the shear connection could be made after the slab is prestressed. For instance, holes could be left in the slab through which connectors could be welded to the steel beam and then grouted into the slab. The holes could either be made small enough to receive individual connectors or large enough to receive a concentrated group of connectors. The connector groups could then be spaced a certain distance apart.

Still a third construction method may be to use transverse precast slab elements in the negative moment regions which are prestressed in the longitudinal bridge direction either before or after these elements are made composite with the steel beams.

With this method, it is likely that the connectors would be grouped or concentrated into spaced out connector groups. Full width precast elements could be placed transversely across the beams between the connector groups. Shorter elements spanning across only two beams could be used adjacent to connector groups. The precast elements would be prestressed in the longitudinal direction of the bridge either before or after grouting of the connector groups.
The particular construction method used would probably depend on such factors as relative economies, structural advantages and ease of construction.

A pilot investigation was undertaken at Fritz Engineering Laboratory to determine the feasibility of prestressing the slab in the negative moment region of continuous composite bridge beams and to evaluate two methods of providing the required shear connection; i.e., equally spaced connector pairs and concentrated connector groups spaced apart. The investigation was primarily designed to provide analytical and experimental evaluation of the first two methods of construction mentioned above which do not involve precast slab elements. However, the experiments were designed to provide useful information applicable to prestressed-precast slab elements.

Two composite test beams designated PSC-1S and PSC-2S were designed and constructed to simulate the negative moment region of a prismatic two-span continuous composite Tee-beam having the same span lengths and load positions as the beams previously described in Refs. 1, 2, 4 and 6. Except for the length of each test beam and the slab prestressing and reinforcing details, each beam was constructed identical to the beams previously described in Refs. 7 and 8. They were also tested in the same manner (slab in tension) so that comparative studies could be made. The shear connection for each test beam was identical and consisted of equally spaced pairs of stud
connectors to one side of the central load position and an equal number of studs placed in three concentrated stud groups on the other side of the load position.

Preliminary studies using computer programs previously developed and reported in Ref. 4 were used to predict the behavior of the beam with the different stud arrangements on either side of the central load position. The design of the studs was partly based on an attempt to ensure zero slip at the load position so that each shear span would behave independently. This same condition would be expected for test beams with symmetrically placed connectors.

The major difference between the two test beams was in the procedure for making the prestressed slab composite with the steel beam. For beam PSC-1S, the concrete slab was cast and made composite with the steel beam in the usual way. After the concrete had reached the desired strength the slab was prestressed. Therefore, the transformed section resisted the prestress force and the applied test loads. For beam PSC-2S the connectors were isolated during casting and prestressing of the slab. The connectors were then grouted so that the transformed section would resist only the applied test loads.

The design and construction of the two test beams therefore enabled an evaluation of the test results to be made within one of four categories as follows:
1. Prestressing after obtaining composite action; equally spaced shear connectors,
2. Prestressing before obtaining composite action; equally spaced shear connectors,
3. Prestressing after obtaining composite action; grouped shear connectors, and
4. Prestressing before obtaining composite action; grouped shear connectors.

The results obtained with grouped shear connectors are particularly useful in evaluating construction methods employing prestressed-precast slab elements since some grouping of shear connectors is envisioned with such a scheme.

This report documents the results of tests of beams PSC-1S and PSC-2S and presents a discussion of some of the more significant results.
2. TEST BEAMS, INSTRUMENTATION AND TESTING PROCEDURE

2.1 Description of Test Beams

Figure 1 describes the two test beams, PSC-1S and PSC-2S. Each test beam was constructed as shown in Fig. 1c to simulate the negative moment region of a two-span prismatic continuous composite beam shown in Fig. 1a. The bending moment diagram for the two-span beam corresponding to the slab prestress force $F$ and the applied loads $P$ is shown in Fig. 1b. The maximum positive and negative bending moments are shown as $M_1$ and $M_2$ respectively. For the two test beams shown in Fig. 1c the slab prestress force is $F$ and the test loads are shown as $V$ and are applied at the theoretical dead load inflection points of the two-span beam. A typical cross-section of the test beams is shown in Fig. 1d (also the same for the two-span beam).

Each test beam consisted of a 60-in. wide by 6-in. thick reinforced concrete slab connected to a W2lx62 (A36) rolled steel beam by 3/4 in. x 4 in. headed steel stud shear connectors. The location of the stud connectors is shown in Fig. 2 and described in detail later in this chapter (Art. 2.3). A prestress force was applied to the concrete slab of each beam by tensioning four 7/8 in. diameter regular grade STRESSTEEL bars centered in 1-3/8 in. diameter thin steel tubes as shown in Fig. 1d. The transverse slab reinforcement consisted of No. 5 reinforcing bars at 6-in.
spacing top and bottom as described in previous reports.\(^{(1,6,7)}\)

Each beam had an overall length of 17'-0". This length included the 15'-0" negative moment region between dead load inflection points of the two span continuous beam and a 12-in. projection on each end as shown in Fig. 1c.

### 2.2 Design Criteria

The design of the two test beams PSC-1S and PSC-2S was based on the design of the two-span continuous composite beam shown in Fig. 1a. Both the static and the fatigue design criteria were considered.\(^{(9)}\) The resulting design, except for the slab prestress and the shear connection continuous through the negative moment region, was essentially the same as for those beams reported earlier.\(^{(1,6,7)}\)

Previously, with no slab prestress in the negative moment region of the two-span composite beam, the design of the shear connectors was controlled by fatigue requirements based on a working load \(P\) of 60 kips and a life of 2,000,000 cycles.\(^{(1)}\) The working load level was determined by the allowable stress in the steel beam.\(^{(1)}\)

On the assumption that slab prestress would be used primarily to eliminate transverse slab cracking under at least the working live loads, the prestress force \(F\) (Fig. 1a) was determined on the basis that the previously obtained working
load level $P$ of 60 kips would remain constant and that at the working load level, the total stress in the slab over the interior support would be equal to zero. The resulting design was checked to ensure that the stress in the steel beam and the remaining concrete slab under $P$ and $F$ was equal to or less than the allowable stress. Assuming that for the steel beam $f_y = 36$ ksi and for the concrete slab $f_c' = 5$ ksi, the prestress force $F$ was found to be 250 kips. It was also assumed that this prestress force would be applied to the continuous composite beam at the dead load inflection points as shown in Fig. 1a.

Therefore, for test beam PSC-1S, which was made composite before prestressing, the prestress force of 250 kips was applied, as shown in Fig. 1c, prior to testing. For test beam PSC-2S which was made composite after prestressing it was decided to maintain the same value of prestress force in the slab; i.e.: $F = 250$ kips.

### 2.3 Design Details and Fabrication

Fabrication details for test beams PSC-1S and PSC-2S are shown in Fig. 2. The rolled steel beams were cut from a 43-ft. length by a local fabricator. The bearing stiffeners and headed stud shear connectors shown in Fig. 2a and 2b were also welded to the beams by the fabricator. The excess length of the rolled shape was used for control test purposes and to test the stud welds. The welding procedure is described in Ref. 7 and the inspection procedure is outlined in Ref. 10.
Figure 2b shows the distribution of stud shear connectors for the two test beams. On one half of each beam the connectors were equally spaced in pairs along the top flange of the steel section. On the other half the connectors were arranged in concentrated groups. Each half of the beam contained the same number of stud connectors since the shear transfer requirements were the same for each half. This arrangement made it possible to study the effects of the two connector patterns and observe any differences. A typical cross section of the fabricated steel beam with connectors, stiffeners and loading pins is shown in Fig. 2c. The details for the stiffeners and loading pins were the same as in previous tests. (7)

2.4 Construction

Figure 3 shows the slab formwork with the transverse reinforcement and steel tubes for the STRESSTEEL bars in place for beam PSC-2S. Beam PSC-1S was identical with the exception that there was no formwork around the connectors. The connectors in beam PSC-2S were formed out so that the beam would remain non-composite while the slab was cast and prestressed for the reasons previously mentioned (see end of Chapter 1). As shown in Fig. 3 extra top and bottom transverse reinforcement was placed at each end of the test beams to provide additional anchorage for the prestressing.

Two concrete mixes were used in the construction of
these beams. The first mix was proportioned and transit mixed for a 28 day compressive strength of 3000 psi with a slump of 3-1/2 in. This concrete was used in lieu of the 5000 psi concrete used in design so that a better comparison could be made with the results of previous tests for which 3000 psi concrete was also used for the slab of the test beams.\(^{(6,7)}\) A second mix used as a grout for beam PSC-2S was proportioned for a 28 day compressive strength of 6000 psi with a slump of 1 in. and was mixed in a mechanical mixer in the laboratory.

Test beam PSC-1S was poured in one operation using the transit mixed 3000 psi concrete. Beam PSC-2S was also poured using the same concrete mix with the exception of the areas surrounding the stud connectors which were boxed out as shown in Fig. 3. After the slab of beam PSC-2S was prestressed, concrete made from the 6000 psi mix was placed in the voids around the connectors. This high strength, low slump mix was used to obtain a minimal amount of shrinkage.

Consolidation of the concrete on all of the pours was accomplished by internal vibration and the final finish was obtained by hand trowelling. The various pours took place on different days and control cylinders were made for each pour. Twelve cylinders were prepared from each batch of the transit mix concrete while eight cylinders were made from the high strength mix. In all cases half of the cylinders were made at the beginning of the pour and half at the end.
The concrete in the slabs was moist cured for seven days by covering the exposed surface with wet burlap and a polyethylene sheet. The forms were stripped at the end of the moist curing period and the slabs were allowed to cure under dry conditions until the beams were tested. Testing of each beam occurred approximately two months after the initial pouring of the slab. A number of control cylinders from each pour were cured in the same manner as the slab while the remainder were cured as prescribed by ASTM (11) (see Table 2).

2.5 Properties of the Test Beams

A test program was conducted to determine the mechanical properties of the materials used in the test beams. Properties of the structural steel were determined using tensile coupons cut from the excess lengths of the rolled beam (Art. 2.3). These coupons were tested according to the procedures outlined in Ref. 12. The mechanical properties of the rolled steel beam are shown in Table 1.

The concrete used for the slabs of the test beams was made of type 1 Portland cement, gravel crushed to a maximum size of 3/4 in. as coarse aggregate and natural bank sand as fine aggregate. Control cylinders were made for each batch of concrete (Art. 2.4). The results of the compression tests on these cylinders are shown in Table 2.

The physical dimensions of the test beams were obtained
and the cross-sectional properties determined. The calculated properties are shown in Tables 3 and 4. In Table 3 the calculated properties for the rolled shapes are compared with the standard properties tabulated in the 1969 AISC Manual of Steel Construction.

2.6 **Instrumentation**

The instrumentation for test beams PSC-1S and PSC-2S was identical except that beam PSC-1S had strain gages mounted on the concrete slab while beam PSC-2S did not. Instrumentation details are shown in Figs. 4 and 5. A combination of electrical resistance strain gages, calibrated electrical slip gages and mechanical dial gages was used.

Figure 4 shows a detail view of one of the calibrated electrical slip gages. A 2-in. x 2-in. angle bolted to the underside of the concrete slab provides support for a small screw which bears against a metal strip which is secured to the top flange of the steel beam. Slip between the slab and the steel beam causes bending of the metal strip. Electrical resistance strain gages mounted on the metal strip convert the bending strain in the extreme fibres of the strip to deflection at the screw tip by means of calibration curves.

A mechanical dial gage used to measure slip at the ends of the beam, and to provide a check on the nearest electrical slip gage, is also shown in Fig. 4.
Figure 5 shows the locations of the nine instrumented cross sections used for both test beams. For both beams electrical slip gages were located at all nine locations. Ten strain gages were also mounted around the steel beam cross-section at each of the locations except 1, 6 and 9. The position of each of the ten gages on the cross-section of the beams is the same as in previous tests. They are shown in Fig. 4 of Ref. 7.

Test beam PSC-1S also had additional strain gages mounted on the concrete slab at locations 2, 3, 4, 5, 7, and 8. Locations 5 and 8 had a total of eight strain gages mounted on the concrete. At these locations, strain gages were mounted on the surface of the slab directly above and below each of the STRESSTEEL bars. At the remaining locations gages were mounted above and below only the two interior STRESSTEEL bars.

One of the interior STRESSTEEL bars was strain gaged for each beam in order that the prestressing force could be monitored throughout the test. The vertical deflection at each end of the test beams was measured by means of Ames 0.001 in. mechanical dial gages.

The recording equipment consisted of a B and F 100 channel automatic strain recorder and a Budd Datran unit to read the electrical resistance strain gages. A fifty power microscope was also used to measure the width of the cracks which developed in the concrete slab near the end of the test.
2.7 Prestressing, Test Procedure and Loading

The four $\frac{7}{8}$-in. diameter STRESSTEEL bars were prestressed to the desired level by means of the jacking system shown in Fig. 6. The system consisted of a fifty ton Simplex hydraulic jack with a jacking chair and a calibrated pull bar with a coupler, pulling head and wedge. The pull bar was instrumented and calibrated to measure the applied tensioning force.

The prestressing operation was carried out in the following manner:

The pull bar was connected to the STRESSTEEL bar and the 50 ton jack with chair was placed over the pull bar and against the end plate. The pulling head and wedge were set in place and the jacking was started. When the load reached the required level in the pull bar, the nut on the STRESSTEEL bar was tightened until it was snug and bearing on the end plate. The jack was then released and the jack force was completely transferred to the prestressing bar. The prestress force was applied in several stages to each bar in turn rather than all at once to any one bar to minimize the prestress losses due to the jacking sequence.

The loading procedure for the test beams was similar to that used in previous tests. The center of the test beam was supported on a roller (this simulates the center reaction of the two-span beam) while vertical forces were applied to the
web of the rolled steel beam as shown in Fig. 1c (this simulates the shear forces at the inflection points of the two-span beam). The test set-up is essentially the same as that used in previous test and is shown in Figs. 9 and 10 of Ref. 7.

Each test was performed approximately two months after the slab was cast. Approximately 15 hours were required for testing which was carried out on 3 consecutive days. Load increments of ten kips were applied throughout each test except after the yielding of the steel beam was apparent. At this point small increments of deflection were used until the end of the test. The tests were carried out until the unloading behavior of each beam was obtained.
3. TEST RESULTS

3.1 Load-Deflection Behavior

Figure 7 shows the load-deflection behavior of test beams PSC-1S and PSC-2S. The curves show the relationship between the center reaction \( R \) and the deflection \( \Delta \) of the center support relative to a line joining the load points as shown in the figure.

The deflection \( \Delta \) of the test beam at the center support was computed as shown in the figure from the measured deflections \( \Delta_1 \) and \( \Delta_2 \) at the ends of the beam. This deflection is also assumed to represent the relative deflection of the inflection points and the center support of the two-span beam as shown in Fig. 7.

The predicted behavior of the test beams is shown by the dashed line. Since the properties of the two test beams were quite close (see Table 4), average values were used to determine the prediction curve for both test beams.

For this prediction curve the full concrete slab was assumed to act compositely with the steel section until the first crack appeared. After initial cracking, it was assumed that only the prestressing steel would contribute to the composite behavior of the beam. For this reason, bilinear behavior was predicted for the elastic range and the transition occurs at the average cracking load of the two test beams.
Equations from the 1969 AISC Manual of Steel Construction were used to determine the deflections of the two span beam shown in Fig. 7 for the elastic portion of the theoretical curve. The predicted ultimate load was based on the plastic moment capacity of the composite steel section.

It can be observed from Fig. 7 that both test beams exhibited approximately the same behavior. This was expected since the section properties of both beams were almost the same. The slightly greater stiffness of beam PSC-1S can be seen at the higher load levels. The load-deflection curves for the two test beams are in good agreement with predictions and indicate that the full transformed section acts compositely up to approximately the cracking load when the slab is prestressed.

First yielding occurred in beam PSC-1S at a slightly higher load level than in beam PSC-2S. This behavior was expected and is explained as follows. Beam PSC-1S was prestressed after the shear connection was made and, therefore, the entire transformed section was subjected to the prestressing force. This caused the extreme bottom fibers of the steel section to be in tension when testing began and thus these fibers reached their compressive yield stress at a higher load level than the extreme bottom fibers in beam PSC-2S which were not initially in tension. First cracking also occurred at different load levels in the two test beams and this will be discussed in detail in Art. 3.3.
In Fig. 7 it can be seen that both test beams exceeded the theoretical maximum load by about five percent which is probably due to strain-hardening. Due to local buckling, unloading took place immediately after the maximum load was reached; however, both beams were able to carry a large percentage of the maximum load for deformations considerably in excess of those at which local buckling occurred. The buckling was more severe in beam PSC-2S and probably explains why the unloading rate for beam PSC-2S was greater than that for PSC-1S. The local buckling of the two test beams will be discussed in Art. 3.4.

3.2 Slip

Figures 8 and 9 show the slip distribution for the two test beams. The slip measurements were made using electrical slip gages as described in Art. 2.6. The least accurate of the slip gages was capable of recording a change in slip of 32 micro inches. In both figures, slip to the east is shown as positive.

Figure 8 shows the slip distribution obtained in both beams, due to the applied test loads only (effect of prestress not included). The slip curves shown in the figure correspond to center reactions of 60 and 150 kips so that comparisons with previous test results are possible.\(^{(6,7)}\)

The magnitude of the slip in beam PSC-1S was generally greater than that in beam PSC-2S. This can probably be explained
by the fact that in beam PSC-2S the concrete surrounding the connectors had a much higher compressive strength than the concrete surrounding the connectors in beam PSC-1S (see Table 2).

Unlike previous tests, no attempt was made to eliminate the bond between the steel beam and the concrete slab for either test beam prior to testing. The effect of this bond may be observed in Figs. 8 and 9 in the areas between locations 2 and 3 and between locations 4 and 5. Since there were no shear connectors in these two areas, the change in slip was probably due to this bond between the steel and the concrete.

The slip distribution in both beams followed similar patterns with the exception of the east half of beam PSC-1S at the 150 kip load level. However, at the prior load level of 140 kips the slip distribution in the last half of beam PSC-1S was similar to that in the east half of beam PSC-2S. While the test load was increased from 140 kips to 150 kips a significant failure in the bond between the slab and the steel beam was observed and may account for the above noted variation in slip distribution.

It should also be noted in Figures 8a and 8b that although the magnitude of the slip in the west half of each beam was slightly greater than that at corresponding sections of the east half, the magnitudes were approximately the same. This seems to indicate that the grouping of connectors in the
west half of each beam did not greatly affect the slip behavior of that half of the test beam as compared to the half with equally spaced connectors.

Figure 9 shows the slip distribution in beam PSC-1S including the slip prior to testing caused by the prestress force and the losses which occurred during the time which elapsed between the prestressing operation and the application of the test loads. This figure indicates that initially the prestress force was carried by the connectors at the ends of the beam. With increasing time this force was redistributed across the length of the beam.

3.3 Cracking Behavior

Figures 10 and 11 show the crack patterns in the concrete slabs of test beams PSC-1S and PSC-2S. Figure 10 shows the crack patterns at a load level of 90 kips center reaction which is about 1-1/2 times working load level. Figure 11 shows the crack patterns at the maximum load level. In both figures, the rectangles shown within the slab of beam PSC-2S are the areas where the stud shear connectors were isolated from the main slab pour, and then grouted later using the high strength grout.

At the 90 kip load level, beam PSC-1S had one crack approx. 0.005 in. wide across the entire width of the slab directly over the center support while beam PSC-2S had 3 hairline cracks (approx. 0.01 wide) in the area where the grout
was placed. The reason that hairline cracks occurred in the grouted areas of beam PSC-2S is due to the fact that the grout was placed after the slab was prestressed and consequently these sections of the slab were not actually prestressed.

It was expected that the slab of beam PSC-1S would crack before that of beam PSC-2S since the slab of beam PSC-2S carried the entire prestress force while the full transformed section carried this force in beam PSC-1S. For this reason, the slab of PSC-2S had a greater compressive stress at the start of the applied test loading.

It can be seen from Fig. 11 that even at the maximum load level there were very few cracks in the slab of either test beam. Beam PSC-1S contained three cracks across the width of the slab while beam PSC-2S had the equivalent of two cracks. The crack near the center support of beam PSC-2S was the first to form as was expected.

The cracks near the center support of each beam attained the greatest width. The average width of these cracks at the maximum load level was about 0.1-in. Most of the other cracks were hairline cracks with average widths generally less than 0.01 in. Both beams had short longitudinal hairline cracks at the east and west ends due to the high anchorage stresses induced by the prestressing bars.

The slab cracking in both beams is quite symmetrical.
about the center support. This is significant since each half of the two test beams had a different distribution of stud shear connectors. The symmetrical crack patterns indicate that the beam behavior was symmetrical about the center support.

3.4 Failure Mode

Test beams PSC-1S and PSC-2S both failed in essentially the same manner as the non-prestressed beams which were previously tested.\(^{(6,7)}\) For both test beams, failure occurred after local buckling of the compression (lower) flange of the steel beam adjacent to the center support. Local buckling of the flange was accompanied by web buckling in both beams. Web buckling in PSC-2S was quite pronounced as can be seen in Fig. 12. In both tests local buckling occurred shortly after maximum load was reached as shown in Fig. 7.

Figure 13 shows beam PSC-2S shortly after failure. This was typical of both test specimens. Local buckling of the lower flange and web of this beam can be seen near the center support.
4. SUMMARY AND CONCLUSIONS

Two simply supported composite steel-concrete beams with prestressed slabs were tested under negative moment (slab in tension). These beams were designed to simulate the prestressed negative moment region of a two-span continuous composite beam. Each of the simple span beams reported herein had a span length of 15'-0". The cross sections of each beam were identical and consisted of a 6-in. x 60-in. reinforced concrete slab connected to a W21x62 rolled steel beam by means of 4-in. high by 3/4-in. diameter headed steel studs. The stud connectors were attached to the steel beams in two patterns; concentrated groups and equally spaced.

A prestress force was applied to the slab of each beam by means of four 7/8-in. diameter STRESSTEEL bars placed in four thin steel tubes which were cast into the slab. In one beam the prestress force was applied prior to making the shear connection and in the other the prestress force was applied after making the shear connection.

The beams were tested under static loading and the results of these tests are presented herein. The following conclusions are based on the test results:

(1) Composite action of the complete transformed section can be obtained in the negative moment
region of a continuous composite beam by placing shear connectors throughout the negative moment region and applying a suitable prestress force to the slab.

(2) When the slab in the negative moment region is prestressed, slab cracking can be eliminated up to loads which depend upon the level of prestress. After the prestress has been overcome, the slab will crack, but the crack widths prior to the ultimate load level can probably be reduced by increasing the prestress force.

(3) Prestressing of the slab before making the shear connection is desirable in delaying the formation of slab cracks since the initial compressive stress in the slab will be higher.

(4) The test results indicated that when same number of connectors is used, grouped connectors provide a satisfactory shear connection provided the concrete can properly be placed around the connectors. This concept may have a significant application in the use of precast slabs in composite construction.

(5) The test results also indicated satisfactory composite behavior using a low shrinkage grout around the connectors providing that adequate shear transfer is obtained between the grouted sections and the remainder of the slab.
5. ACKNOWLEDGEMENTS

The study described in this report was part of an investigation on composite beams that was conducted at Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. L. S. Beedle is Director of Fritz Engineering Laboratory and D. A. VanHorn is Chairman of the Department of Civil Engineering. The project was sponsored by the Pennsylvania Department of Transportation and the Federal Highway Administration.

The writers wish to thank: Drs. R. G. Slutter and J. W. Fisher for their advice and help in the planning and conduct of these tests; Messrs. K. R. Harpel and H. Sutherland and their staff at Fritz Laboratory for their work in preparing the test set-up and instrumentation; Mr. R. N. Sopko for the photographic coverage; Mr. J. Gera and Mrs. S. Balogh for preparing the drawings; and Mrs. Charlotte Yost for typing the manuscript.
6. TABLES AND FIGURES
## TABLE 1

MECHANICAL PROPERTIES OF STEEL

<table>
<thead>
<tr>
<th>Type of Specimen</th>
<th>No. of Test</th>
<th>Yield Point (ksi)</th>
<th>Static Yield Stress (ksi)</th>
<th>Tensile Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web W21x62</td>
<td>3</td>
<td>37.5</td>
<td>34.4</td>
<td>60.8</td>
</tr>
<tr>
<td>Flange W21x62</td>
<td>4</td>
<td>34.4</td>
<td>32.0</td>
<td>58.8</td>
</tr>
</tbody>
</table>

## TABLE 2

RESULTS OF CONCRETE CYLINDER TESTS

<table>
<thead>
<tr>
<th>Test Beam</th>
<th>No. of Tests</th>
<th>f'c (psi)</th>
<th>No. of Tests</th>
<th>E_c (ksi)</th>
<th>f'c (psi)</th>
<th>E_c (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC-1S</td>
<td>8</td>
<td>4760</td>
<td>2</td>
<td>3665</td>
<td>4810</td>
<td>3235</td>
</tr>
<tr>
<td>PSC-2S (Slab)</td>
<td>10</td>
<td>4330</td>
<td>4</td>
<td>3380</td>
<td>4370</td>
<td></td>
</tr>
<tr>
<td>PSC-2S (Grout)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7070</td>
</tr>
</tbody>
</table>
### Table 3

**Rolled Steel Beam Properties**

<table>
<thead>
<tr>
<th>BEAM</th>
<th>AREA (in²)</th>
<th>DEPTH (in)</th>
<th>FLANGE WIDTH (in)</th>
<th>FLANGE THICKNESS (in)</th>
<th>WEB THICKNESS (in)</th>
<th>MOMENT OF INERTIA (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC-1S</td>
<td>17.431</td>
<td>21.063</td>
<td>8.230</td>
<td>0.570</td>
<td>0.404</td>
<td>1251.465</td>
</tr>
<tr>
<td>PSC-2S</td>
<td>17.602</td>
<td>21.063</td>
<td>8.293</td>
<td>0.575</td>
<td>0.405</td>
<td>1267.489</td>
</tr>
<tr>
<td>W21x62</td>
<td>18.300</td>
<td>20.990</td>
<td>8.240</td>
<td>0.615</td>
<td>0.400</td>
<td>1330.000</td>
</tr>
</tbody>
</table>

*From 1969 AISC Manual of Steel Construction*

### Table 4

**Properties of the Test Beams**

<table>
<thead>
<tr>
<th>BEAM</th>
<th>CROSS-SECTIONAL SLAB AREA (in²)</th>
<th>LOCATION OF N.A. Yb (in)</th>
<th>MOMENT OF INERTIA TRANSFORMED SECTION (in⁴)</th>
<th>MOMENT OF INERTIA STEEL SECTION* (in⁴)</th>
<th>COMPUTED ULTIMATE MOMENT (kip-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC-1S</td>
<td>366.54</td>
<td>20.303</td>
<td>3702</td>
<td>1638</td>
<td>7072</td>
</tr>
<tr>
<td>PSC-2S</td>
<td>365.83</td>
<td>20.042</td>
<td>3663</td>
<td>1654</td>
<td>7127</td>
</tr>
</tbody>
</table>

*Including STRESSTEEL bars.
FIG. 1 TEST BEAMS PSC-1S AND PSC-2S
West Grouped Studs | Equally Spaced Studs
---|---
2 P 4"x1" | 2" φ Pin
2 P 4"x½" | W21 x 62
12" | 7'-6" |
7'-6" | 12"

(a)

6⅜" 6 @3" 1'-10⅜" 7 @3" 1'-10⅜" 4@3"

(b)

Equal Spaced Stud Distribution

(c)

FIG. 2 FABRICATION DETAILS
FIG. 3  FORMWORK FOR BEAM PSC-2S

FIG. 4  ELECTRICAL SLIP GAGE
FIG. 5 LOCATION OF NINE INSTRUMENTED CROSS SECTIONS FOR BEAMS PSC-1S AND PSC-2S

FIG. 6 PRESTRESSING SET-UP
FIG. 7 LOAD-DEFLECTION BEHAVIOR OF BEAMS PSC-1S AND PSC-2S
FIG. 8 SLIP DISTRIBUTION DUE TO APPLIED LOADS
FIG. 9 TOTAL SLIP IN PSC-1S
FIG. 10 CRACKING AT 90 KIPS

FIG. 11 SLAB CRACKING AT THE MAXIMUM LOAD
FIG. 12 LOCAL BUCKLING FAILURE MODE

FIG. 13 TEST BEAM AFTER FAILURE
7. REFERENCES

1. Daniels, J. H., and Fisher, J. W.
   FATIGUE BEHAVIOR OF CONTINUOUS COMPOSITE BEAMS,
   Fritz Engineering Laboratory Report No. 324.1,
   Lehigh University, December 1966

2. Daniels, J. H., and Fisher, J. W.
   STATIC BEHAVIOR OF CONTINUOUS COMPOSITE BEAMS, Fritz
   Engineering Laboratory Report No. 324.2, Lehigh
   University, March 1967

3. Garcia, I., and Daniels, J. H.
   NEGATIVE MOMENT BEHAVIOR OF COMPOSITE BEAMS, Fritz
   Engineering Laboratory Report No. 359.4, Lehigh
   University, April 1971

   ANALYSIS OF CONTINUOUS COMPOSITE BEAMS, Fritz
   Engineering Laboratory Report No. 359.5, Lehigh
   University, May 1971

5. Fisher, J. W., Daniels, J. H., and Slutter, R. G.
   CONTINUOUS COMPOSITE BEAMS FOR BRIDGES, Fritz
   Engineering Laboratory Report No. 359.6, Lehigh
   University, June 1971

   CONTINUOUS COMPOSITE BEAMS UNDER FATIGUE LOADING,
   Fritz Engineering Laboratory Report No. 359.2,
   Lehigh University, September 1970

7. Garcia, I., and Daniels, J. H.
   TESTS OF COMPOSITE BEAMS UNDER NEGATIVE MOMENT,
   Fritz Engineering Laboratory Report No. 359.1,
   Lehigh University, February 1971

8. Garcia I., and Daniels, J. H.
   VARIABLES AFFECTING THE NEGATIVE MOMENT BEHAVIOR OF
   COMPOSITE BEAMS, Fritz Engineering Laboratory Report
   No. 359.3, Lehigh University, June 1971

   FATIGUE STRENGTH OF SHEAR CONNECTORS, Highway
   Research Record No. 147, Highway Research Board,
   1966, pp. 65-88
10. American Association of State Highway Officials
   STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, Tenth

11. American Society for Testing and Materials
   STANDARDS, Part 10: Concrete and Mineral Aggregates,
   1969

12. Desai, S.
    TENSION TESTING PROCEDURE, Fritz Engineering
    Laboratory Report No. 237.44, Lehigh University,
    February 1969