February, 1992

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Report to the

BRIDGE GRID FLOORING
MANUFACTURERS ASSOCIATION

on the

# Physical Testing of Deck Samples from the Cedar Street Bridge, Youngstown, Ohio

## Abstract

Three of several pieces of filled grid taken from the deck of the Cedar Street Bridge in Youngstown, Ohio were tested in various modes at the University of Pittsburgh and are reported herein. The bridge was a three lane, multiple simple span structure serving as a viaduct near downtown Youngstown, built in 1941 and dismantled in 1991. If there was ever any supplemental or superimposed wearing surface there was no evidence of it on the samples reported here. The location of the samples within the bridge is unknown except that they came from the main span.

Two of the specimens were rectangular panels tested in flexure in the strong direction with strain and deflection gages and one was a square block tested for debonding in the weak direction. The panels demonstrated complete strain compatibility between the main bar steel and the concrete in positive bending at low loads but exhibited what might have been concrete cracking on the tension side in negative bending. The ultimate capacity of the panels in bending was essentially the same in negative and positive moment and corresponded to about one and one half times the plastic moment strength of the main bars. There was no debonding evident in the block or in the panels.

## The Test Articles

The test articles supplied by BGFMA consisted of two panels roughly 40 inches wide in the weak or cross bar direction by about 7 feet 10 inches in length in the strong or main bar direction, and one square block about 13 inches on a side. The panels designated as Panels 11 and 12 have dimensions and bottom appearances as shown in Figures 1 and 2.

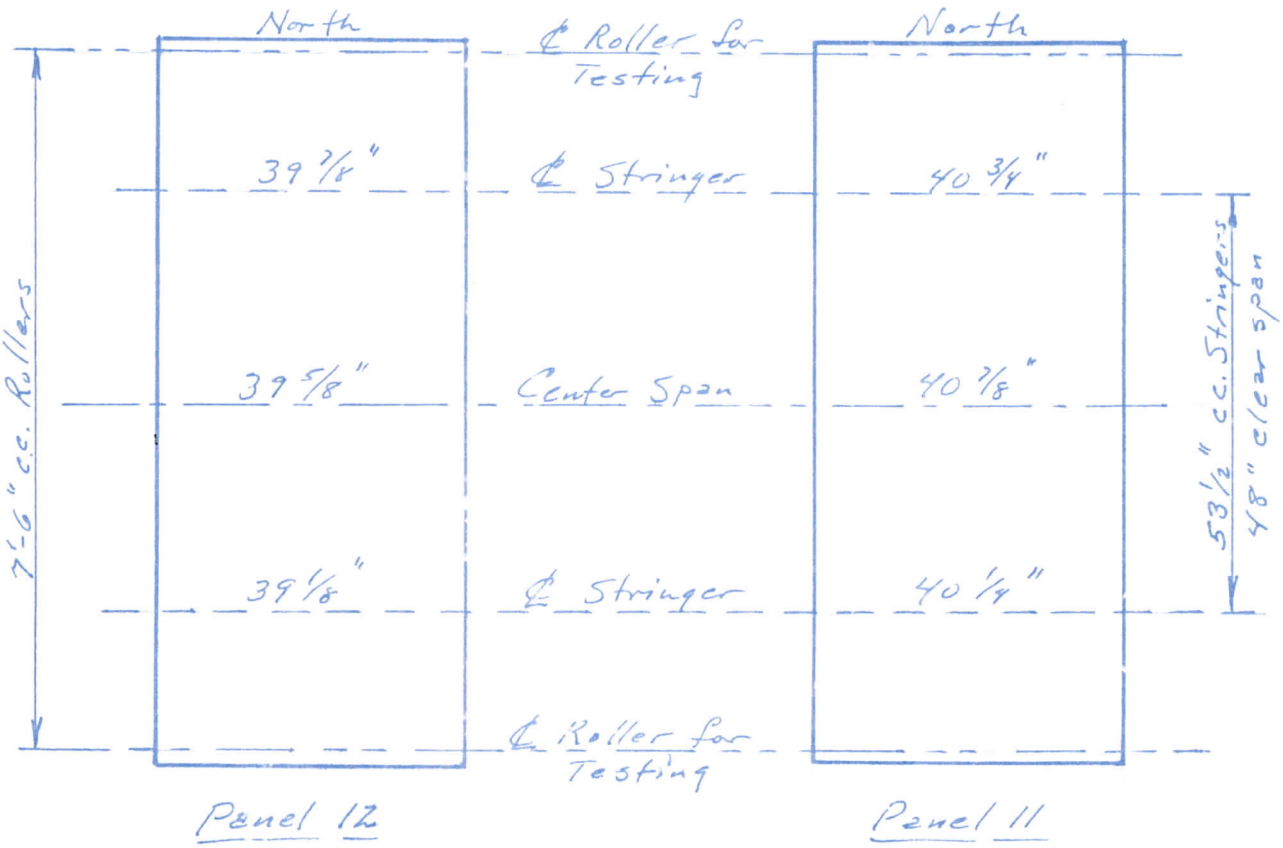



Figure 1. Dimensions of Test Panels

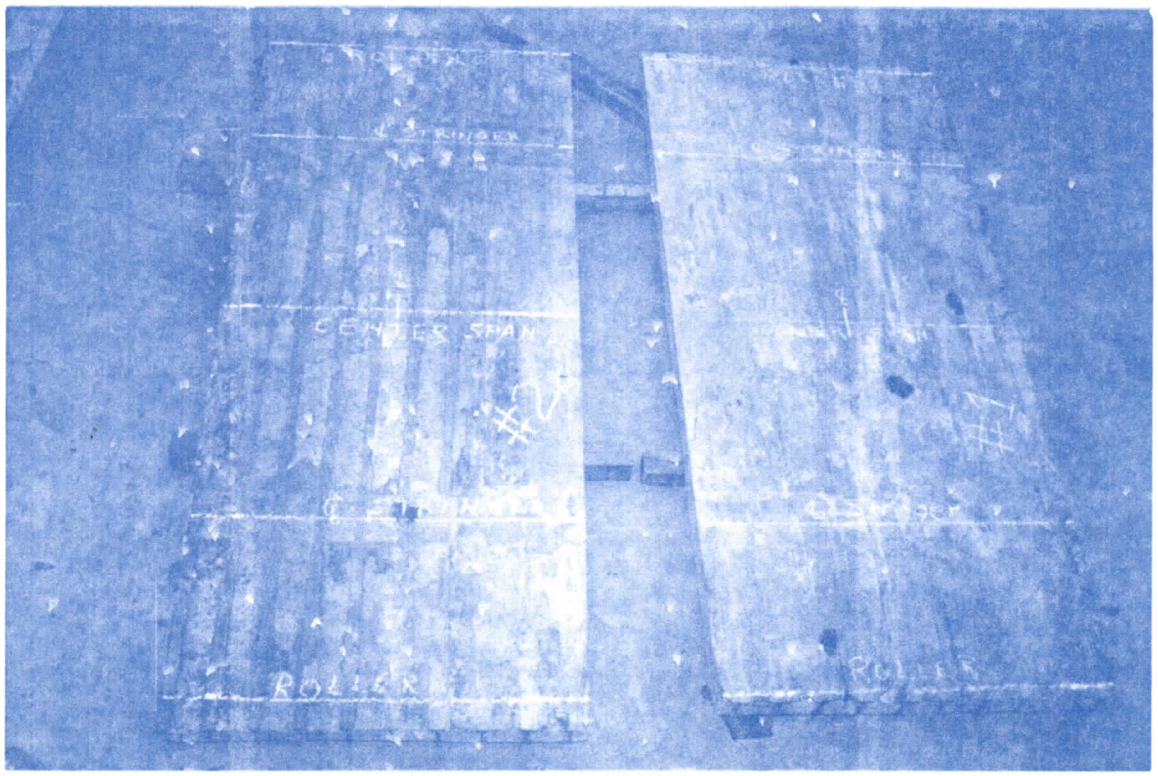



Figure 2. Appearance of Panel Bottoms
(Note: Numbers subsequently changed to 11 and 12.)

The main bar spacing was 4 inches and the main bar section appears to have been a 3-1/2 inch bar with an original and final shape as shown in Figure 3. Wear and corrosion had reduced the top flange from very little at the cross bar intersections to as much as 2/10 inches between cross bars.

The cross bars, also at 4 inch spacing, were probably of a triangular shape with the apex down, press-welded into the top ridges of the main bars. About 50% of the cross bars were completely worn through between main bars. Parallel with the cross bars were two round bars, 9/16 inches in diameter, 16 inches apart, 3/4 inches up from the fill pan, and symmetrically placed with respect to the center line between stringers.

The concrete was eroded and pitted another 1/10 to 1/8 inches at the center of the cells. In the southeast corner of Panel 12 the concrete was poorly compacted (Figure 4) while in the northwest corner a fracture due to a fault or inclusion (Figure 5) had been initiated. Longitudinal cracks at or near the steel/concrete interface were apparent over much of the top surface. Those shown in Figures 6 and 7 were near the center span of Panels 11 and 12. Similar cracks were visible on the square block. The fill pans, originally 20 gage steel sheet, were largely rusted away in Panel 12 but completely sound in Panel 11 (Figures 2, 8, and 9).

## Instrumentation

A total of four elecrical resistance strain gages per panel were mounted on the two panels near center span. On a central main bar of each panel a 1/4 inch gage was mounted top and bottom. Within an inch laterally of these two gages were placed 2 inch long concrete gages parallel with the main bars (Figures 8 and 9.) In the case of Panel 11 a section of fill pan had to be removed so that the bottom concrete gage could be mounted directly on concrete. Gage locations are given in Table 1 below. In addition to strain gages a single dial gage measured the center deflection of each panel.

Table 1
Strain Gage Locations

| Panel | Gage | Material | Position | Distance from Center Span, inches |
|---|---|---|---|---|
| | 1 | Steel | Top | 3.0 |
| 11 | 2 | Steel | Bottom | 2.5 |
| | 3 | Concrete | Top | 3.0 |
| | 4 | Concrete | Bottom | 2.5 |
| | 5 | Steel | Top | 1.5 |
| 12 | 6 | Steel | Bottom | 3.0 |
| | 7 | Concrete | Top | 1.5 |
| | 8 | Concrete | Bottom | 3.0 |

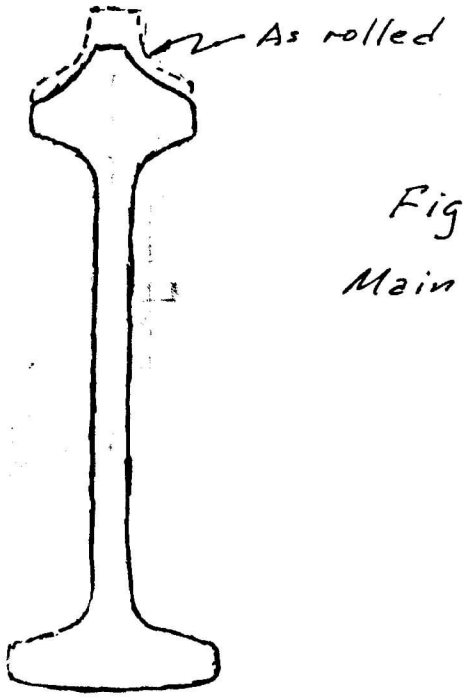



Figure 3
Main Bar Section

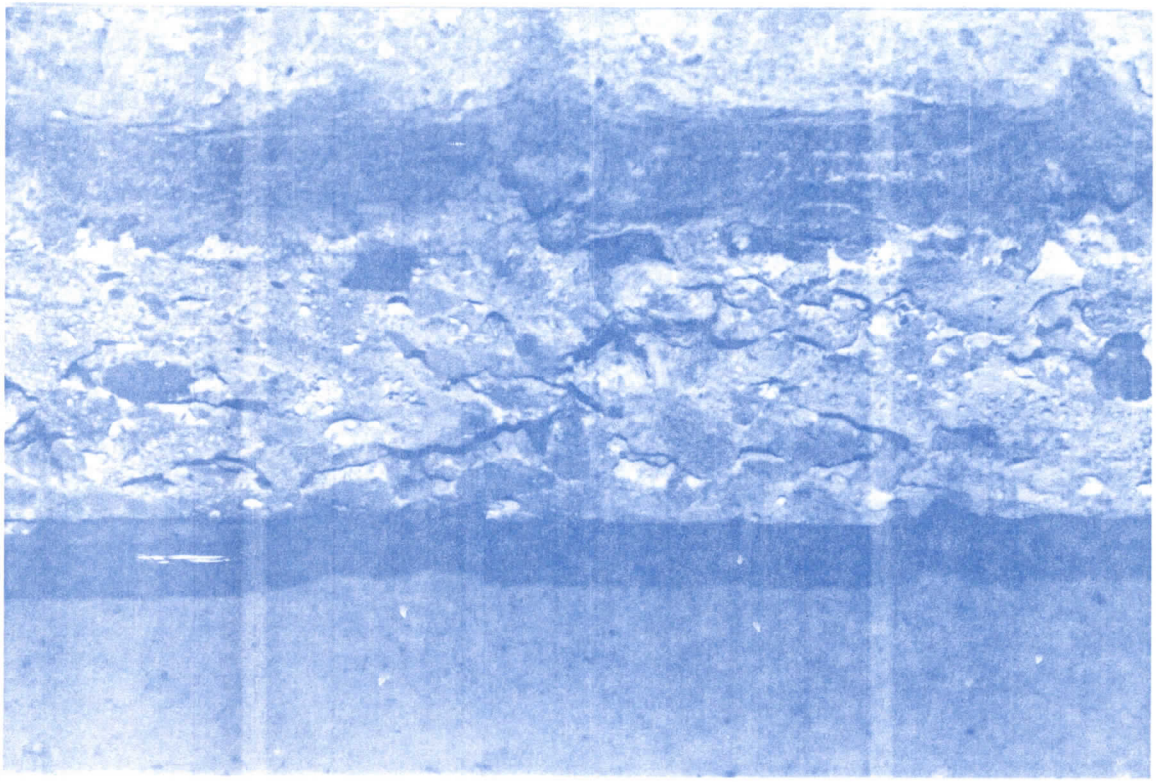


Figure 4. Southeast Corner of Panel 12

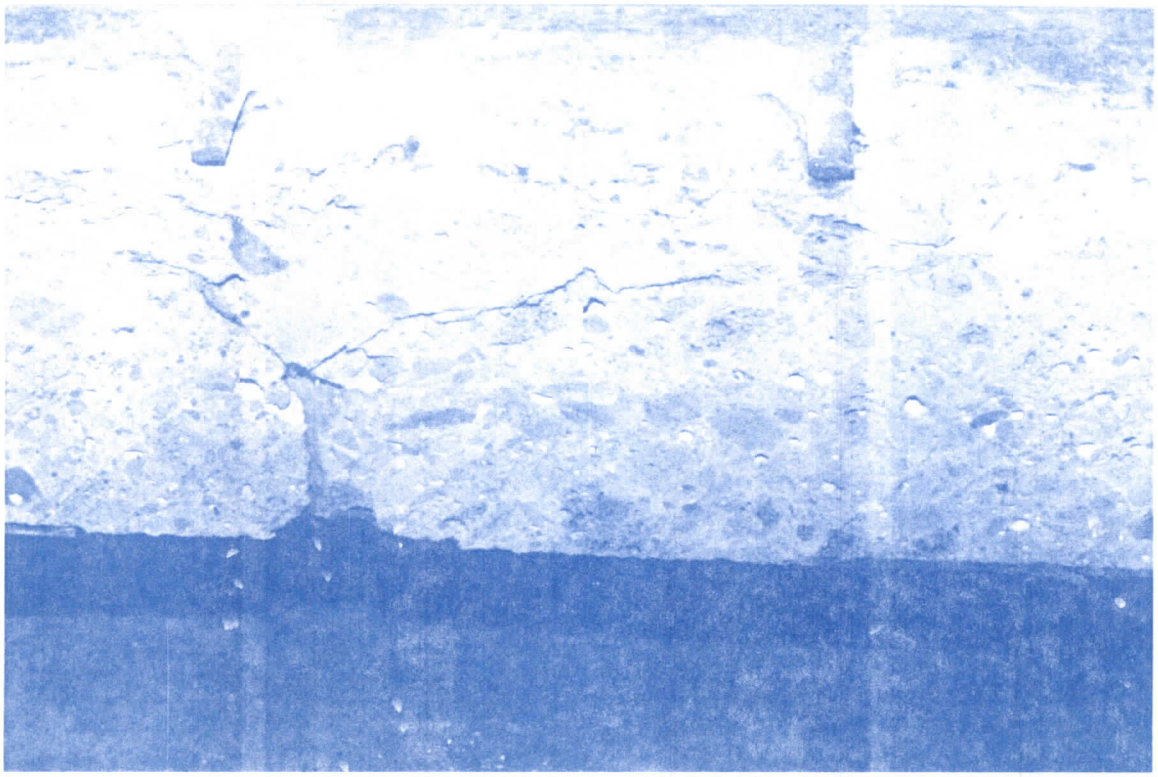


Figure 5. Northwest Corner of Panel 12

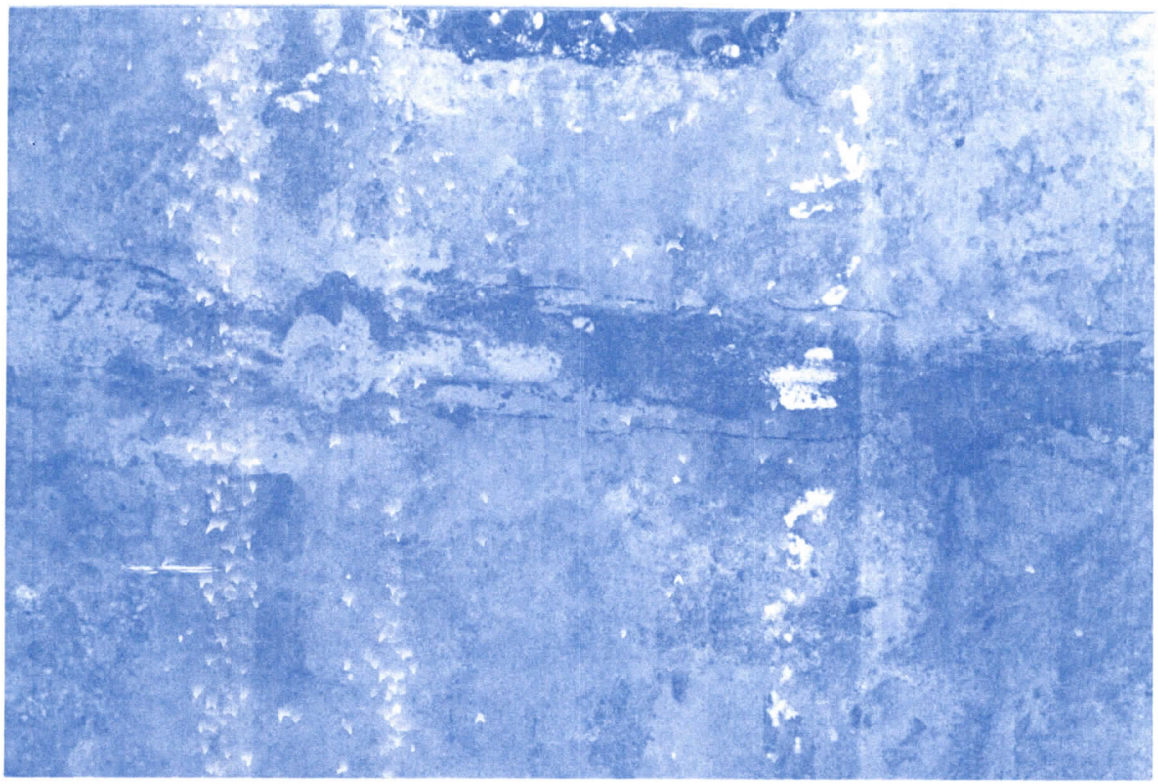


Figure 6. Interface Cracks, Panel 11

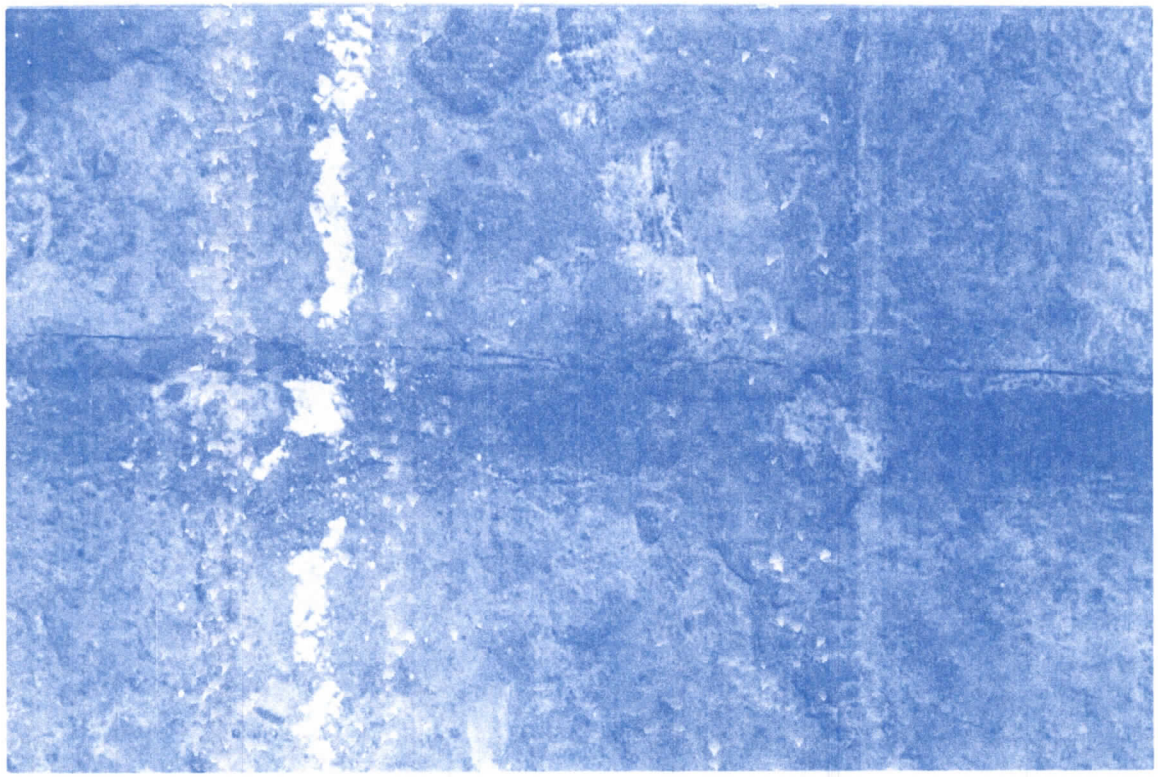


Figure 7. Interface cracks, Panel 12

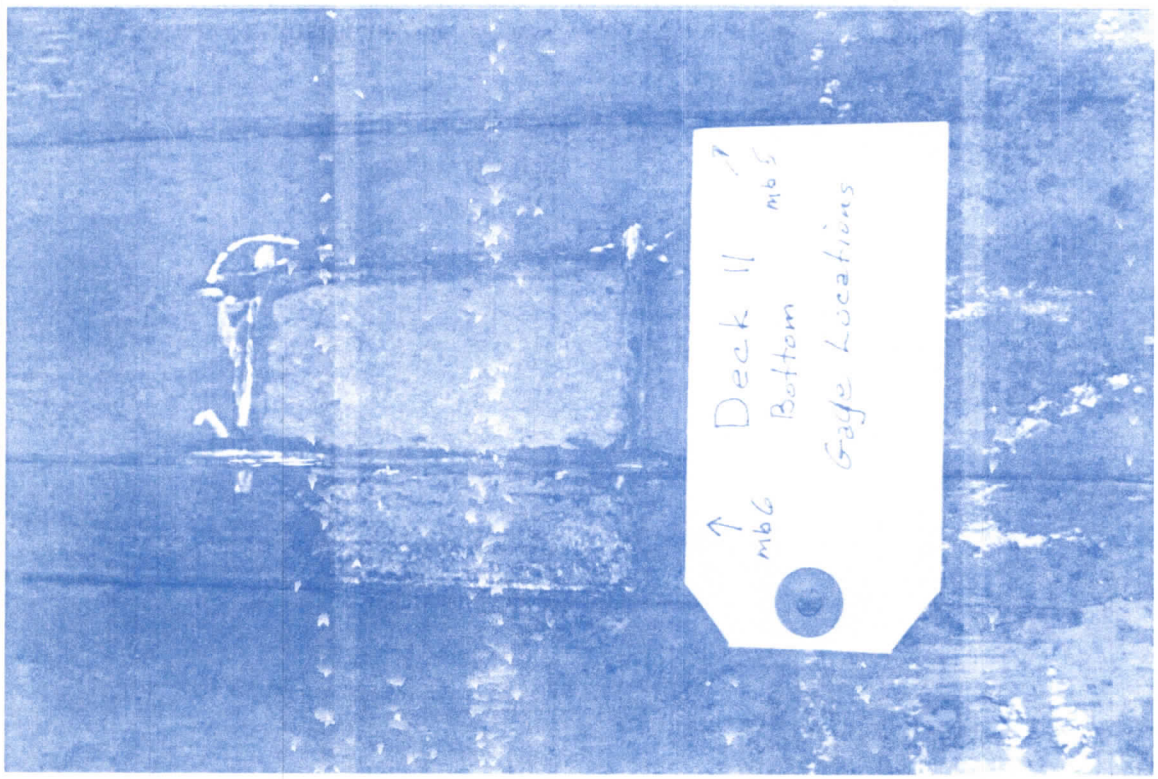



Figure 8.

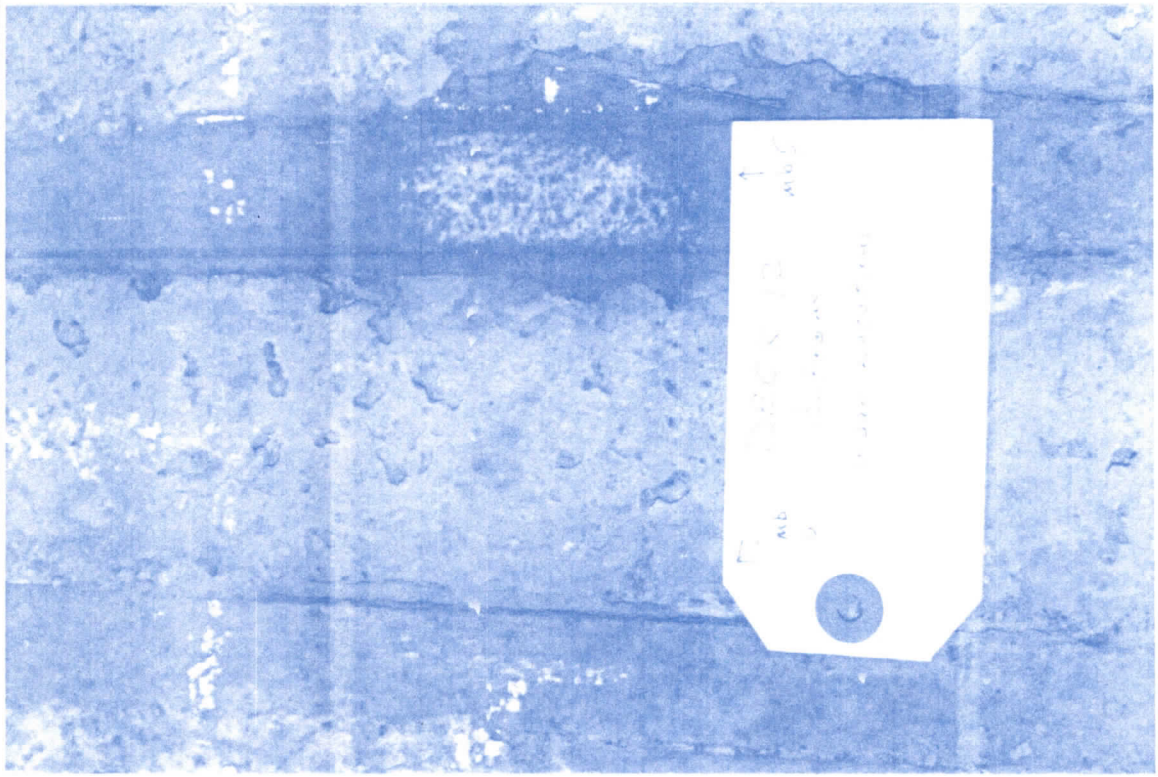


Figure 9.

Loading

The panels were simply supported side by side over a span of 7 feet 6 inches and loaded with a simulated line load consisting of interconnected jacks. The ends of the main bars were shimmed to achieve some uniformity of support. The jacks were loaded by a loading beam (Figure 10) which in turn was loaded by a single large ram with a load cell in series with it. The interconnected jacks were not connected to any active oil pressure source but merely responded uniformly to being squeezed between the loading beam and the test articles Due to the width of the panels and the number and spa-cing of main bars the jacks (spaced at 8 inches) were halfway between alternate pairs of main bars, in order to maintain symmetry within the loading system. It was found that loading the concrete directly or spanning over it with a steel rod be-tween each pair of main bars for each jack made little difference for loading in the elastic range. For ultimate loading 40 inch long steel channels were laid across all of the main bars of each panel as a bearing surface to prevent the possibilty of the jacks punching through the concrete before the main bars had reached their full plastic moment strength.

In shake-down elastic tests both panels were first tested right side up (RSU) to determine whether they behaved in identical fashion. Then Panel 11 was turned upside down (USD) for the final elastic and ultimate strength tests. The elastic loadings reached a maximum total load of 11.56 kips (for both panels combined). The ultimate loading was 51.25 kips total.

The 13 inch square block was tested USD as a wide, short simple beam spanning 11 inches in the weak direction. Load was applied by a universal testing machine through a 12 inch rod resting on the central main bar and in full contact with the loading head of the machine. Any surviving cross bars had been cut to guarantee that the strength on the tension side (normally the top) would be entirely that of the concrete or the steel/ concrete interface. The purpose of the test was to determine any tendency for debonding. Only the failure load was recorded.

The specimen failed abruptly at 3700 pounds by a vertical fracture (Figure 11) through the concrete beginning at the boundary between the concrete and vertical edge of what would normally be the top flange of the center main bar. Except at that edge which was quite rust stained there was no evidence of debonding. Assuming that the failure section consisted of a rectangle of concrete from the bottom outside corner of that edge to the fill pan ( a depth of some 2-3/4 inches) and that the stress was linear from top to bottom, the tensile strength of the concrete was 600 psi. If the tensile strength of concrete is taken as 10% of the compressive strength, the compressive strength would be around 6000 psi.

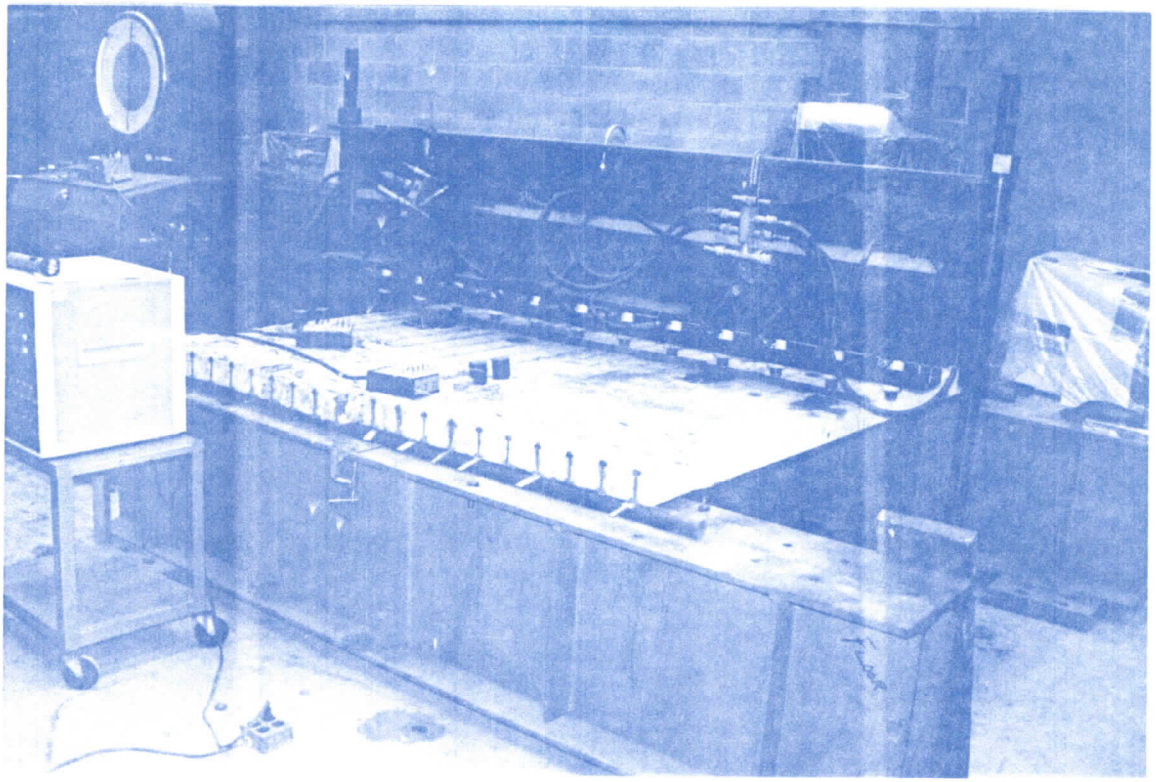


Figure 10. Panel Loading System

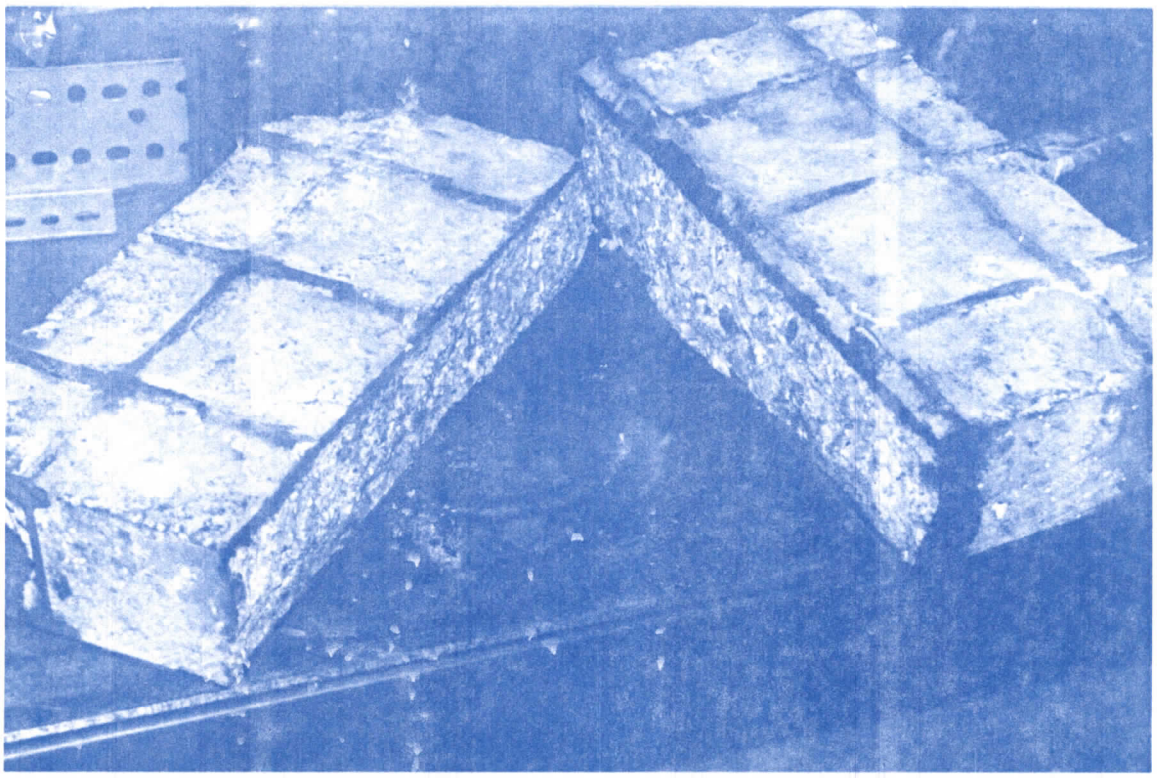


Figure 11. Square Block Fracture

## Panel Results

There was initially some difficulty in obtaining reasonable results from gages 4 and 8 due to either some surface contamination or slow setting of the adhesive. However, by using separate strain recorders for these gages their behavior became linear and repeatable with no residual strain showing upon unloading. As a consequence, only the results of run #8 are presented. There was no significant degradation in the other strain or deflection measurements of Panel 12 as a consequence of the previous tests. After Panel 11 was turned USD there seems to have been some cracking of the tension concrete, as will be noted below. No difference in the center deflection was observed, however.

Figure 12 depicts the strain records for run #8. It will be noted that all gages on Panel 12 are linearand those on 11 are reasonably so. For comparison the strains at 11.56 kips are plotted in Figure 13 against the gage positions vertically on the cross section. Because the gage positions horizontally along the length of the panel were not precisely identical (Table 1) the values have been corrected to correspond to a position 3 inches away from center span, assuming linear moment variation between the load and the supports.

It will be noted that Panel 12 demonstrated complete strain compatibility between concrete and steel, as did Panel 11 before it was turned USD. In the USD position gage 3 on Panel 11 showed evidence of debonding, or proximity to a flexural tension crack although none was visible during the elastic tests.

In Figure 14 are the deflection results which remained essentially unchanged over the series of elastic tests. Panel 11 was consistently stiffer than Panel 12 by about 5%. This difference may have been due to both the greater width of Panel 11 (about an inch) and the survival of the fill pan.

In the ultimate strength test the strain and deflection recordings at low loads did not always agree with those reported above due to the presence of the channel sections. Because of the non-uniformity of wear on the top of Panel 12 and the uneven fill pan surface on the bottom of Panel 11 the main bars did not load up uniformly. In Figure 15 are the deflection data recorded for the ultimate strength test. Figure 16 shows the corresponding strains.

## Discussion

In the elastic range the compatibility of strain demonstrated by Panel 12 in tension as well as compression invites an estimation of elastic section properties. The reduced

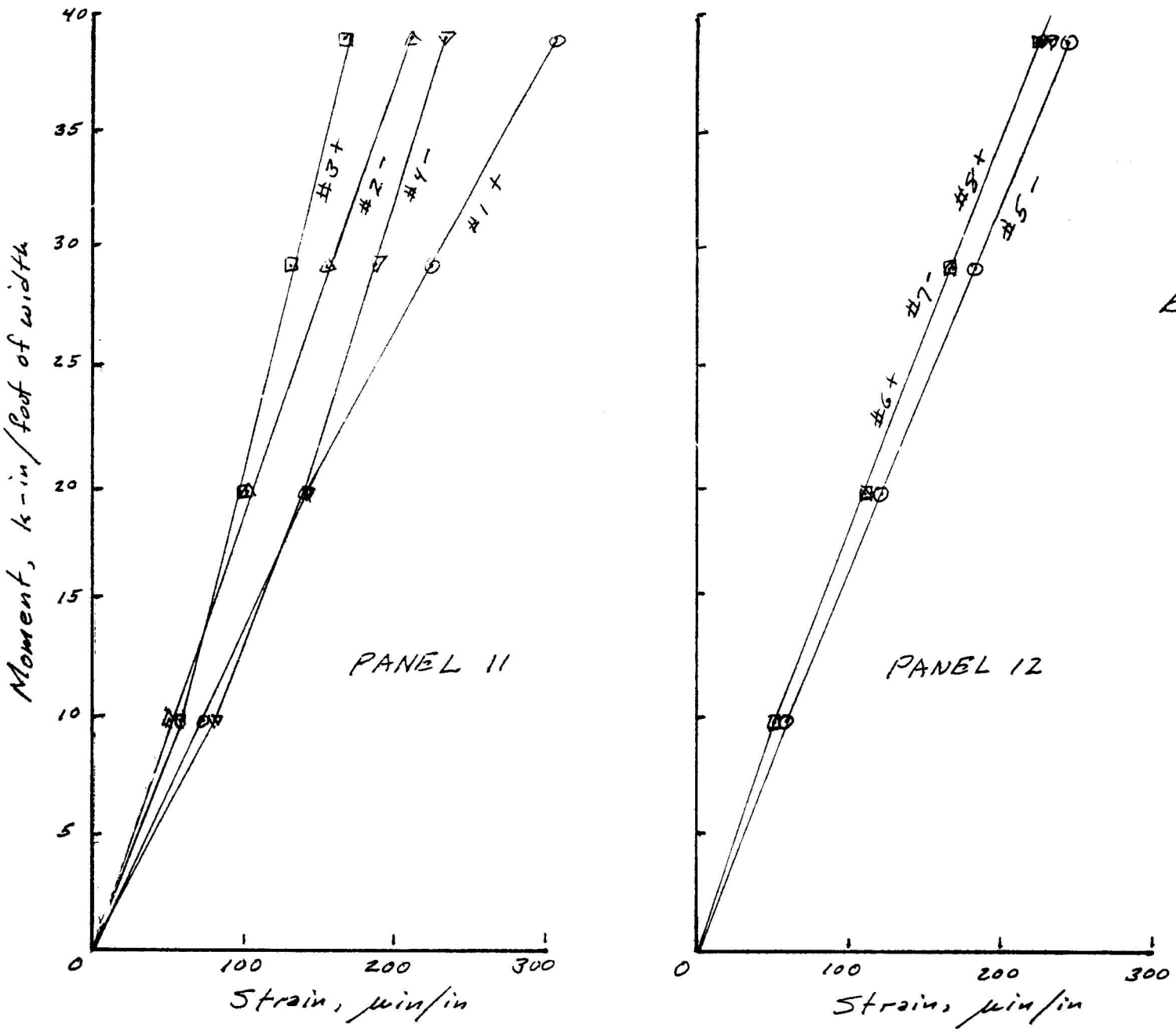



Figure 12.
Elastic Strains

11.56$^k$ total load
produces
39.0 $^{k\text{-}in}/_{ft}$
moment

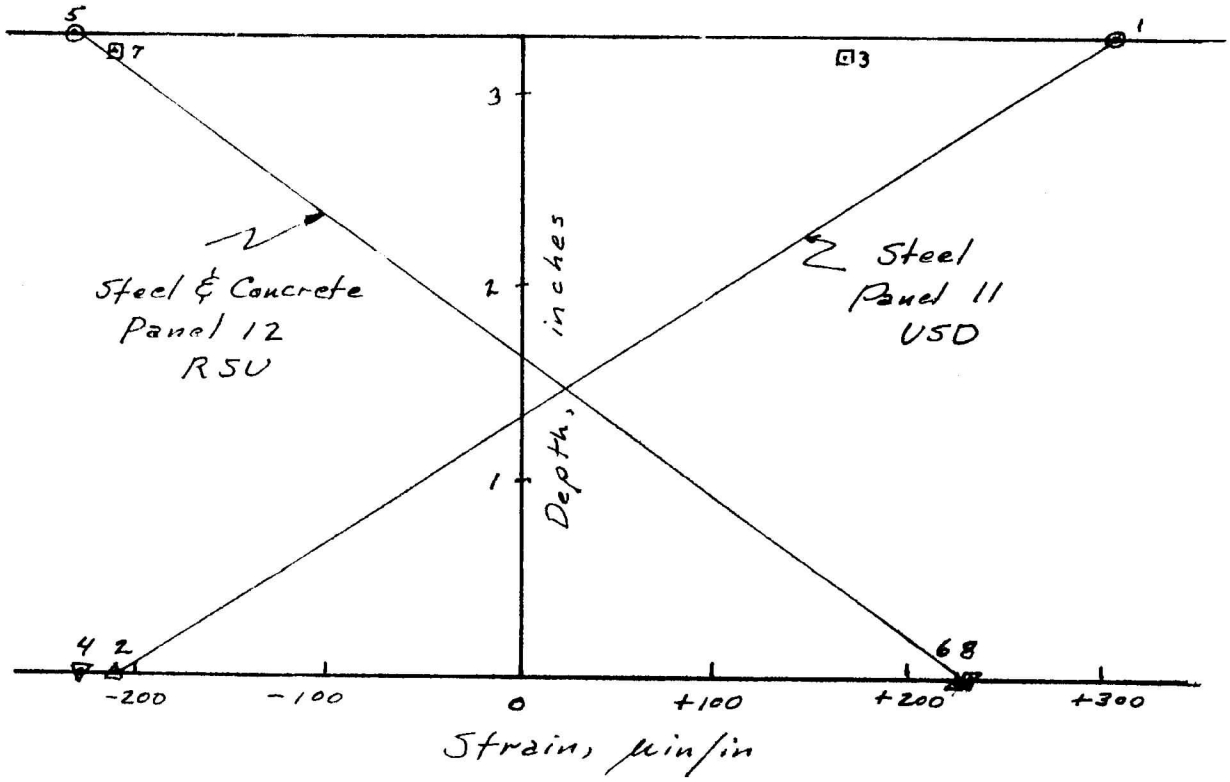



Figure 13. Strain Variation @ 1156$^k$ Total Load
3" from center span

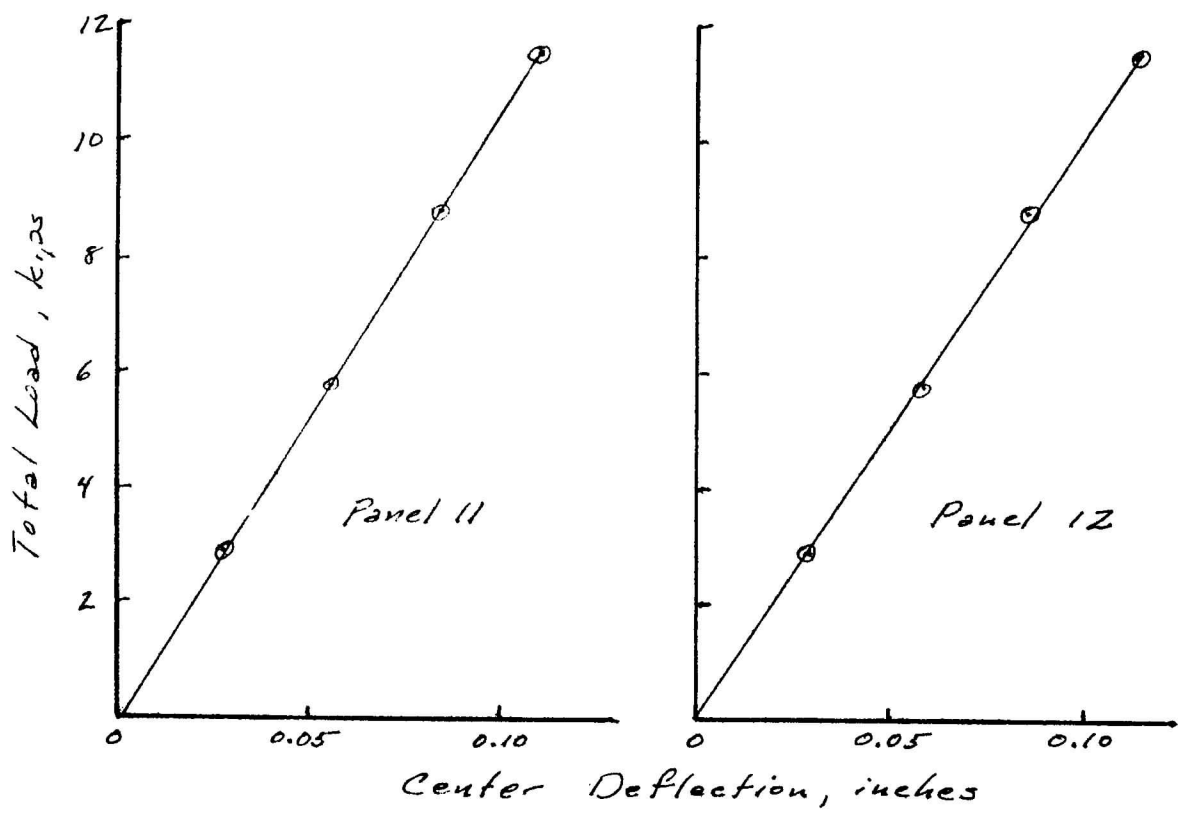



Figure 14. Elastic Deflections

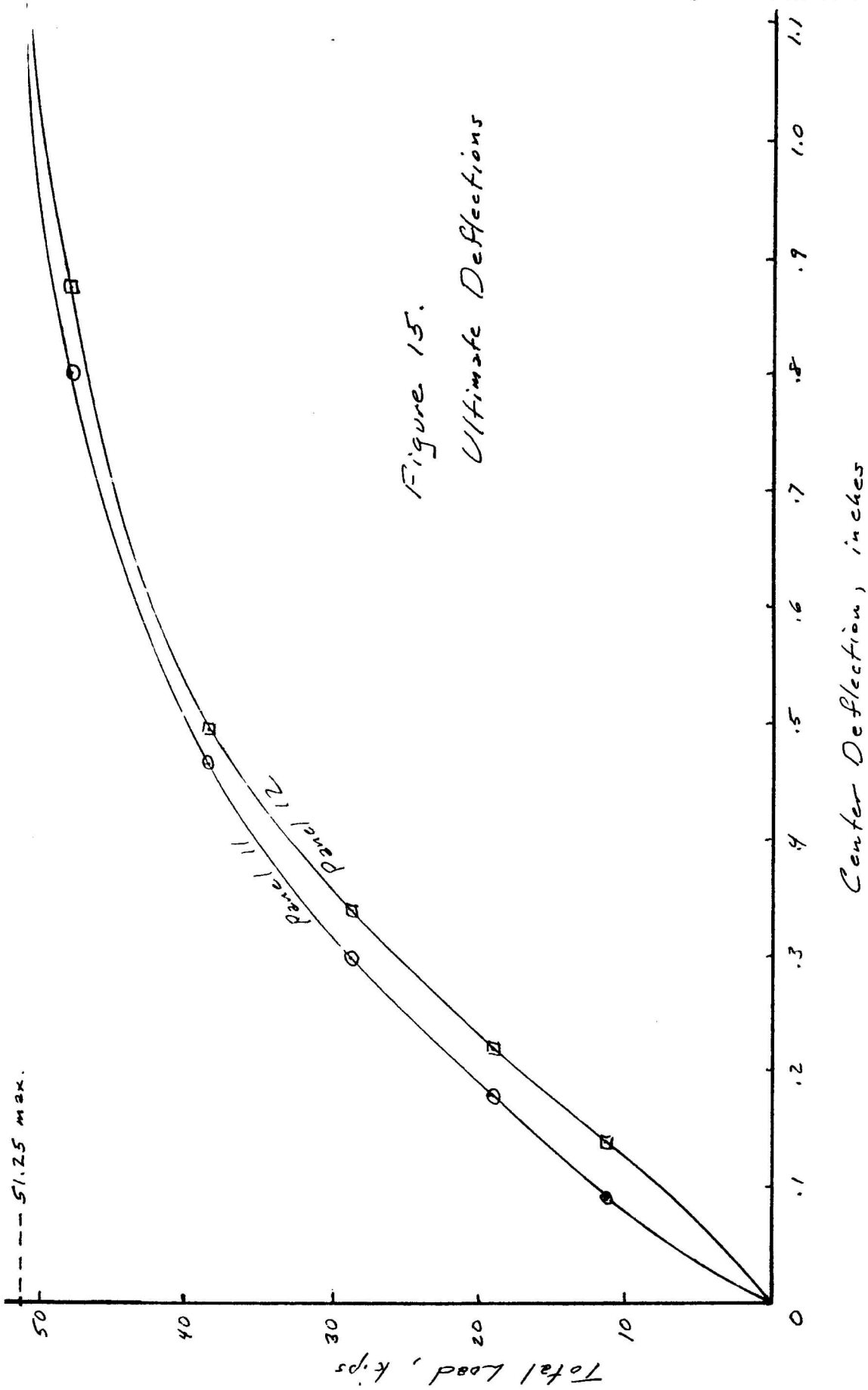



Figure 15.
Ultimate Deflections

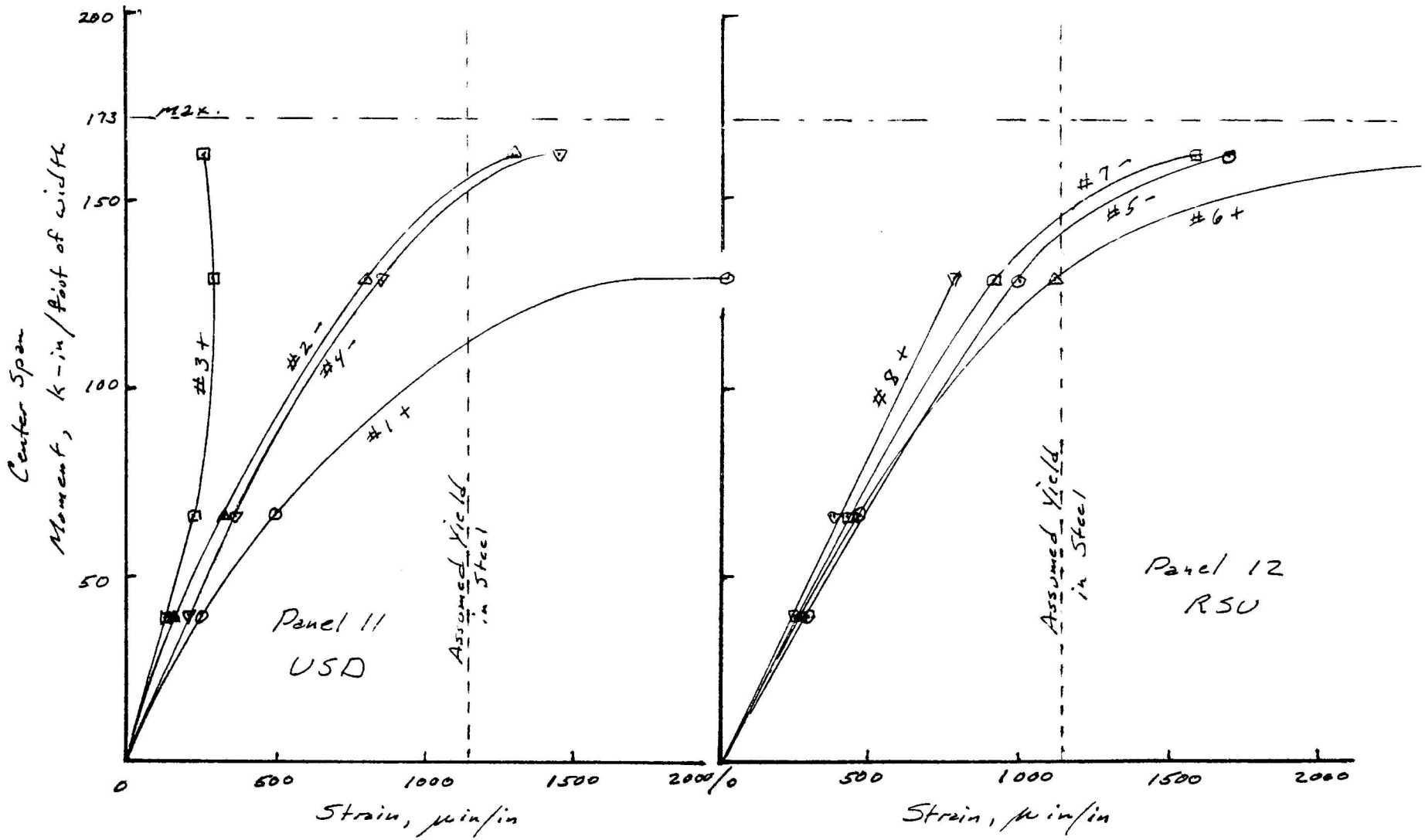



Figure 16. Ultimate Strains

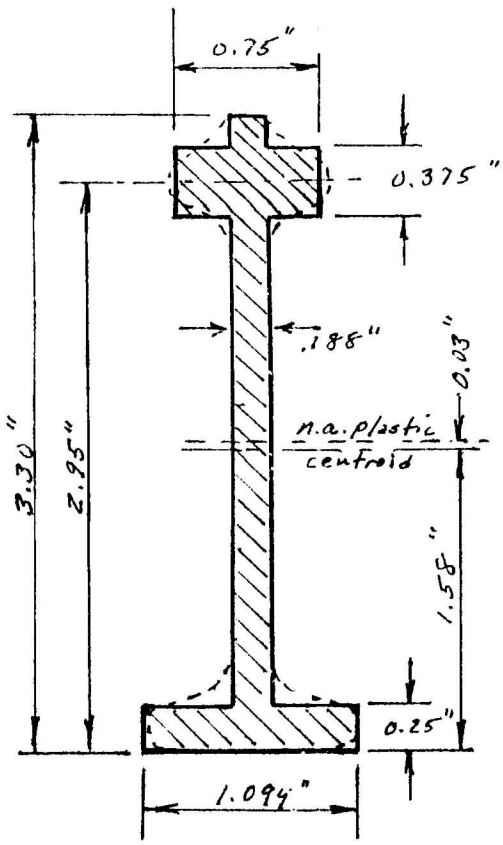
main bar section of Figure 3 has been approximated in Figure 17. The section properties derived therefrom are also shown. The neutral axis is estimated to be at 1.58 inches up from the bottom. The neutral axis of the composite section for Panel 12 as shown in Figure 13 is approximately at 1.62 inches. If the depth of concrete is taken as 3.2 inches its neutral axis would also be at roughly 1.6 inches. If the neutral axes of both components taken separately lie at the same location as the neutral axis for composite action, the two materials can be treated as acting independently.

Using the steel strains in Figure 12 (gages 5 and 6) it is possible to calculate stresses. With the stresses and the section properties in Figure 16 one can calculate the bending moment in one main bar. By subtracting that moment from the total bending moment per 4 inches of deck one can find the moment resisted by the concrete. Assuming a rectangular section for the concrete with cut-outs for the main bar section it is possible to calculate the section properties and the corresponding concrete stress due to the concrete moment. The ratio of calculated stress to measured strain gives the concrete elastic modulus with which the stiffness, EI, of the concrete can be found. Using the estimated moment of inertia of the main bar the total section stiffness can be approximated. A calculated deflection for the panel can then be compared with the measured one, which is done in Figure 17 with an apparent error of about 13%. Actually the error is probably greater because the calculated deflection, which is less, is based on a minimum cross-section while the measured one takes into account the average of properties over the entire panel. Approximate as they are, these calculations never-the-less show an internal consistency between the strain and deflection measurements. Included in Figure 17 is an estimate of the concrete strength, 6800 psi, based on the ACI formula for calculating E from concrete strength.

The deflections in Figure 15 demonstrate the typical load-displacement relationship for ductile materials. Panel 12 shows a significantly lower initial stiffness but later tracks Panel 11 quite closely. The initial difference was most likely due to the effect of the channel on the uneven surface of Panel 12 such that the central part of the panel where the dial gage was did not deflect as much until contact was made with the channel.

The strains in Panel 11 (Figure 16) show that compatibility between concrete and steel in compression continued almost up to ultimate. In tension the concrete (#3) actually ceases to participate while the steel (#1) takes more stress, reflecting the effect of the cracking of the concrete which became visible on either side of the gage as failure approached. Similarly, the compression gages in Panel 12 (5 and 7) remained compatible while 6 and 8 diverged. (Gage 8 actually went off scale before the last reading and it could not be determined which way it went.)

# Figure 17. Approximate Section Properties




## Elastic Properties Panel 12

@ 11.56 k total or 0.578 k/m.b.

Steel

$I_s = 1.443\ in^4$ $E_s I_s = 41{,}800\ k\text{-}in^2/m.b.$

$S_{top} = 0.840\ in^3$ $\epsilon_5 = 251\ \mu\epsilon$ @ center span

$S_{bot} = 0.912\ in^3$ $\epsilon_6 = 242\ \mu\epsilon$

$M_s = M_{AVG} = 6260\ \#\text{-}in/m.b.$

$M_{Total} = 13{,}000\ \#\text{-}in/m.b.$

Concrete

$M_c = 6740\ \#\text{-}in$

$I_c = 9.53\ in^4$ depth of concrete 3.2"

$S_c = 5.95\ in^3$

$\epsilon_7 = 229\ \mu\epsilon$ Avg $\epsilon = 239\ \mu\epsilon$ @ center span.

$\epsilon_8 = 249\ \mu\epsilon$

$\sigma_c = 1130\ psi$

$E_c = 4.7 \times 10^6\ psi.$

$E_c I_c = 45.1 \times 10^3\ k\text{-}in^2/4''$

Composite $EI = 81.0 \times 10^3\ k\text{-}in^2/4''$

Calculated Deflection 0.101"

Measured Deflection 0.114"

$f_c' = 4800\ psi$ based on the ACI formula.

## Plastic Properties

$Z_s = 1.13\ in^3$

$M_p = 37.3\ k\text{-}in/m.b.$ or $112\ k\text{-}in/foot$

Assuming $F_y = 33\ ksi$

In Figure 18 are shown the steel strain variations in Panel 12 at two loadings approaching ultimate. It is obvious from the shift of the neutral axis that the tension concrete is ceasing to participate and that composite action is using some of the steel to work with the compression concrete. It is impossible in these states to calculate any estimated properties as was done for the elastic case. However, by using the reduced main bar section as shown in Figure 17 it was possible to estimate a plastic section modulus for the main bar and hence an ultimate moment resistance for the steel alone. When compared to the total moment resistance at ultimate it was found that the composite section for both panels was about 50% stronger than the steel alone. Finally, the fractured states of both panels are depicted in Figures 19, 20 and 21.

## Conclusions

It is clear that the 50 year old deck has been behaving effectively as a composite material with completely compatible strain, at least in positive bending. The apparent loss of compatibility in negative elastic moment was more likely due to incipient adjacent tensile cracking than to debonding. If this is so then it may be concluded that both panels behaved in identical fashion right up to ultimate strength with no evidence of debonding.

The elastic properties of the steel grid were approximately doubled by the addition of the concrete. In part this increase may be due to a particularly high strength concrete (for bridge decks) though the strength may not have been that high during the early life of the deck. Under certain circumstances concrete strength has been known to increase slowly but indefinitely over time. The ultimate strength of the filled deck appeared to be about 50% greater than the plastic strength of the steel grid alone.

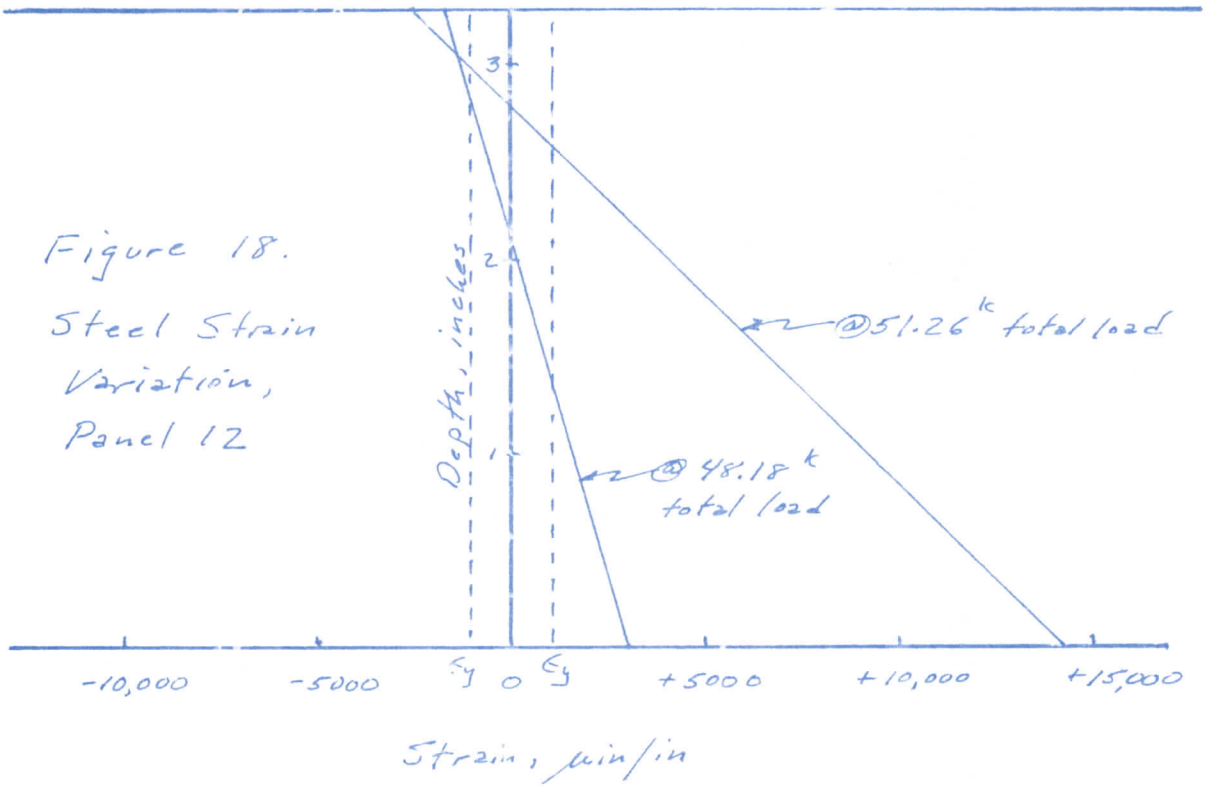



Figure 18. Steel Strain Variation, Panel 12

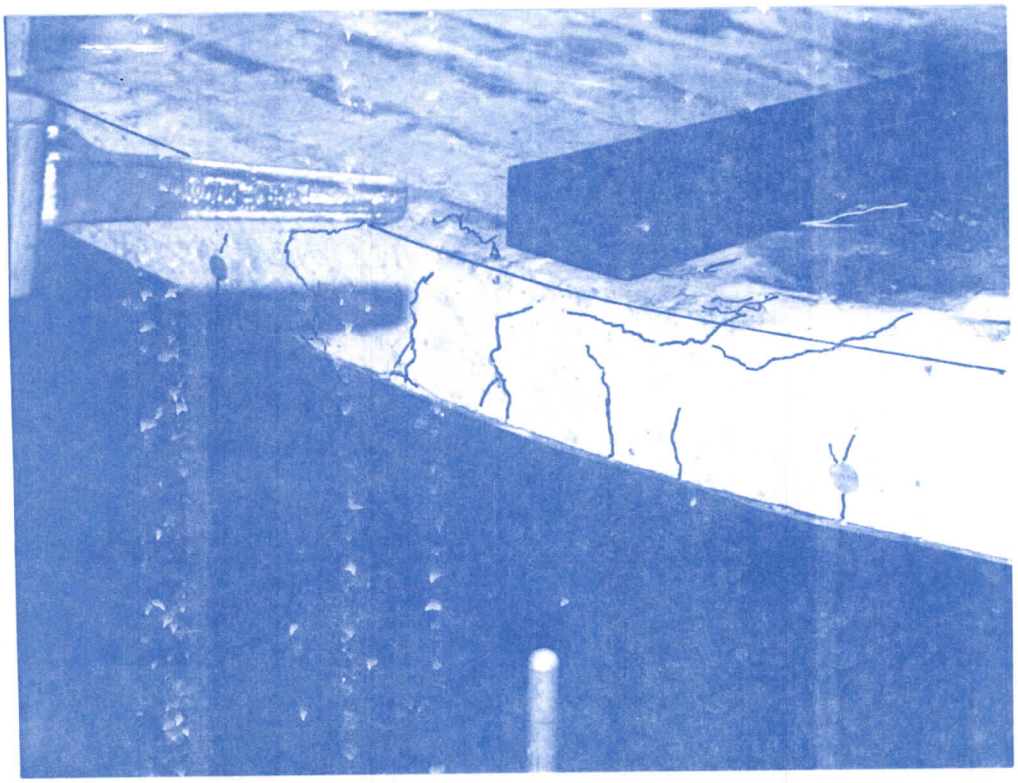


Figure 19. Cracking in Panel 12 (enhanced)

Figure 20. Cracking in Panel 11
(enhanced)

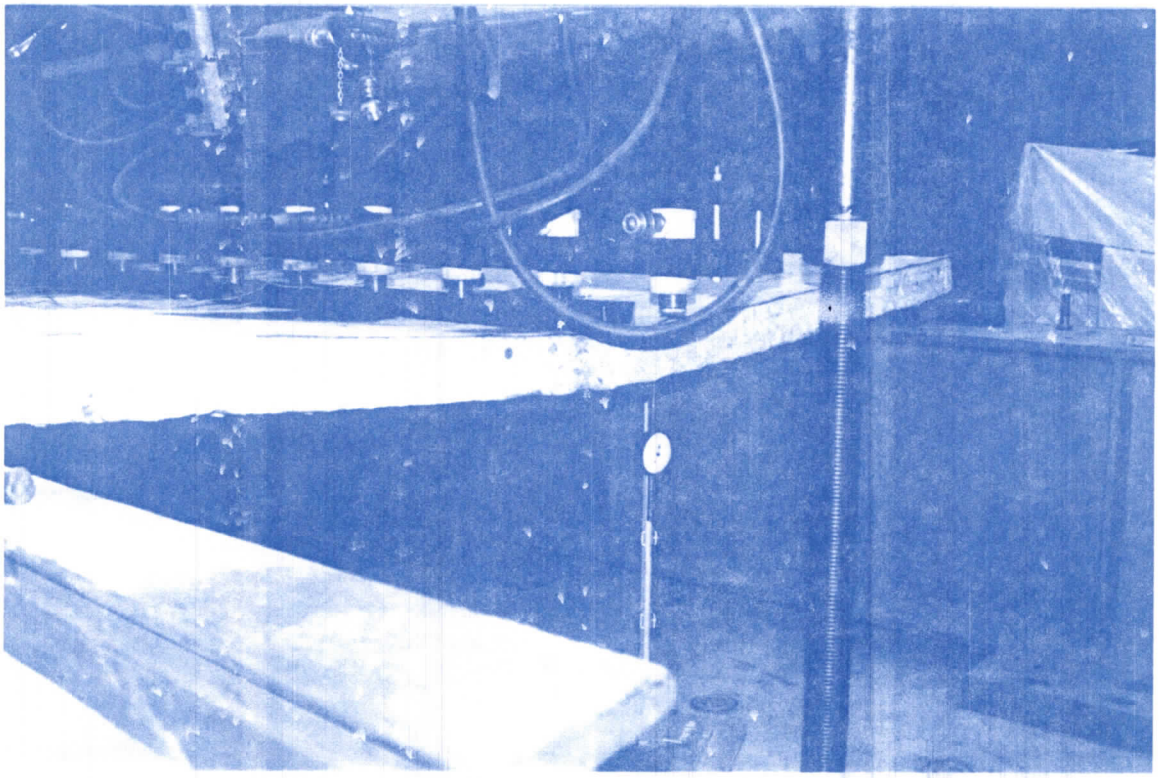


Figure 21. Final Deflection of Panel 11