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The degree of composite action at beam-to-column joints (honors thesis, June 1968)

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THE DEGREE OF
COMPOSITE ACTION AT
BEAM-TO-COLUMN JOINTS

by

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A THESIS

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CERTIFICATE OF APPROVAL

This Honors Thesis is accepted and approved in fullfilment of the requirements of Creative Concepts 190.

May 18, 1968
Date

Dr. John W. Fisher
Professor in Charge

Dr. D. A. VanHorn, Chairman
Department of Civil Engineering

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1. INTRODUCTION

Plastic design methods have been successfully employed for the design of low steel building frames[1,2] and in recent years, for the design of braced multi-story frames.[3] Accordingly, interest in the extension of these methods to unbraced multi-story frames has grown. Plastic design employs the concept of the maximum or plastic strength of a structure as the basis for design. It is founded on the unique ductility of structural steel and on the ability of steel structures to redistribute moments as plastification occurs. Its application usually results in more efficient use of material, a more uniform factor of safety and relatively simple design procedures.

With today's demand for increasingly economical building frames, which at the same time must provide adequate strength and stiffness for gravity and lateral loads, attention has been focused on the use of composite steel-concrete beams in multi-story frames. The use of composite beams has been recognized by the AISC specification for many years.[2] However, their use has been limited to frames designed by allowable-stress methods. Investigations into the extension of plastic design methods to steel-concrete beams have been made, but these studies were limited to continuous beams subjected to gravity loads.[4,5,6]

The ultimate moment capacity of composite beams in the positive moment regions is determined by the plastification of the steel beam and by crushing of the concrete slab over an effective width. In the negative moment regions, the ultimate moment capacity is determined by the plastification of the steel beam and of the longitudinal slab reinforcement over a certain slab width.[6] Although the results of these investigations may be used for the design of composite beams in braced or unbraced multi-story frames subjected to gravity loads, they are not generally applicable for design of unbraced frames which consider combined gravity and lateral wind loads.[7]

Since no studies are available on the ultimate strength behavior of composite steel-concrete beams in unbraced frames which are subjected to combined loads, a small pilot investigation was initiated to explore the variables involved.[8] Two joint assemblies were designed and loaded to simulate the combined loading conditions in composite beams near the interior and exterior joints of an unbraced multi-story frame.

It is the purpose of this honors thesis to describe the results of a part of this investigation and to suggest further studies that are required. Reference 9 presents other aspects of this investigation.

2. DISTRIBUTION OF BENDING MOMENTS

An unbraced multi-story frame subjected to combined factored gravity (1.3W) and wind (1.3H) loads will develop a distribution of bending moments similar to that shown in Fig. 1. In this case, it has been assumed that the lateral wind load is large enough to result in positive moments adjacent to the leeward side of the joint. Such a distribution of bending moments will determine four regions which must be considered if an investigation of the ultimate strength behavior of composite beams under combined loads. These regions may be defined as follows: (See Fig. 1)

Region 1. An interior region in which cross-sections are subjected to positive bending moments and in which compressive forces act over the full effective slab width at the ultimate moment capacity.

Region 2. A positive moment region between Region 1 and the cross-section adjacent to the leeward side of a joint where compressive forces act on a reduced slab width. Adjacent to the column compressive forces will be developed only between the column face and the concrete slab which is assumed to be in contact with the column.

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

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

Regions 1 and 3 do not differ appreciably from similar regions of composite beams subjected only to gravity loads. This pilot investigation was designed to provide preliminary experimental data on the behavior of composite beams in Regions 2 and 4 and to provide further study of Region 3.

This thesis is concerned only with the ultimate strength behavior of composite beams in Region 2. Of particular interest is the mechanism by which the cross-sections adjacent to the leeward side of the joint develop their ultimate moment capacity.

3. TEST SPECIMEN DETAILS

Two nearly identical test specimens were used in the experimental phase of this investigation. A schematic view of one of the test specimens is shown in Fig. 2 together with the loading. Since the ultimate strength behavior of the beam cross-section adjacent to the leeward side of a joint were of main importance in this study, only shear type loading was used to simulate lateral wind loading of a joint. Gravity loads were not required. A short composite beam length was used in order to produce a steep moment gradient at the column face, a condition which can occur in the real frame.

The two test specimens used in this study were designated J1 and J2 and are shown in Fig. 3. In both cases, the steel fabrication was identical. The two beams for each test specimen were cut from the same 36-ft. length of 16W40, ASTM A-36 steel. The original beam was sectioned into thirds with the midsection being subsequently used for the various control tests described in Section 5. The beams were welded to the flanges of the column using full penetration butt welds to form the joint.

The two spreader plates shown in Fig. 3 and 4 which were welded to the flanges of the column adjacent to the slab, were so placed to simulate a 16-in. wide column flange. This would enable the columns in the test specimens to be reduced in size while keeping the member sizes at the joint nearer those found in an actual structure. The column section at the joint was stiffened substantially in order to keep joint deformations small. Thus the flexural characteristics of the joint could be eliminated as a variable in the test.

The shear connectors used were ½-in. diameter x 2-in. high headed steel studs and were welded along the top flanges of the beams as shown in Fig. 4. The spacing was kept uniform for most of the beam length at 5-in. in order to maintain nearly practical spacings. However the spacing was decreased near the free end of the beams in order to develop the ultimate moment capacity of the full composite cross-section without flexural failure of the studs.

For test specimen J1, the concrete slab was made continuous at the column in order to duplicate conditions at the windward and leeward exterior columns. The slab for test specimen J2 was continuous through the joint to duplicate the condition at an interior column. In each case the slab width was taken as 72-in to approximately conform with the effective width requirements of the AISC specifications.[2] This choice was arbitrary because no slab width requirements exist for plastic analysis of composite beams; however, Fig. 5 verifies that little additional theoretical moment capacity may be gained by an increased width. The asymptotic relationship may be explained by the fact that once full yielding of the W section has occurred, additional slab width will not result in a greatly increased moment capacity.

The concrete for the slab was transit-mixed and proportioned for a 28 day strength of 400 psi. The slab was reinforced with intermediate grade reinforcing bars as detailed in Fig. 6. The corners of the slab were cut off to eliminate unessential material and to get the test specimen into the test frame. Again for practically the slab thickness was taken as 4-in.

4. 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 beams were instrumented at these cross-sections with both 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 these strains. 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 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 the instrumentation used, one of the tension jacks and the compression dynamometer at the end of the beam in Region 2.

5. CONTROL TESTS

The mechanical properties of the 16W40 were determined from tension test specimens cut from a 2-ft. length of the 12-ft. section left over after the fabrication of the joint test specimens. The three web and four flange tension specimens were tested using a 120 kip mechanical testing machine. The resultant mechanical properties are shown in Table 1. Standard 6-in. x 12-in. concrete cylinders cast during the pouring of the concrete slab were tested to determine their compressive strength and modulus of elasticity. Table 2 presents 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 was used to determine the plastic moment capacity of the steel beam as shown in Fig. 10, 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. The resulting moment-curvature relationship is shown in Fig. 14. This test was carried out on a 300 kip hydraulic testing machine. The instrumentation used was similar to that used for J1 and J2.

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

6. TEST PROCEDURE

The test set-up is shown in Fig. 11. The applied joint moment was maintained by the hydraulic jacks, one at each end of the column. Beam reactions were taken by the pinned connection in the negative moment region of the test specimens (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 connection and rocker support allowed the joint the freedom of horizontal and vertical movement during loading.

Loading increments of from nearly zero to five kips at each tension jack were applied to the test specimens up to failure. The initial loading increments were fairly uniform and strain controlled. As plastification of the joint area occurred, the tension jacks maintained a nearly constant load up to the development of the moment plateau. The test specimens were unloaded and reloaded at different stages of testing for various problems in the test frame. Photos in Fig. 12 further detail the test set-up.

Due to the fact that there is a lower theoretical moment capacity in the negative moment region, a bracing or stiffening procedure as shown in Fig. 13 had to be employed to allow the testing to failure of the joint. The W-section diagonal braced was clamped into position only after excessive deformation of the negative moment region cross-section threatened to halt the test.

Excessive deformation occurred in the loading frame used with the test specimens (see Fig. 11) 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 but the testing was finally halted when the rocker support again collapsed. The joint was therefore not tested completely to failure, but ended with partial slab crushing and beam plastification.

The loading frame problems were corrected prior to the testing of J2 and a roller support replaced the rocker support to give greater flexibility of horizontal movement. Joint J2 was subsequently tested until failure caused by crushing and pushing out of the slab near the joint and the resultant failure of the W section which was unable to carry the flexural stresses alone.

7. TEST RESULTS

The concluding discussion in this report reflects mainly the behavior of the positive moment regions of the test specimens, with the negative moment regions being discussed in detail in Ref. 9. Fig. 14 summarizes the moment-curvature relationships for test specimens J1 and J2 and the control beam. These plots are based on measurements adjacent to the leeward side of the column face. The moments were calculated neglecting the dead load of the frame and considering only the live load beam reactions. J2 reached an ultimate moment of 5050 in-kips while J1 was tested only to a maximum moment of 4725 in-kips. Local buckling was not a problem in the development of the moment capacity of either specimen. The curvature calculations were taken from either the electrical or mechanical strain measurements depending on which one provided the more reliable result. At lower levels of strain generally electrical measurements were used, but at higher strains a graphical procedure was employed to judge the reliability of the various measurements.

Calculations of either axial force or moment from strain data have been omitted for reasons explained in Section 8 of this report. Slip patterns for the positive moment side of each test specimen as shown in Fig. 15 were similar. Initial slab movement under load was toward the column face but eventually the pattern reversed as significant pressure was exerted by the column on the slab. J2 experienced much

much greater slip than J1 even with the presence of a continuous slab. Likewise, J2 underwent much more joint rotation reflecting this behavior.

Figures 16 to 18 show the various conditions of the joint specimens after failure. The cracking and crushing patterns for the positive moment (marked north) slabs are shown clearly in Figs. 16 and 17. The slab crushing for J2 was more extensive and continued for the depth of the slab; however it should be recalled that J1 was not tested to failure. The cracking patterns for J1 and J2 vary considerably. In J1 the slab underwent significant compression cracking over the full width. However, in J2 the cracking was limited to two parallel planes lying symmetrically over either side of the beam flange. It thus appears that the slab for J2 was subjected to other outside influences, some of which are discussed more fully in Section 8.

Figure 18 details the conditions of the steel beam near the joint after failure.

8. ANALYSIS OF RESULTS

The moment-curvature relationships summarized in Fig. 13 are shown as non-dimensional plots in Fig. 19 assuming that a theoretical yield and ultimate moment are calculated assuming that a 16-in. slab width is effective for a full depth of 4-in. Also shown in Fig. 19 is the calculated value of initial joint stiffness considering the same reduced cross-section.

It was apparent from Fig. 14 that a flexural capacity based on the 72-in. slab width was unrealistic as a design criterion for a cross-section near the leeward face of the column. Likewise, there was a lack of correspondence between observed and theoretical initial stiffness of the joint when considering the full slab width. It was assumed in Section 2 of this report that the positive moment region adjacent to the joint would be subjected to compressive forces only between the slab and the column flange. This fact may be born out upon an examination of the mode of failure of the joint. The joint was so fabricated that failure of either the column or shear connectors would not be a controlling factor. Likewise, once the negative moment region of the test specimen underwent significant deformation, the installation of the diagonal brace allowed testing of the joint to continue. Thus joint failure would result only from a crushing or wedging out of the concrete slab in compression and the resultant failure of the beam under positive moment. Visual observations, as shown in Figs. 16

and 18, indicate that slab crushing occurred only in the region of the column face and that the remaining concrete contributed little except for providing anchorage for the continuous reinforcement in the case of J2. Thus it was assumed that the width of the column flange, or in this particular instance, 16-in, was a governing criterion.

Based on the results shown in Fig. 19 it is apparent that the observed and theoretical initial stiffness values based on a 16-in. width are in agreement. Furthermore, the theoretical ultimate moment of a composite section with the 16-in slab could be obtained without excessive deformation. It has been suggested that the ultimate moment should be reached within curvatures not exceeding 10 to 15 times yield curvature to coincide with the onset of strain hardening.[10]

The significant increase in rotation capacity at sustained moment for J2 over J1 was possibly a result of the greater degree of plastification of the joint due to the presence of a continuous slab.

In actuality, both joint test specimens were able to develop a moment capacity approaching that for a cross-section with a 72-in. slab. However, it was just shown that only that region of the slab in direct contact with the column face, a 16-in. width, was effective in flexure. One explanation of this phenomenon is that the slab adjacent to the column was in a bi-axial or tri-axial state of stress such that confining pressures allowed large compressive stresses, well above the crushing strength of concrete as determined by standard cylinder tests (f'c.) The existance of such pressures is reasonable considering the confining effects resulting from the maze of reinforcing bars, shear connectors and the flanges of both the beam and column. This observation will itself require much extensive study, however the use of improved confinement techniques could be the key to reaching higher useful moments.

If these observations are shown to be true then to take full advantage of this strength would require a means to control deformation. It is possible that this may be accomplished by using wider spreader plates on the column flanges, but this also must be investigated.

In Section 7 it was mentioned that some of the results were left unreported due to certain uncertainties. Explanations of this as well as several recommendations for future testing may be as follows: The majority of the problems concerned the strain data from the steel W sections-both electrical and mechanical readings. It was noted earlier that the strain gages were applied to both sides of the web of the W. It is felt that more reliable data would have resulted if the strain gages had been applied to the flanges. This appears to be the conclusion also reached from past experience.

It was hoped to make more conclusive use of the slab strain data, but due to a lack of a sufficient number of data points, this was not possible. This is important because a greater indication of the state of slab stress must be known before any serious design recommendations can be made. Likewise, for the continuous slab the reinforcing bars passing through from one region to another should be strain gaged. Both strain readings and studies of crack patterns indicate that a portion of the positive moment side of the slab outside the 16-in. strip was actually in tension. Whether this is due to cracking of the slab surface or as a result of the continuous reinforcement is not known.

Thus the discussion of axial force distributions throughout the beam length has been omitted from this report. It was expected to find an increasing axial force in the beam toward the column. In this case however at higher loads the slab and beam force appeared to fluxuate irregularly. Until future studies can verify that this is possible due to a force transfer from the column face or from the reinforcement, it is improper to present this as a true phenomenon.

It is apparent from Figs. 14 and 19 that there is a definite variance between theoretical and measured yield curvatures. This reduction of initial joint stiffness may be initially accounted for by the failure of the beam and slab to behave completely compositely. This is probably the result of the initial slips due to the presence of a shrinkage gap between slab and column.

9. RECOMMENDATIONS AND CONCLUSIONS

Although this particular study indicated that the ultimate moment capacity of the test specimens need be reduced to that based on a cross-section with a slab width equalling the width of the column flange, it is quite possible that future research may show that a greater slab width can be relied upon. Thus the design criterion presented here should provide a lower bound to the true strength. It was apparent from these two tests that many factors effecting the capacities of the joints, such as the state of stress near the column face, need thorough study. It appears that the use of devices to provide better confinement of the concrete such as spreader plates offer much promise.

The recommendations suggested in the last portion of the analysis should be followed in future testing.

10. ACKNOWLEDGMENTS

The author wishes to acknowledge the guidance and encouragement of Professors John W. Fisher and J. Hartley Daniels who supervised this honors thesis.

The work described in this thesis 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. This study was supported by funds provided by the National Science Foundation for undergraduate research.

The manuscript was typed by Miss K. Philbin.

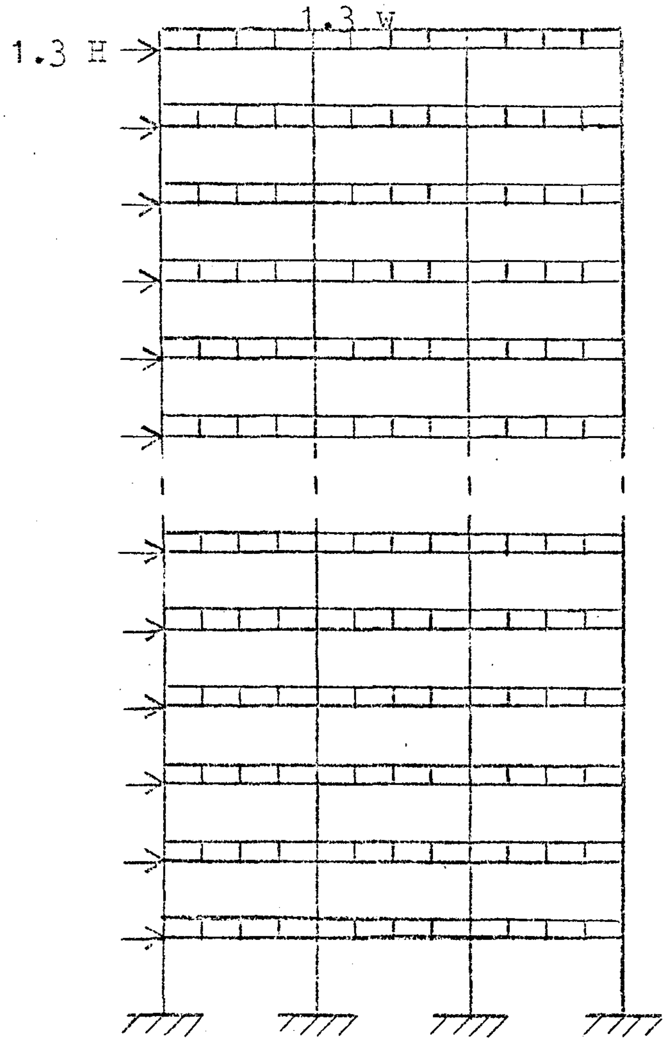
11. TABLES AND FIGURES

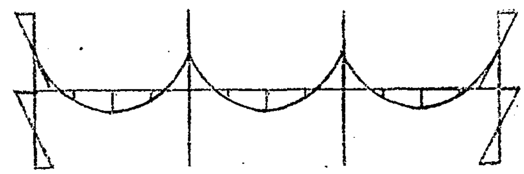
Mechanical Properties of 16WF40		
Property	Flange	Web
Yield Stress	36.4 ksi	40.0 ksi
Modulus of Elasticity	29266.6 ksi	29900.0 ksi
Strain Hardening Strain	.02520 in/in	.02394 in/in
Strain Hardening Modulus	466.0 ksi	492.0 ksi

TABLE 1

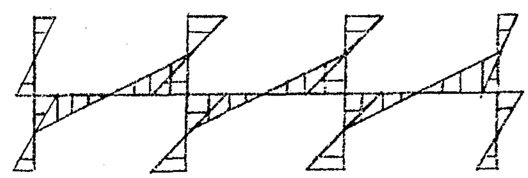
Mechanical Properties of Concrete	
Compressive Strength	4110 ksi
Tangent Modulus	2480 ksi

TABLE 2

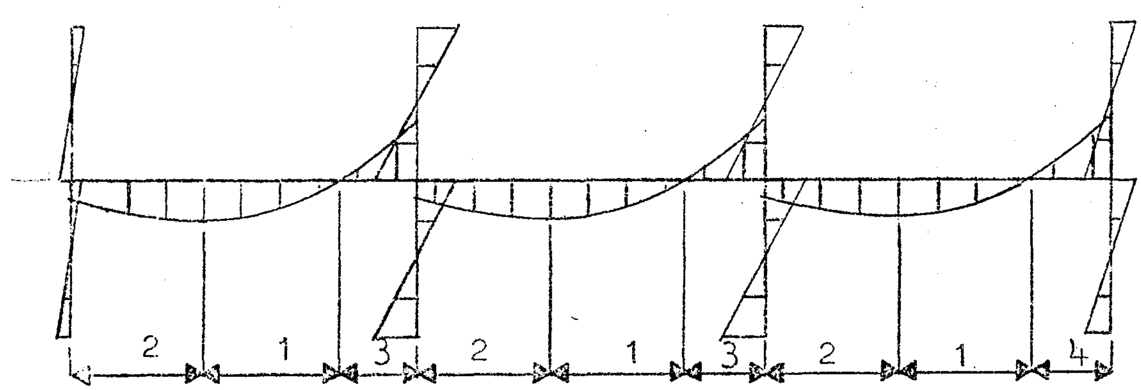




BENDING MOMENTS DUE TO FACTORED GRAVITY LOADS



BENDING MOMENTS DUE TO FACTORED LATERAL WIND LOADS



BENDING MOMENTS DUE TO FACTORED LOADS

FIG. 1- BENDING MOMENTS IN UNBRACED MULTI-STORY FRAMES

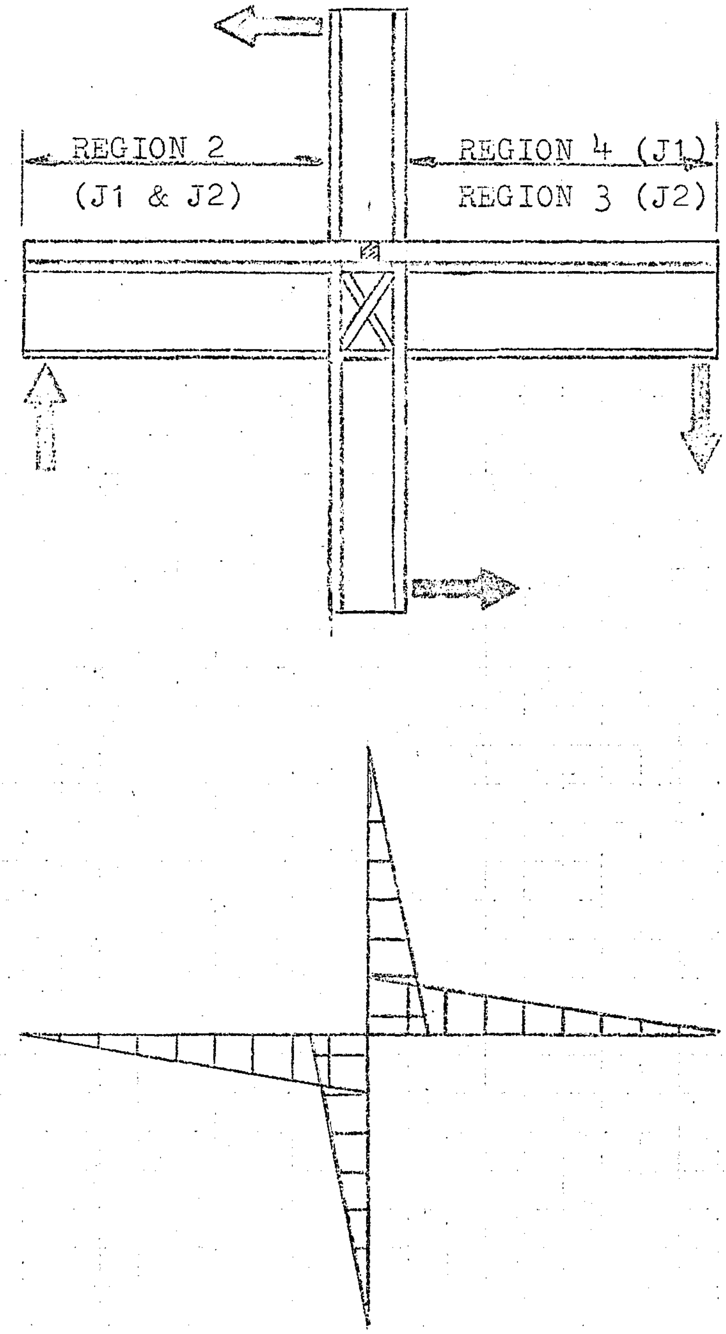


FIG. 2- SCHEMATIC VIEW OF TEST SPECIMENS AND TYPICAL LOADING

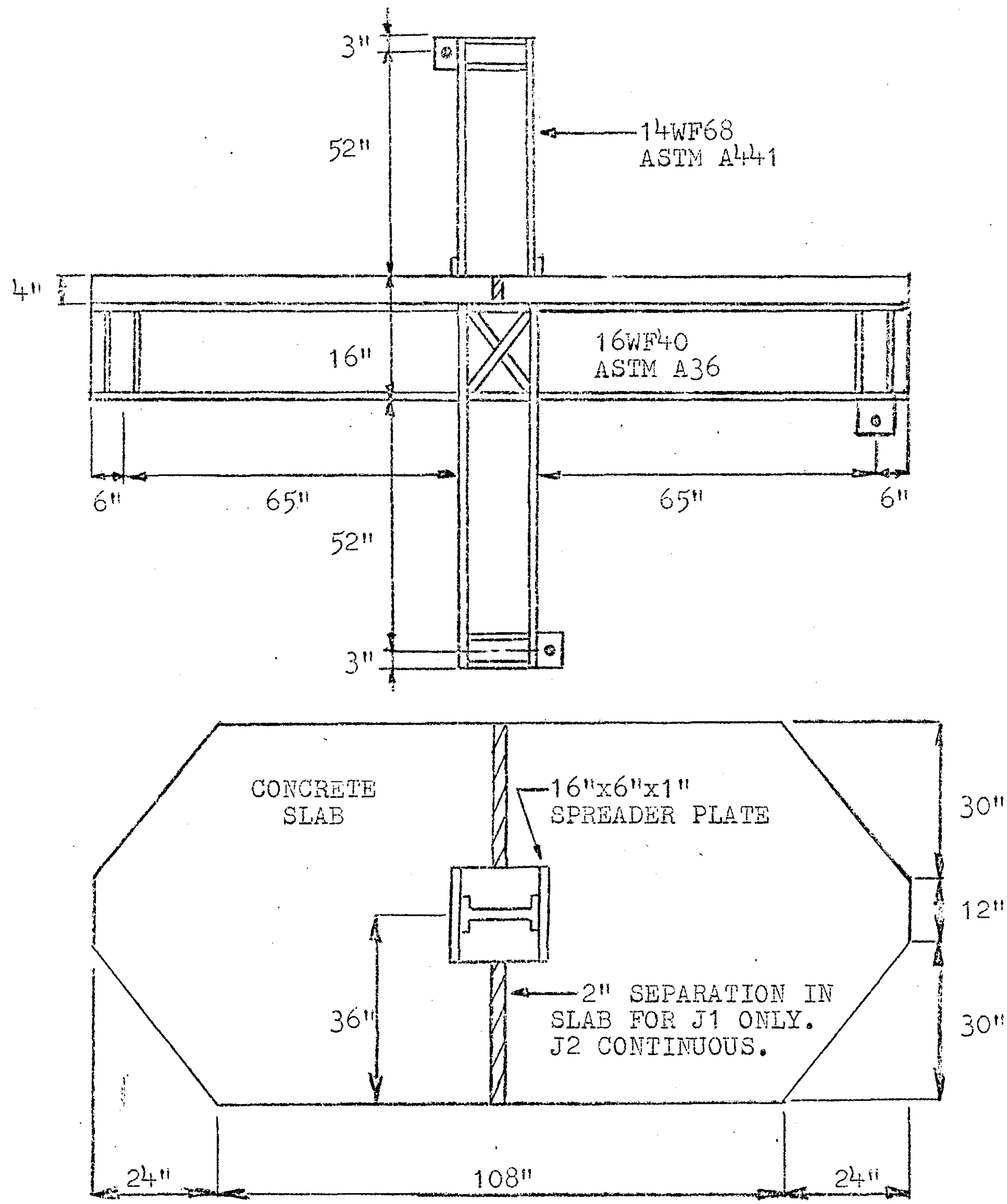
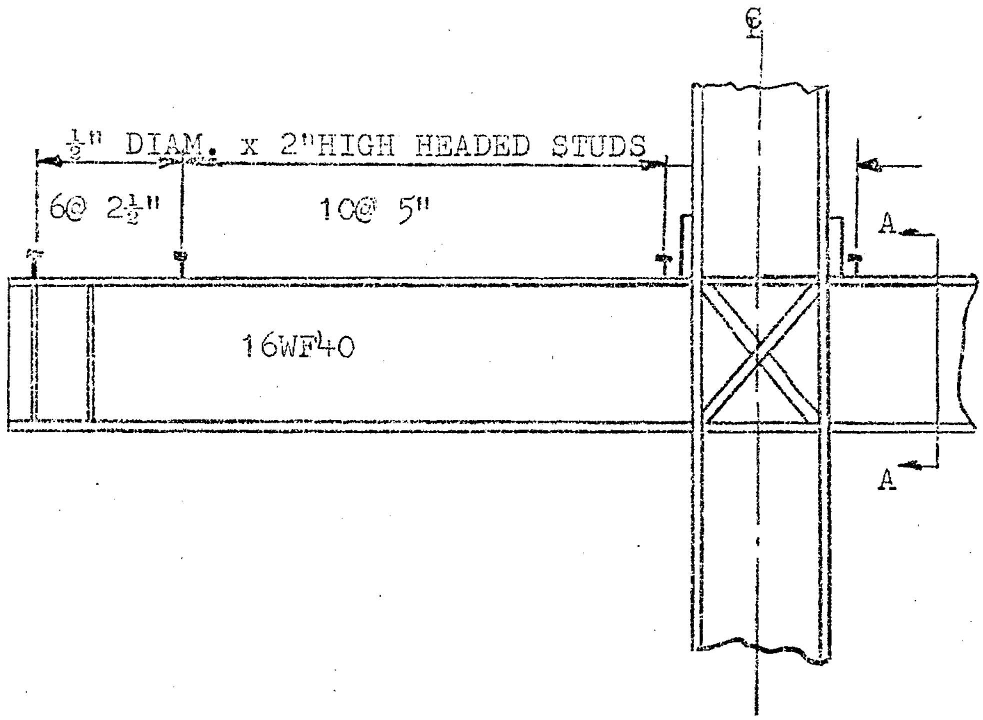
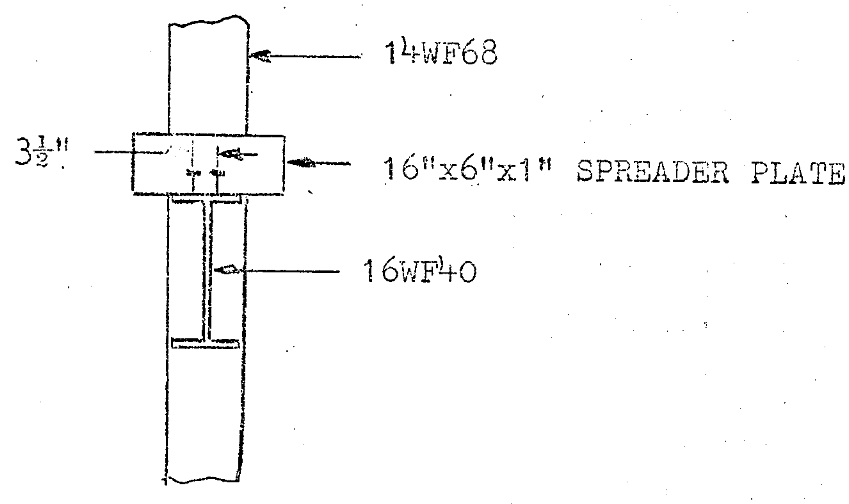


FIG. 3- DIMENSIONS OF TEST SPECIMENS J1 AND J2





SECTION A-A

FIG. 4- DETAILS OF TEST SPECIMENS J1 AND J2

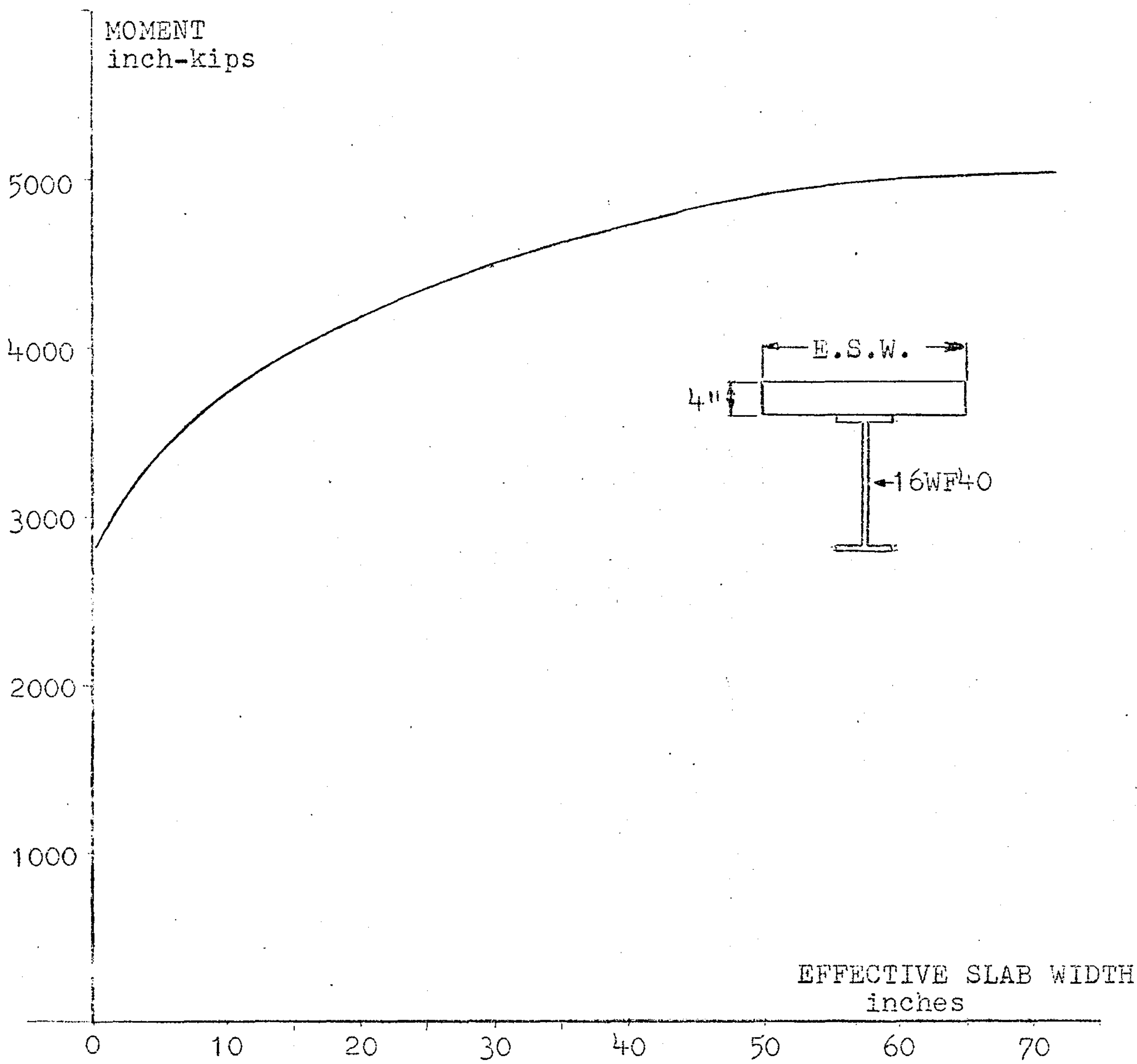
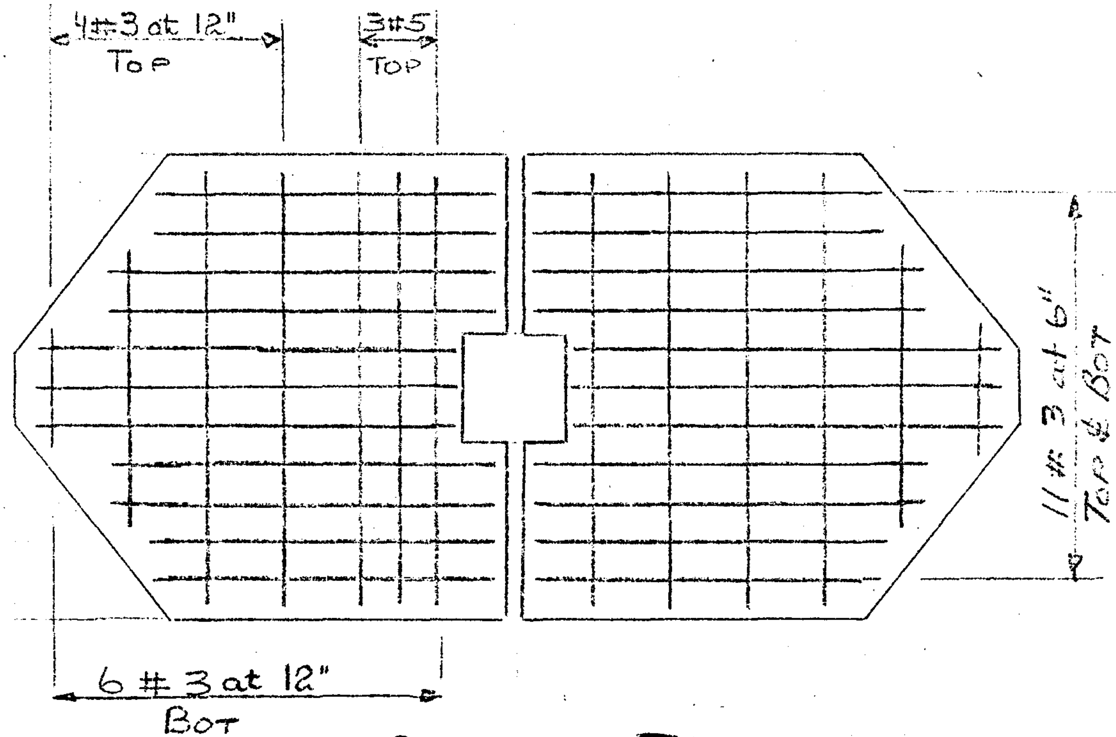
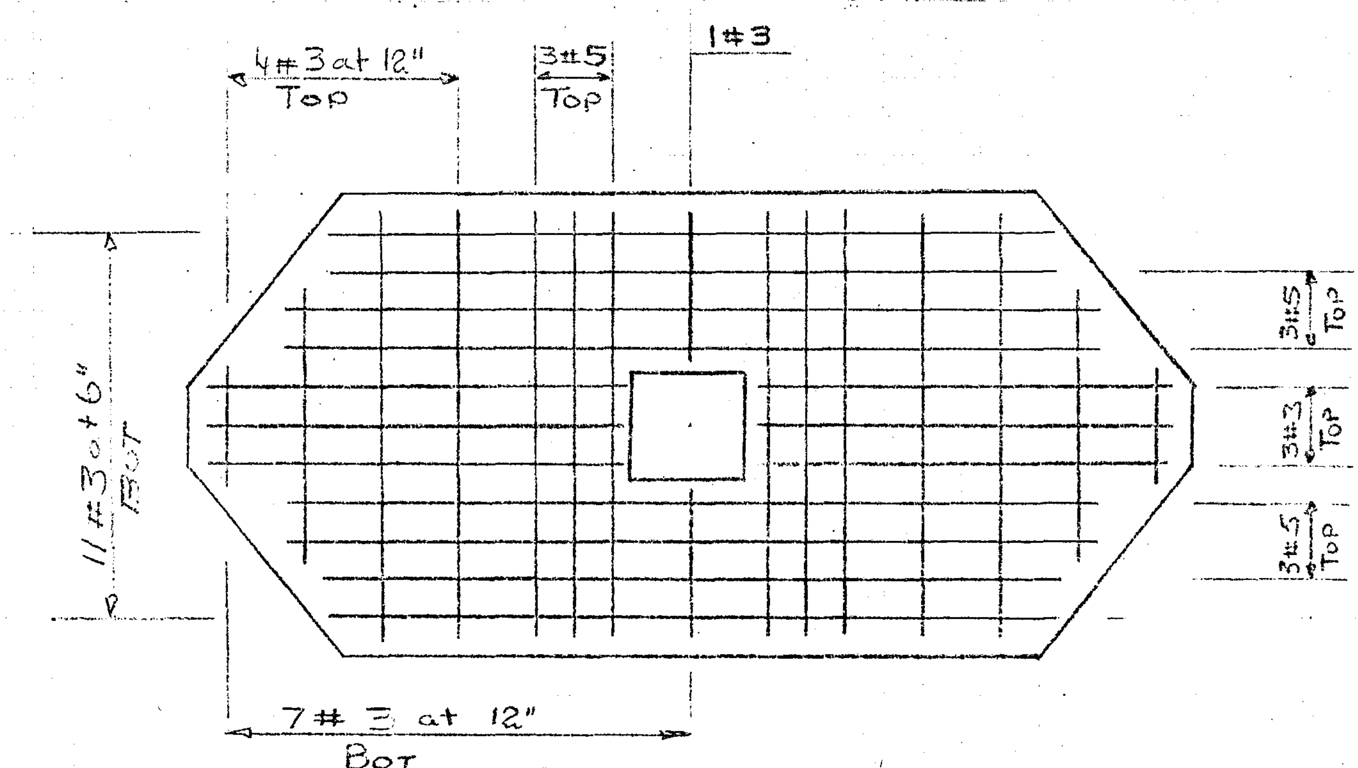


FIG.5- ULTIMATE MOMENT - EFFECTIVE WIDTH RELATIONSHIP



SPECIMEN J1



SPECIMEN J2

FIG. 6- SLAB REINFORCEMENT FOR TEST SPECIMENS J1 AND J2

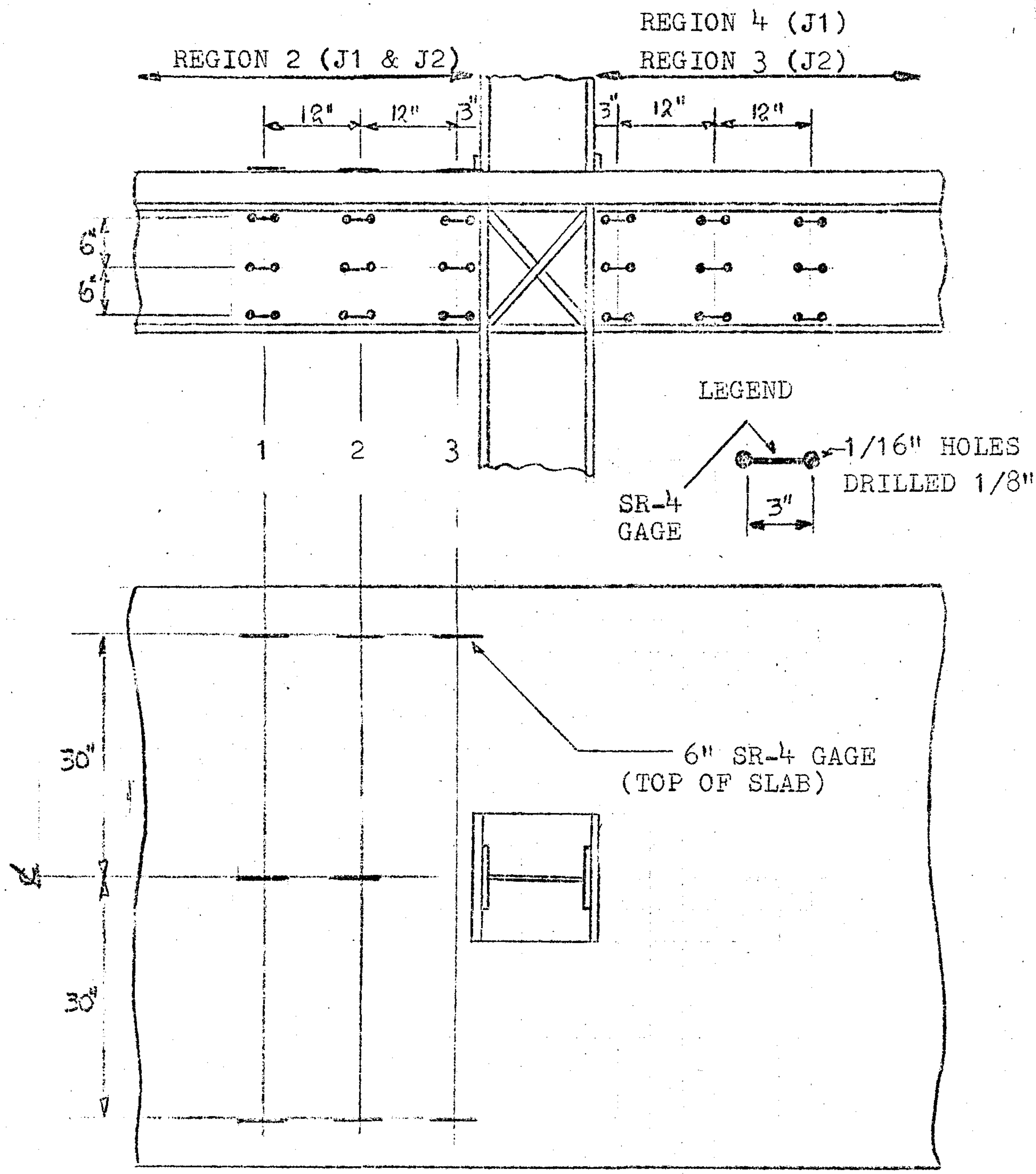


FIG. 7- INSTRUMENTATION LOCATION - STRAIN GAGES

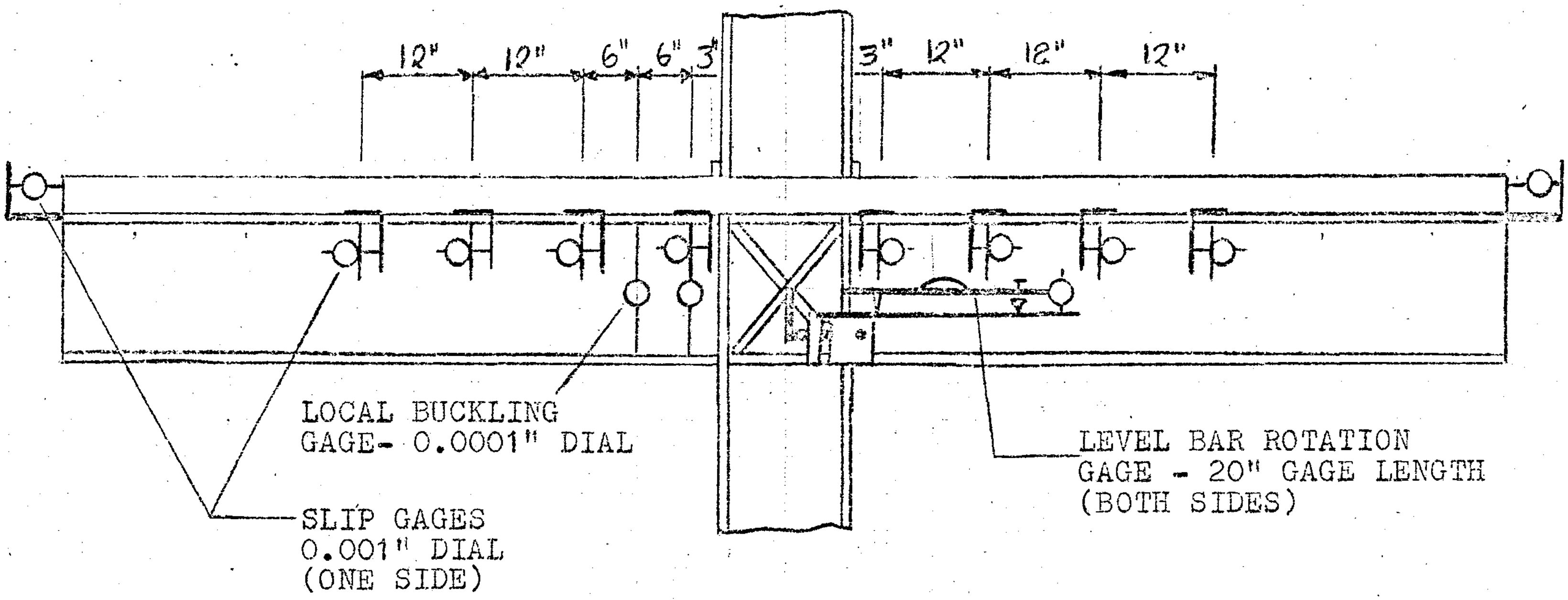


FIG. 8- INSTRUMENTATION LOCATION - SLIP, ROTATION AND LOCAL BUCKLING GAGES

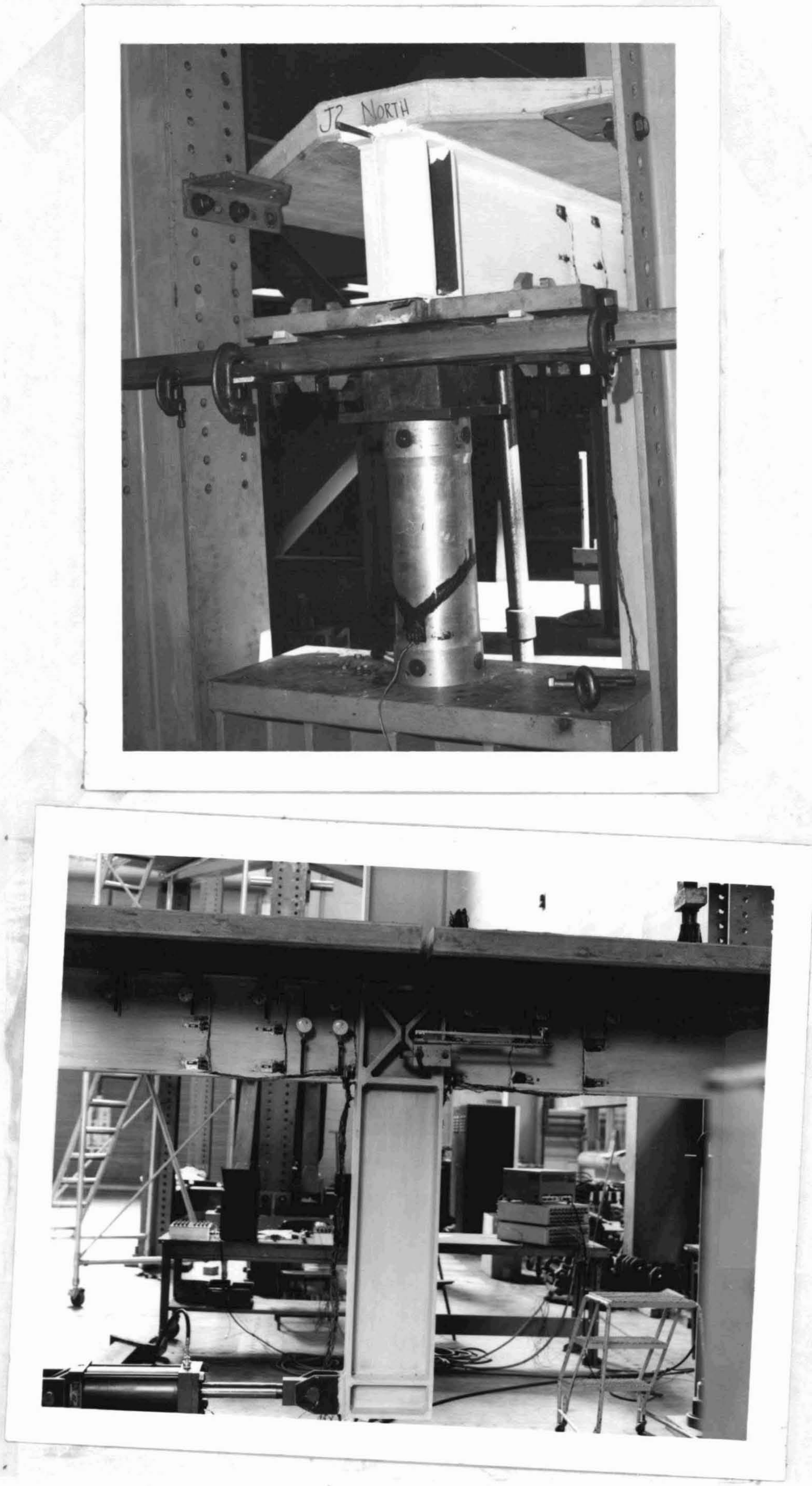


FIG. 9- INSTRUMENTATION DETAILS

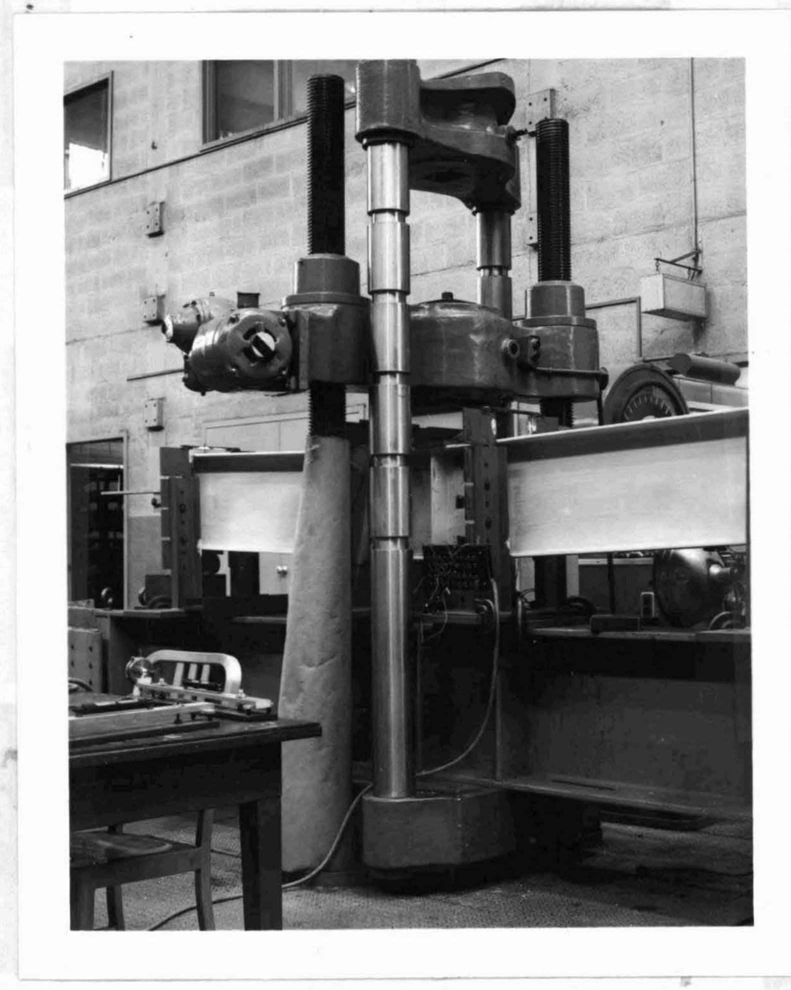


FIG. 10- CONTROL BEAM TEST

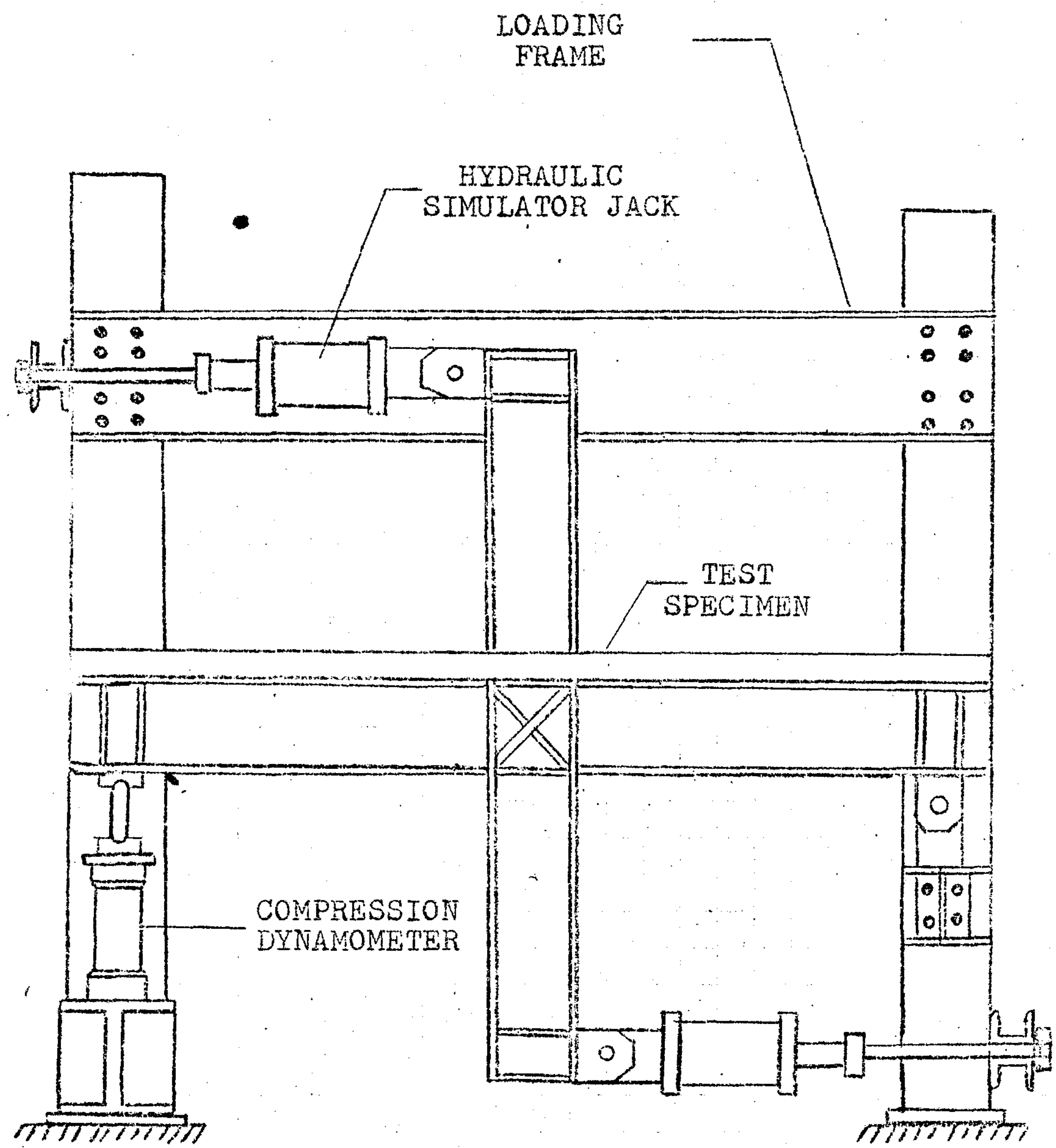


FIG. 11- TEST SET-UP

FIG. 12- DETAIL OF TEST SET-UP

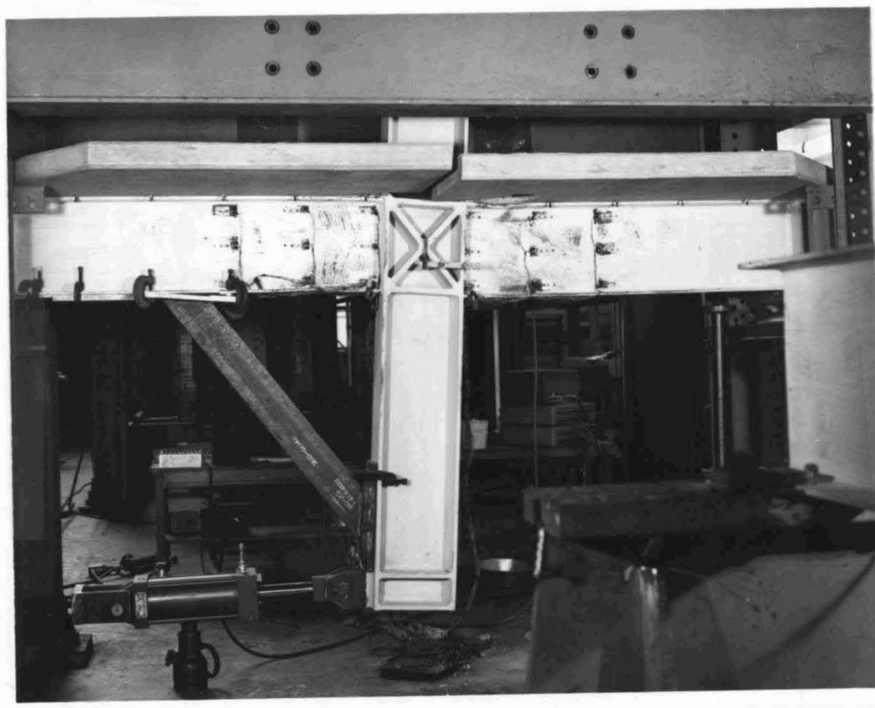


FIG. 13- BRACING OF NEGATIVE MOMENT REGION

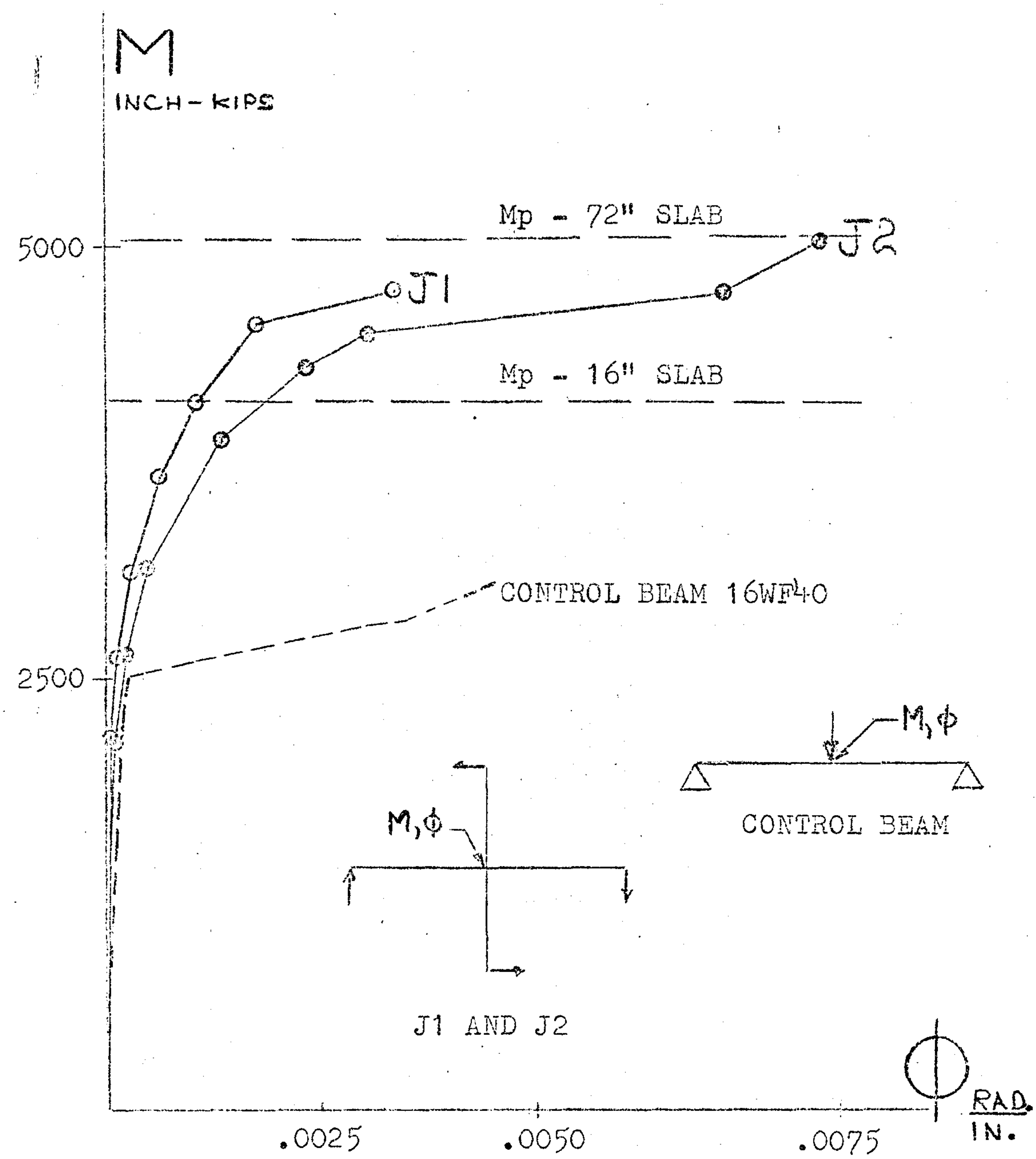


FIG. 14- MOMENT-CURVATURE PLOTS

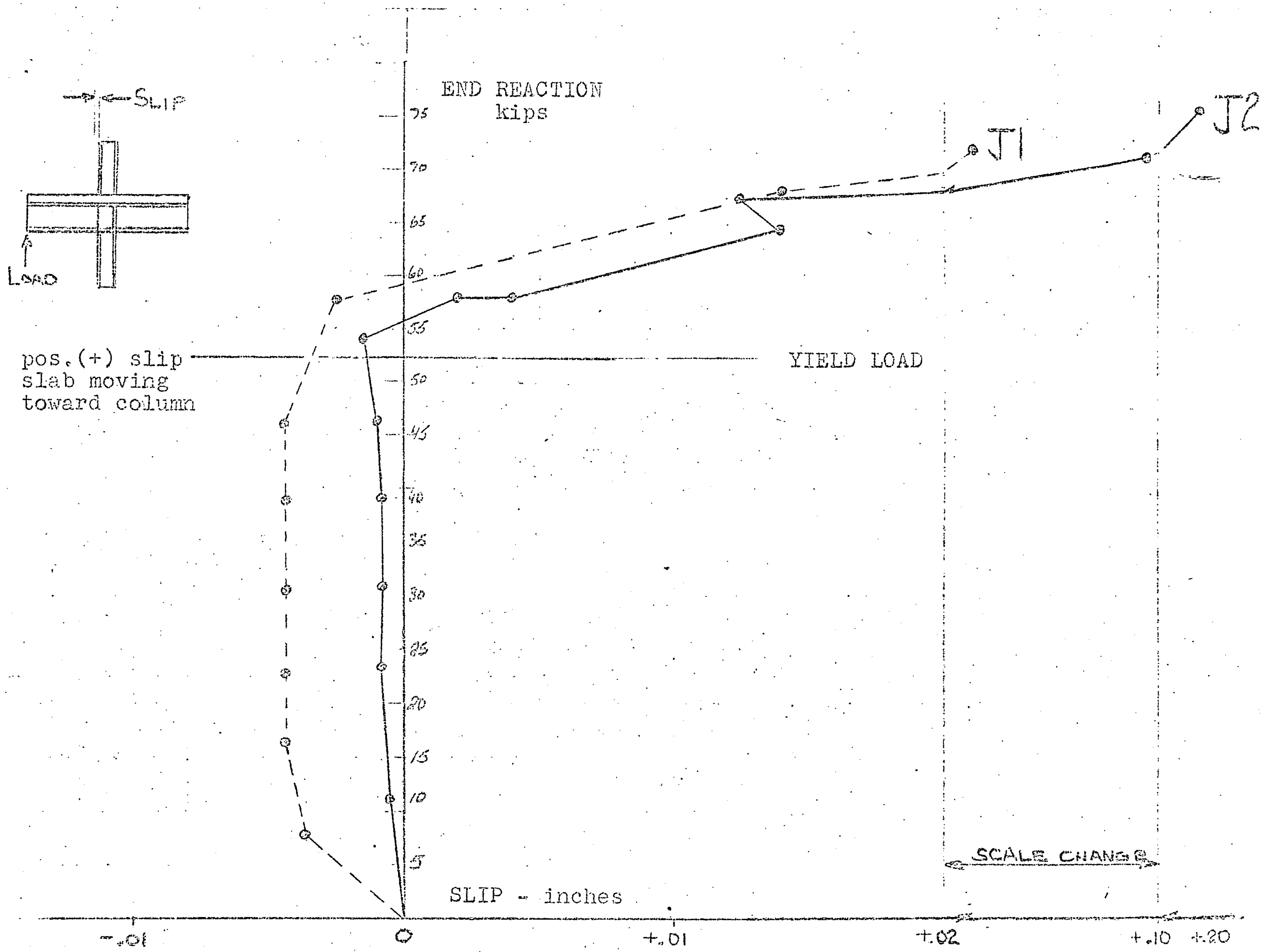


FIG. 15- END REACTION - SLIP RELATIONSHIPS

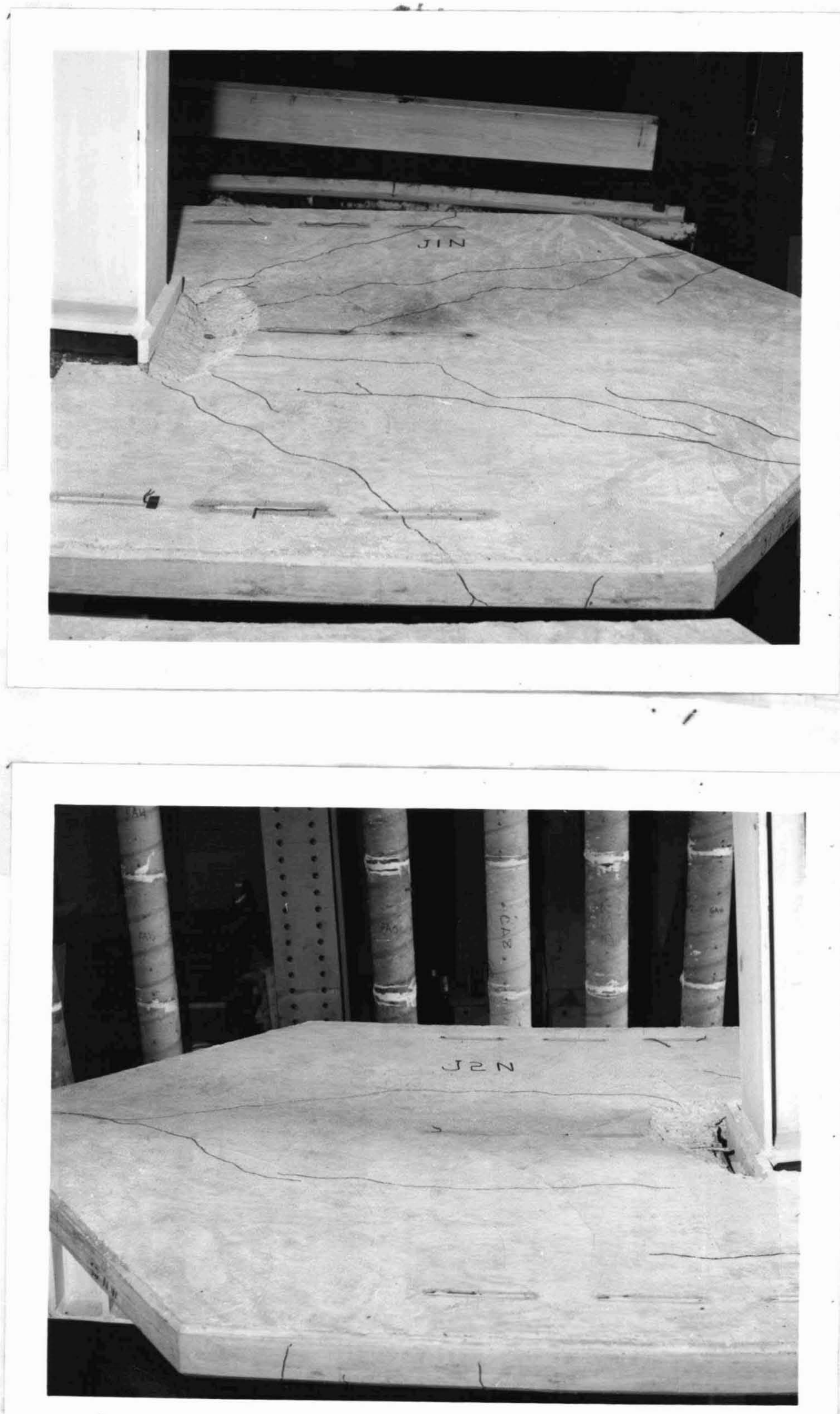


FIG. 16- SLAB CRACKING PATTERNS

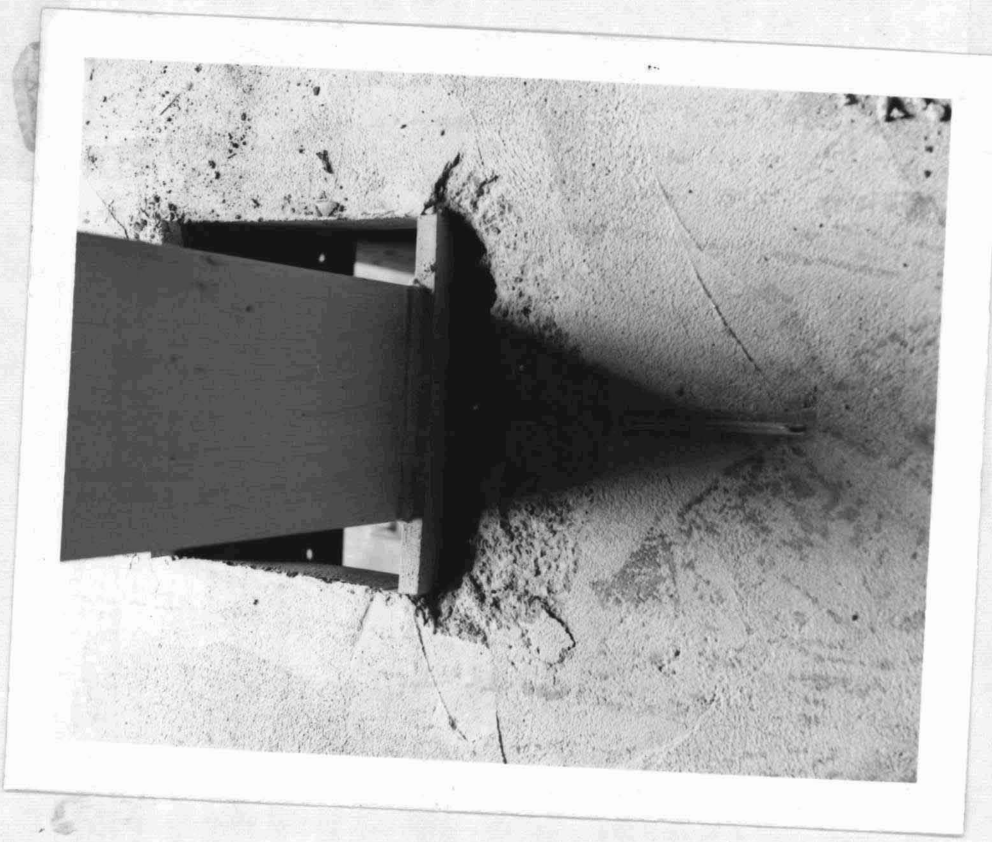
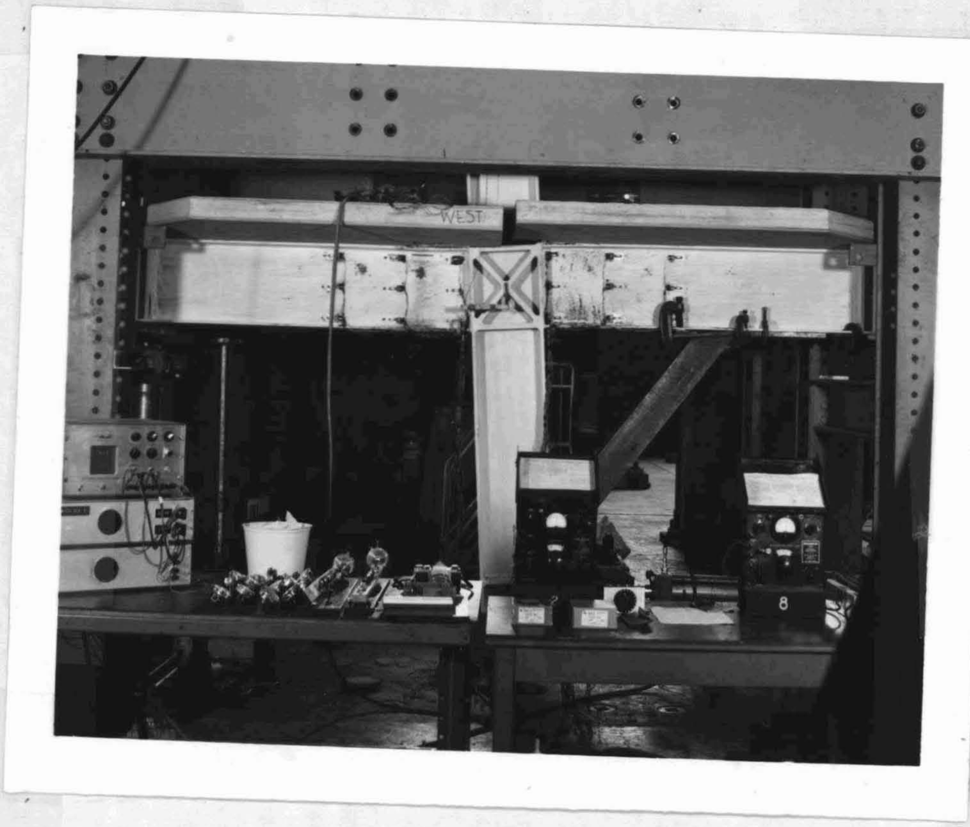


FIG. 17- SLAB CRUSHING DETAILS



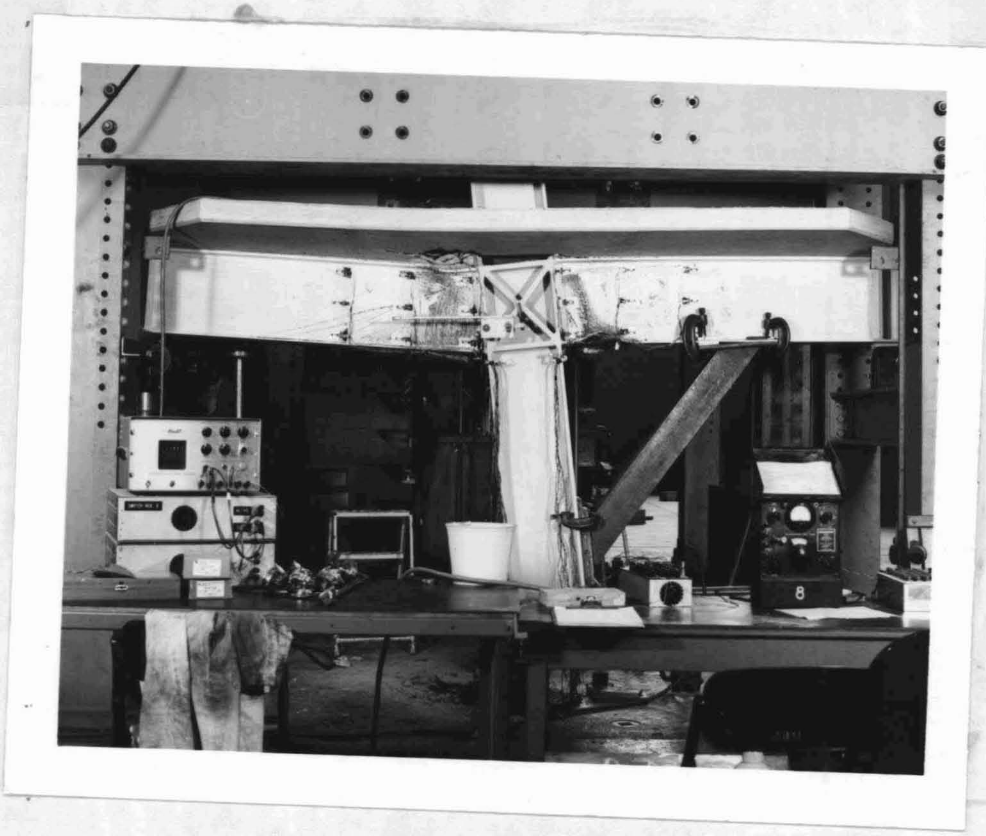


FIG. 18- JOINT FAILURE DETAILS

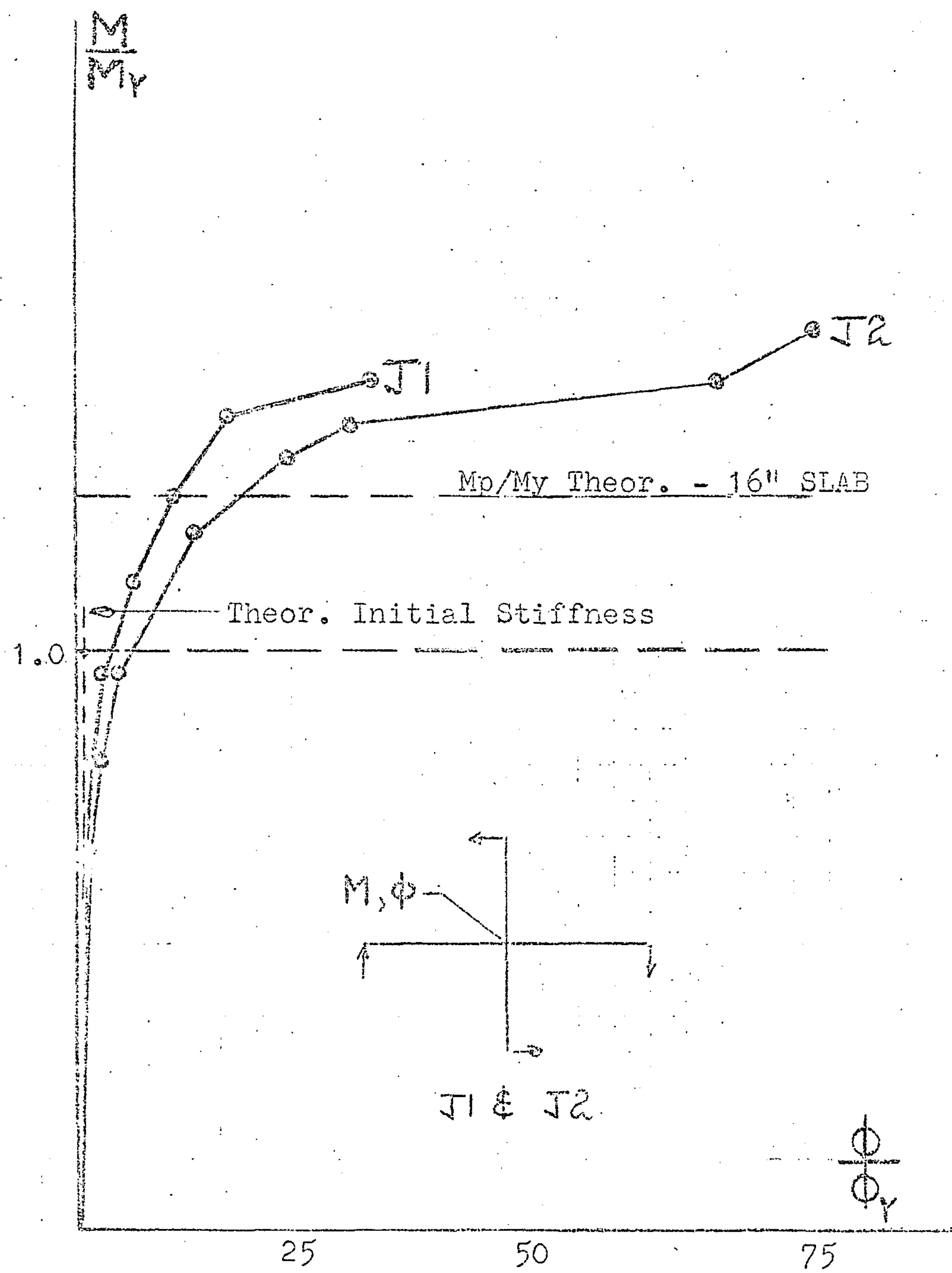


FIG. 19- MOMENT-CURVATURE PLOTS

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