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BEHAVIOR OF COMPOSITE-BEAM TO COLUMN JOINTS[a]

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INTRODUCTION

Plastic design methods have been successfully employed in the design of low steel building frames (2, 8) and during recent years in the design of braced and unbraced multi-story building frames (5, 4, 10). Plastic design uses the concept of the maximum or plastic strength of a structure as the basis of design. Its application will result in more efficient use of material, a more uniform factor of safety and relatively simple design procedures.

With todays demands for increasingly economical structures, which at the same time must provide adequate strength and stiffness to resist gravity or combined gravity and wind loads, attention is being focused on the use of composite steel-concrete beams in multi-story frames for buildings (6, 12). The use of composite beams in frames has been recognized by the AISC Specification for many years (8). However, such use

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4. Numerals in parenthesis refer to corresponding items in the Appendix - References.

has been limited to frames which are designed by allowable-stress methods. Investigations into the extension of plastic design theory to include composite beams are being made, but these studies are limited to continuous beams subjected to only gravity loads (3, 1, 11). Although the results of these investigations may be used in the design of braced frames with composite beams, they will not generally be applicable for unbraced frames in which the beams are subjected to combined gravity and lateral wind loads. Neither will the results be applicable at exterior joints where continuity of the composite beam is not present.

Since no studies were available on the ultimate strength behavior of composite beams subjected to combined loads, a small pilot investigation was initiated at Lehigh University to explore the variables involved (7, 9). Two joint assemblies were constructed and loaded to simulate the combined loading conditions in composite beams within the regions adjacent to the interior and exterior columns of an unbraced frame.

Combined Loading Conditions

Figure 1(a) shows an unbraced multi-story frame which is subjected to gravity loads w at each floor level and lateral wind loads H assumed concentrated at each exterior joint. A typical distribution of bending moments at level n produced by each loading system is shown in Fig. 1(b). A typical distribution of bending moments at level n produced by the combined loading condition is shown in Fig. 1(c). In this case it has been assumed that the lateral wind load is large enough to result in positive bending moments adjacent to the leeward side of one or more columns. Such a distribution of bending moments will determine at least four regions which must be considered in an investigation of the ultimate strength be-

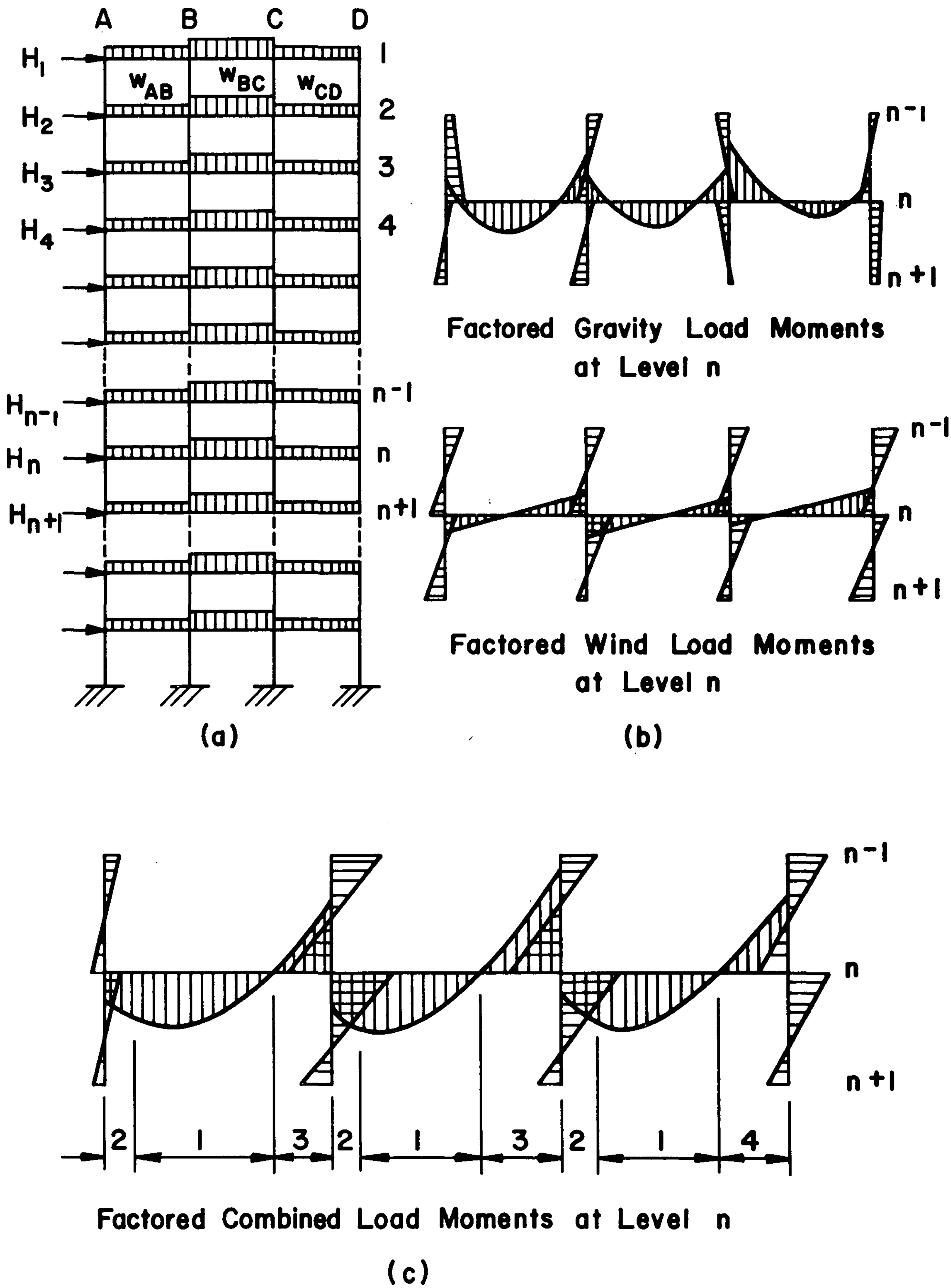


Fig. 1 Bending Moments in Unbraced Multi-Story Frames

havior of composite beams under combined loads. These regions are defined in Fig. 1(c) as follows:

Region 1. An interior region in which the cross-sections are subjected to positive bending moments (slab in compression) and where compressive forces can act over the full effective slab width at the ultimate moment capacity. (The term "effective slab width" denotes the usual slab width of the Tee section used to compute the maximum ultimate moment capacity of the composite beam).

Region 2. A transition zone consisting of the positive moment region between Region 1 and the cross-section adjacent to the leeward side of an interior or a windward exterior column where the ultimate compressive forces in the concrete act on a reduced slab width. Adjacent to the column, compressive slab forces can be developed only between the column face and the concrete slab which is assumed to be in contact with the column. The reduced slab width at this point then is the width of the column face.

Region 3. A negative moment region between Region 1 and a cross-section adjacent to the windward side of an interior column.

Region 4. A negative moment region between Region 1 and a cross-section adjacent to the windward side of the leeward exterior column.

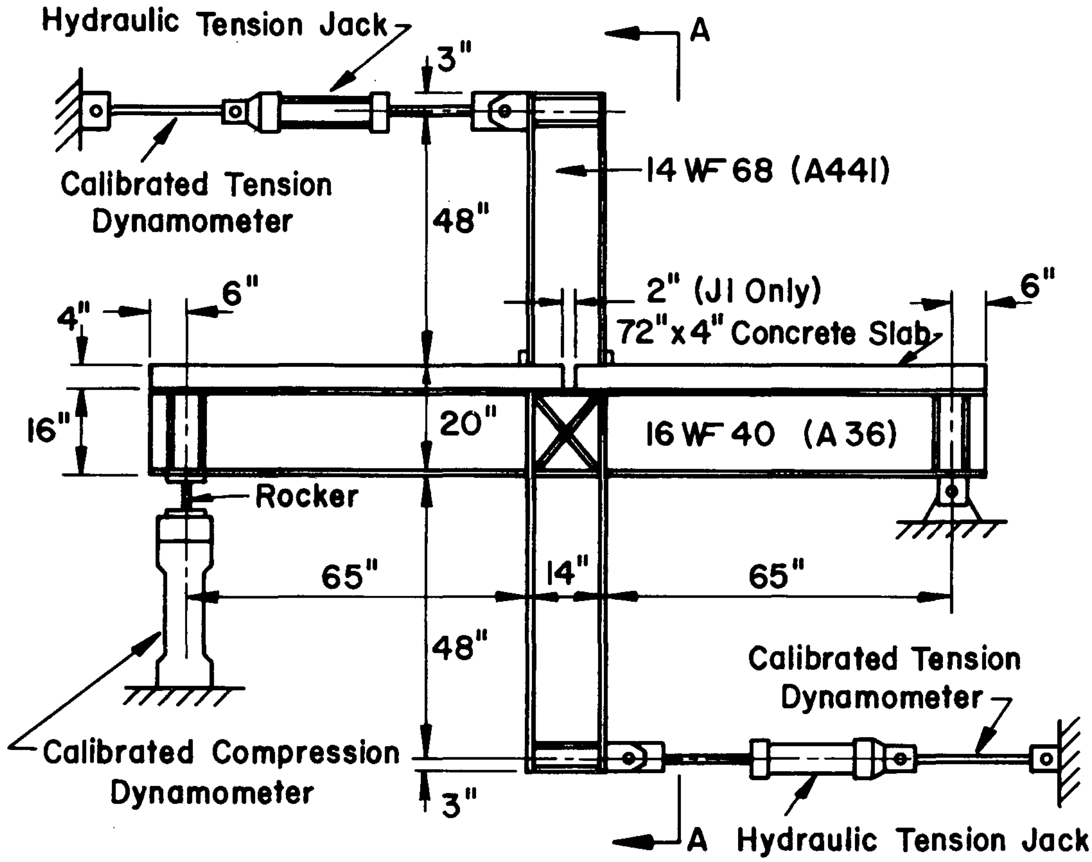
Regions 1 and 3 do not differ appreciably from similar regions of continuous composite beams subjected only to gravity loads. The ultimate moment capacity in the positive moment regions (Regions 1) will be determined by plastification of the steel beam and by crushing of the concrete slab over an effective slab width as defined above. In the negative moment regions (Regions 3 on both sides of an interior column) the ultimate moment capacity will be determined by plastification of the steel beam and the longitudinal reinforcement over a certain slab width (11).

This pilot investigation was essentially designed to study the behavior of composite beams subjected to the loading conditions represented by Regions 2 and 4, and to provide further study of Region 3 where continuity of this region beyond the column does not exist. Of particular interest in this paper is the mechanism by which cross-sections in Regions 2, adjacent to the column face develop their ultimate strength. Reference 9 presents the remaining results of the investigation which was concerned with the local buckling behavior of composite beams.

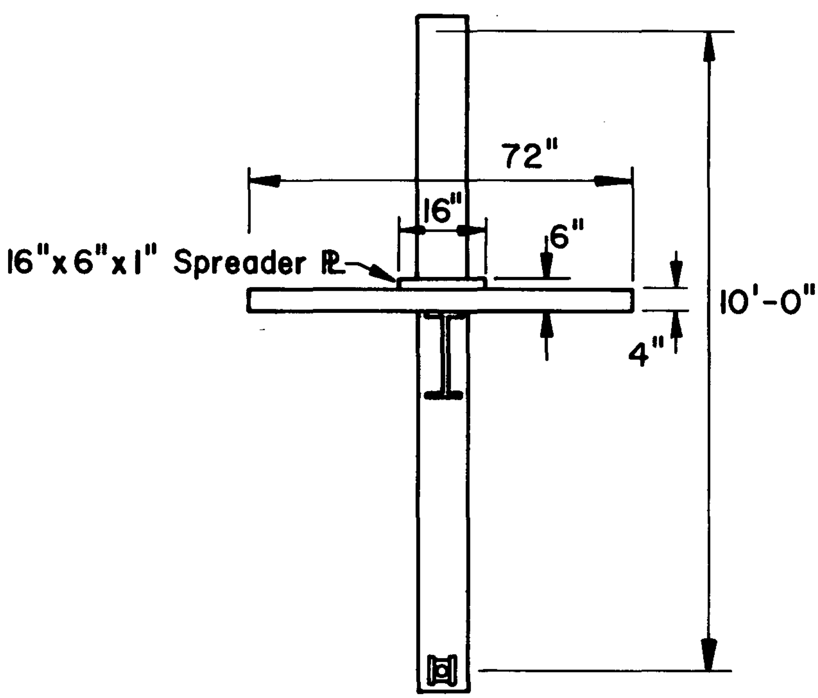
TEST SPECIMEN DETAILS

Design:

Two nearly identical test specimens, designated J1 and J2, were used in the experimental phase of the investigation. They were designed to simulate as nearly as possible the portion of an actual unbraced frame in the vicinity of the windward and leeward columns as well as in the vicinity of an interior column. A schematic view of the test specimens is shown in Fig. 2. The support or loading points at the ends of the beams and columns represent possible locations of inflection points in an actual frame. Horizontal shear forces were applied at the column ends by means of hydraulic tension jacks. Positive and negative bending moments were therefore produced in the composite beams on either side of the column. In the lower stories of a tall unbraced frame, the gravity load moments will be small compared to the moments produced by the lateral wind loads. Therefore for simplicity no gravity loads were applied to the two test specimens. Although combined load conditions were not attained in the test specimen, the loading used could be further justified since the ultimate strength behavior of cross-sections adjacent to the column face



SCHEMATIC ELEVATION OF J1 AND J2



SECTION A-A

Fig. 2 Schematic View of Test Specimens

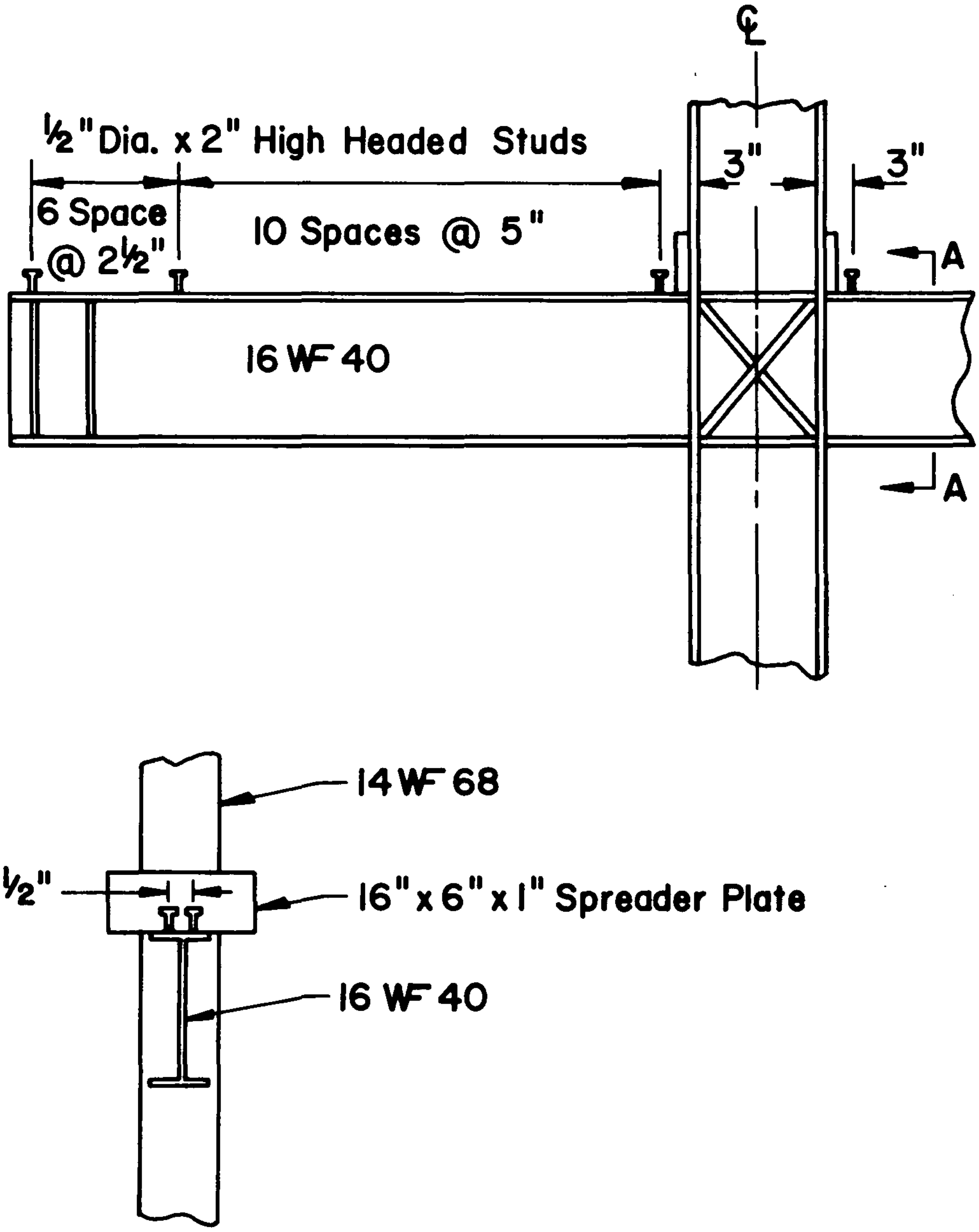
was of primary importance in this study. A short composite beam length was used in order to produce a steep moment gradient at the column face, a condition which frequently occurs in an actual frame, especially in Regions 3 and 4.

It was considered desirable to test nearly full size joint specimens to eliminate scale factor effects. The slab width and thickness were determined by the AISC Specifications and present practice. The column behavior was not being studied in this investigation. However, a 16-in. column face width was required in order to duplicate practical column dimensions. Therefore an A441-14W68 column was used and two 16" x 6" x 1" spreader plates were welded to the column faces so that the contact width between the column faces and the concrete would be increased from 10-in. to 16-in. Details of the spreader plates are shown in Fig. 3.

The only difference between the two test specimens was in the continuity of the slab at the column as shown in Fig. 4.

In J1 the slab was discontinuous at the column. Therefore under loading, Regions 2 and 4 would be produced in the positive and negative moment sides of the beam respectively. Thus the behavior at the windward and leeward joints could be obtained from the results of one test specimen. The slab in J2 was made continuous so that the behavior at an interior joint (Regions 2 and 3) could be obtained. The corners of the slabs were clipped so that the specimens could be fitted within the loading frame. The column section at the joint was considerably over-stiffened to eliminate any joint deformation from the test results.

In each specimen the slab width was made 72-in. to approximately



SECTION A–A

Fig. 3 Details of Test Specimens J1 and J2

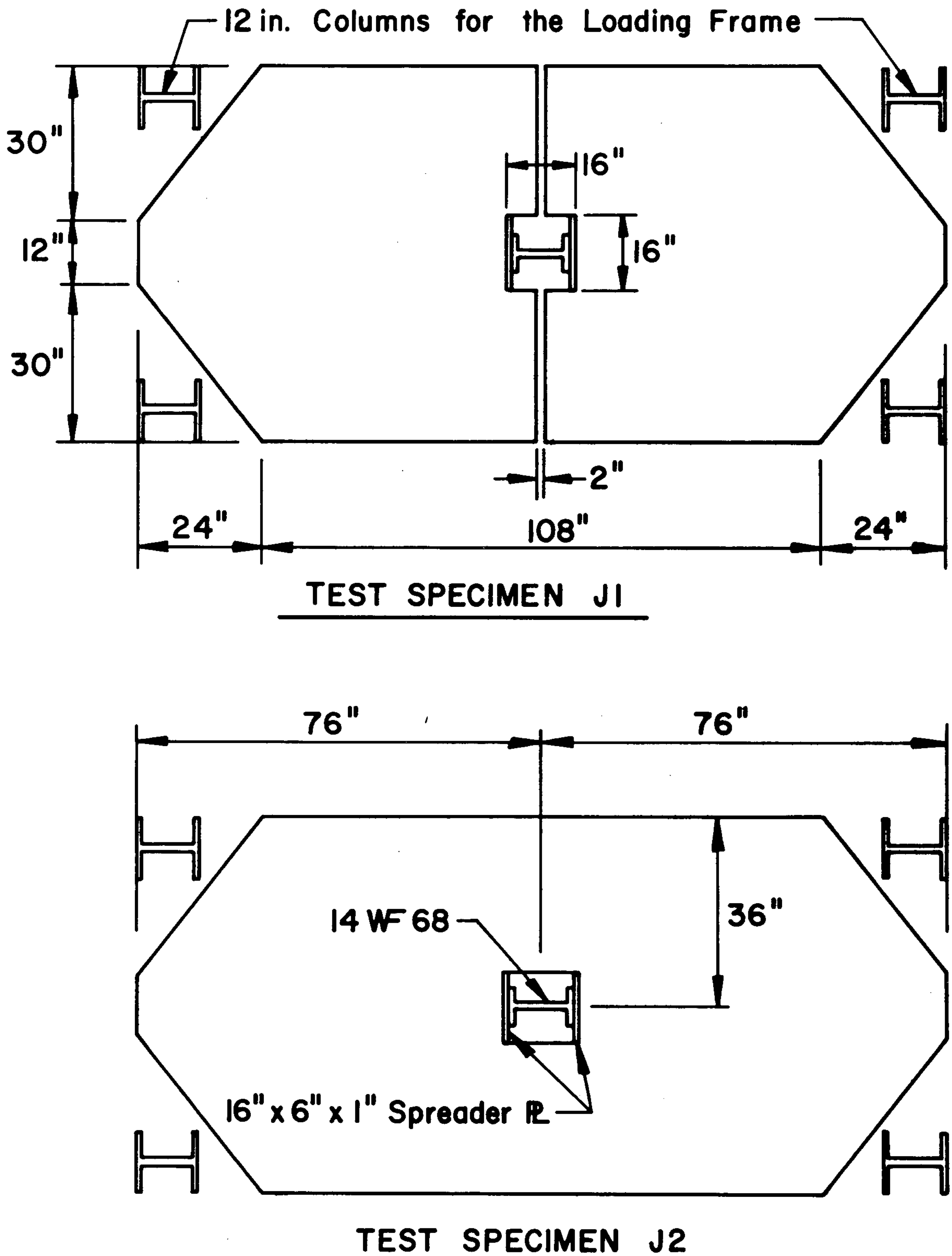


Fig. 4 Dimensions of Test Specimens J1 and J2

conform with the effective width requirements of the current AISC Specifications. This choice was arbitrary because no slab width requirements exist for a plastic analysis of composite beams. Figure 5, however shows that little additional theoretical moment capacity may be gained by an increased width. This is explained by the fact that once full yielding of the W section has occurred, additional slab width will not result in substantially increased moment capacity.

Construction:

The steel fabrication for each test specimen was identical. The two beams for each specimen were cut from the same 36-ft length of 16W40, ASTM A-36 steel. The 36 ft. length was sectioned into thirds with the midsection being subsequently used for various control test purposes which are described later. The steel beams for J1 and J2 were cut from the end sections and welded to the flanges of the column using full penetration butt welds. The columns were fabricated from short lengths of 14W68, ASTM A441 steel.

A 72-in. x 4-in. reinforced concrete slab was connected to the top flange of the steel beam with pairs of 1/2 in. diameter x 2-in. high headed steel studs. The shear connector spacing was kept uniform at 5-in. for most of the beam length in order to maintain a practical spacing of the studs near the joint. However the spacing was decreased near the free end of the beams in order to simulate the boundary conditions at points of inflection and to insure development of the flexural capacity. Details of the stud shear connectors are shown in Fig. 3. The concrete for the slab was transit-mixed and proportioned for a 28-day strength of 400 psi. The slab was reinforced with ASTM A615 Grade 40, (Intermediate Grade) reinforcing bars as detailed in Fig. 6.

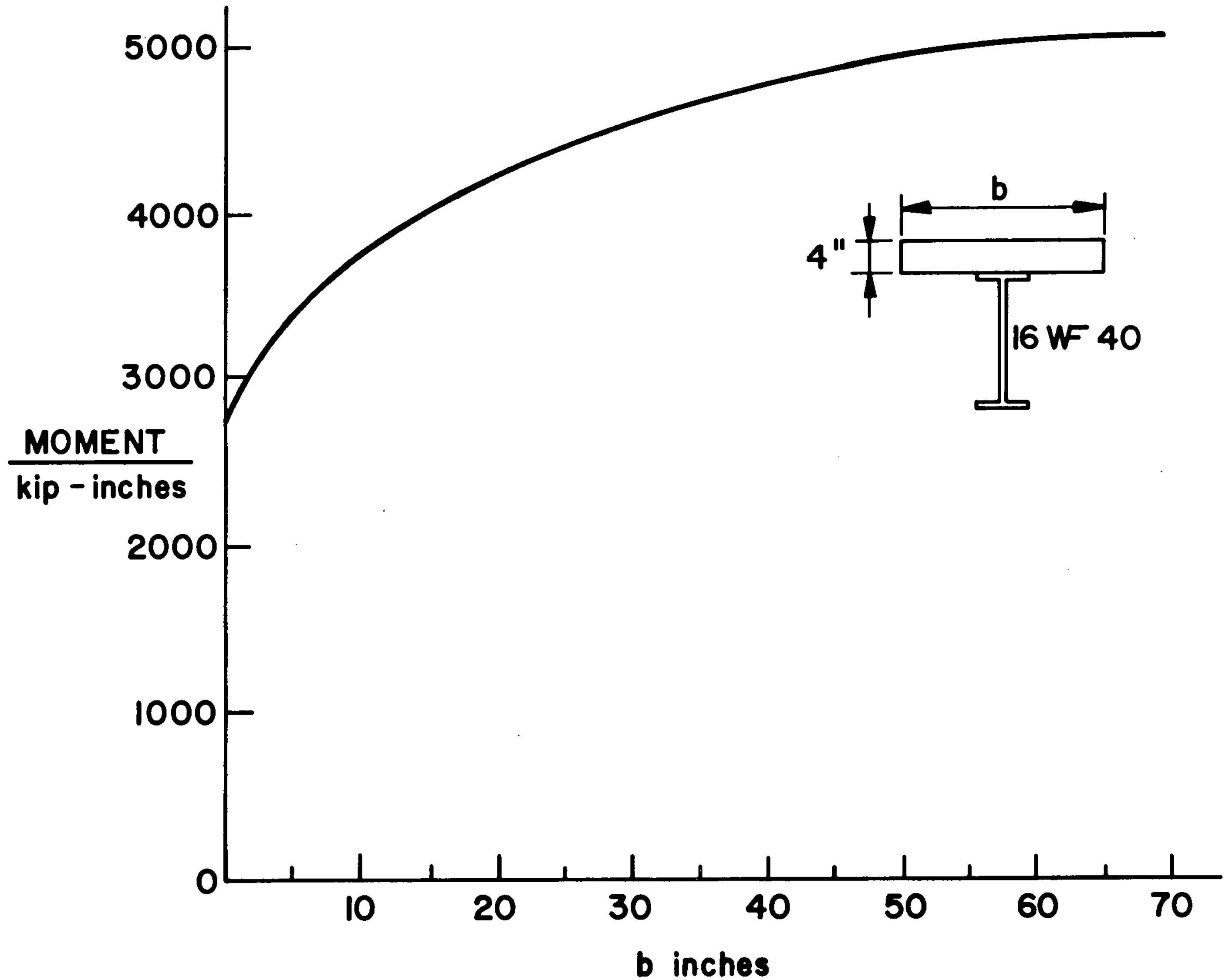
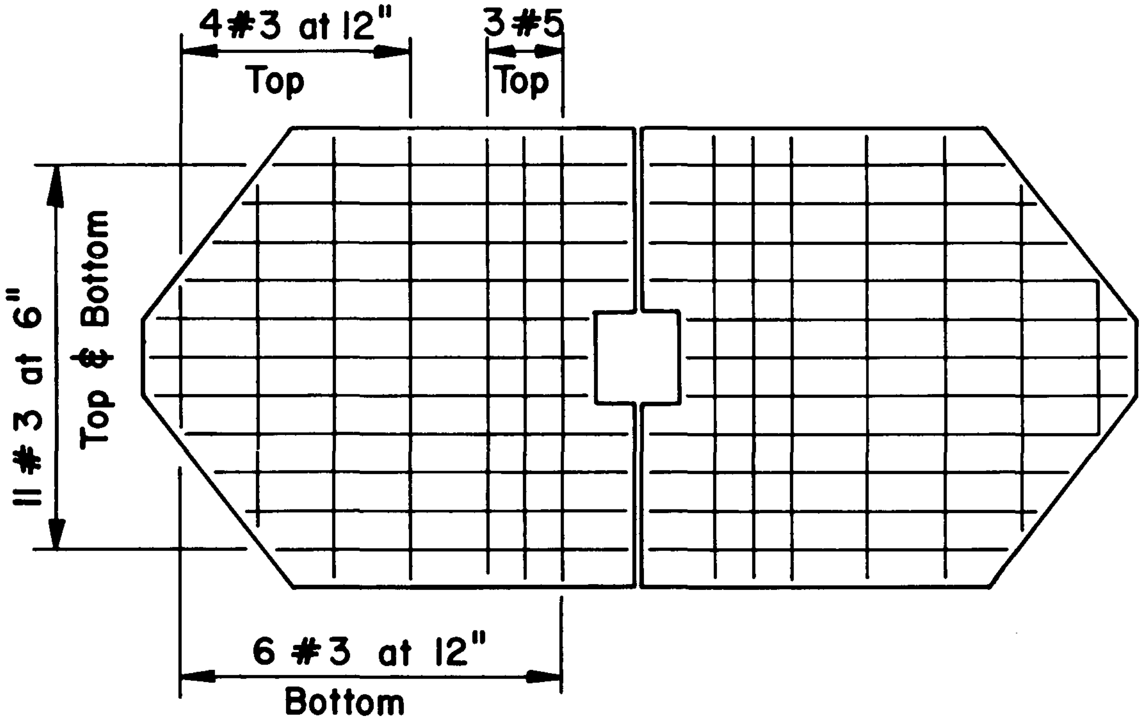
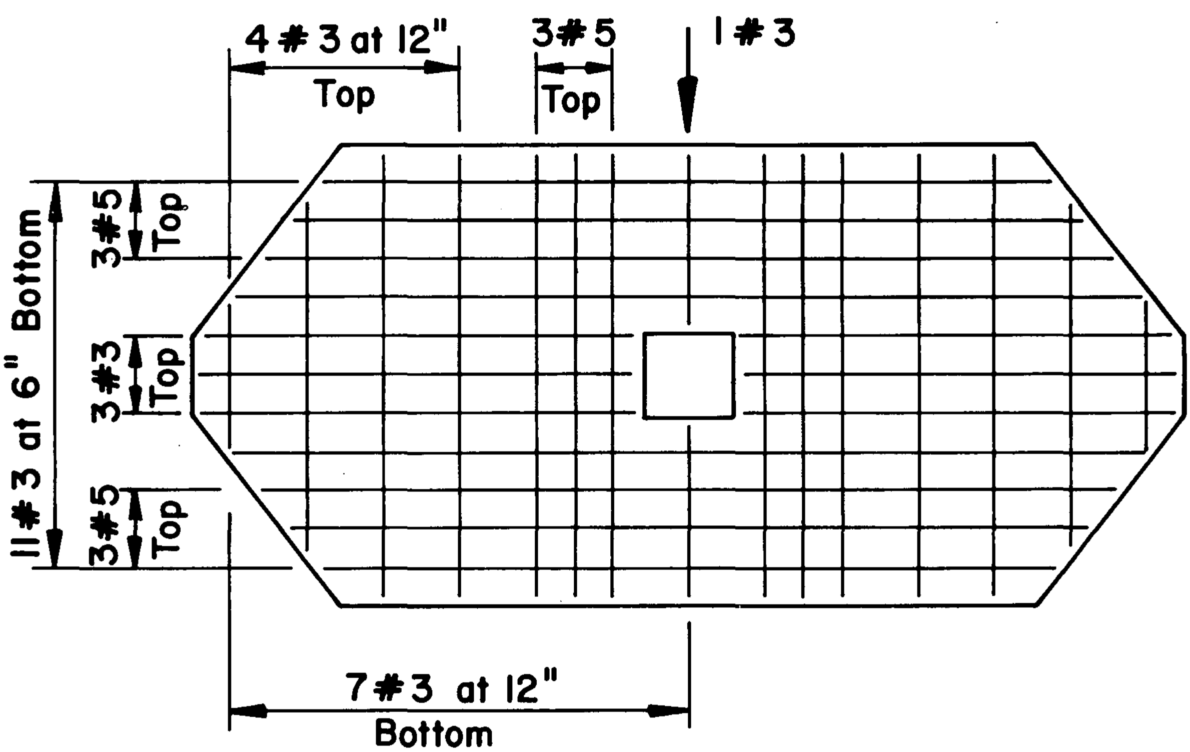


Fig. 5 Ultimate Moment Versus Slab Width Relationship



TEST SPECIMEN J1



TEST SPECIMEN J2

Fig. 6 Slab Reinforcing Details

INSTRUMENTATION

Four different types of instrumentation were used: electrical resistance strain gages, dial deflection gages, level bar rotation gages and slide bar extensometers. Cross-sections located at 3-in., 15-in., and 27-in., from each column face were chosen as the cross-sections for major instrumentation as shown in Fig. 7. The W beams were instrumented at these cross-sections with SR-4 electrical resistance gages as well as mechanical gage points with a three inch gage length located symmetrically with respect to the electrical gages. A slide bar extensometer fitted with a 0.0001-in. dial gage was used to measure the strains from the mechanical gage points. The SR-4 electrical resistance gages provided reliable results in the elastic range while the mechanical measurements were more accurate once plastification of the beam began. SR-4 gages were also placed on the top of the slab as shown in Fig. 7 but only within Region 2. The limit of usefulness of these gages was determined either by excessive compressive strains in the concrete or by cracking of the concrete slab.

The remaining instrumentation is shown in Fig. 8 and was used to determine the amount of slip of the steel-concrete interface, rotation of the joints, and the onset of local buckling. Reference 9 presents the results obtained from the local buckling studies and additional behavior in the negative moment region.

Shear loads were applied to the columns as shown in Fig. 2 by means of hydraulic tension jacks. The force was measured using calibrated load dynamometers. The beam reaction in Region 2 was also measured using a calibrated compression dynamometer. Figure 9 shows a view of the instru-

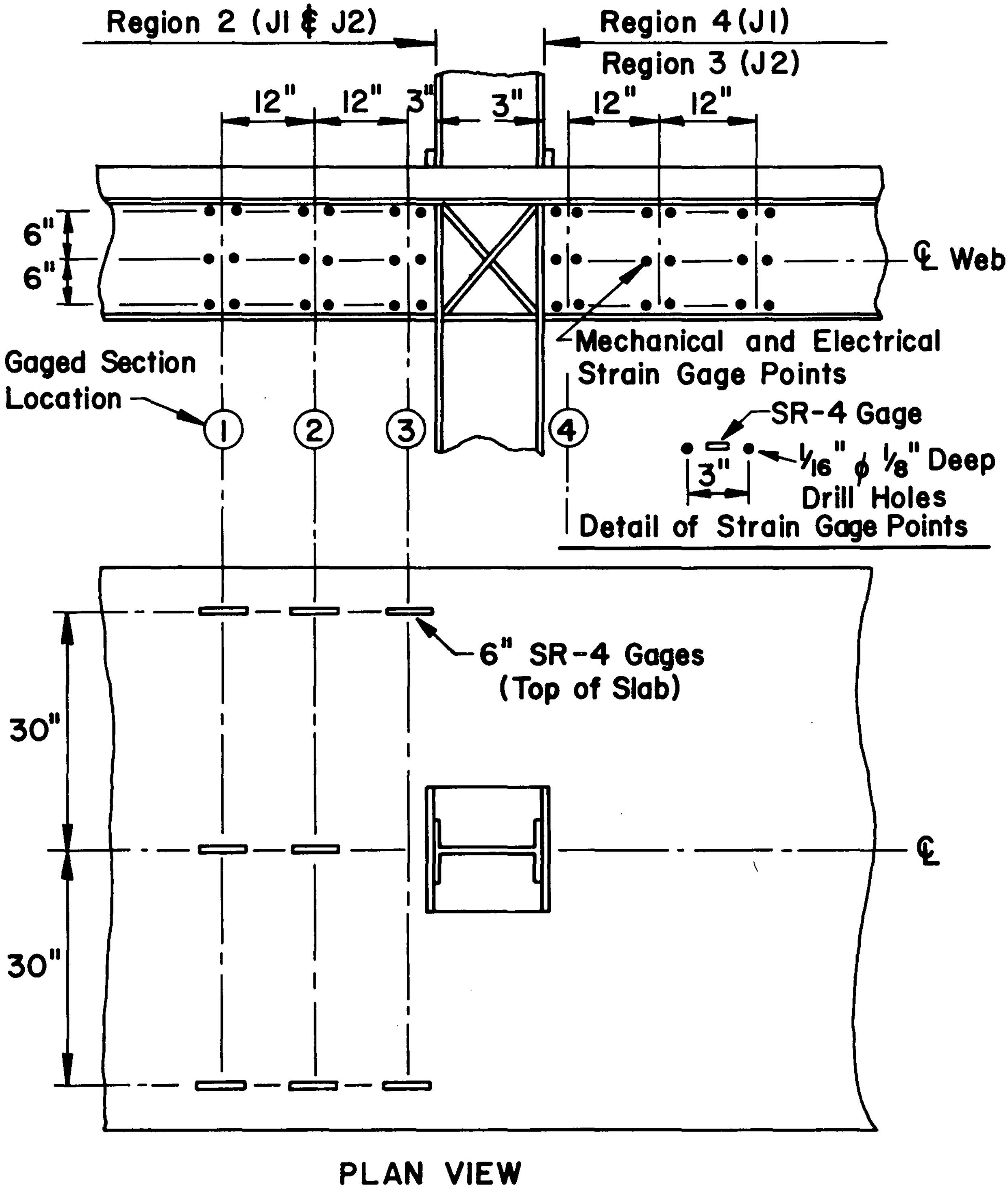


Fig. 7 Details of Instrumentation Used

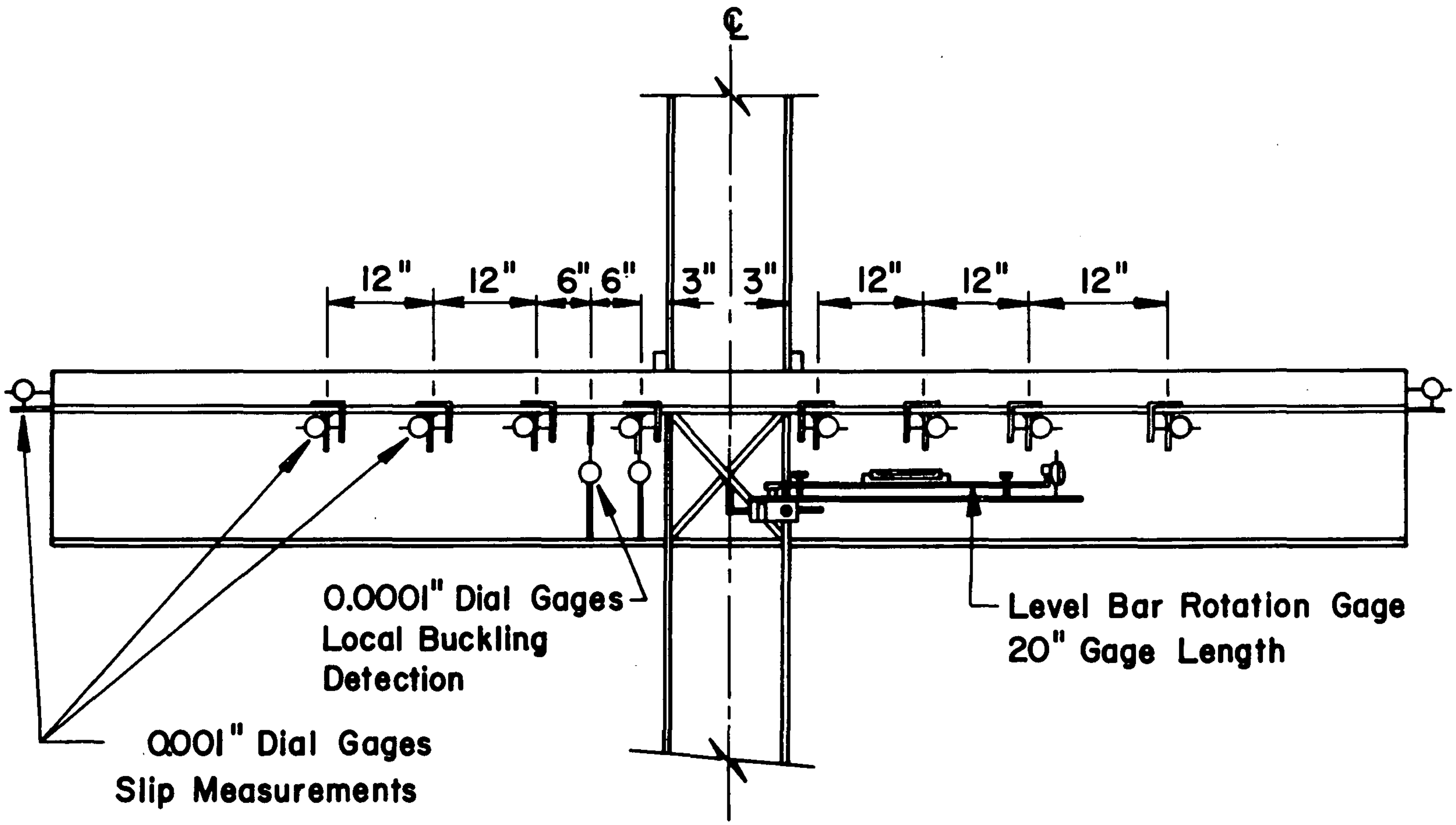


Fig. 8 Details of Instrumentation Used

mentation used, one of the tension jacks and the compression dynamometer at the end of the beam in Region 2.

CONTROL TESTS

The mechanical properties of the 16W40 steel section were determined from tension test specimens cut from a 2-ft. length of the 12-ft. mid-section described earlier. The mechanical properties are summarized in Table 1. Standard 6-in. x 12-in. concrete cylinders case during the pouring of the concrete slab were tested to determine their compressive strength and modulus of elasticity. Table 2 summarizes the results of the cylinder tests which were carried out 58 days after pouring to correspond with the testing of test specimens J1 and J2.

The remaining 10-ft. length of the 16W40 steel section was used to determine the plastic moment capacity of the steel beam with point loading and to determine the moment-curvature behavior of the steel beam under high moment gradient, similar to that which would occur in the tests of J1 and J2.

Control tests were not carried out on the shear connectors or the reinforcing steel.

TABLE 1

Mechanical Properties of 16W40

Property	Flange	Web
Yield stress	36.4 ksi	40.0 ksi
Modulus of Elasticity	29,000 ksi	30,000 ksi
Strain Hardening Strain	0.02520 in/in	0.02394 in/in
Strain Hardening Modulus	450 ksi	500 ksi

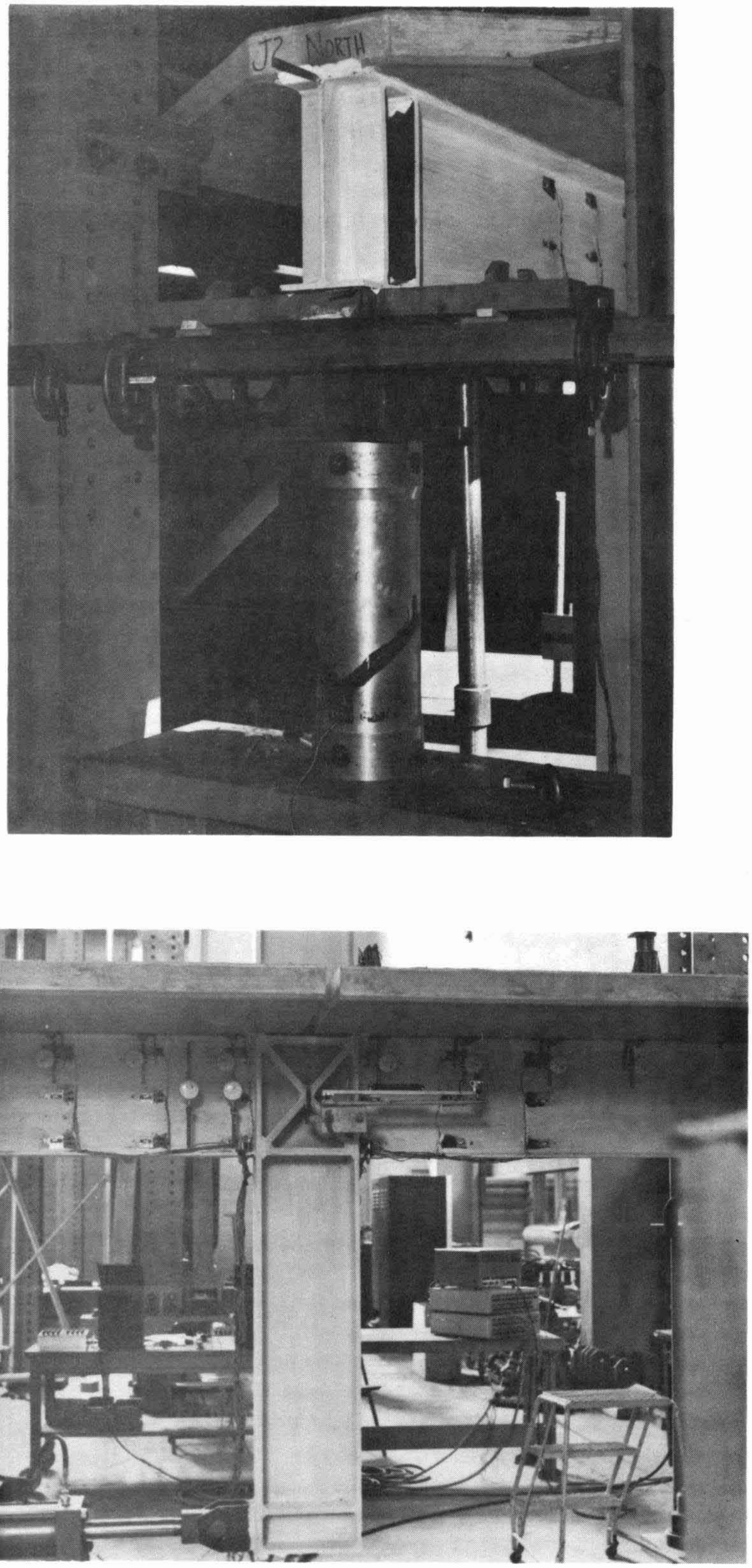


Fig. 9 Details of Instrumentation Used

TABLE 2

Mechanical Properties of Concrete

Compressive Strength	4100 psi
Tangent Modulus	2480 psi

TEST PROCEDURE

A schematic view of the test set-up is shown in Fig. 2. The applied joint moment was maintained by the hydraulic jacks, one at each end of the column. Beam reactions were taken by the pinned support in the negative moment region (Regions 3 and 4) and by a rocker support on a compression dynamometer in the positive moment region (Region 2). The combination of the pinned and rocker supports allowed the joint the freedom of horizontal and vertical movement during loading.

The load increments which were applied to each test specimen varied up to five kips at each jack. The initial loading increments were the largest and nearly linear behavior resulted. As plastification of the steel-beams occurred near the joint much smaller increments in load were applied with ever increasing deformations. Because of some small problems which developed with the loading apparatus the test specimens were unloaded and reloaded at various stages of testing. Figure 10 shows a view of the test set-up for specimen J1 during testing.

Due to the fact that there is a lower theoretical ultimate moment capacity in the negative moment region, a bracing or stiffening procedure as shown in Fig. 10 had to be employed to allow the ultimate moment capacity in the positive moment region to be reached. The diagonal brace was

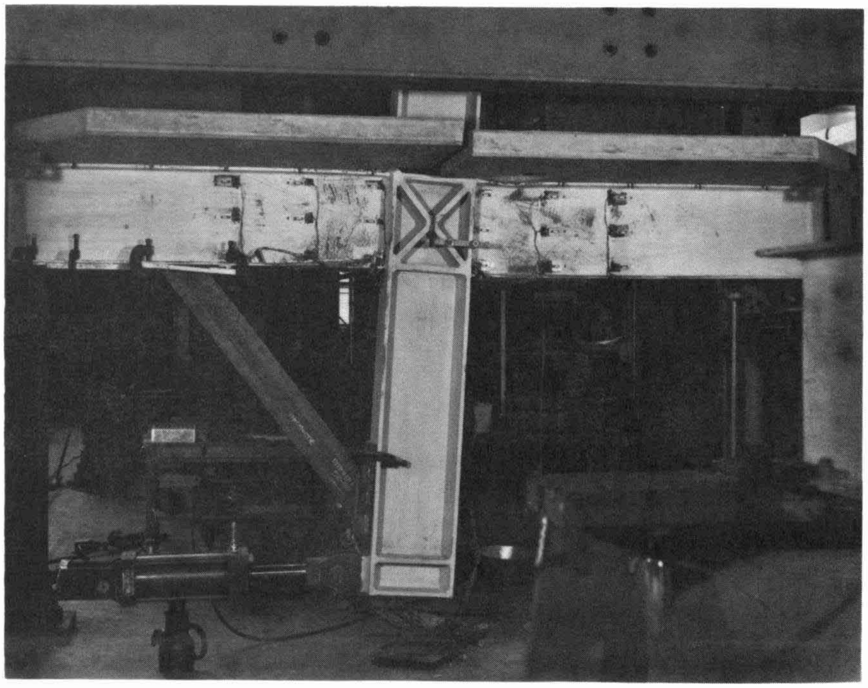


Fig. 10 Specimen J1 After Testing Showing Brace in Region 3

clamped into position in the negative moment region only after excessive deformation of this region threatened to halt the test. The presence of the diagonal brace did not alter the ultimate strength capacity of the positive moment region.

Excessive deformation occurred in the loading frame during the test of specimen J1. This resulted in a misalignment of the loading jacks and excessive movement of the rocker support. These conditions were partially corrected by stiffening the loading frame and repositioning the rocker support over the sompression dynamometer, but the test was finally halted when the rocker support collapsed. The joint was therefore not tested completely to failure. However, substantial slab crushing and beam plastification were obtained.

The loading frame was stiffened prior to testing J2. A roller support over the compression dynamometer replaced the rocker support to allow greater horizontal movement. Joint J2 was subsequently tested until failure which was caused by crushing of the concrete slab adjacent to the column face, complete plastification of the steel beam and fracture of the steel beam section adjacent to the weld (Fig. 16). It should be emphasized that fracture occurred only after considerable deformation and strain hardening at the cross-section.

TEST RESULTS

Figures 11 and 12 summarize the moment-curvature behavior which was obtained at gage locations 3 and 4 (Fig. 7) for test specimens J1 and J2.

Figure 11 shows the moment-curvature behavior at Location 3, in the positive moment region of each test specimen. Bending moments are

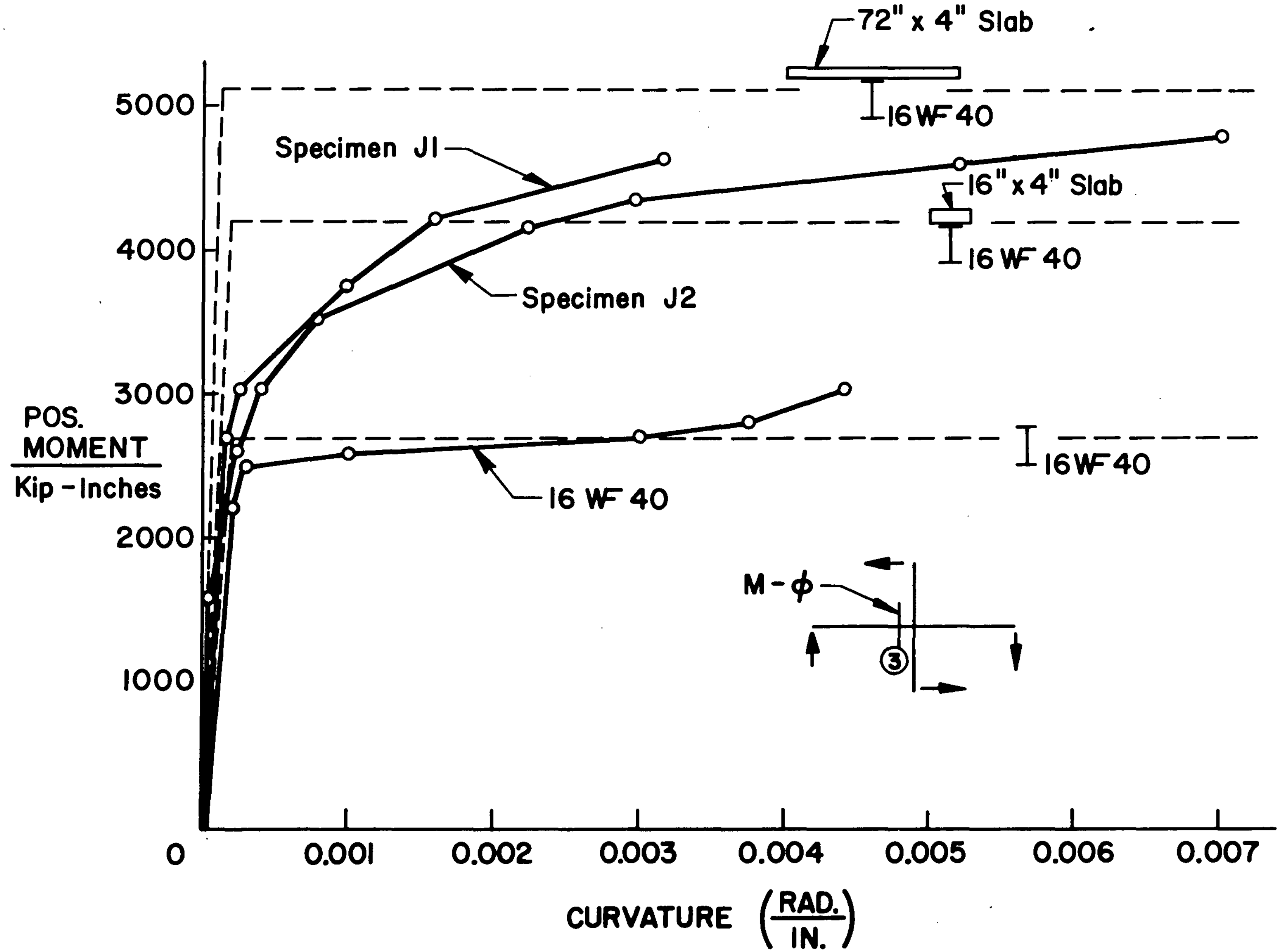


Fig. 11 Moment-Curvature Behavior at Location 3 For Specimens J1 and J2

based on the measured beam reactions but neglect the dead load moments of the specimens. The curvatures are based on measurements taken from either the electrical or mechanical strain gages or both depending on which technique appeared to provide the more reliable result. At lower levels of strain the electrical strain measurements were more accurate. At higher strain levels both sets of data were used, with the obvious poor data being eliminated and the remaining data average on a least-squares basis.

Figure 11 also shows the moment-curvature behavior of the 10-ft. length of 16W40 steel beam section described earlier which was subjected to a concentrated load at mid-span. For comparative purposes the calculated theoretical moment-curvature relationships for three beam cross-sections are also shown. These curves are shown dashed and are based on the actual material properties.

Figure 12 shows the theoretical and experimental moment-curvature curves at location 4, in the negative moment region of each test specimen. The theoretical curve shown dashed for the composite beam accounts for the strengths of the longitudinal slab reinforcement for test specimen J2 only. Since the reinforcement for J1 was not continuous at the joint it was expected that the theoretical behavior would be the same as for the 16W40 steel beam section.

Figure 13 shows the observed slip between the steel beam and the concrete slab at strain gage location 3 (Fig. 7) for each test specimen. Initial movement of the slab was toward the column face but reversed when significant pressure was exerted by the column on the slab. The initial slip behavior indicated the presence of a small shrinkage crack between

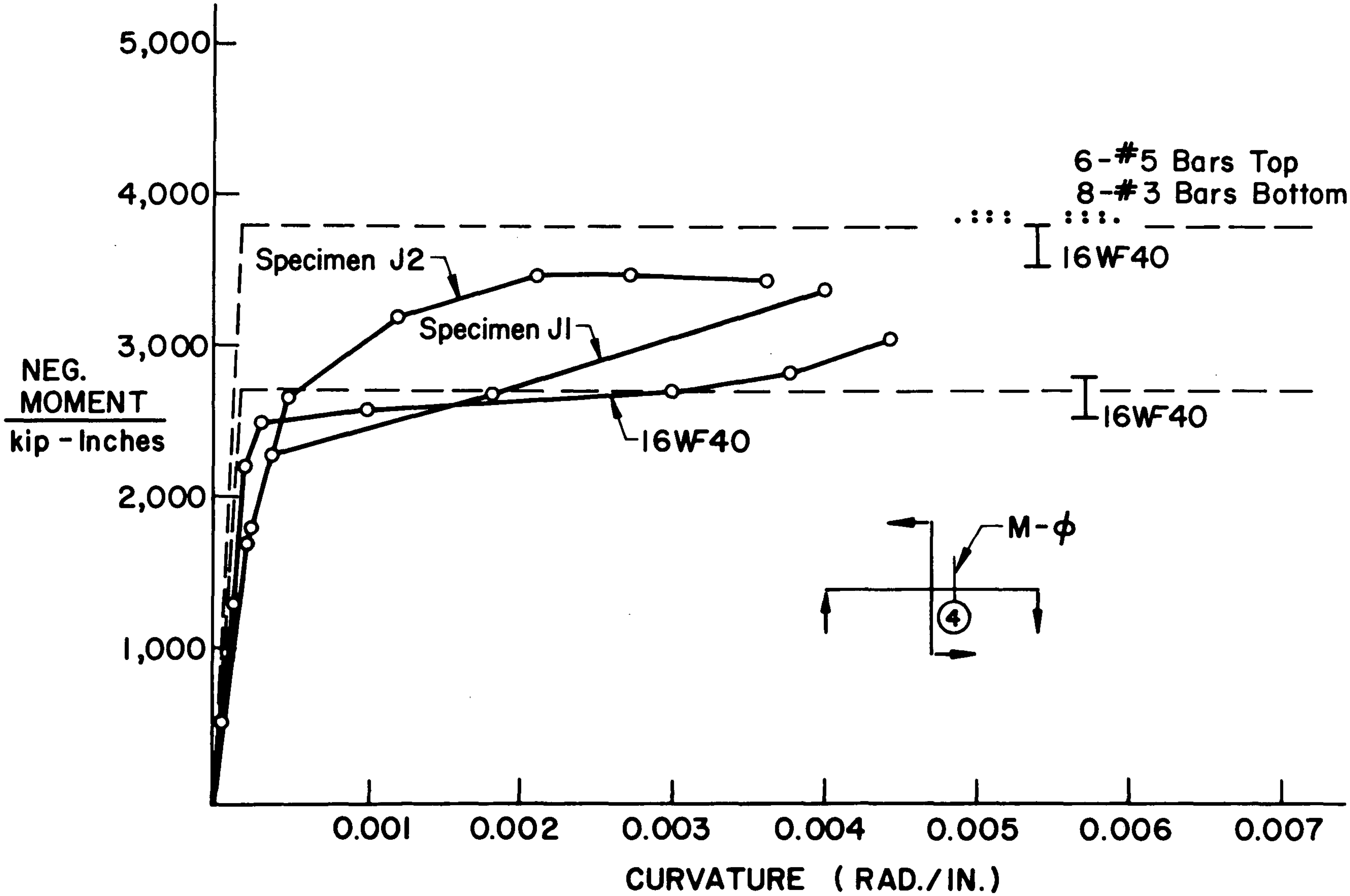


Fig. 12 Moment-Curvature Behavior at Location 4 for Specimens J1 and J2

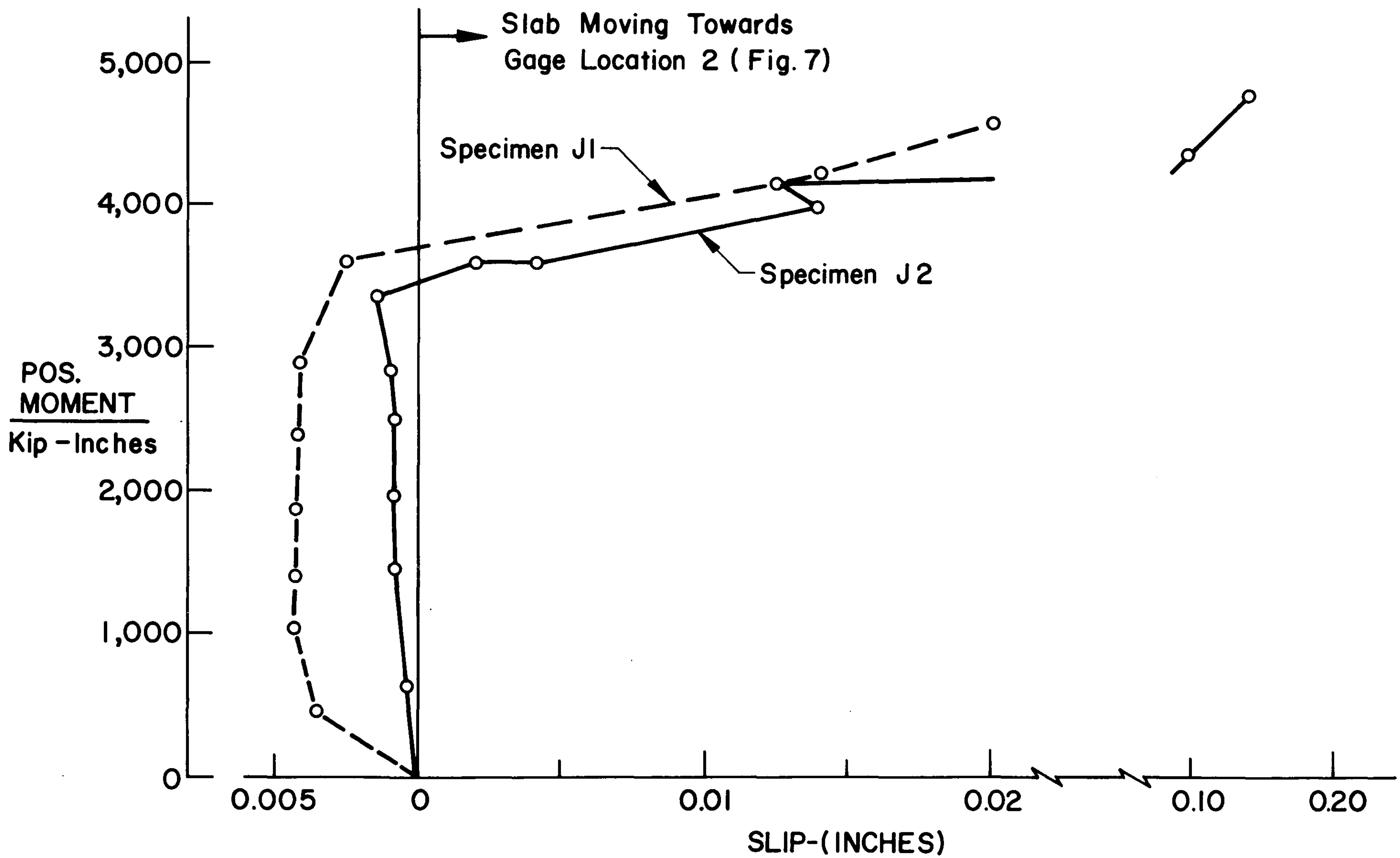


Fig. 13 Slip Behavior at Location 3 for Specimen J1 and J2

the concrete and the spreader plates. No attempt was made to ensure that the slab was initially tight against the spreader plates. This could easily be accomplished both in the laboratory and in actual frame by providing for grouting at this location.

Figures 14 and 15 show the condition of the test specimens after testing. The patterns of slab cracks and the degree of crushing adjacent to the column are evident in Fig. 14. Specimen J1 is shown in the upper photo while J2 is shown in the lower photo. The slab crushing for specimen J2 was more extensive and occurred over the whole slab depth. Crushing in specimen J1 was not so extensive, however J1 was not subjected to as much deformation as J2. The cracking patterns varied considerably. In J1 the slab was extensively cracked along the diagonal lines which extended to the edges of the slab. However, in J2 the cracking was mainly limited to two parallel lines symmetrically placed on each side of the steel beam flange.

Figure 15 shows the amount of deformation that each test specimen was subjected to at the conclusion of testing. Specimen J1 is in the upper photo.

Figure 16 shows the fracture which occurred in the 18W40 steel beam of specimen J2 at maximum load. This fracture occurred only after substantial deformation and strain hardening of the cross-section as mentioned previously.

ANALYSIS OF TEST RESULTS

As mentioned in the Introduction, this study was particularly interested in the ultimate strength behavior of cross-sections in Regions 2, adjacent to the column face. The moment-curvature results shown in Fig. 14 indicate that it would not be realistic to base the flexural

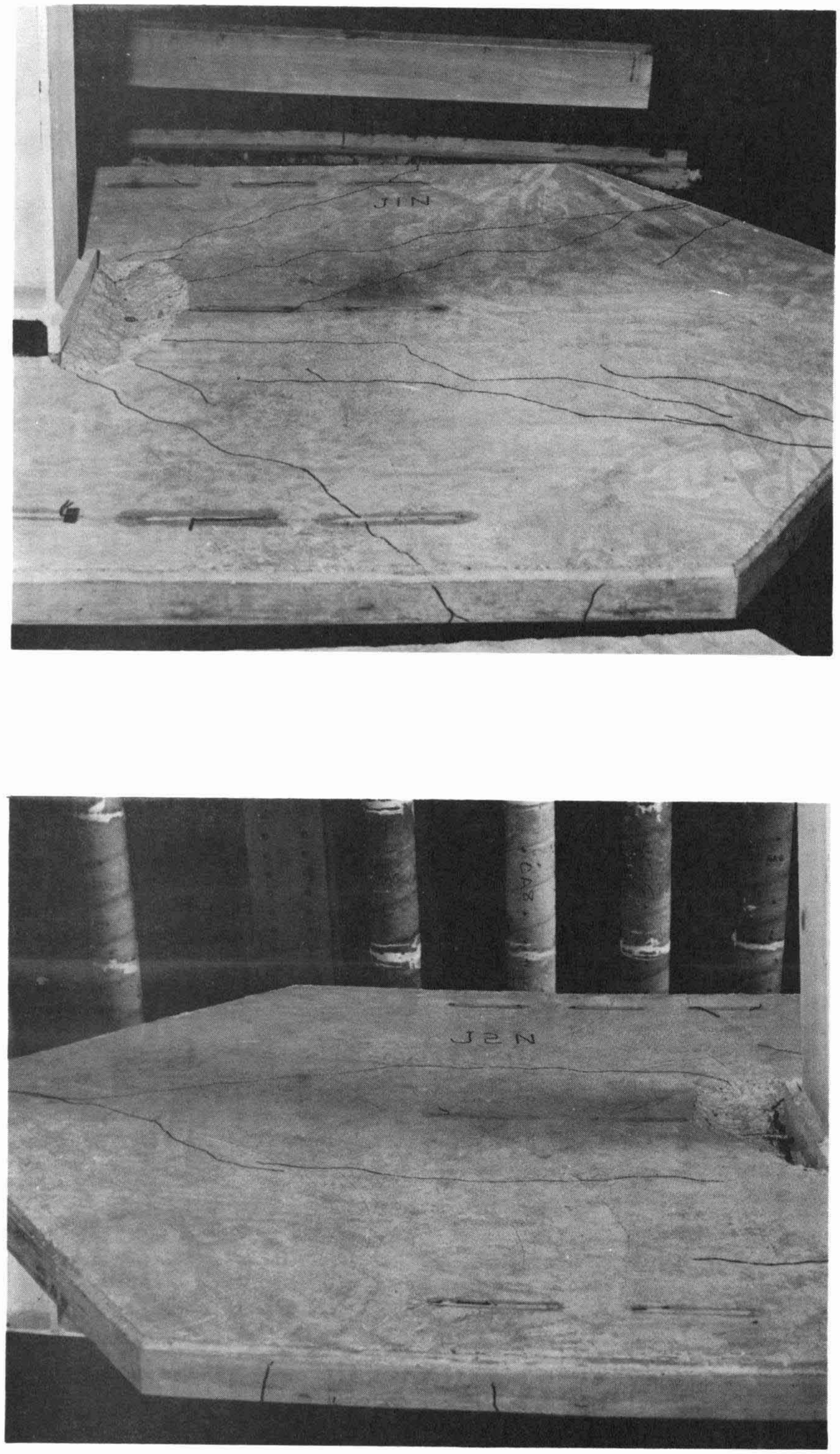


Fig. 14 Cracking of Slabs and Crushing of Concrete at Face of Column

capacity near the column face on the full 72-in. slab width. This would be true either for a windward or a leeward column. In addition the initial stiffness at the joint was either equal to or less than the stiffness predicted on the basis of the 72-in slab width. However, Fig. 11 indicates that the initial stiffness of the joint and the ultimate strength of the cross-section near the column face could be based on a composite cross-section with a slab width equal to the column face width. Of equal significance, Fig. 11 indicates that considerable ductility is available at this cross-section. It has been suggested that the ultimate moment should be reached within curvatures not exceeding 10 to 15 times the yield curvature if plastic design theory is to be used (Ref. 2). The moment-curvature behavior of both J1 and J2 indicates that redistribution of bending moments would occur in a frame with composite steel-concrete beams if a plastic hinge were to form in Region 2 near the column face. In addition the plastic hinge moment could be computed on the basis of ultimate strength theory with a slab width equal to the column face width.

The significant increase in rotation capacity exhibited by J2 was possibly a result of the greater degree of plastification of the joint due to the presence of the continuous slab. In addition, as mentioned earlier, a premature failure of the rocker support resulted in a lower total curvature being reached in J1.

It can be seen further from Fig. 11 that both test specimens were able to develop a moment capacity which approached that based on a 72-in. slab width. However, this was possible only after large deformations. One explanation of this phenomenon is that the slab adjacent to the column was subjected to a semi-confined state of stress. Such a state would allow compressive stresses to be developed in the slab

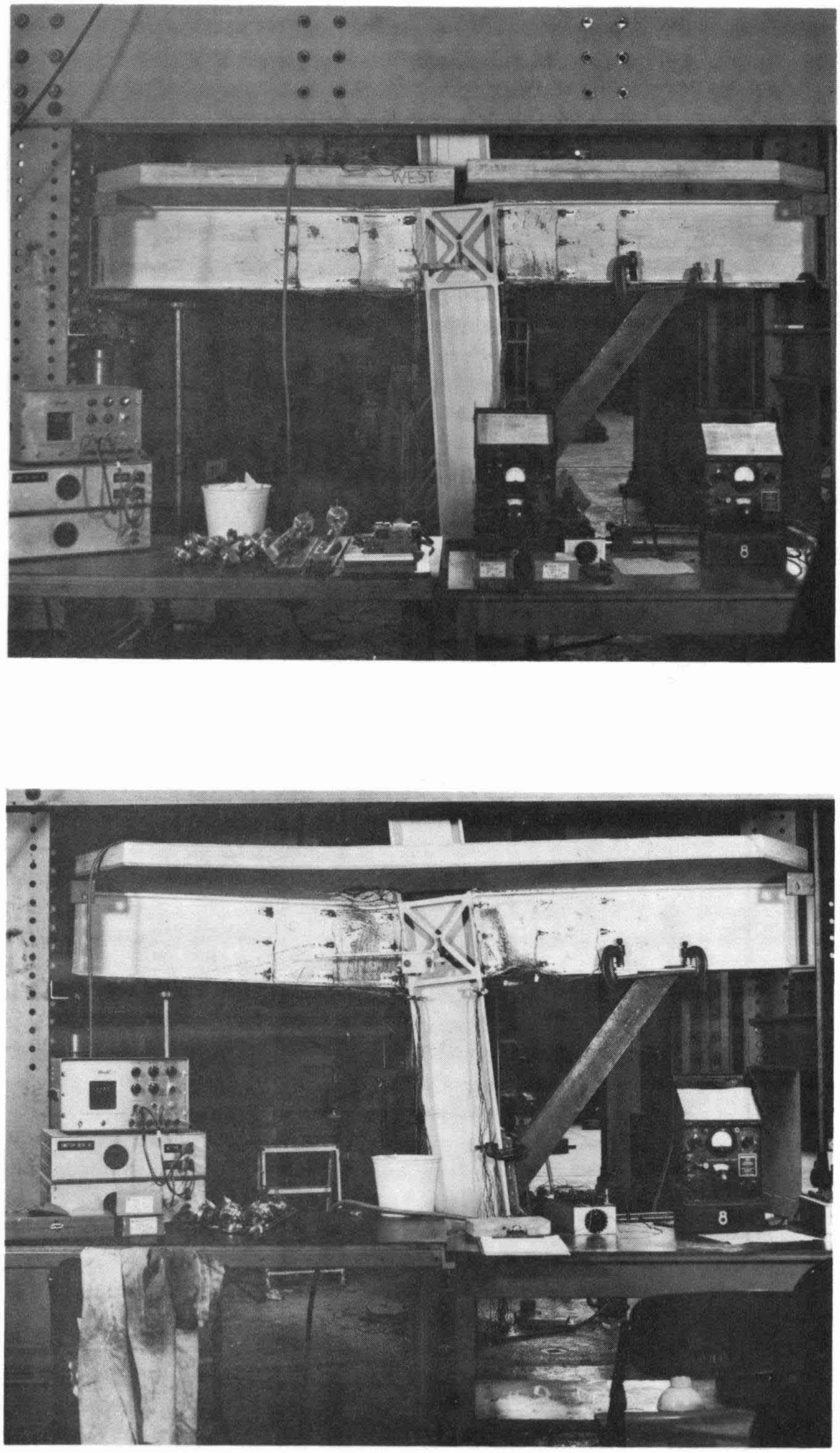


Fig. 15 Specimens J1 and J2 After Testing

near the column face which would be in excess of the maximum stresses as determined by standard cylinder tests. The confining effects would be produced by the large mass of concrete surrounding the column face, which was tied together with reinforcement, as well as the column and beam flanges. The deformation of the beam flange below the crushed concrete adjacent to the column face is shown clearly in Fig. 16. This deformation was exagerated after yielding of the steel beam due to compression in the top flange. However, prior to yielding when the top flange was observed to be in tension a small downward deflection of the flange tips was observed.

Further experimental and theoretical studies are required which would indicate if it is possible to develop higher predictable moments by using wider spreader plates at the column face, stiffer beam flanges, or other means to take advantage of the confining effects near the joint.

It was hoped that sufficient data could be obtained in this study to locate the cross-section in Region 2 at which the full ultimate moment capacity of the 72-in slab width.would be available. This cross-section would then define the boundary between Regions 1 and 2 in the test specimens. However, the data was inconclusive.

The moment-curvature behavior in the negative moment regions, Regions 3 and 4 were also of importance in this study. Figure 12 shows the results obtained. Region 4 was produced in test specimen J1 (Fig. 7). The test results clearly indicate that the behavior near the column face in this Region is not significantly different from that of the steel beam. Such behavior would be expected since the slab is discontinuous at the leeward column.

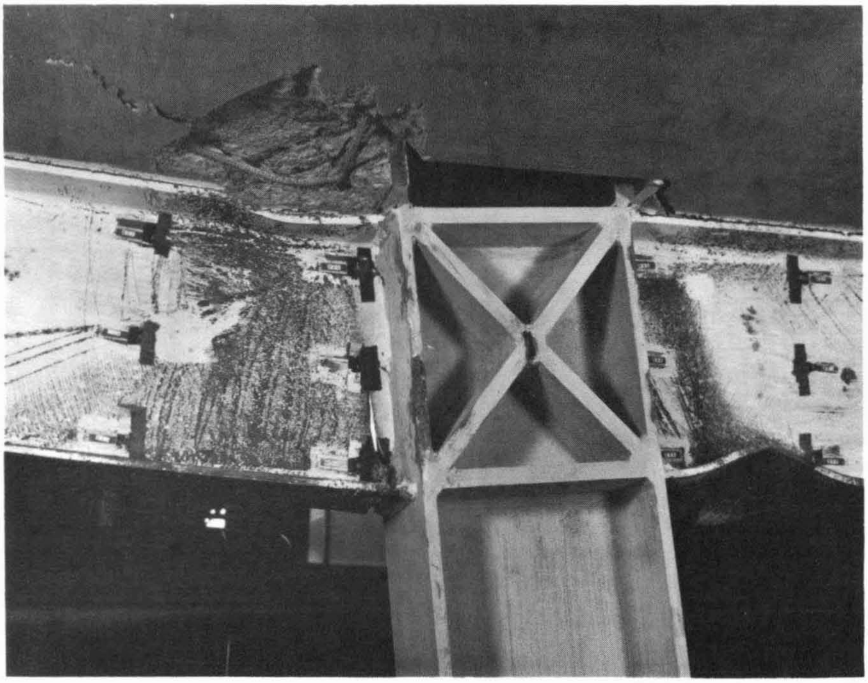


Fig. 16 Specimen J2 After Testing Showing Crushing of Slab and Deformation of Beam Flange

In Region 3, somewhat better behavior was obtained as shown in Fig. 12. Region 3 was produced in test specimen J2 (Fig. 7). The maximum moment capacity at location 4 of J2 was reached at the onset of significant local buckling of the compression flange of the steel beam. For larger curvatures the moment capacity decreased slightly. It was evident that the capacity was limited by local buckling. However, Fig. 12 indicates that if local buckling was prevented that the ultimate strength at location 4 would have approached the theoretical capacity based on the steel beam and the continuous longitudinal capacity based on the 72-in. slab width at this point. In addition, considerable rotation capacity would likely also have been available allowing the use of plastic design theory.

CONCLUSIONS AND RECOMMENDATIONS

A small pilot investigation was conducted at Lehigh University to explore the ultimate strength behavior of composite steel-concrete beams in frames which are subjected to combined loads. Two joint assemblies were constructed and loaded to simulate the combined loading conditions in the composite beams within the regions adjacent to the interior and exterior columns of an unbraced frame. The following conclusions are based on the results of this investigation:

(1) The ultimate strength of a cross-section of the composite beam adjacent to the column face and subjected to positive bending moment can be conservatively based on the strength of the steel beam plus a portion of the concrete slab which has a width equal to that which in contact with the column.

(2) The ultimate strength of a cross-section of the composite beam adjacent to the column face and subjected to negative bending

can be based on

a) the strength of the steel beam alone if the column is an exterior leeward column, and

b) the strengths of the steel beam plus the continuous longitudinal slab reinforcement over a certain width of slab. In this investigation the reinforcement appeared to be effective for the full slab width.

(3) Local buckling of the steel beam in negative moment regions must be prevented if the ultimate strength of the composite cross-section is to be reached.

(4) Sufficient rotation capacity of the composite beam exists in the vicinity of the joint to enable plastic design theory to be used in the analysis.

(5) The results of this investigation have indicated that the preliminary load-deflection analysis method developed for unbraced frames and described in Ref. 4, if extended to include multi-story frames with composite beams, would be valid.

The following recommendations for future studies are based on the results of this pilot study:

(1) Additional studies, both analytical and theoretical, are required to explore the range of ultimate strength which is possible in the vicinity of the column when the composite beam is subjected to positive bending moment. It appears that the use of devices which would provide increased confinement of the concrete near the column face would be desirable.

(2) Studies should be made which would indicate the length of Region 2 and the variation of ultimate strength behavior within this length. Such studies are required before the method presented in Ref. 4 can be used for frames with composite beams.

(3) The influence of an initial gap between the column face and the concrete slab requires investigation.

(4) The influence of joint detail, placement of shear connection and distribution of slab reinforcement on the width and distribution of slab cracking in the negative moment regions (Regions 3 and 4) should be studied

ACKNOWLEDGMENTS

The work described in this report was conducted as part of a general investigation into the plastic design of multi-story frames at Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. The experimental investigation upon which this study was based was sponsored by the American Iron and Steel Institute and Lehigh University. The study was supported by funds provided by the National Science Foundation for undergraduate research.

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