TEST OF UPPER ANCHORAGE OF #14S COLUMN BARS

IN PYLON DESIGN

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P R E F A C E

This report is on one phase of the general project "Splices and Anchorage of Reinforcing Bars." It covers a test of an upper anchorage detail used for the vertical column steel (#14S) in a pylon design. For this test it was necessary to cast a full scale specimen (in two dimensions) representing a slice one foot thick out of the six-foot pylon thickness.

The test was conclusive in indicating the anchorage was adequate even without extending it through the top beam steel. It was adequate even for A432 steel bars rather than the intermediate steel used in construction to date.

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

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S U M M A R Y

A test run on a full size model representing a one-foot slice through the upper part of a pylon showed that #14S column bars of A432 grade steel could be adequately anchored in 4200 psi concrete for their full yield strength in tension produced by unbalanced live load, even without extending them through the top beam tensile reinforcing steel. The failure was in flexure (plus direct stress) in a manner giving both warning and toughness.

THE PROBLEM

Objective of Investigation

Anchorage of large bars can raise questions as to the behavior under the actual varying stress conditions present over the extreme required length. The upper end anchorage of the vertical #14S bars in the single column multiple level pylon bent used in expressway interchanges raises such a question. Large tension forces must be carried by these bars just below the balanced cantilever arms which support the unbalanced live load on the roadway. The upper part of the anchorage may be weakened because it falls where flexure cracks in the cantilever beam tend to open along the plane of these bars. The lower part of the anchorage is improved by perpendicular flexural compression from the same cantilever beams. The net effect of two such diverse factors is uncertain. This test was set up to explore the specific case in more detail and to modify design procedures if such seemed desirable.

Pertinent details of such a pylon bent with #14S bars as the main column steel are shown in Fig. 1. The particular objective of this investigation was to assess the adequacy of the top anchorage of the vertical #14S tension bars into the upper cantilever beams.

Scope of Investigation

Only one model test specimen was made, although the program provided for a second in the event significant questions remained after the first test. Since bond strength was heavily involved, only a full size model with regard to the bars could be trusted. Hence a model could be used only in the sense that the thickness used for tests was not the full thickness of the construction in the field.

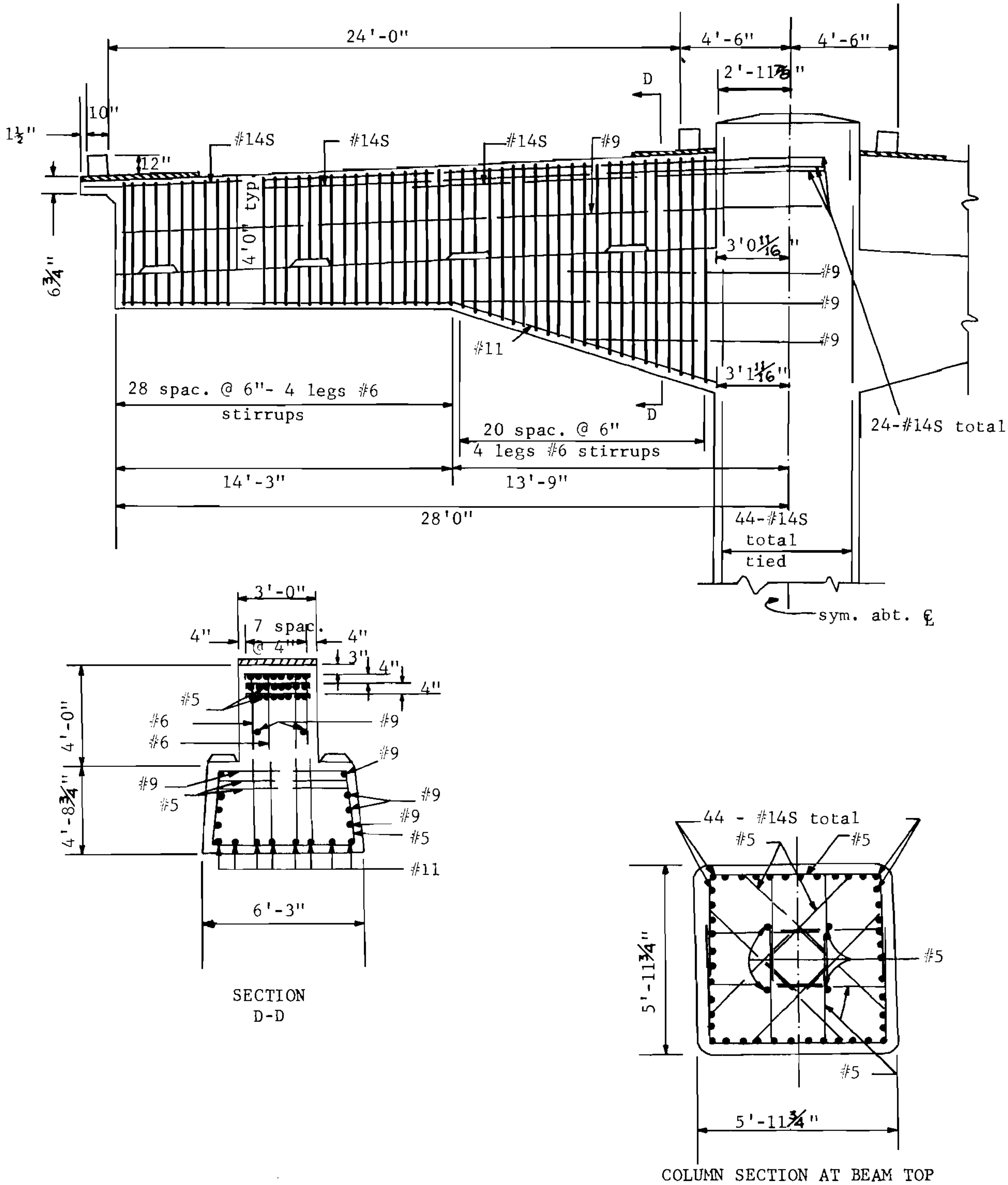


Fig. 1. Details of pylon (prototype).

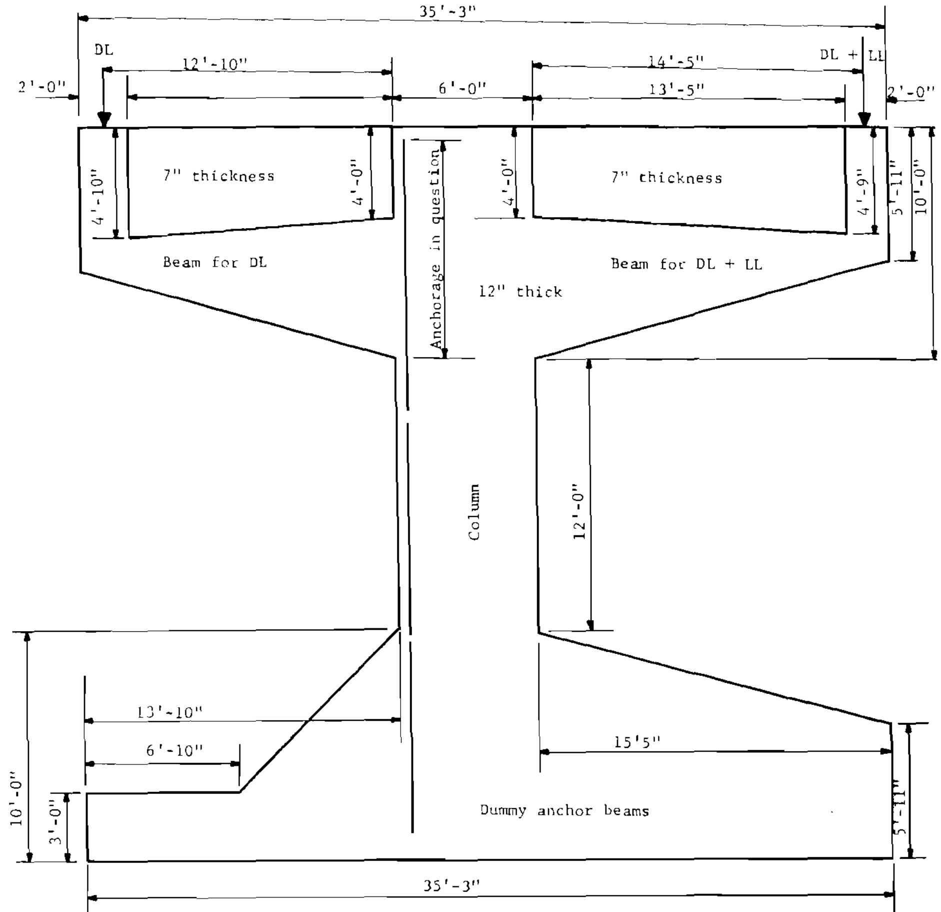
Test Specimen

A slice one-sixth the thickness of the balanced cross beams and the upper part of the pylon was constructed to full size, with a dummy anchor beam built as the lower part of the specimen, as shown in Fig. 2. The cantilever beams were constructed without a sloping top and modified as to length to permit a single concentrated load to give essentially the proper ratio between moments and shears at the joint. The entire specimen was cast in a horizontal position, raised and placed on rollers, and tested still in the same horizontal position. "Vertical" loads were applied by jacking the cross beams and anchor beams toward each other by means of hydraulic rams reacting on strap-type loading frames. The details of the test specimen are shown in Fig. 2b.

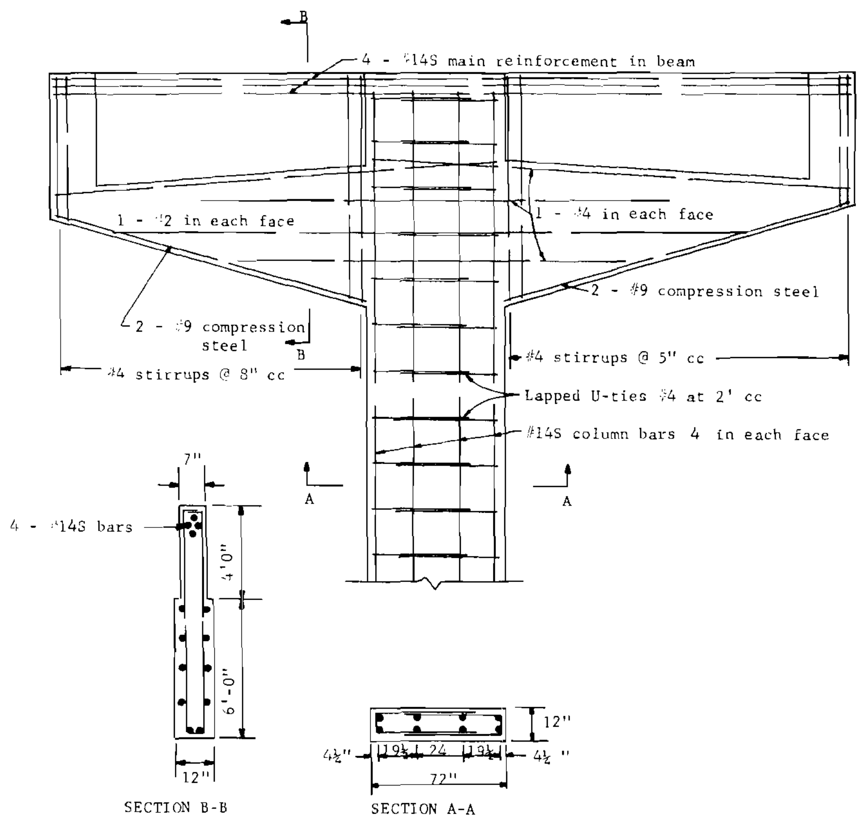
The test specimen, shown before concreting in Fig. 3, departed in several details from being an exact model of the actual pylon. First, the #14S vertical bars were stopped 9 in. below the top of the beam. Second, the end of each cantilever was thickened to receive a top load from the test jacks and the transverse steel which would have been in the beam brackets was omitted because they were not loaded. Third, the reduced thickness of the upper part of the transverse beam, as compared to the column thickness, was made 7 in., whereas an exact model would have been 6 in. This gave more placing room for the concrete around the big transverse bars. Also, the web steel in the transverse beam was redesigned for the larger expected overloads which the model might be called on to handle. These changes did not extend into the joint which was being tested and should not have influenced the behavior of the joint in any fashion.

In one way the test was deliberately changed from the prototype. The #14S bars were used as A432 steel with a yield strength (at 0.005 strain) of 62.7 ksi, instead of intermediate grade as designed.

In the column #4 bars were used for ties rather than #5 bars in the prototype. To the extent that the longitudinal bars influence tie stress, the #4 ties would be more representative of the number of bars



(a) Outline showing location of critical bars.



(b) Detailed reinforcement.

Fig. 2. The test specimen.

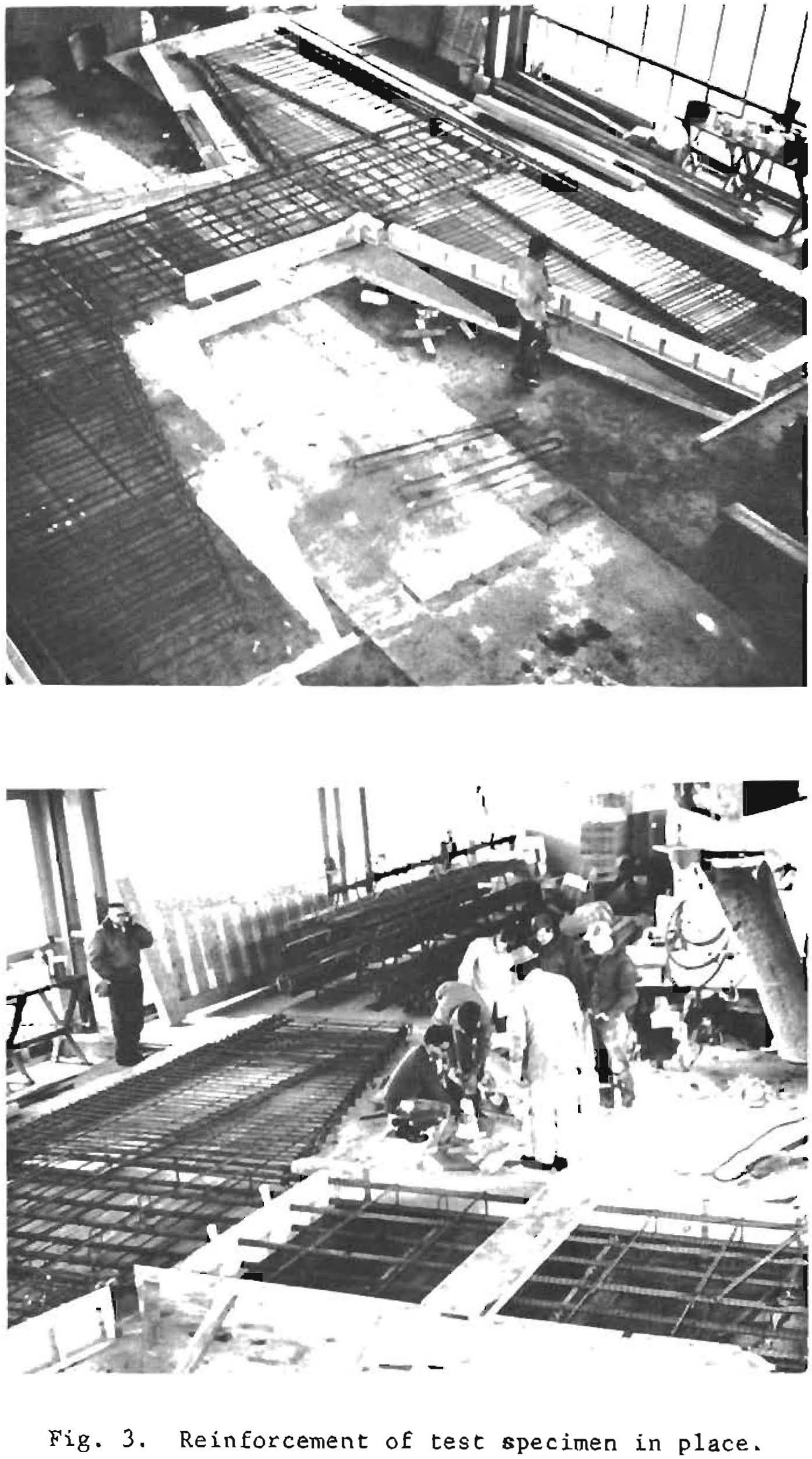


Fig. 3. Reinforcement of test specimen in place.

used. Strain gages were mounted on one short leg of the "upper" four column ties, within the beam depth along the critical anchorage.

Electrical resistance strain gages were attached to each #14S bar on the tension side of the column, 11 on each outer bar and 7 on each inner bar, as diagrammed in Fig. 4a. These gages were distributed over the entire anchorage length of these bars and about a foot into the top of the column below the transverse cantilever beams. The leads were taken out through the compression face of the beam in such a manner as not to influence the test results (Fig. 4b).

Concrete made with Type III cement was placed from a transit mixer and the specimen was cast in two parts, with a construction joint just below the transverse beams as shown in the foreground of Fig. 3, to simulate construction in the field.

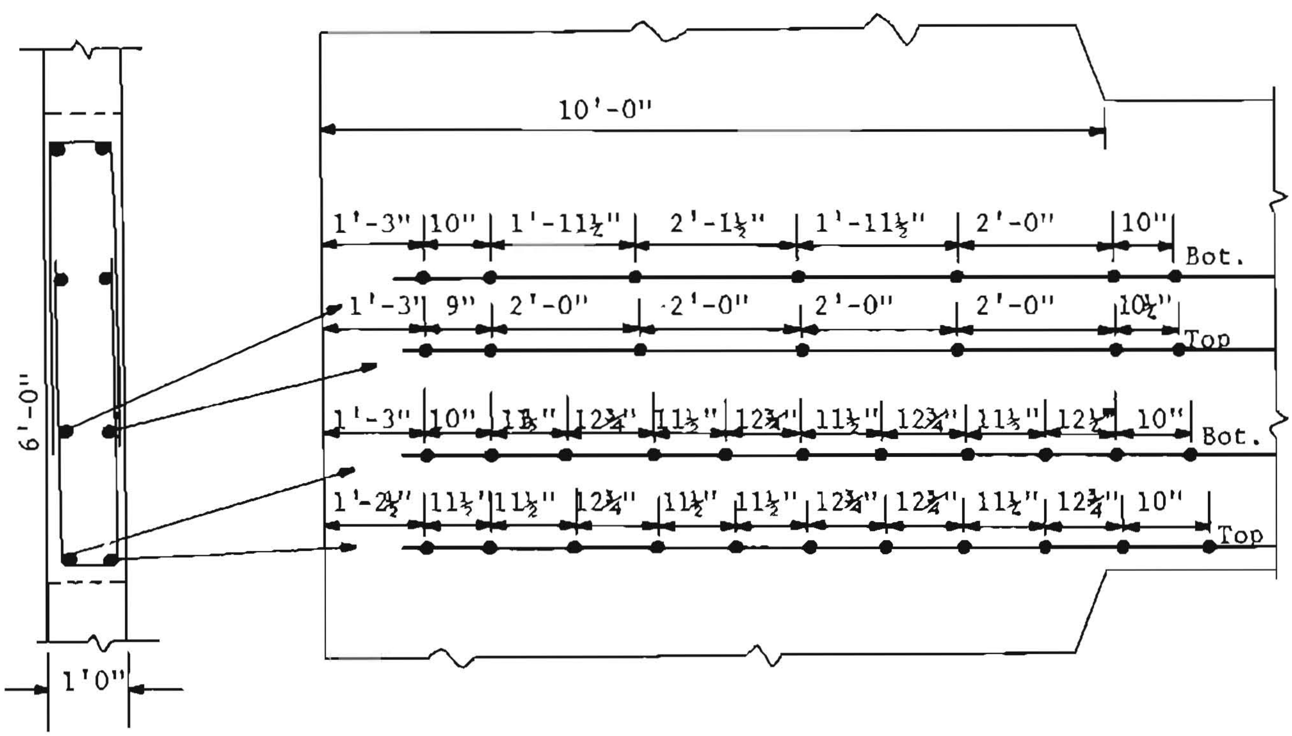
TEST PROCEDURE

Preparation for Test

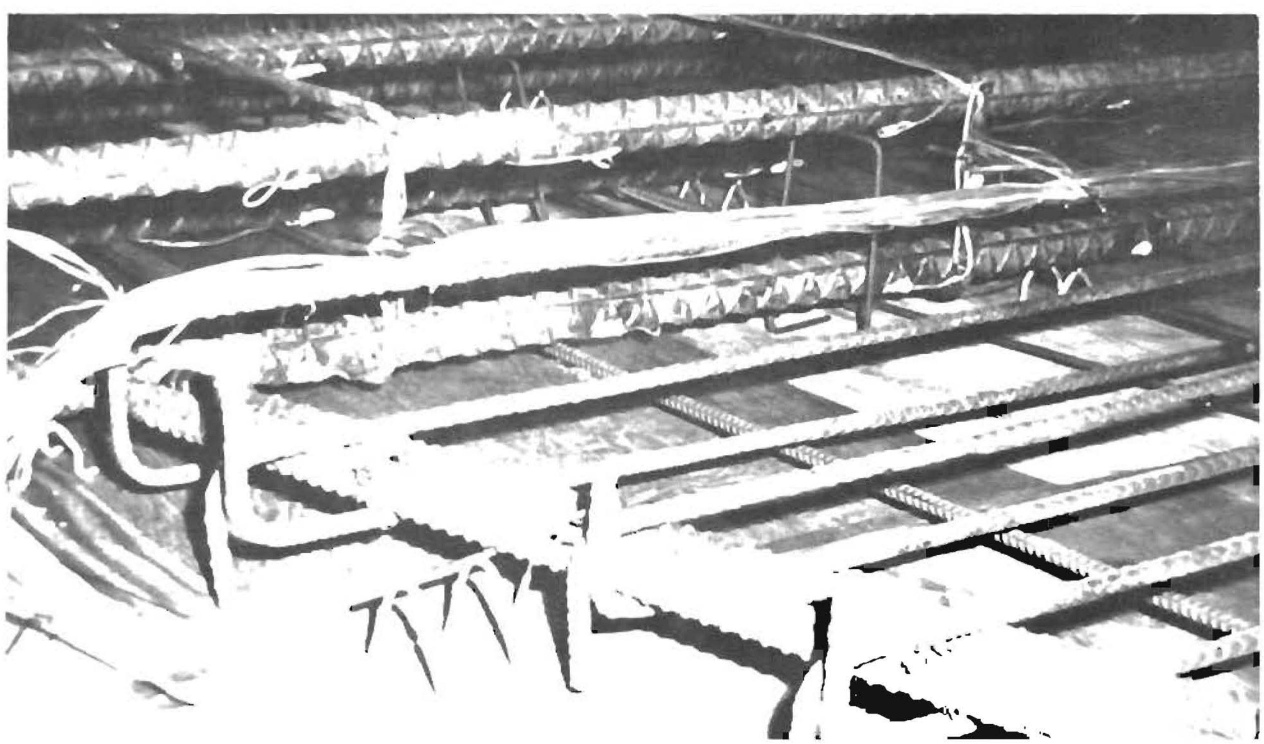
The test specimen, after curing, was lifted from the floor at five days of age, using as a strongback a pair of steel channels bolted through the specimen. Lifting was carried just far enough to permit the placement of 7-in. rollers made from concrete-filled steel pipe, as shown in Fig. 5. One of the loading cantilevers was cracked by an accidental torque loading during lifting, as shown by the cracks specially noted in Fig. 9. These cracks had no observable influence on the failure; the loading arm functioned precisely as planned.

The lifting rods are still in place in Fig. 6, which shows the general setup for the test. The steel straps in the foreground and in the background are the loading straps, with the load applied by jacking between the cantilever arm and the channels which form the verticals of the loading yoke, as shown in Fig. 7. The single jack was at the lightly loaded end and the double jack at the heavy end, with both loads controlled through load cells.

The transverse cantilever beams were cracked in flexure prior to the test proper by loading to the equivalent of dead load plus 1.25 live loads



(a) Location of strain gages.



(b) Lead wires coming out through the beam.

Fig. 4. Strain gages on reinforcement.

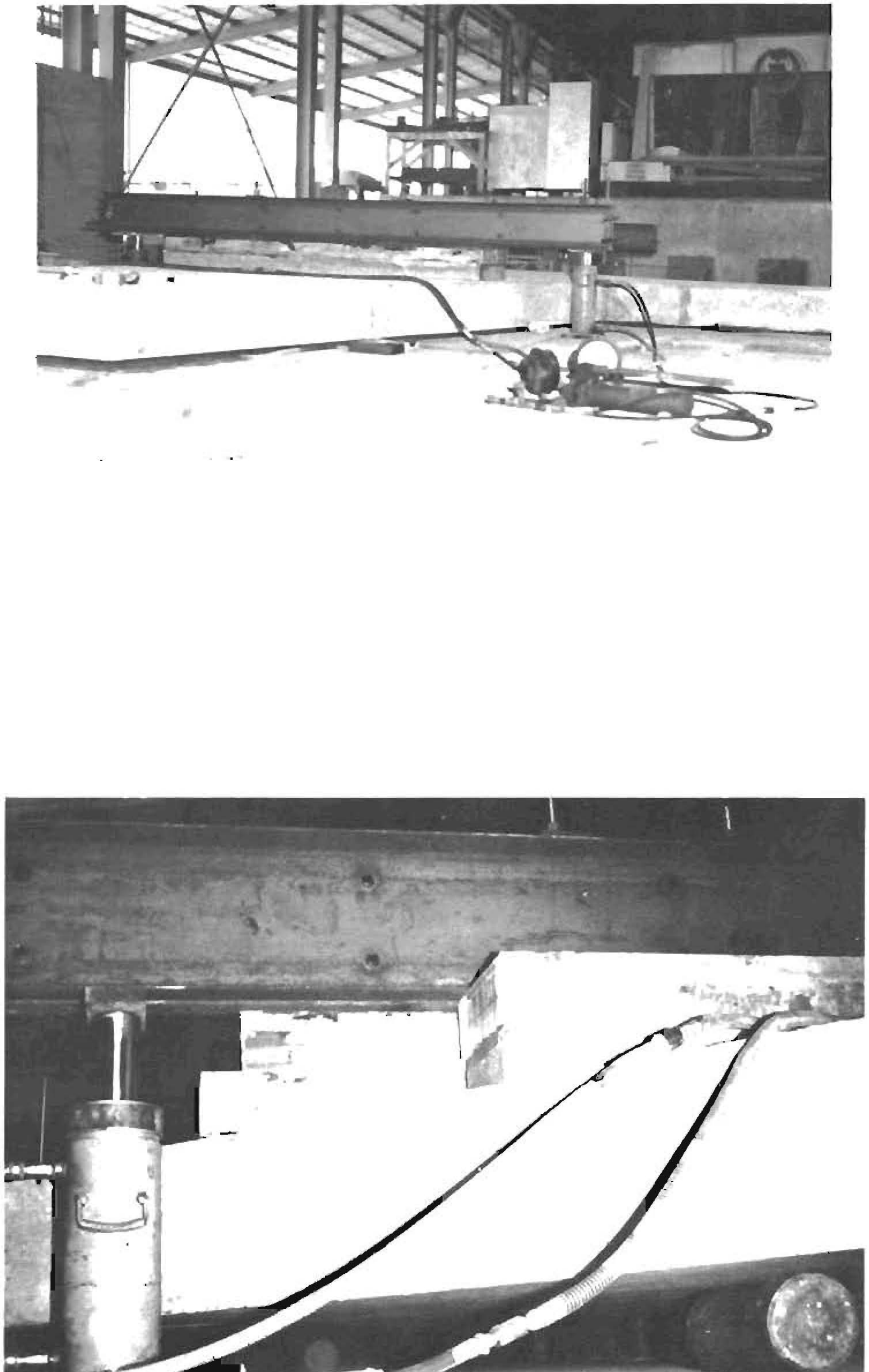


Fig. 5. Lifting of the specimen and placing on rollers.

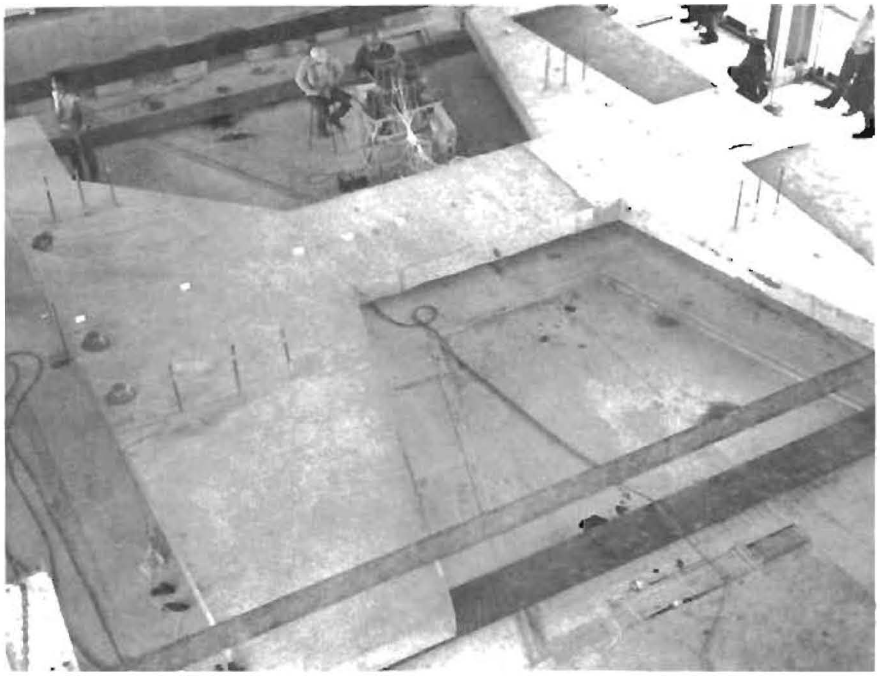


Fig. 6. Overall view of the test.

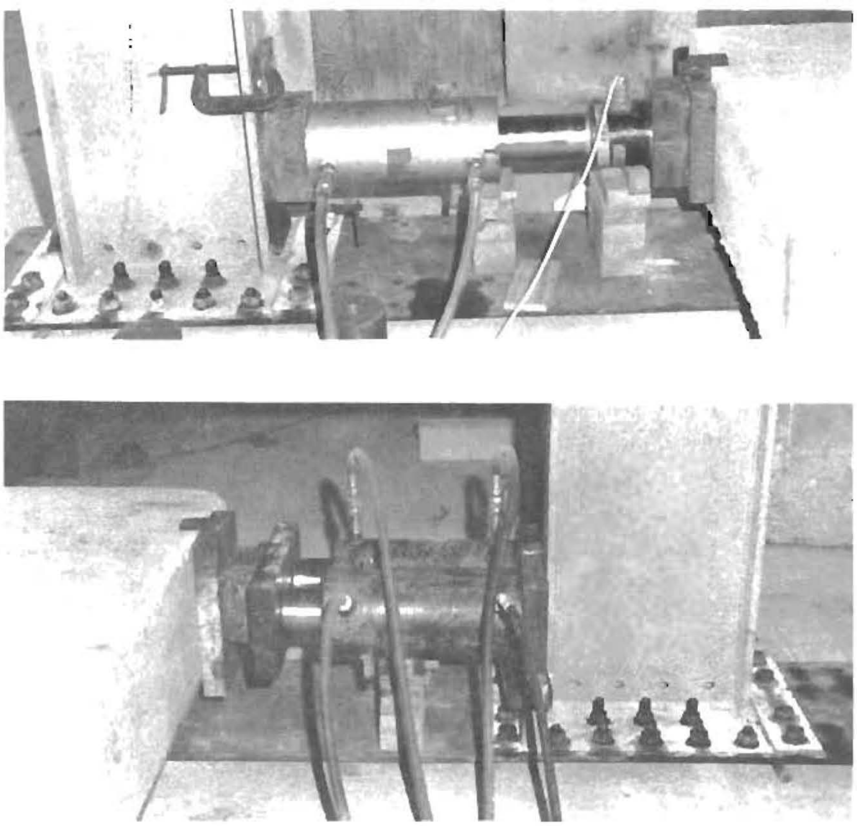


Fig. 7. The jacking arrangements.

(DL + 1.25LL),* and then unloaded. For the test proper, the equivalent of dead load was applied to each arm and then the right arm was loaded progressively by steps up to the equivalent of dead load plus 5.5LL. At each loading stage the strain gages on the bars were read and cracks were marked on the surface of the specimen as they had occurred.

At 5.5LL the loading system at the heavily loaded end became unstable and released the load, bringing the first part of the test to an end. The strains still remaining in the bars when the specimen was unloaded were recorded.

Four days later, after some correction to the loading system, the testing was resumed in the same manner except with slightly larger load increments. This brought the heavily loaded beam to a level of dead load plus 6.75LL before failure occurred, equal to 2.54(DL + LL). The failure was the result of the yielding of the tensile steel in the column and the subsequent secondary compression failure at the opposite face of the column above the construction joint. The crack pattern in the joint at this stage is shown in Fig. 8, where the crushing of the concrete at the right foreground indicates the start of the compression failure. The loading, continued further at no higher level, finally resulted in breaking off the compression concrete outside the compression steel as shown in Fig. 8b.

The final crack pattern over the joint is shown in Fig. 9. One can note the vertical splitting which occurred over the lower half of the left tension bar of the column, where it was anchored into the beam. It will be noted, however, that this does not continue all the way to the top even at this last stage. There was no bond or anchorage failure.

One of the practical difficulties of this test was the removal of the specimen. It had to be broken up into truck-sized units as shown in Fig. 10.

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*Throughout this report live load is considered as the sum of the live and impact loads and is noted by the abbreviation LL.

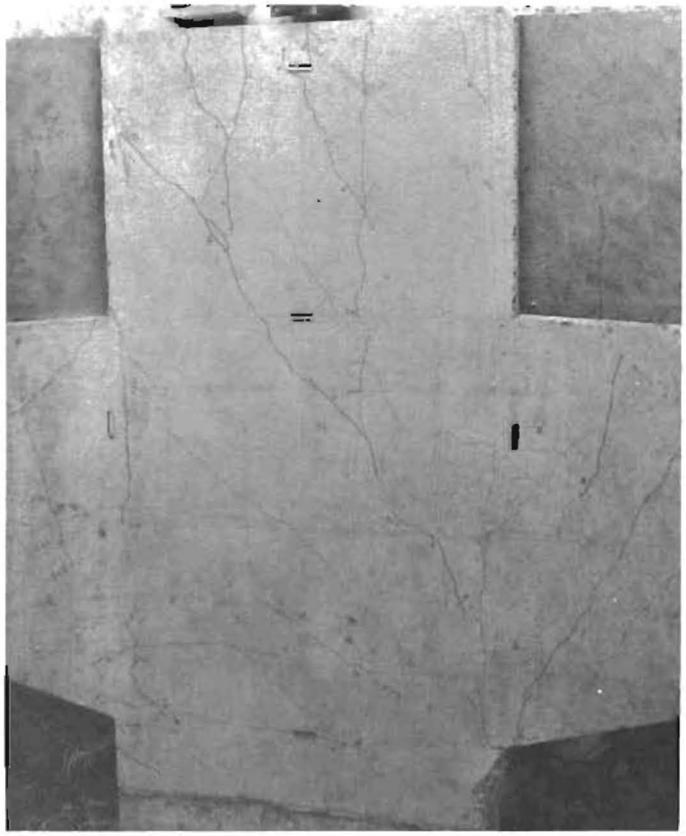

Fig. 8(a). Cracking pattern at the junction of the pylon and crossbeam.

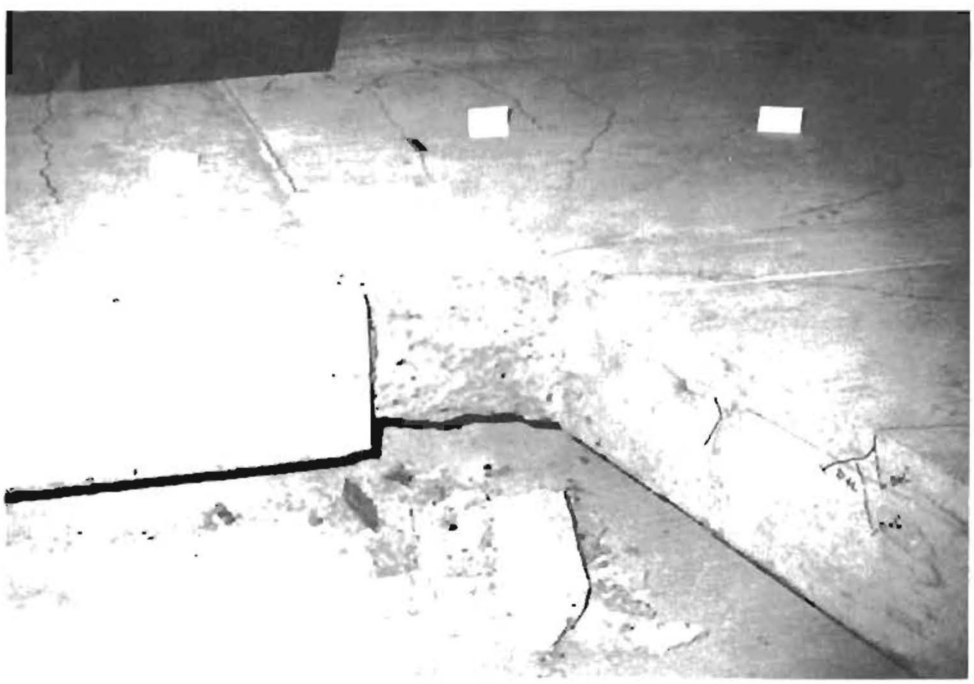

Fig. 8(b). Final failure in secondary compression after yielding of steel in tension.

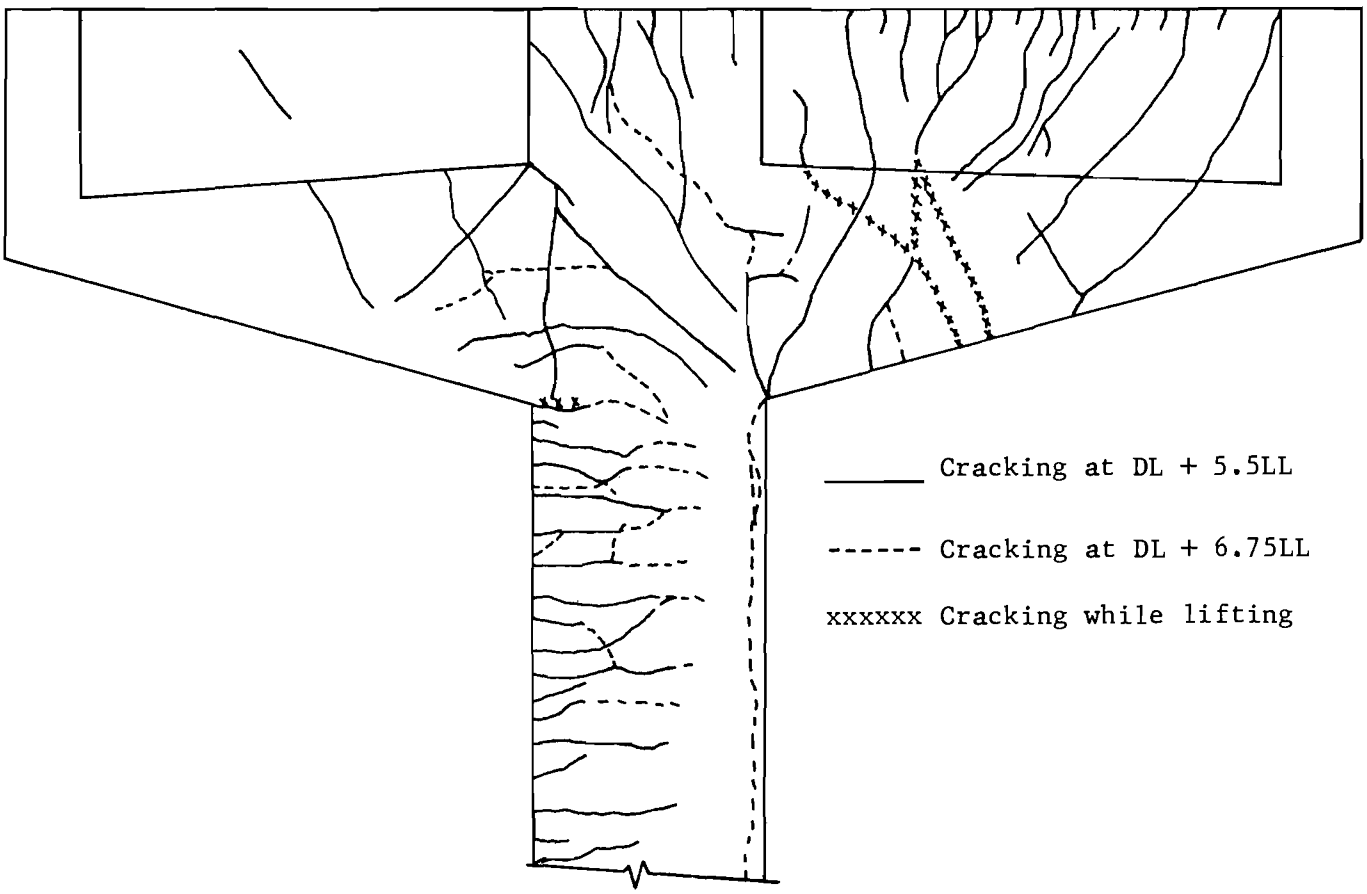

Fig. 9. Crack pattern of the specimen.

Fig. 10. Specimen broken up for removal.

DATA AND ANALYSIS OF TEST RESULTS

Overall Factor of Safety

In analyzing the test the following must be kept in mind.

1. In the test specimen the main bars in the column and the cross beam were of A432 steel, whereas in the structure they were specified as intermediate grade. This change permitted the study to go beyond the normal loading implied in the design.
2. Shear failure of the arms of the cross beam was avoided by added stirrups.

The primary purpose of the test was to check the anchorage (or development length) of the #14S tension bars in the pylon column where these extended into the upper cross beam. With $f'_c = 4200$ psi, the construction carried dead load on each arm and 6.75LL* on the right arm without any failure in anchorage or development length. This load was equivalent to 2.54(DL + LL). The actual failure of the column was in secondary compression after the A432 #14S bars had yielded. Although the strain gage record is not as good at this level as might be desired, it appears that the outside tension bars were stressed to at least 65 ksi and at least one of the inner bars (at about the quarter point of the depth of the column) was at 58 ksi stress.

The bent was designed for intermediate grade steel. At DL + 4.5LL, equivalent to 1.97(DL + LL) the steel stress in the outer bars was 43 ksi and in one of the inner bars was 37 ksi. This indicates

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*At face of column (on the 1-ft. thickness) the design moments were:

DL M = 1122 KF
LL M = 430 KF (including impact)

an overall factor of safety of nearly 2 for a design based on intermediate grade steel.

Although there was some splitting (Fig. 9), it is not evident that the anchorage was very seriously in danger even at the ultimate load.

Strain Gage Data

The chief subsidiary data were strain gage readings on four bars, the two in the outside tension face of the column and two adjacent bars at about the quarter point of the depth of the column. These are interpreted as stresses and shown in Figs. 11 through 14. The data in each case are in two parts, an upper set of curves which represents the first loading (before it became unstable) and the lower set of curves the data taken during the final load test, including the locked-up strains which existed after removal of the first loading. These curves are discussed in the next section.

Interpretation of Bar Strain Data

The stresses in Figs. 11-14, inclusive, were obtained by matching the observed strains against the stress-strain data of the bars shown in Fig. 15. The data around gages 6 and 7 (Fig. 11) are questioned, since it is not reasonable to expect either gage to unload as the external load increases as these appeared to do. Some trouble also exists with the last reading at gages 10 and 11, which would be the critical ones. However, it is obvious from the data at gage 9 that the steel has reached the yield stress and at gages 10 and 11 must be more highly stressed. The bulge in the stresses in the neighborhood of gage 5 probably occurs because of a diagonal crack which opened in that vicinity and pointed radially toward the inner corner between the compression side of the column and the heavily loaded beam. The data of Fig. 12 appear more consistent, but are marred by the loss of readings near the maximum loading.

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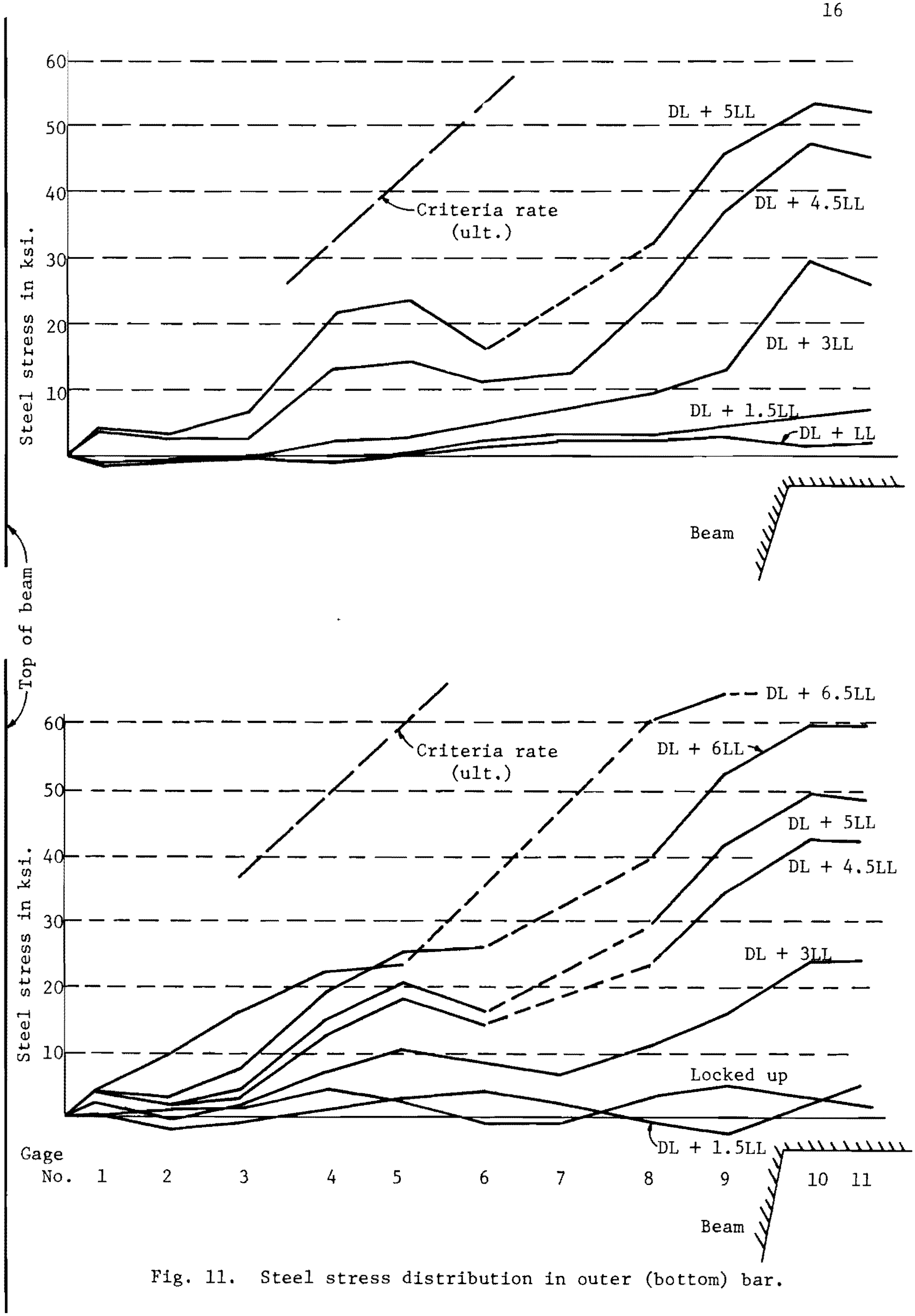

Fig. 11. Steel stress distribution in outer (bottom) bar.

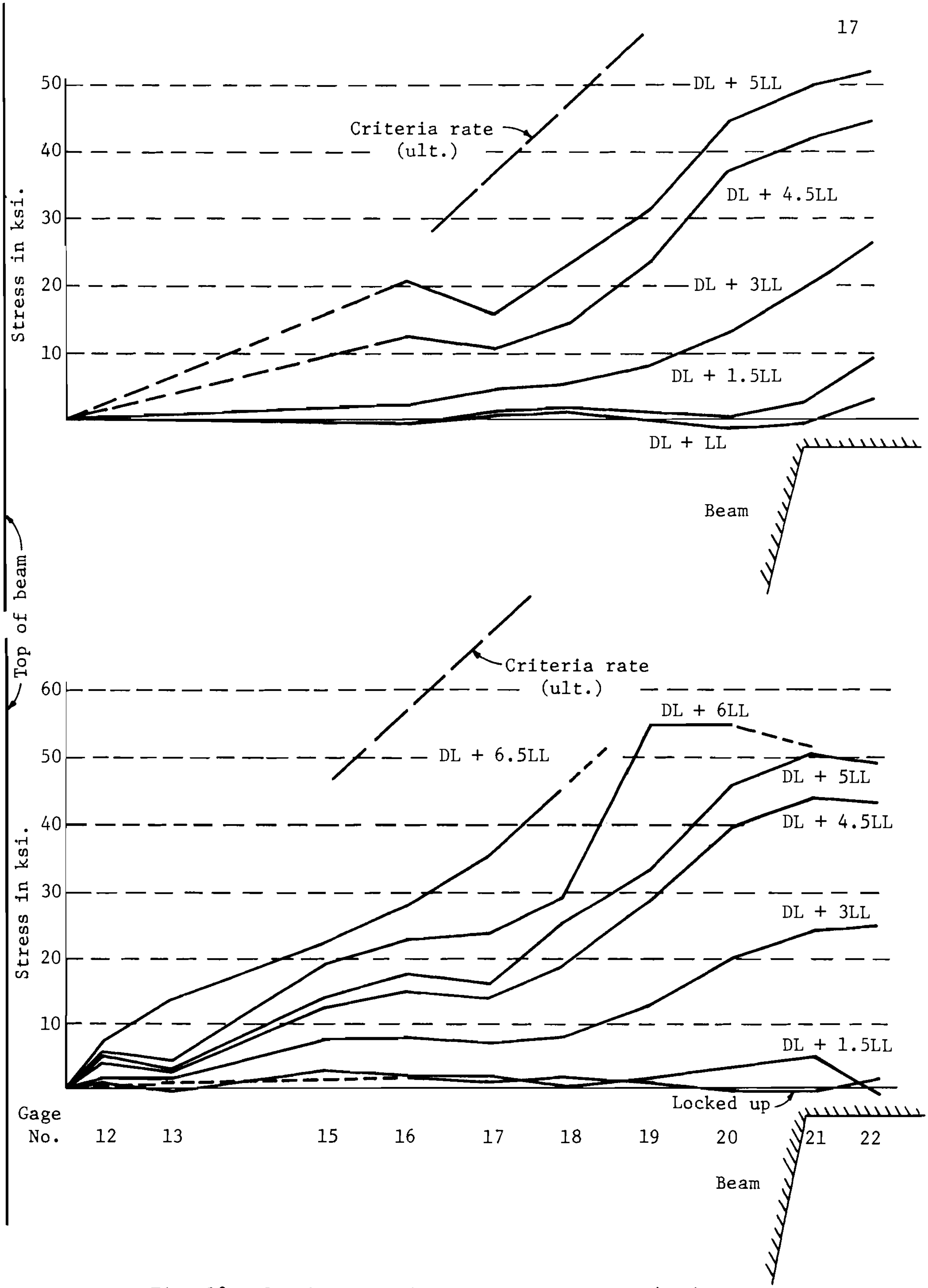

Fig. 12. Steel stress distribution in outer (top) bar.

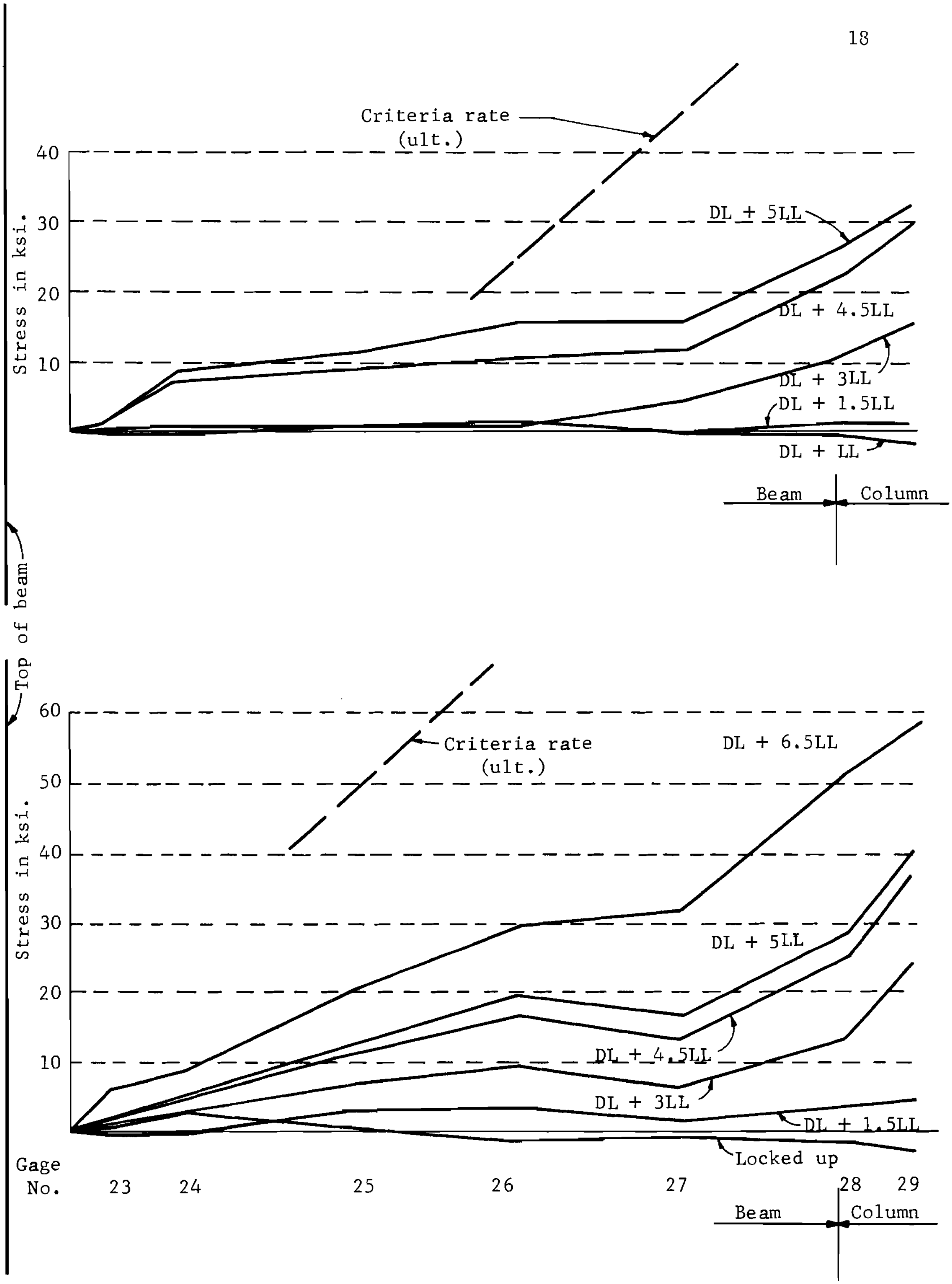

Fig. 13. Steel stress distribution in inner (top) bar.

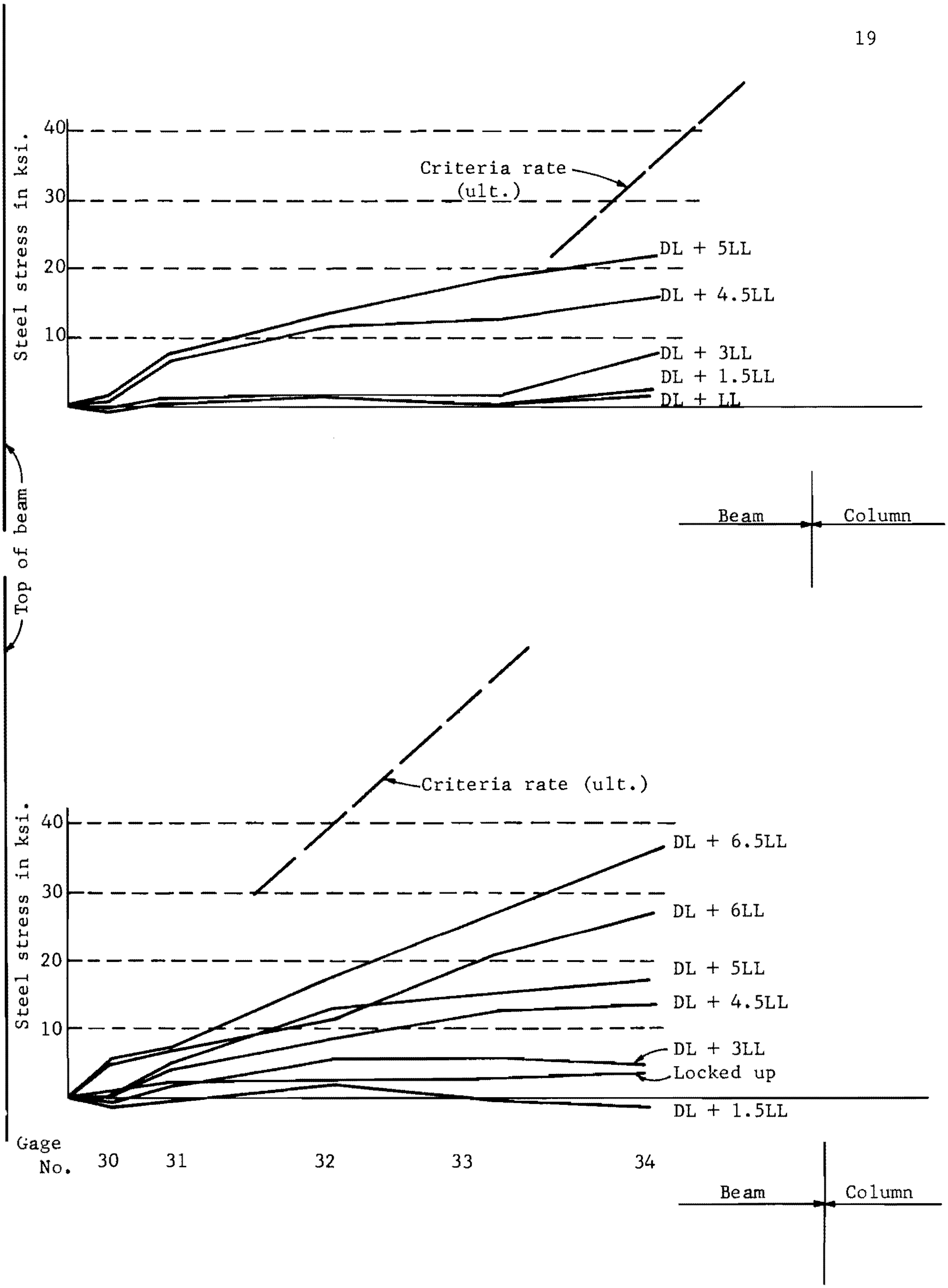

Fig. 14. Steel stress distribution in inner (bottom) bar.

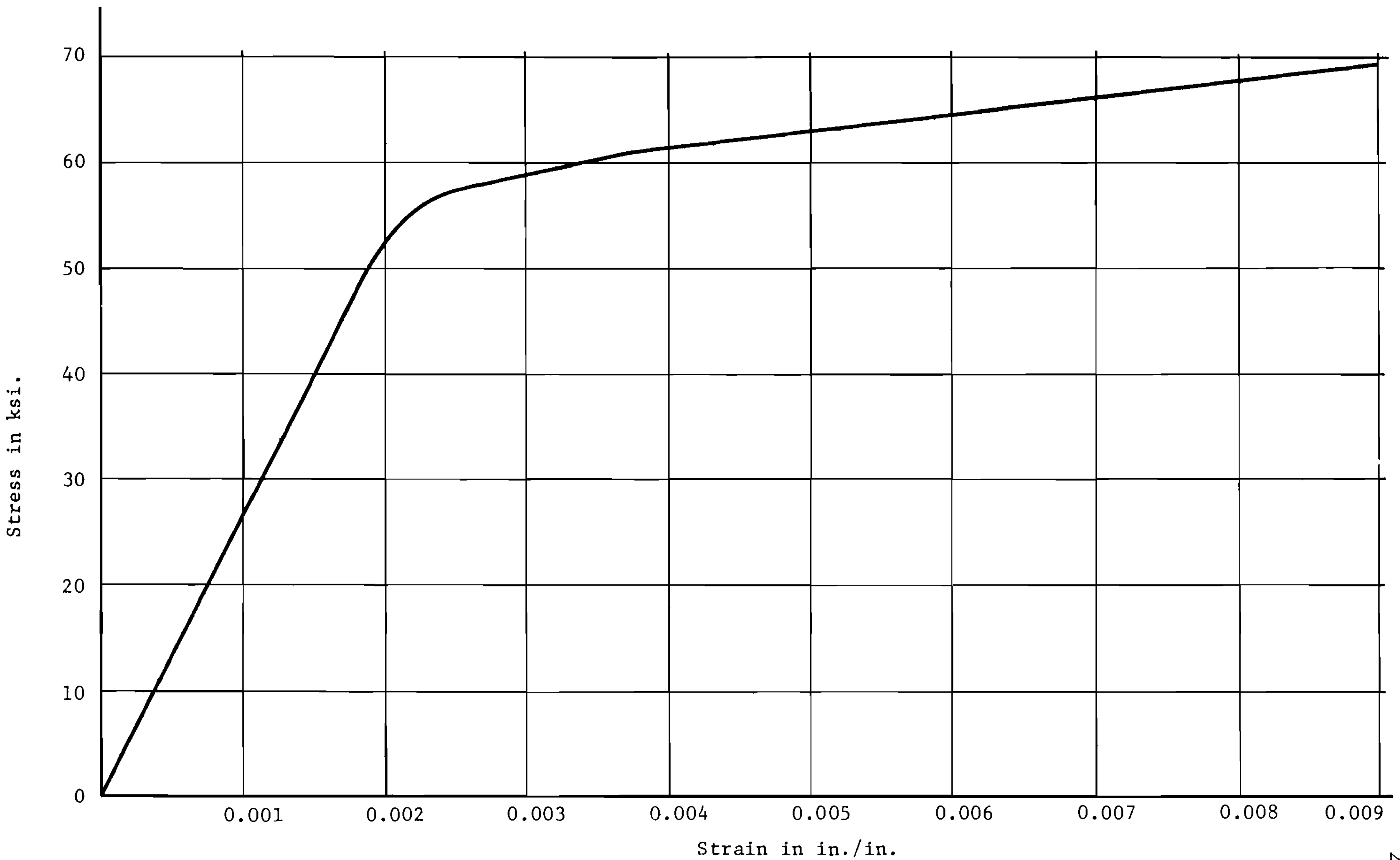

Fig. 15. Steel stress-strain curve for #14S bar.

The stresses on the bars at about the quarter point of the depth from the tension face (Figs. 13 and 14) appear more consistent, although those in Fig. 14 lack readings of critical gages near the bottom of the beam. The stresses in Fig. 13 drop substantially as the bar leaves the simpler stress pattern at the top of the column and enters into the more complex joint area.

So far as end anchorage is concerned, it would appear that these inner bars could be stopped off somewhat shorter than the outside bars, possibly at about two-thirds of the anchorage length of the outer bar. However, the longer length does serve a very necessary function in controlling the diagonal cracking which occurs within the joint, marked by the bulge in stress at station 26. The crack pattern of Fig. 9 indicates the need for a reinforcing grid throughout the joint, or area common to the beams and column, to control cracking.

Effective Bond Resistance

The design stresses in bond which establish the required anchorage length are the same in the Bureau of Public Roads' criteria for ultimate strength design* and in the 1963 ACI Building Code, namely, $6\phi\sqrt{f'_c}$. For these bars (and $\phi = 1$ for a known f'_c of 4200 psi) this stress predicts the rate of change of bar stress indicated by the slope of the lines marked "criteria" in Figs. 11 through 14. The inner bar of Fig. 13 reaches this rate only in the short section that was gaged below the beam; Fig. 14 does not cover the corresponding length. The exterior bars (Figs. 11 and 12) on the first unbalanced loading reached approximately this rate of change in stress and in the final loading seem to have exceeded it at least locally.

Although this degree of correlation might look encouraging, little generalization is possible because of the following:

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*U. S. Department of Commerce, Bureau of Public Roads, Strength and Serviceability Criteria, Reinforced Concrete Bridge Members, Ultimate Design (Washington, D.C.: U. S. Government Printing Office, August, 1966).

1. No bond failure (only some splitting) occurred; one can only guess how closely failure was approached.

2. The bars were closely spaced, #14S at 6-in. centers, which logically might lead to low bond resistance (low splitting resistance).

3. The beam compression was a compensatory factor which would reduce the net splitting tension.

Since roughly one-half of the anchorage length had split at the highest loading, this suggests that these two effects may simply have offset each other and led to a normal bond resistance, possibly close to ultimate, possibly not. This was a test of a specialized condition. It is valid only in interpreting similar situations qualitatively.

Tie Stress

The stress in the short leg of the #4 ties parallel to the tension face was low within the #14S anchorage length even at ultimate load, a maximum of 18.8 ksi. The ties seemed not to be critical as to strength anywhere, but it was noted that a column flexural crack did open over each tie in the column proper. Cracking over stirrups or ties has been frequently noted in tests of flexural members.

C O N C L U S I O N S

The investigation covered a special anchorage detail and cannot be generalized. However, the objectives were met and the test showed:

(1) The upper anchorage detail of the #14S vertical bars was adequate in 4200 psi concrete, not only for the design with intermediate grade steel, but also for A432 grade

steel. This conclusion is based on stopping these bars just under the main transverse beam steel.

(2) The column failure was a secondary failure in flexural compression after yielding of the tension steel, the most desirable type of failure with adequate warning and toughness.

(3) The cracking in the joint, or area common to beam and column, justified a grid of reinforcing at least equal to that furnished by ties and column steel.

(4) The reasonably tight spacing of #14S bars (6-in. centers), which might cause weakness in splitting, seems to have been about offset by the flexural compression from the beams. Bar stress gradients were reasonably consistent with those the BPR criteria for ultimate strength design or the 1963 ACI Building Code specify, in spite of the special circumstances present. It should be noted that the ultimate in anchorage bond was not reached and the remnant of untapped strength cannot be evaluated. It is suspected it was not large, but this is only a surmise.