

An aerial photograph of a bridge deck, showing a grid of reinforcement lines. The lines are arranged in a rectangular pattern, with some lines being thicker than others. The background is a light gray, and the lines are dark gray. The overall image has a grainy, high-contrast appearance.

## Test Report

# **Splicing Grid Reinforced Concrete Bridge Deck Panels Without Welding Using Conventional Rebar Methods**

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This test was conducted  
under the technical supervision of

**Ahmad K. Ahmadi, P.E., Ph.D.**

and funded by the



**Bridge Grid Flooring Manufacturers Association**

Tests were conducted at Harmarville, PA  
by Clark Mangelsdorf, P.E., Ph.D.

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NOTE: The information contained herein has been prepared in accordance with generally accepted engineering principles. However, the Bridge Grid Flooring Manufacturers Association is not responsible for any errors that may be contained herein. The user of the information provided herein should check the information supplied and make an independent determination as to its applicability to any particular project or application.



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## Abstract

Two 5- $\frac{3}{16}$ " deep steel grid panels, one 10' long (in the direction of the main grid I-beam) x 3'-8" wide, and one 10' long x 3' wide, were spliced together by using 18" long #4 rebars @ 8" c/c, resulting in a single panel 10' long x 6'-8" wide. The panel was then half-filled with concrete to the top of the steel grid section (flush-filled) and the splice constructed in the manner described was tested. The spliced panel was subjected to static and dynamic testing (over 1 million cycles). It was also tested for shear and ultimate loading. The completed panel contained 11 grid I-beams, and deflectometers were placed under the seven middle I-beams to measure displacement under the load. Results of the test showed that the splice performed satisfactorily, which was predicted, based on theoretical analysis. No cracks were observed after 1 million cycles. Finally the panel was loaded to failure (ultimate loading test). During the ultimate loading test, at a test load of 79 kips, a longitudinal crack, along the center line of the splice, developed. As a result of this crack, the differential displacement between opposite sides of the splice detail was 2" to 3" under the load, and decreased to 0" at the ends of the panels.

The load at failure exceeded the design capacity of the deck. Note that concrete-filled steel grid decks have been shown to demonstrate, and are analyzed, as orthotropic plates, distributing load in both directions. The test panel had a finite width of 6'-8". In actuality, panels on a bridge are much wider and the load is distributed over a much larger width. Test results shows that the grid I-beams farthest away from the load have a relatively large displacement, indicating that a wider panel would have distributed the load much farther.

## 1. Introduction

In bridge deck construction using a concrete filled steel grid, it is standard practice to put deck panels side by side and connect them in some fashion. Typically, transportation requirements restrict deck panels to approximately eight foot wide. Therefore, where grid I-beams are transverse to the direction of traffic, a transverse panel splice occurs about every eight feet. If grid I-beams are parallel with the direction of traffic, a panel splice occurs longitudinally. Common methods of splicing crossbars are to weld or bolt them together in the field once panels are positioned and leveled on the bridge. While each of these methods has performed quite satisfactorily, alternate methods are being explored in order to both speed the operation and reduce field costs.

Use of unwelded rebar for splicing components of concrete filled steel grid decks is not a completely new idea. This detail was used to splice the cross bars on the main lower roadway deck of the Manhattan Bridge, New York City (1938), and the main grid I-beams of sections of the Gold Star Memorial Bridge, Groton, CT (1973). More recently, cross bars of the half filled grid on the Fordís Island moveable bridge, Pearl Harbor, Hawaii were spliced using unwelded rebar. The purpose of this test is to design, develop, and test a rebar detail which provides full continuity between adjacent panels, capable of transferring loads from one panel to the other.

The splice detail was tested for strength and serviceability. For strength, the criteria was for the splice rebars to transfer the load across the splice. This was checked by applying the load on one side of the splice and measuring displacements of both sides of the splice. As long as the displacement on both sides of the splice is nearly symmetrical, this indicates that the splice rebars are transferring the load across the splice. Serviceability was checked by applying cyclic loads and looking for cracks in the deck along the splice line.





## 2. Theoretical Calculation

For design of continuity between adjacent panels, the theoretical “shear friction” concept of AASHTO Section 8.16.6.4 and section 5.15 of “Reinforced Concrete Design” by Chu-Kia Wang and Charles G. Salmon, Fourth Edition were utilized. Grid crossbars are secondary reinforcement and act mainly as distribution reinforcement, except when considered in the calculation of composite section properties of the supporting members, i.e. stringers or girders. Shear friction equation is:

$$V_n = A_{vf} f_y \mu$$

Where;

$V_n$  = Shear strength

$A_{vf}$  = Area of rebars

$f_y$  = Yield strength of rebars

$\mu$  = Coefficient of friction, and is  
= 1.4 $\lambda$  for concrete placed monolithically  
= 1.0 $\lambda$  for concrete poured against  
hardened concrete with intentionally  
roughened surface

$\lambda$  = 1.0 for normal weight concrete  
= 0.85 “sand lightweight” concrete  
= 0.75 “all lightweight” concrete

For this test, number 4 rebars (Area = .2 in<sup>2</sup>) with  $f_y$  of 60 ksi were considered to transfer the HS-20 wheel load (16 ksi) from the loaded side of the splice to the unloaded side. Assuming impact is 30%, concrete is normal weight and poured monolithically:

$$\text{Load} = 16 \times 1.3 = 20.8 \text{ kips}$$

Assume that half of the load should be transferred across the splice.

Therefore;

$$V_n = 20.8 / 2 = 10.4 \text{ kips}$$

computing area of rebars needed to transfer this shear:

$$10.4 = A_{vf} \times 60 \times 1.4 \times 1 \quad A_{vf} = 0.124 \text{ in}^2$$

According to AASHTO section 3.30 for HS-20 live load, tire print in the direction perpendicular to traffic is 20 inches (note that for actual distribution of live load section 3.24.3 should be used). Conservatively assume 20" live load distribution:

$$0.124 \times 12 / 20 = 0.075 \text{ in}^2/\text{ft}$$

For this test #4 rebars @ 8" (0.30 in<sup>2</sup>/ft) were provided. Therefore, rebar provided is more than required rebar for HS-20 loading. Ratio of rebar area provided to the rebar area required (0.30/0.075 = 4) indicates that splice should not fail, either yield or slippage, up to 4 x 20.8 = 83.2 kips.

To simplify the theoretical analysis of the deck an isotropic finite element model of the test panel was prepared. A concentrated load of 10 kips was applied at the middle of the model. The finite element model showed a maximum moment of 51.74 in-kips/ft. under the 10 kip load.

The composite plastic moment capacity of the test deck was computed. Material mill certificates indicated that the grid I-beams had an  $F_y$  of 54 ksi, and  $F_u$  of 75 ksi. Using these  $F_y$  and  $F_u$  values resulted in composite moment capacity of 298.0 and 408.6 in-kips/ft. respectively.

The results of the finite element analysis were used to compute the loads required to cause moments of 298.0 and 408.6 in-kips/ft:

$$(298.0/51.74) \times 10 = 57.6 \text{ kips}$$

$$(408.6/51.74) \times 10 = 78.97 \text{ kips}$$

As noted, the panel failed, with large displacement, at a 79 kip load. Also, Table 3 indicates that non-linear displacement started at about a load level of 56 kips. Therefore, there is very good correlation between the theoretical and actual section capacity of the deck tested. This also indicates that the panel performed as expected and that the splice did not fail. Note that the 79 kip load is much larger than actual live loads to which the deck would be subjected.



**Table 1: Displacement for Static Loading, inches**

		Displacements, inches						
Cycles	Load	101	102	103	104	105	106	107
@ 3,880	14	0.113	0.141	0.169	0.176	0.167	0.139	0.112
@ 203,890	16	0.130	0.162	0.196	0.203	0.192	0.160	0.126
@ 403,890	18	0.148	0.186	0.226	0.233	0.222	0.184	0.144
@ 613,470	20	0.149	0.191	0.234	0.242	0.229	0.187	0.144
@ 813,480	22.8	0.186	0.238	0.293	0.303	0.286	0.233	0.178
@ 1,010,560	22.8	0.189	0.247	0.309	0.320	0.301	0.242	0.182

**Table 2: Displacement for Test with Prop Support, inches**

		Displacements, inches						
Cycles	Load	101	102	103	104	105	106	107
@ 1,010,560	25	0.112	0.140	0.165	0.151	0.0	0.051	0.017
@ 1,010,560	30	0.136	0.170	0.200	0.184	0.0	0.064	0.023
@ 1,010,560	35	0.157	0.198	0.235	0.216	0.0	0.080	0.031
@ 1,010,560	40	0.187	0.236	0.282	0.262	0.0	0.103	0.044

**Table 3: Displacement for Ultimate Loading Test, inches**

		Displacements, inches						
Cycles	Load	101	102	103	104	105	106	107
@ 1,010,560	24	0.215	0.284	0.359	0.373	0.343	0.277	0.204
@ 1,010,560	32	0.280	0.371	0.472	0.498	0.450	0.361	0.262
@ 1,010,560	40	0.348	0.470	0.618	0.556	0.582	0.455	0.322
@ 1,010,560	48	0.418	0.585	0.661	0.559	0.680	0.564	0.415
@ 1,010,560	56	0.509	0.763	0.661	0.559	0.924	0.730	0.441
@ 1,010,560	64	0.631	0.810	0.660	0.559	1.218	0.923	0.525
@ 1,010,560	72	0.836	0.810	0.661	0.558	1.482	1.124	0.527

Note: Gauges 103 and 104 show the same displacements after 56 and 48 kips respectively, indicating malfunctioning gauges. Actual displacements increased.



### 3. Test Panel and Test Setup

The steel grid deck panel which was used for this test was 10'- 0" (direction of main bars) long and 6'- 8" wide (see Figure 1). It was constructed by splicing a 3'- 8" and a 3'- 0" wide panel and was half-filled with concrete (see Figures 1 & 2). The grid design tested consisted of 5- $\frac{3}{16}$ " special section I-beams spaced 8" c/c, with two 1 x  $\frac{5}{16}$ " supplemental bars parallel with and spaced equally between grid I-beams, and 2 x  $\frac{1}{4}$ " cross bars spaced 4" c/c. All material was ASTM A588 (grade 50) steel. As shown in Figures 1 & 3 two panels were spliced by using 18" long rebar (#4, grade 60) with 9" of each rebar on either side of the splice. For this particular test panel, rebars were inserted through the same fabrication slots in the main bar through which the 2 x  $\frac{1}{4}$ " cross bars are inserted. Separate slots to accommodate this splice detail can also be used, depending on manufacturer's fabrication preference (see Photos 1 & 2). Concrete used  $\frac{3}{8}$ " maximum aggregate and had an ultimate strength of 5450 psi.

As shown in Figure 1, the center-to-center dimension of panel supports is 8'-11" (simple span). Loading consisted of a single ram at the center of the span (see Figure 2). Loads were applied through a 1" x 8" x 20" steel plate and  $\frac{1}{2}$ " rubber pad to simulate a HS-20 truck wheel. The loading pad was situated in the 3'-8" panel with the 20" long side parallel to the main bar and was positioned along the edge of the splice length.

A total of 7 displacement gauges were placed under the 7 center grid I-beams (see Photo 3). Note that panel contained 11 main I-beams (see Figure 2). Results of gauge readings are presented in Appendix A.

### 4. Test Procedure

Loading of the panel was started 62 days after concrete was poured. As shown in the table below, the panel was subjected to over 1 million load cycles. Loading began with a static test; after every 200,000 cycles the cyclic load test was halted, and a subsequent static test was performed.

At the end of the 1,000,000 cycle a steel prop (see Figures 4-6) was placed under the main bar to the other side of the panel splice, opposite the load, and the load was increased to 40 kips. Finally, the prop was removed, a larger ram was installed (160 kips), and the deck was loaded up to 79 kips, at which point the deck failed. The failure mode consisted of a longitudinal crack along the center line of the splice and large differential displacement between the loaded and unloaded sides of the splice. The panel at the loaded side of the splice had a maximum of 2" to 3" differential downward movement under the load. This displacement was smaller along the center line of the splice away from the load.

Cycles Thousands	Test Type	Load or Load Range, kips
0	Static	14
0 - 200	Cyclical	12
200	Static	16
200-400	Cyclical	14
400	Static	18
400-600	Cyclical	16
600	Static	20
600-800	Cyclical	18
800	Static	22.8
800-1000	Cyclical	22.8
1000	Static	22.8



## 5. Evaluation of Test Results

Test results are presented in Appendix A. Measured displacements are shown in Tables 1-3 and are plotted in Plots 1-3.

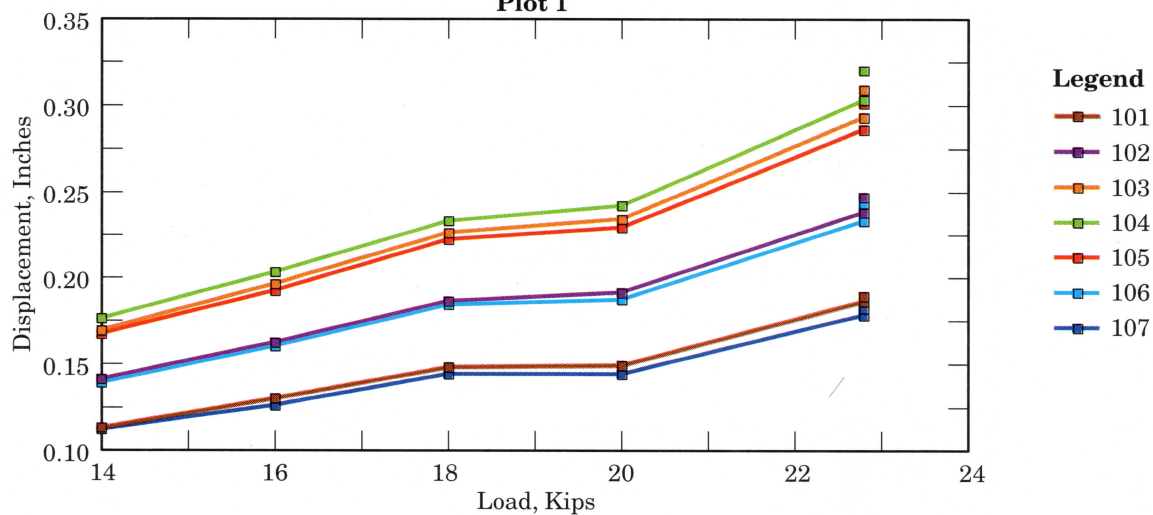
From the data and plots it is clear that the splice rebar carried the static load across the splice. Also, no evidence of any serviceability issues were noticed after the 1,010,560 cycles of loading. When the prop was placed under the main bar of the unloaded side, to simulate a more severe shear case, and the load was increased to 40 kips, the splice did not crack or show any sign of distress. For the static loads, displacement is almost symmetrical which indicates that load was transferred across the splice and was shared by both segments of the spliced panel. These results indicate that the splice performed as expected and as demonstrated theoretically in Section 2.

From Table 3 and Plot 3, ultimate loading, it is evident that the displacements for loads between 48 and 56 kips are symmetrical. This indicates that the splice rebars transferred the load across the splice and load was shared between the two segments of the panel. Also, it should be noted that the rebar splice transferred the load across the splice at loads above 56 kips. However, gauges 103 and 104 malfunctioned beginning at the 56 kip load level. Actual displacements of the grid I-beams were greater. As indicated earlier, these loads exceeded the design capacity of the deck; higher loads were intentionally applied in order to fail the panel. As discussed in Section 2, Theoretical Calculation, yielding was predicted at about 56 kips.

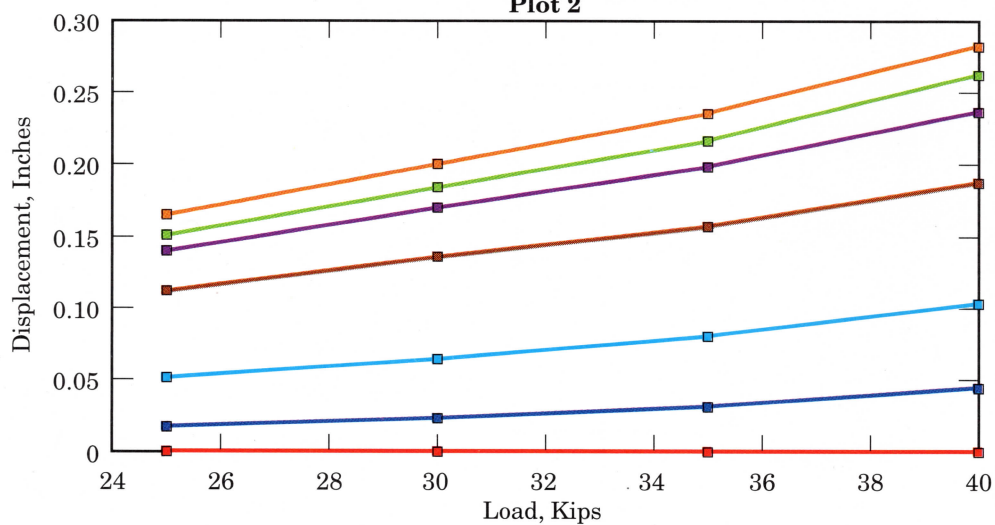
The parameters of the test program were established to provide the most severe condition for a splice joint. Hence, a flush filled, half depth steel grid was used, without the benefit of an integral overfill or separately applied overlay (see Photo 4). Therefore, the successful use of a similarly designed and constructed splice joint, incorporating rebar for splicing grid panels, using either an overfilled half depth grid or any full depth system, is confidently predicted.



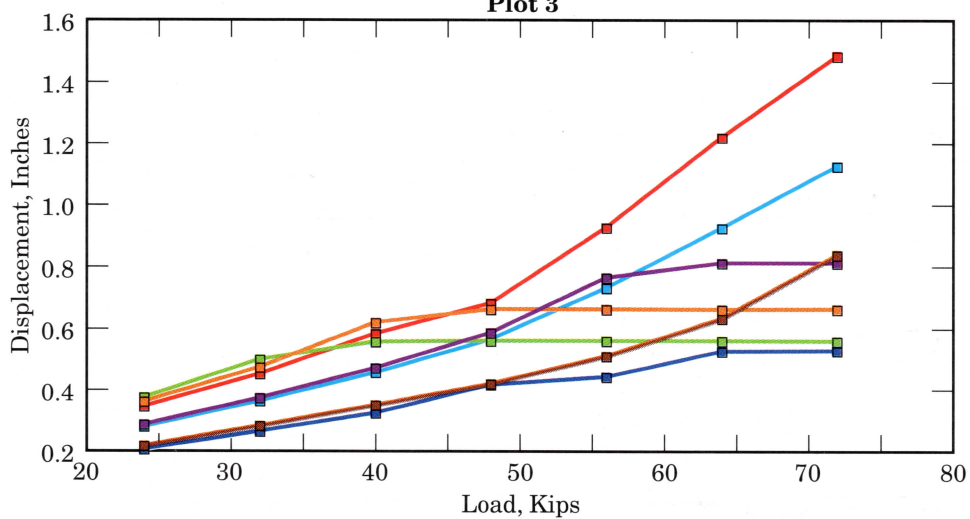
Displacement for Static Test  
Plot 1

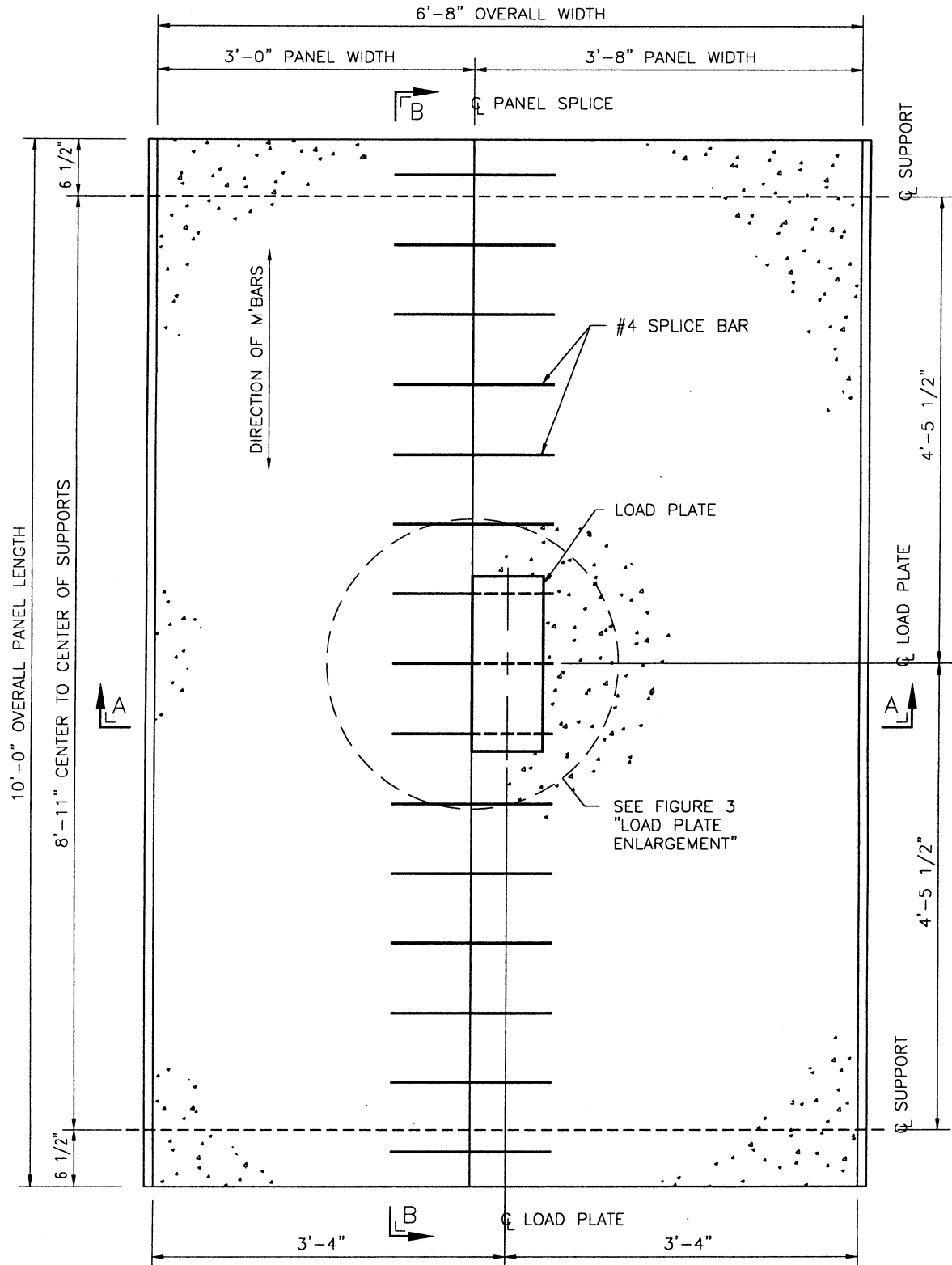


Displacement for Prop Support Test  
Plot 2



Displacement for Ultimate Loading Test  
Plot 3



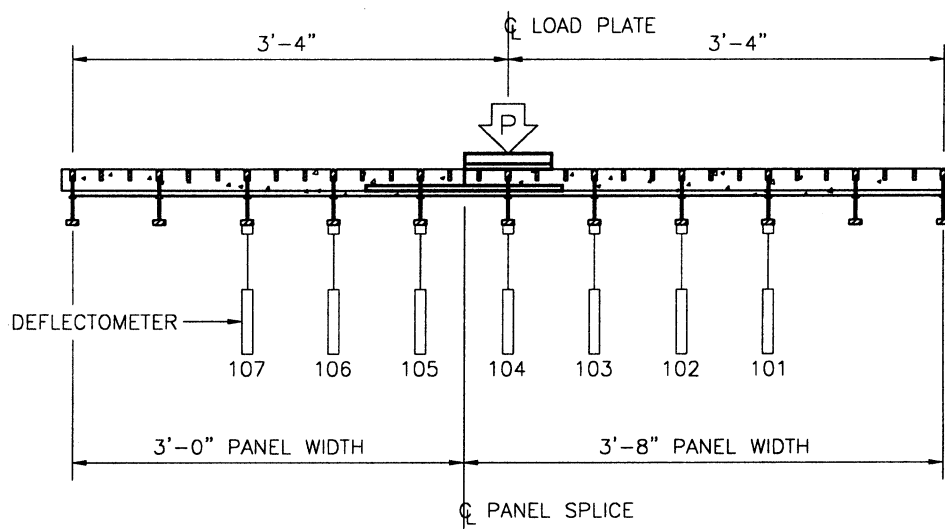


**NOTE**

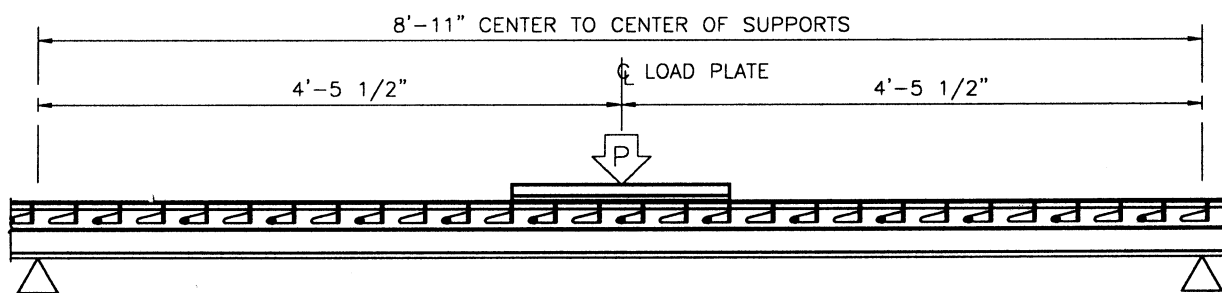
1. SEE FIGURE 2 FOR CROSS SECTIONS A-A AND B-B.

**Figure 1 Test Panel Layout (Weldless Splice Load Text)**



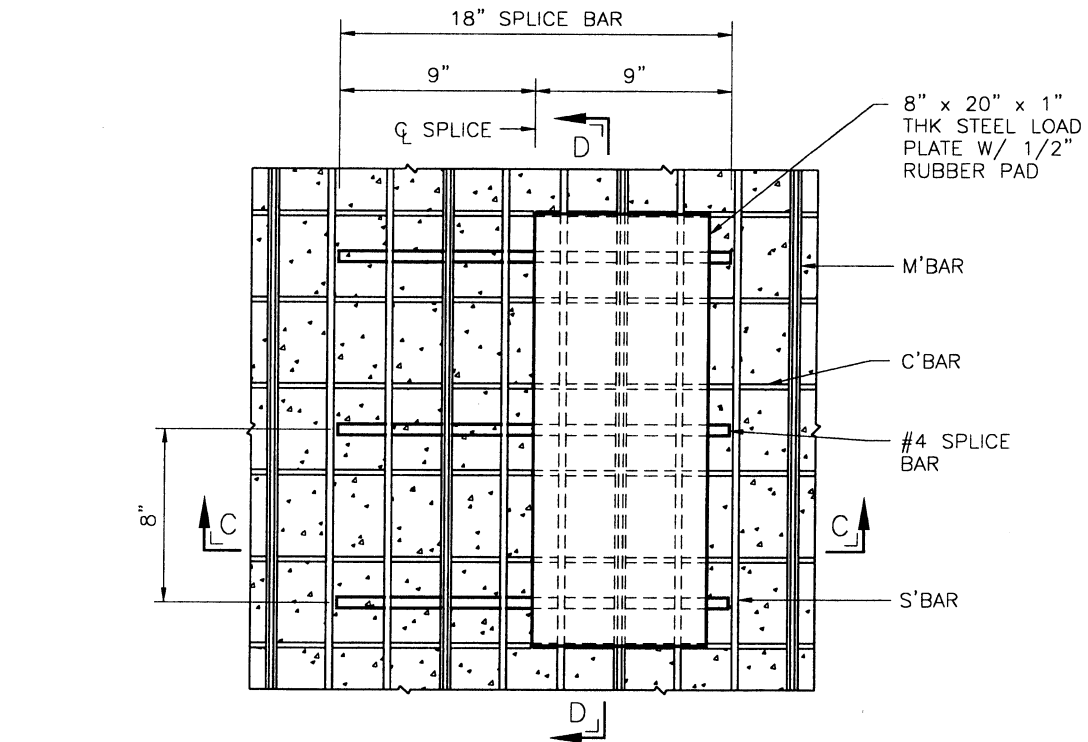


SECTION A-A

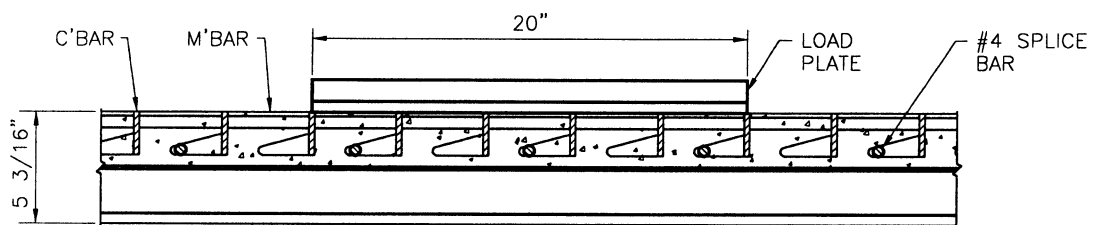


SECTION B-B

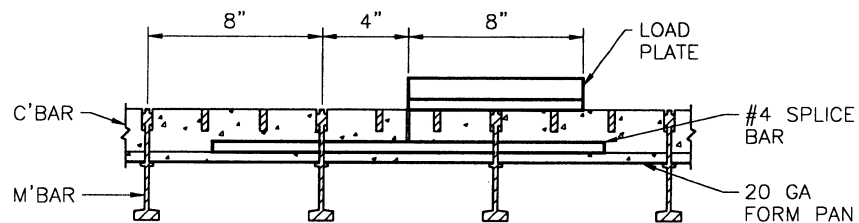
**Figure 2 Test Panel Cross Sections (Weldless Splice Load Test)**



LOAD PLATE PLAN VIEW



SECTION C-C



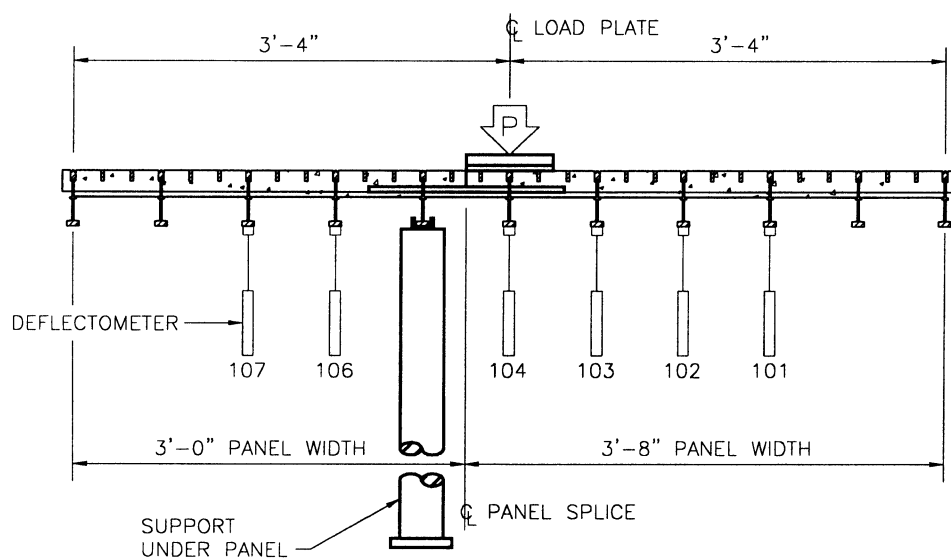
SECTION D-D

Figure 3 Load Plate Enlargement (Weldless Splice Load Text)

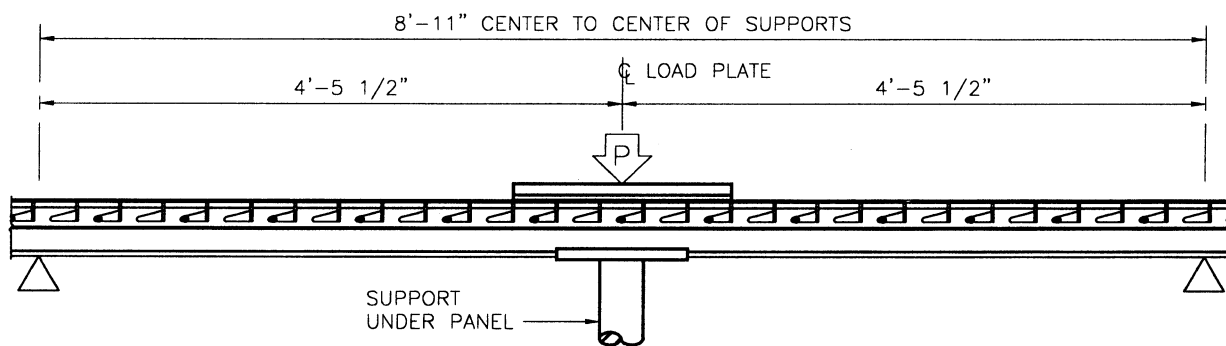


NOTE  
1. SEE FIGURE 5 FOR CROSS SECTIONS A-A AND B-B.

A-7

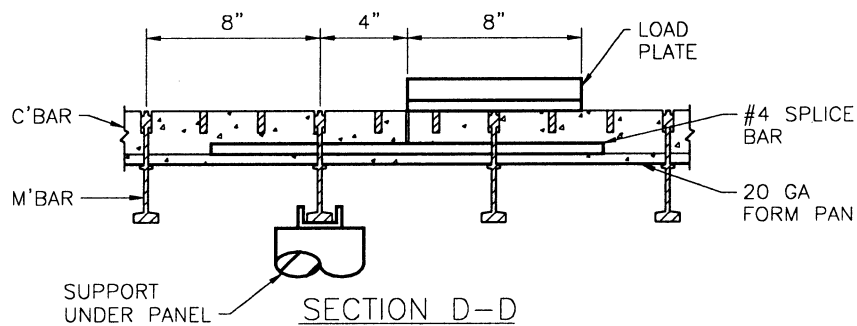
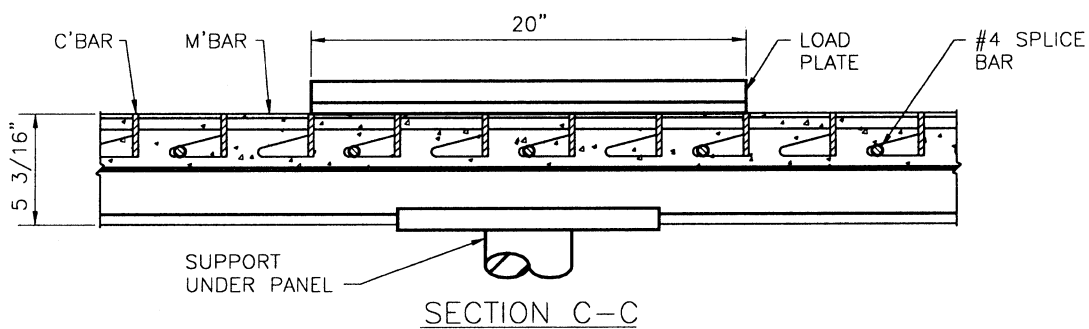
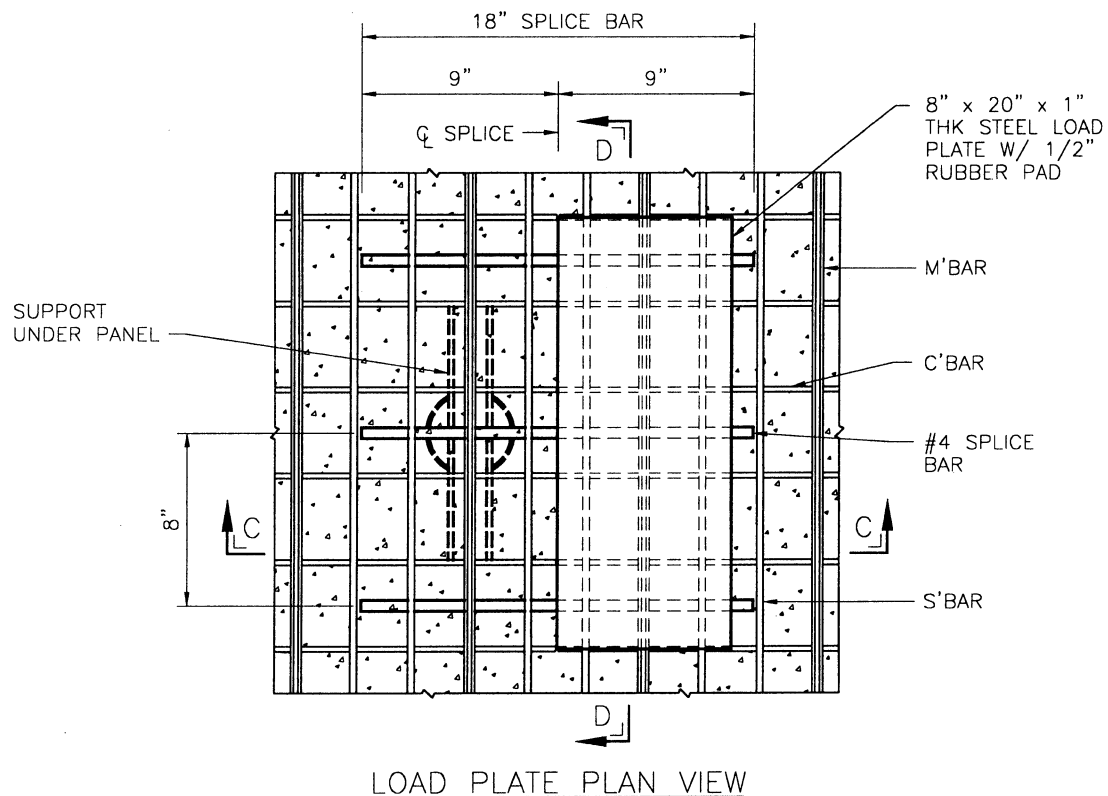


SECTION A-A



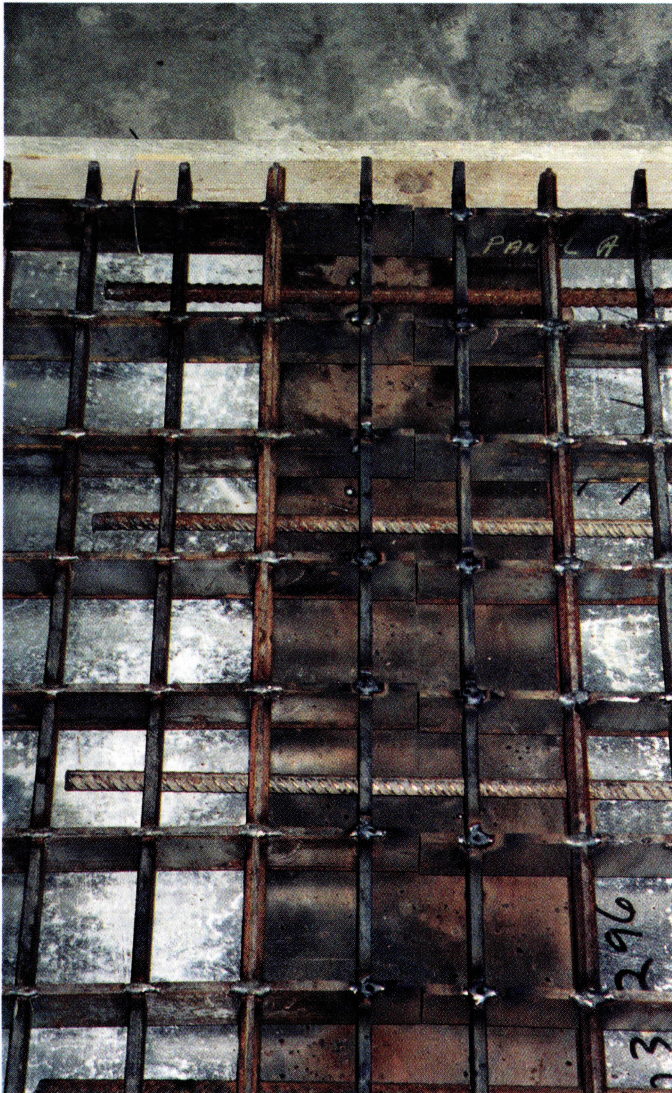
SECTION B-B

Figure 5 Test Panel Cross Sections (Weldless Splice Shear Test)



**Figure 6 Load Plate Enlargement (Weldless Splice Shear Test)**



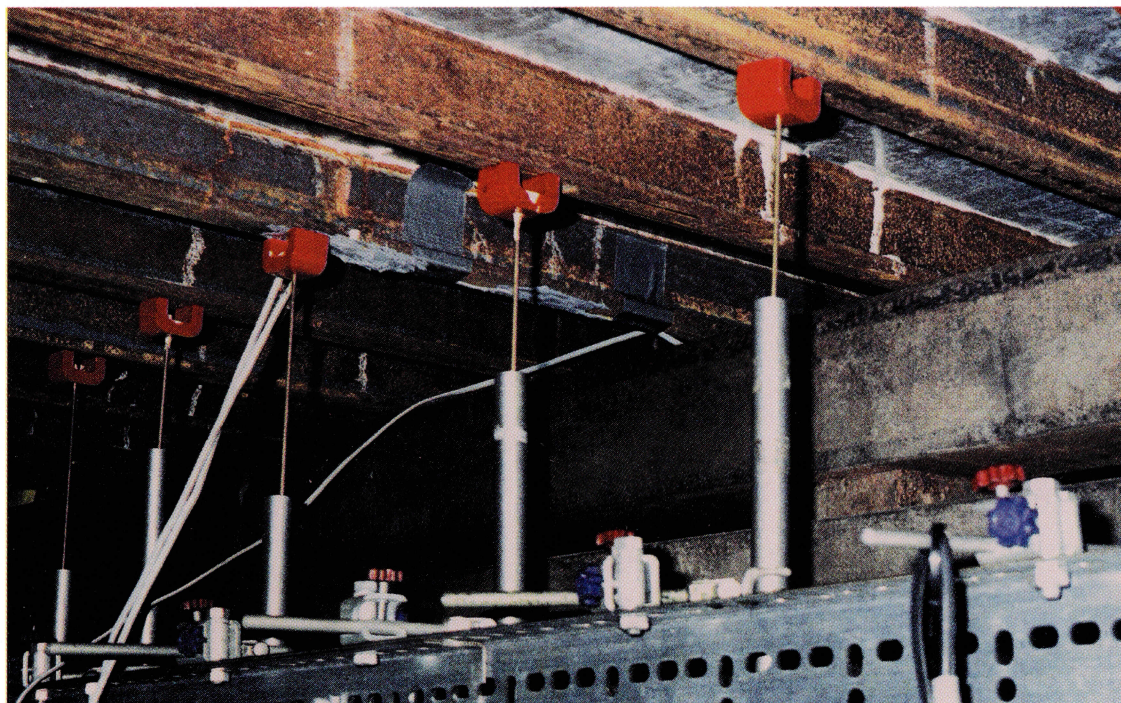


**Photo 1** Panel View Along Splice Joint  
Showing Splice Rebar Spaced  
8" c/c.

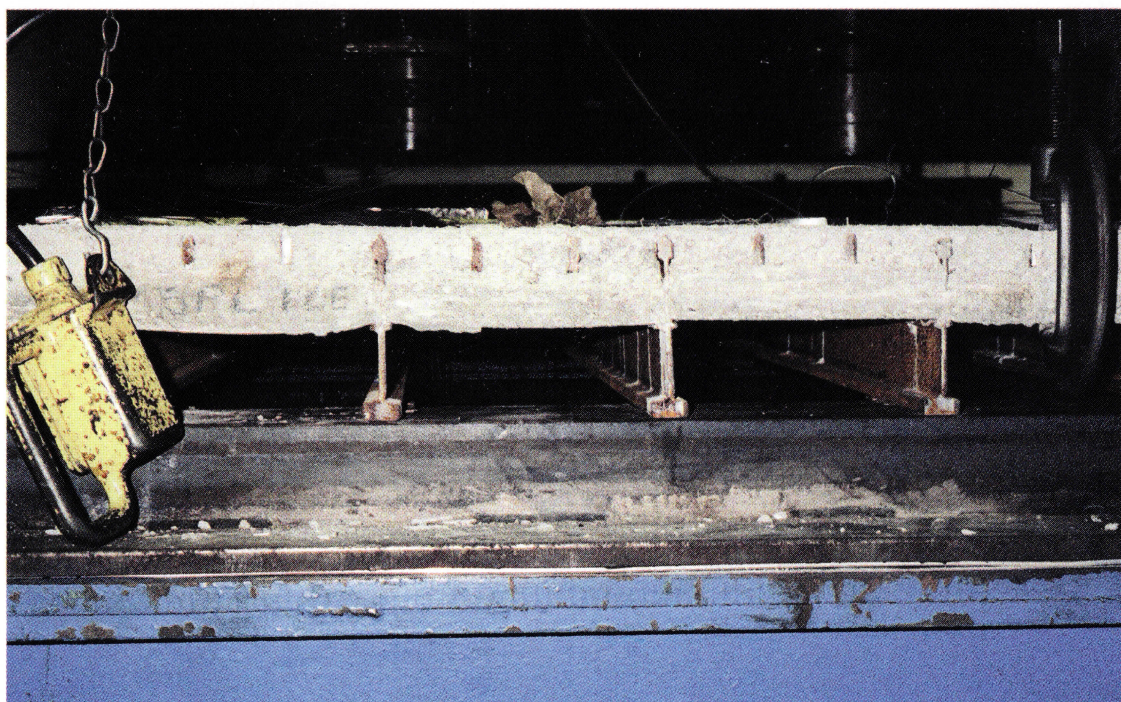


**Photo 2** Splice Rebar May be Inserted  
Through Grid Cross-bar slot, or  
Through Separate Opening at  
Option of Grid Manufacturer.





**Photo 3 Deflection Gauges in Place Beneath Panel.**



**Photo 4 Cross Section of Half Depth, Flush Filled Grid.**