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Precast Chemically Prestressed Concrete Frame

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by

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PRECAST CHEMICALLY PRESTRESSED CONCRETE FRAME

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ABSTRACT

This report describes in detail the fabrication and experimental studies of two prestressed concrete frame models. The fabrication of the models was carried out in order to investigate the feasibility of constructing concrete frames by connecting simple precast chemically prestressed structural elements. All elements had a 2.7/8 by 4-inch rectangular cross section and a nominal length of about 40 inches. Total percentage of reinforcement was 1.74. The structural elements were all self stressed by chemical prestressing through the use of an expansive cement developed recently at the University of California, Berkeley. History of the expansion and the corresponding stresses developed during the curing process of these elements are presented.

To assure continuity at the knees of the frames, the connections between the precast prestressed elements were developed by the use of special end-bearing plates and high-strength steel bolts.

The experimental work consisted of a series of tests programmed to study the mechanical behavior of the simple structural elements and of the frames under proportional loading.

According to the moment-rotation curves obtained in the tests carried out on two simple structural elements, it is shown that the real moment-curvature relationship can be approximated by the idealized elastic-perfectly plastic curve used in the conventional "plastic hinge theory" as suggested by Baker. That this approximation introduces only minor errors is demonstrated by comparing the results obtained in the tests carried out on the frames with the corresponding analytical values calculated on the basis of such theory.

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PRECAST CHEMICALLY PRESTRESSED CONCRETE FRAME

I. INTRODUCTION

Statement of Problem

A recent trend in reinforced concrete construction in U.S.A. is to use high-strength deformed reinforcing bars ranging in yield point from 55 to 120 ksi.(1)* The architectural advantages and economy of this kind of reinforcement have been outlined in a paper in 1941.(2)

The main problem encountered in using high-strength steel is the control of cracking for the service load level. This problem may be partially overcome by several means such as using well-deformed bars and distributing the bars as uniformly as possible over the concrete zone of maximum tension. Another possible solution toward inhibiting the formation of cracks is to eliminate or to reduce tensile stresses by appropriate prestressing. Presently the three possible prestressing methods are mechanical, electrical, and chemical. Of these, the only one that up to the present has been developed to an adequate stage is mechanical or conventional prestressing. However, to the best of the authors' knowledge, no attempt has been made in U.S.A. to fabricate precast prestressed structural elements by mechanically prestressing high--strength deformed bars. This lack may be due to the fact that the percentage loss of prestress due to shrinkage and creep may be so high that the process will be uneconomical.

Recently an expansive cement which was developed at the University of California, Berkeley,(3) has been successfully used in pilot studies for production of what may be called "chemically prestressed concrete structural elements" such as slabs, pipes, and hyperbolic paraboloid shell (4) (5). Therefore the authors decided to investigate the possibility of using this expansive cement in the prefabrication of chemically prestressed structural elements for framed structures using high-strength deformed bars.

Since the technique of eccentric prestressing with expansive concrete is yet to be developed, the study reported herein was reduced to the investigation of the potentialities of the chemical method for prestressing concentrically structural elements. Although this may be thought to be a severe limitation in the possible application of these elements, actually it is not since in the case of buildings subjected to remote blasts, wind gusts, or earthquake effects, the structure may be subjected to loading patterns which are nearly fully

*Superscript numbers in parentheses refer to corresponding entries in the references.

reversible, and therefore require designing of structural elements with the same amount of compressive and tensile reinforcement giving opportunity to obtain concentric self prestressing. Moreover, it is felt that by proper combination of precast chemically concentrically prestressed structural beams with cast-in-place slabs, final eccentrically prestressed structural elements can be obtained.

General Objective of the Investigation

The general objective of the investigation was threefold:

1. To investigate the feasibility of fabricating precast-prestressed double reinforced structural elements--beams and columns--by a chemical method and through the use of high-strength deformed bars and the expansive cement developed at the University of California, Berkeley.

2. To study means of connecting the prefabricated structural elements to built-up rigid frames.

3. To study the mechanical behavior of precast prestressed rigid frames under load.

This report describes in detail the fabrication of the precast-prestressed structural elements and the manner in which these elements were connected to build up two rigid frames. The history of expansion observed during the curing of the structural elements is presented as well as that of the control specimens. The tests carried out on control specimens to determine the mechanical properties of concrete and steel, and the mechanical behavior of the precast-prestressed structural elements--which were required to predict mechanical behavior of the frames--are also described herein. Finally, the report presents the results of the tests carried out on two rigid frames and the comparison of their actual behavior with the predictions of theory.

General Considerations Regarding the Selection of Type of Rigid Frame

The type of rigid frame selected for the investigation reported herein is shown schematically in Figure 1. According to the nominal dimensions indicated in this figure, the rigid frame can perhaps be considered a model of a simple one-story, single-bay rigid frame. The dimensions were chosen to yield the smallest structural elements--beams and columns--that could be conveniently fabricated and tested using the minimum size of commercially available deformed bars.

The study of the mechanical behavior of the frame was carried out by loading the structure by means of a vertical (V) and a horizontal (H) concentrated force as indicated in Figure 2. Proportional loading was adopted, i.e., the loads were increased in fixed proportion from zero up to the magnitude which brought the structure to collapse. This loading condition might be thought of as a simplification of a combination of dead load and vertical live load with the effects of remote blast, wind gusts, or earthquake.

II. ANALYTICAL STUDIES

Prediction of Instantaneous Collapse Load

As a guide to developing an adequate testing procedure and to permit comparison of experimental results with the predictions of theory, a limit analysis of the frame was carried out. This analysis was made according to the simplified assumptions of the "Plastic Hinge Theory" suggested by Baker.[6]

In this theory Professor Baker uses the idealized moment curvature relationship shown in Figure 3a. In the application of this theory to the frame of Figure 1, it is assumed that at collapse the frame can be considered as a series of elastic members, joined by sufficient number of frictionless plastic hinges, to make the structure as a whole or part behave as a mechanism. The frictionless hinges are concentrated at points, for which the moment--load relationships follow the diagram of Figure 3b.

To determine the progressive formation of the plastic hinges along with the progressive theoretical values of rotations and deflections, a step-by-step calculation was carried out on the idealized frame indicated in Figure 4. From this figure it can be seen that the actual location of the plastic hinges has been taken into account by considering the steel joints at the knees and bases as blocks of infinite moment of inertia.

The results of the step-by-step calculations are summarized in Figure 5 where the computed values of the deflections--horizontal, ΔH, and vertical, ΔV--and those corresponding to the plastic-hinge rotations $\theta_{E'}$, $\theta_{D'}$, $\theta_{A'}$, $\theta_{D''}$, and $\theta_{B'}$ expressed in dimensionless terms, are plotted against the nondimensional load parameter $P\frac{L}{M_p}$.

It is interesting to note that hinge rotation at Section D' has started when $P\frac{L}{M_p} = 3.665$. However, when the load reaches a value such that $P\frac{L}{M_p} = 4.093$, rotation at this section ceases and the bending moment there is reduced below its fully plastic value. Therefore we have a case that points out the importance of tracing the successive formation and rotation of the plastic hinges by means of a step-by-step analysis. Any of the conventional methods that have been proposed for estimating deflections at the point of collapse[7] would have given an incorrect answer in this case, because no analysis which is concerned solely with conditions at collapse can possibly reveal the fact that hinge rotation at Section D' has occurred.

The resulting collapse mechanism and a check of the collapse load using the principle of virtual work are presented in Figure 6.

Evaluation of the Plastic Moment

To obtain numerical values of the results given in Figures 5 and 6, it was necessary to evaluate the plastic moment, M_p, corresponding to the cross--section selected for the members of the model frames. The value of M_p for a prestressed reinforced concrete section can be adequately estimated only after the probable actual relationship between resisting moment and curvature of such section is evaluated as the load is increased from zero up to collapse.

For preliminary design of the joints and connectors to be used to build up the model frame, as well as the loading apparatus and instrumentation, an approximate value of M_p was computed assuming that due to the final expansion of the concrete the steel would be pretensioned up to its yielding point, and assigning a value to the modulus of rupture of concrete in accordance with results obtained in previous investigations[4].

However, in order to have a more realistic value for M_p it was decided to determine the actual moment-curvature relationship of the selected section by testing in pure bending two chemically prestressed concrete structural elements similar to those to be used in the fabrication of the actual model frames.

It has to be noted that the analysis of the results given in Figure 6 indicates that the axial forces and shear forces are very small in comparison to the bending moments. Consequently, it might be expected that the behavior would be very close to that of a member under pure flexure. However, it has to be recognized that even very small values of axial forces have appreciable effect on the value of M_p, and prediction of behavior of the frame neglecting this effect may be too conservative[8].

III. EXPERIMENTAL INVESTIGATION

III-1. Test Specimens

Description of Test Specimens

Two different types of test specimens were fabricated: structural elements and control specimens.

All specimens were symmetrically restrained to produce concentric prestressing during expansion of the concrete. They had a constant 2-7/8 by 4-inch rectangular cross-section. The restraint was supplied by means of four No. 2 deformed bars which gave a total percentage of reinforcement of 1.74. A brief description of each type of specimen follows:

a. Structural Elements. Altogether eight structural elements were fabricated. Six of them were required to make up the two model frames, and the other two to determine their mechanical behavior under pure bending.

Details of the structural elements used to build up the model frames are given in Figures 7, 8, and 9. As indicated in Figure 7, the total length of the structural elements was 44 inches for those used as beams and 38 inches for those used as columns. The stirrups were made of 12-gage wire bent into the form of column ties and spaced at 1 1/2-inch intervals along the whole length of the structural members. The reinforcing bars were welded to end bearing plates which were part of the steel details shown in Figures 8 and 9. This detail was designed to facilitate the joining of the structural elements and to assure the continuity required for building up the rigid frames. A typical connection is illustrated in Figure 10.

The two structural elements which were fabricated to determine their mechanical behavior under pure bending were similar to the others except that at the ends only bearing plates were used.

b. Control Specimens to Determine Properties of Concrete. Two different types of control specimens were cast to determine:

(i) Modulus of Rupture. For this purpose two specimens were fabricated. These specimens were similar to the structural members except that the restraining steel was placed externally and consequently no stirrups were used. As illustrated in Figure 11a, one of the end bearing plates was adjustable in position through the use of bolted end connections supplied to the restraining bars.

(ii) Stress-strain Relationship in Compression. Four identical specimens were prepared for this purpose. They were similar to those used for the determination of the modulus of rupture except that the length was reduced to 12 inches. Figure 11b shows the restraining bars and forms used for these specimens.

Properties of Materials

a. Reinforcing Steel. No. 2 deformed bars were used as principal reinforcement and/or restraining bars for all specimens. The average properties for these bars are given in Table 1.

TABLE 1. - Properties of Reinforcing Bars

Bar Size	Area Sq.in.	Proportional Limit, ksi	Modulus of Elasticity E_s, ksi	Yielding Stress f_y, ksi	Strain at Onset of Strain Hardening ε_h, in/in	Ultimate Strength f_{su}, ksi	Strain at Ultimate Strength ε_{su} in/in
No. 2 (deformed)	0.0505	47.50	30,100	49.74	0.0183	67.25	0.175

Figure 12 shows a typical tensile stress-strain curve for this reinforcement. As the yield point of these bars is around 50 ksi, they must be classified as intermediate-grade steel rather than the high-strength grade which would have been desirable to work with. As these were the only No. 2 deformed bars available commercially and as the principal purpose of the experimental work was to investigate the feasibility of fabricating structural elements and the potentialities of the use of chemical prestressing rather than the use of a specific steel, it was considered acceptable to carry out the investigation with this steel.

b. Concrete. The chemical prestressing was accomplished through the use of a concrete mix containing expansive cement. After some preliminary tests with different trial mixes, the one given below was considered to be adequate to produce an expansion sufficient to force the steel up to its yield point.

The proportions by weight were 1 part of cement to 1.01 parts of Coast Range sand--containing almost 60% of graywacke and sandstone--and 1.52 parts of gravel with a maximum size of 1/2 in. The water-cement ratio of the concrete was 0.277. The expansive cement consisted of 66.7 percent of Type II portland cement and 33.3 percent of calcium-alumino-sulfate expansive compound. A water-reducing retarder was used in the amount of 4 oz. per sack of blended expansive cement.

c. High-strength Steel Bolts. High-tensile- strength bolts 3/8 in. in diameter, with yield point stress of 98,500 psi, were used to connect the different structural members of the rigid frame.

Manufacture and Curing of Test Specimens

a. Manufacture. The cage of reinforcement for each of the structural members was fabricated by using special steel jigs which insured the maintenance of proper steel spacing throughout the length of the members. After the electrical-resistance strain gages to be used to record the history of expansion had been placed on the longitudinal bars and waterproofed, the reinforcement cage of each structural member was firmly secured to the form at the ends and center.

All structural members and control specimens were cast from one batch of concrete. This was mixed at approximately 70°F. After the concrete was placed, it was consolidated by means of form vibration.

b. Curing. Subsequent to casting, all specimens were stored in fog at 70°. The forms were removed at age of 18 hours. At age of 15 days the specimens were removed from the fog room and stored under drying conditions at 70°F and 50 percent relative humidity.

III-2. Expansion of Concrete

Since in chemical prestressing the
during the curing of the concrete, through the energy of the expanding concrete, it is very important to know the history of the expansion. This information permits determination not only of the final amount of prestressing achieved but also of the time required to reach the final state of stabilization.

The record of the rate of expansion of the different specimens was obtained through the use of electrical-resistance strain gages placed on the restraining bars and of gage points which were provided on the surface of the concrete for observation of length changes by means of Whittemore gages.

Control Specimens

In Figure 13 is shown the average observed expansion as a function of time, for the two different types of control specimens. There was no significant difference in the behavior of these specimens. Most of the total expansion occurred during the first three days, and the expansion terminated essentially at the value of 0.15% at the age of ten days. This expansion induced a tensile prestress amounting to about 45,000 psi in the steel and a corresponding compressive prestress of 780 psi in the concrete. As indicated in Figure 13, when the control specimens were stored at 50% relative humidity and $70^{\circ}F$ the concrete contracted due to drying shrinkage. Volumetric equilibrium was reached at the age of 35 days with a loss in prestress of about 27 percent. The final total net expansion was about 0.11 percent; the final tensile prestress in the steel was 33,000 psi; and the corresponding compressive prestress in the concrete was 575 psi.

Structural Elements

Figure 14 shows the average history of expansion of the structural elements. The expansion terminated practically at an average of 0.184 percent at the age of 15 days. This expansion stretched the restrained steel into its plastic range.

When the specimens were stored at 50% relative humidity the concrete contracted; the final average net expansion was 0.147 percent, the loss in prestress due to drying shrinkage being approximately 20 percent. According to the expansion history of these specimens and the stress-strain relationship of the steel (Figure 12) it was estimated that the final average tensile prestress in the steel was approximately 39,000 psi, and the corresponding compressive prestress in the concrete was 679 psi.

Comparison of the curve given in Figure 14 with those shown in Figure 13 indicates that the history of expansion of the structural elements was similar to that obtained for the control specimens but considerably larger. This difference was expected because the existence of closely spaced ties in the structural elements did not allow the concrete to expand freely in the lateral directions and therefore forced it to expand more longitudinally.

III-3. Testing of Specimens

The specimens were tested in the sequence given below:

Control Specimens for Determination of Stress-Strain Relationship in Compression

a. Instrumentation: Besides the electrical-resistance strain gages attached to the reinforcement, and the gage points provided on the surface of the concrete for observation of length changes for determinating the history of expansion, additional electrical-resistance gages were placed on the surface of the concrete to determine the stress-strain relationship of the concrete under compression.

b. Loading Apparatus: The specimens were tested in a universal testing machine of capacity 120,000 lb. Details of the general set-up are shown in Figure 15.

c. Test Procedure: Two of the four specimens were tested directly under compression by loading them in increments up to failure. Strain readings of the steel and concrete were taken at the end of each load increment.

In the other two specimens, the prestressed force developed during curing was released by unscrewing the nuts that held the end plates in position. The release was done in several steps, and strain readings on the steel and on the concrete were taken after each step. Once the concrete was relieved of all of its prestressed force, the specimens were loaded incrementally in compression up to failure.

d. Test Results: The most important results obtained from the data recorded in these tests are summarized in Table 2 and presented in the form of stress-strain curves in Figure 16. These curves represent the average values obtained in the tests on two specimens for each case.

TABLE 2. - Test Results From Compression Tests of Control Specimens

Specimen No.	Age at Test days	Initial Prestress, psi		Compressive Strength of Concrete, psi	Strain at Maximum Stress, Millionths	Initial Tangent Modulus of Elasticity, Million psi
		Steel	Concrete			
1	41	36,300	632	8,260	2,390	5.25
2	41	32,250	561	7,570	2,070	5.55
3+	41	33,000	575	7,830	2,700	4.85
4+	41	30,100	525	7,650	2,530	5.05

+ Tested after prestressed force was released.

Control Specimens for Modulus of Rupture

a. Instrumentation: To obtain the modulus of rupture of the concrete, the two beams fabricated for this purpose were loaded in pure bending and the instrumentation was designed to give a record of the tensile strains of the concrete and of the deflections of the beam versus load.

b. Loading Apparatus: The beams were loaded at the third-points in a universal testing machine of capacity 60,000 lb.

c. Test Procedure: The two beams were first relieved of their prestress by unscrewing the nuts holding the end plates, and were then loaded incrementally up to failure. Readings of strains and deflections were taken after each increment of load.

d. Test Results: Results obtained in these tests are presented in Table 3.

TABLE 3.- Control Specimens for Modulus of Rupture - Test Results

Specimen No.	Age at Test days	Initial prestress, psi.		Modulus of Rupture, psi
		Steel	Concrete	
1	37	29,000	505	660
2	37	31,710	551	690

Structural Elements

a. Instrumentation: The instrumentation was designed to provide a record of strains, rotations, and deflections versus load. Besides the strain gages attached to the reinforcement for use in observing the history of expansion during curing, additional electrical-resistance wire gages were placed on the surface of the concrete to determine the variation of strains. The relative rotations of two sections 6 inches apart were determined through the use of a Whittemore gage by placing special gage points on the surface of the concrete.

Midspan deflections were measured by means of dial gages.

b. Loading Apparatus: The two beams were loaded at the third-points universal testing machine of capacity 60,000 lb. Figure 17 illustrates the set-up used for the testing of this specimen.

c. Test Procedure: One of the beams was incrementally loaded to failure. The other was incrementally loaded up to first crack, then incrementally unloaded, and finally incrementally reloaded to collapse. Strains, changes in length between gage points, and deflections were observed at each load increment. Careful visual inspection for cracks and for crushing was made at each load increment.

d. Test Results: These are presented in the form of moment-deflection curves in Figure 18 and as moment-rotation curves in Figure 19.

The most significant results obtained from the data collected in these tests are summarized in Table 4.

TABLE 4. - Structural Elements - Test Results

Beam No.	Age at test days	Initial prestress,* psi		Moments, kip-in., at			Strain at crushing of concrete Millionths	Maximum+ Rotation (radians)
		Steel	Concrete	1st Crack	1st yielding	Max.		
1	46	39,000	679	16.76	18.10	22.50	3,310	----
2	46	35,750	622	14.01	15.10	20.60	3,810	0.18

*Estimated in accordance with history of expansion and stress-strain diagram for steel.
+Maximum measured as limited by capacity of instrumentation.

Limitations imposed by the test set-up and instrumentation prevented loading the specimens up to complete or catastrophic failure. The moment-deflection curves for the two beams show that before reaching the maximum observed displacement the specimens were deflecting continuously under a constant moment of approximate 18 kip-in. for beam No. 1 and 19 kip-in. for beam No. 2. This relationship and visual observation of the test suggest that, had it been possible to continue the tests, the beams would have continued deflecting under the last recorded loads until complete collapse would have occurred. It is also interesting that the data obtained from the strain gages placed on the reinforcement at the ends of the beams show that the initial strains of the steel did not change during the complete loading process of these beams. This result is significant, because it indicates that very good bond was developed between the longitudinal reinforcement and the expansive concrete. The authors believe that this close bond was possible due to the lateral restraint offered by the closely spaced ties.

Model Frames

At the age of 41 days, the structural members were connected together to build up the model frames. The assembly was accomplished by joining the ends of the members with high-tensile-strength bolts as illustrated in Figure 10. Full continuity at these connections was obtained by tightening the high-tensile-strength bolts up to 5000 lb. This force was considered high enough to supply a clamping force which would keep the connected plates in complete contact even at collapse, i.e., when the fully plastic moment would be acting at the joint. Therefore, the increase in the initial tensile force in the high-strength bolts, even at collapse of the frame, was expected to be very small.

a. Loading Apparatus: Figure 20 shows schematically the loading device used in the testing of the model frames. The bases of the columns of the specimen A to be tested were clamped down to a rigid supporting frame, and the loads -- supplied by means of dead weights placed in the steel baskets D_1 and D_2 -- were transmitted to the model frame through the lever arms E_1 and E_2, and the cables F_1 and F_2.

b. Instrumentation: The measuring equipment was designed to provide a record of the applied loads, deflections, strains, and rotations. The loads supplied to the specimen were measured through the use of specially designed load cells,[8] indicated as I_1 and I_2 in Figure 20. The horizontal deflection at the beam level was measured by means of two 0.001-in. mechanical dial gages with 5-in. travel, indicated as J_1 and J_3 in Figure 20. The vertical deflection at the center of the beam was measured with 0.001-in. dial gage (J_2) which had 1-in. travel. The rotations at the bases of the columns were determined by measuring the change in length between special gage points (K) through the use of a 2 1/2-in. Whittemore gage. At the sections where plastic hinges were expected to form, steel and concrete strains were obtained by means of electrical-resistance wire gages. Location of these gages are indicated in Figure 7.

The initial tension in the high-strength bolts and the variation in tension when the frames were loaded were determined through the use of electrical--resistance wire gages.

c. Test Procedure: As illustrated in Figure 2, the model frames were loaded with a horizontal and a vertical concentrated forces. By placing controlled weights in the steel baskets designated as D_1 and D_2 in Figure 20, the two forces were kept equal in magnitude and were incrementally increased. While one of the model frames was directly loaded up to what was considered the apparent instantaneous collapse load, the other was first loaded up to a load of 1740 lb, then incrementally unloaded, and finally incrementally reloaded until apparent collapse was reached. Deflections, and Whittemore, and electrical-gage readings were taken at each load increment. Careful visual inspection for cracks and crushing of concrete was made at end of each load increment.

d. Test Results: The results obtained from the readings of the side sway deflection, ΔH, are presented as load-deflection curves in Figure 21, and a brief description of the general behavior of each frame follows:

Frame No. 1: When the bolts used to clamp the bases of the columns of this frame to the supporting frame were tightened, a crack was developed at section E (see Figure 4). This crack occurred at the side of the section which was later under tension during the loading process. The second crack appeared at D' under a load of approximately 1830 lb. At a load of about 2060 lb, cracking at section C and crushing on D' were observed. The fourth crack opened at section A' under a load of approximately 2110 lb.

At a load of 2140 lb a new crack at D" was observed. Under this load, continuous creeping of the horizontal and vertical deflections were observed; and while the new crack at D" became wider, the crack at D' did not develop further. As can be seen from Figure 21, after the horizontal deflection reached a value of 0.5 in. the frame was capable of carrying more load; however, when the load reached a value of 2250 lb an unrestricted deflection under constant load was observed. When the deflection reached a value of about 2.5 in. the hook connecting cable F_2 to the lever arm E_2 failed and the vertical load dropped off. Due to this sudden removal of the vertical load, a considerable increase in horizontal deflection was observed; and the frame came to rest at a horizontal deflection of 3.85 in. and a recorded horizontal load of 1900 lb. At this state the test was discontinued.

From visual observation of the connections during the test, it appeared that full continuity at the joints and rigid connections at the bases of the frame was practically achieved. This finding was corroborated by the analysis of the strain measurements made on the high-strength bolts, which revealed that the maximum change in the initial tensile force--approximately 5000 lb--was less than 300 lb.

Frame No. 2: As seen in Figure 21, this frame was first incrementally loaded up to about 1740 lb and was then unloaded. In spite of the fact that no crack was observed in this test, the analysis of the dial and strain-gage readings indicated that at 1740 lb the frame started to deviate from its linear behavior.

After this first test the specimen was incrementally loaded again. The first crack became visible at section E' under a load of 1770 lb. At a load of 1920 lb a crack occurred at section A'. When the load reached a value of about 1950 lb, a third crack was observed, this at section D'. With increase in load the width of the crack at E' increased considerably, and at a load of 2140 lb crushing of the concrete at this section became visible. Under this load, crushing of concrete at D ' started but a new crack appeared at D". The horizontal deflection increased considerably, and a very fine crack was observed at B'.

In spite of the fact that the horizontal deflections grew up at an increasingly rapid rate, it was possible to raise the load up to about 2400 lb. However, at this stage an unrestricted increase in the horizontal deflection under a constant load was observed. Due to the limitation of the loading apparatus, the frame was unloaded when the horizontal deflection reached 3.4 in. Strain measurements made on the high-strength bolts indicated that during the tests the maximum change in the initial tensile force did not exceed a value of 300 lb. Figure 22 shows the frame after apparent collapse.

IV. EVALUATION OF TEST RESULTS

IV-1 Mechanical Behavior of Structural Elements

Amount of Prestress

Analysis of the history of expansion given in Figure 14 reveals that the expansion of the concrete of the structural elements was capable of stretching the restraining steel beyond its yield point since the maximum average strain was about 1850 microinches per inch, or millionths, whereas the yield strain was only 1660 microinches per inch. Therefore, it may be concluded that there is likelihood of prefabricating chemically prestressed structural elements in which the steel reinforcement will be stretched up to its yield point even when high-strength deformed bars with yield point above 55 ksi are used. It must be recognized that this conclusion is valid if the amount of reinforcement is equal or less than that used in this investigation, i.e., 1.74%. Of course, the lower the percentage of reinforcement the larger will be the magnitude of the expansion. Also, supporting the foregoing conclusion is the fact that the expansive component used in this investigation, being three years old, had lost some of its expansive potentialities (as has been proven by comparison of the results obtained in the measurement of the total expansion on free-expansion specimens that have been cast from time to time in the laboratory).

Comparison of results given in Figures 13 and 14 shows that the maximum expansion of the structural element was about 25 percent higher than that of the control specimens. As explained before, this difference was expected due to the fact that whereas the concrete of the control specimens was capable of expanding freely in the lateral directions, in the case of the structural elements the presence of closely spaced ties confined the concrete laterally and forced it to expand more longitudinally. Thus these results pointed out the convenience of prefabricating what can be called "three--dimensional" chemical prestressed structural elements rather than merely uniaxial prestressed elements.

Mechanical Properties of the Concrete

Analysis of the results obtained in the tests carried out on the control specimens shows that the principal mechanical properties of the self-stressed concrete such as, initial modulus of elasticity, strain at maximum stress, or in general the stress-strain relationship in compression as well as the modulus of rupture were very similar to those to be expected in an ordinary concrete of the same compressive strength.

It must be recognized that the mechanical properties obtained from the control specimens cannot be considered as exact representation of the concrete of the structural elements since, according to the discussion previously presented, the confinement of the concrete produced by the ties suggests the possibility of a considerable improvement in the properties of the concrete.

Flexural Behavior of the Structural Elements

In order to draw some conclusions about the possibility of predicting theoretically the mechanical behavior of chemically prestressed concrete structural elements, the accuracy of the predictions of the numerical values of the characteristic points in the flexural behavior of the two specimens tested will be briefly discussed.

a. Cracking Moment. According to the estimated prestresses developed in the concrete of the two beams--679 and 622 psi-- and assuming for the modulus of rupture the larger of the two values obtained in the tests of the control specimens, i.e., 690 psi, the computed cracking moments resulted to be 10.80 kip-in. for beam No. 1 and 9.80 kip-in. for beam No. 2. When these values are compared with those obtained experimentally, i.e., 16.76 kip-in and 14.01 kip-in, a large discrepancy is observed. It is believed that this discrepancy can be explained by the possibility that the modulus of rupture of the internally restrained and partially confined concrete of the structural elements could be considerably higher than that determined from the control specimens.

Assuming that the initial prestresses were correctly estimated, to obtain agreement between the computed and experimental values for the cracking moment there would have been required a modulus of rupture of about 1260 psi for beam No. 1 and 1000 psi for beam No. 2. It is interesting that modulus of rupture of this magnitude was obtained in previous investigations[4] and for concrete of considerably less compressive strength than that used in this study. Therefore, the foregoing values may be considered as acceptable values for the modulus of rupture for the structural elements. Another factor that might have contributed to the observed discrepancy is the possibility that the initial prestress in the concrete at the cracked section may have been larger than that estimated from the strain readings obtained during the curing of the concrete and the stress-strain relationship for the steel. However, as the maximum possible prestress in the concrete was limited by the yielding of the steel, and as this would amount to about 860 psi, it is evident that this error in its evaluation could not be very significant.

Other minor contributions may be found in the fact that due to lateral expansion the final dimensions of the cross-section of the beam were 2.98 by 4.19 in. rather than the original 2.87 by 4 in. If these actual sizes of the sections are considered in the computation of the modulus they become 1110 and 990 psi for beams No. 1 and No. 2 respectively.

b. Moment-Curvature Relationship. Using in the computations the actual size of the cross-section of the beams, an average value of the estimated initial effective prestress, and a modulus of rupture of 1100 psi, the theoretical predicted moment-curvature relationships are given in Figure 23 together with those obtained from the experimental moment-rotation curves in Figure 19. Comparison of these curves reveals some discrepancy in the magnitude of the moments at the characteristic points, as will be discussed below, but the shapes of the curves are very similar to the idealized moment-curvature relationship shown in Figure 3a.

c. Stiffness in the Elastic Range. The moment-deflection and moment--rotation curves given in Figures 18 and 19 show that up to the cracking moment the beams behaved as linearly elastic elements, for all practical purposes. The stiffness factor for this linear range was evaluated from the rotation and deflection data obtained from the two beams, and the average value was found to be:

From the deflection data: $EI = 50,000$ kip-in^2

From the rotation data: $EI = 66,000$ kip-in^2

Assuming as the modulus of elasticity of the concrete a value corresponding to the initial tangent of the compressive stress-strain curves given in Figure 16, the computed value became 114,000 kip-in^2.

d. Yielding Moment. From the curves in Figure 23, it can be seen that the first yielding was predicted to occur under a moment of 16.90 kip-in. Instead, the experimentally determined values were 18.10 and 15.10 kip-in. If we consider the uncertainties regarding the actual modulus of rupture and effective initial prestress, and the fact that variation in yield strength of the steel reinforcement may be larger than 10%, it may be concluded that the agreement between the theoretical and experimental values for this characteristic moment is fairly good.

e. Ultimate Strength. The moment at the start of crushing of concrete was computed to be 19.3 kip-in. Instead, the moments when first crushing became visible were 22.5 and 20.6 kip-in. respectively. Considering the probable errors both in the estimation of the actual steel stress and in the actual strength of the concrete at crushing, it is evident that the observed discrepancy can be accepted.

f. Rotation Capacity. While first crushing of concrete was observed at a curvature of about $\frac{0.0300 \text{ radians}}{6 \text{ in.}} = 0.0050 \frac{\text{radians}}{\text{in.}}$, the theoretical value predicted according to a maximum concrete strain of 0.0038 was $0.0058 \frac{\text{radians}}{\text{in.}}$.

In spite of the fact that the actual rotation capacity was not measured because the test was discontinued before complete or catastrophic collapse was observed, the maximum value recorded was 0.18 radians which already may be considered more than that required for full redistribution of moments in frames of the type tested in this investigation.

It must be recognized that even after all concrete would have been crushed, due both to the presence of compressive reinforcement in the same amount as the tensile reinforcement and to the close spacing of the ties, the original double-reinforcement concrete section would behave as an ideal (no web) wide-flange steel cross-section where the flanges are represented by the tensile and compressive steel reinforcement. Therefore, the rotation capacity will be limited by the ductility of the reinforcing steel, which as well known is more than sufficient to permit full redistribution of moments in framed structures. Related with this behavior it must be noted that as a result of the strain-hardening phenomenon the moment capacity will increase after a certain amount of rotation. This behavior was clearly observed during the experiments. Assuming that the steel reinforcement would be hardened to its ultimate strength (67.25 ksi), the resisting moment offered by the steel alone was computed to be 17.70 kip-in. which is in good agreement with the values of the moments that the beams were capable of resisting under very large deflections (Figure 18).

g. Idealized Moment-Curvature Relationship. The fact that the shape of the curves in Figure 23 is very similar to that of the simplified curve in Figure 3a, justified the replacement of the actual moment-curvature relationhsip by the idealized relation represented by the bilinear diagram in Figure 23. For the computations of the values which define the characteristic point limiting the two distinctive ranges of the idealized relationship, the authors have followed the recommendations given in the research report on "Ultimate Load Design of Concrete Structures"[9].

1. Idealized Elastic Range: This range defined by determination of the value of the effective EI from the expression $EI = E_c \cdot \frac{Mn_1 d}{c}$ For the data obtained in this study, this expression yields the value $EI = 37 \times 10^3$ ksi.

2. Idealized Plastic Range: This range was defined by selecting as M_p the computed value of the moment at the start of crushing, i.e., $M_p = 19.3$ kip-in.

Comparison of the idealized diagram defined by the above values with the actual moment-curvature curves reveals that values of strength and deformations based on the assumed idealized relation should be safe.

From the foregoing discussion, it may be concluded that the flexural behavior of chemically prestressed concrete structural elements, where the steel is stretched close to its yielding strain, may be represented with fair accuracy by means of the idealized moment-curvature relationship assumed in the conventional plastic-hinge theory.

Comparison of Flexural Behavior of Chemically Prestressed and Ordinarily Reinforced Concrete Structural Elements

A beam similar to the chemically prestressed structural element used in this study was fabricated using conventional concrete. When the tests of the control cylinders of this concrete showed the same compressive strength as that obtained in the control specimens of the expansive concrete, the beam was tested under conditions similar to those for the chemically prestressed structural elements. The moment-curvature relationship obtained in this test is presented in Figure 23. Comparison of the initial part of this curve with the corresponding curve obtained for the prestressed structural elements provides evidence of the advantage of inhibiting the formation of cracks by prestressing. While for the ordinarily reinforced beam the first crack became visible under a moment of 7.3 kip-in.; in the case of the chemically prestressed elements it occurred under a moment of 14.0 kip-in.

IV-2. Mechanical Behavior of the Model Frames

Comparison of Theoretical and Experimental Results

According to the idealized behavior assumed in the analytical studies, the principal characteristic of the theoretically predicted behavior is indicated in Figure 5, i.e., 6 linear stages with a sudden and sharp change in slopes at the end of each stage which coincide with the formation of a new plastic hinge.

To obtain numerical values for the results given in Figure 5, it is necessary to give values to the plastic moment M_p and to the stiffness factor EI. According to the idealized moment-curvature relationship adopted for the structural elements--Figure 23-- M_p is 19.3 kip-in. and EI is 37×10^3 ksi. If these values are introduced into the results given in Figure 5, the values represented in Figure 24 are obtained.

When the experimental curves obtained from the tests of the two frames are compared with the theoretical curve, it is seen that--in spite of the fact that the experimental curves do not have the 6 linear stages which are present in the theoretical curve--the shapes of the curves are very similar. In order to draw some conclusions from the comparison of the theoretical and the actual behavior of the frame, the accuracy of the prediction of the numerical values for the most important characteristics of the behavior will be briefly discussed.

a. Prediction of Strength. The horizontal displacement capacity of the loading apparatus prevented the complete or catastrophic failure of the specimens tested. Then the problem is to define the collapse load. If this is defined as the load at which large deflections begin to develop, the experimental curves in Figure 21 reveal that this stage occurred under a load of about 21.4 kips for each frame. Theoretically, the large deflection will begin to develop when the last plastic hinge required to convert the frame into a mechanism has just formed; this value was estimated to be 21.2 kips.

Therefore, the agreement between the observed and the predicted value for what can be called the "yielding load" of the frame, rather than the actual collapse load, appears to be very good.

The experimental load-deflection curves show increases in load-carrying capacity as the deformations continued beyond first yielding. Frame No. 1 was capable of carrying a load of 2270 lb, and frame No. 2, 2400 lb. These increases may be the result of the combined effects of the presence of compressive axial forces and of the strain-hardening of the steel at the plastic--hinge zones. Although the compressive axial forces were small, it is well known that their effect is to increase the yielding moment appreciably.[8]

b. Prediction of Displacements. Comparison between the theoretically predicted and the observed values of horizontal deflection is presented in Figure 24. From this figure, it can be seen that the horizontal displacement appears to be in good agreement at all load levels.

When the theoretical and experimental values of vertical deflections were compared the agreement was not very good. A large discrepancy was obtained at low-load levels, for example: the theoretically predicted value at formation of the first plastic hinge appeared to be two time as large as those observed. Instead, at the point of formation of the last plastic hinge required to convert the structure into a mechanism, the agreement between theoretical and experimental values was very good.

Behavior of the Connections

The overall behavior of the frames, careful inspection of the connections during the tests, and measurement of the strains made on the high-strength steel bolts indicate that full continuity at the knees and at the bases of the frame was achieved.

V. CONCLUSIONS

According to the experience gained in the fabrication of structural elements and their connection to build up what may be called laboratory--size construction models, the authors believe that chemical prestressing is a promising method for fabrication of precast-prestressed framed structures. No major problems were encountered in the fabrication of the models. However, it must be pointed out that the mix was very rich and stiff and set very quickly. This mix was required in order to have a concrete which developed sufficient strength and stiffness as early as possible to permit that the expansion--most of which is developed during the first three days--be converted into mechanical energy for prestressing. According to the results obtained in this investigation, it seems doubtful that it will be possible to stretch up to the yield point deformed bars having a yield stress above 60,000 psi if these bars are used in a percentage equal to or larger than 1.74. Therefore, to obtain expansion which would provide adequate stressing of the deformed bars with the highest yield point available today, it would require the use of these bars in cases of members with small percentages of reinforcement or, on the contrary, it will be necessary to develop a new expansive component with improved composition which will be able to induce higher expansion. Perhaps an alternative solution would be to find a way of delaying the chemical reaction that produces the expansion until the time when the concrete has gained sufficient stiffness. These last two possibilities are under investigation at the University of California.

The observed effect of the closely-spaced ties in the total expansion of the structural elements, when compared with that of the control specimens, is of interest not only because it offers another possibility of increasing the total longitudinal expansion but also because by confining the concrete--apart from producing considerable increase in its strength and ductility--there must be improvement in the bond between the longitudinal reinforcement and the concrete.

Preliminary results of an investigation that is being carried out by one of the authors seem to indicate that, when structural elements are restrained only in the longitudinal direction, i.e., when no ties are provided, not only the mechanical properties of the concrete in the lateral directions are considerably affected but also the soundness of the concrete may be jeopardized due to the existence of voids between aggregate and mortar. Therefore, the authors believe that the use of expansive concrete in the fabrication of precast prestressed structural elements such as beams and columns holds great promise if the concrete is restrained in all directions.

Regarding the means of connecting the prefabricated structural elements to build up the rigid frames, the use of high-tensile-strength bolts appears to be satisfactory. However, it is recognized that for practical applications the details of the connections may need to be simplified. It may be that the use of field-welded connections would offer the possibility of simplifying the connections.

The behavior of the structural elements under pure flexure indicated that a considerable increase in the modulus of rupture is obtained by the presence of the internal reinforcement and the closely spaced ties, and that the moment-curvature relationship is very close to that assumed in the simplifying assumptions of the "plastic-hinge theory". Comparison of the experimental results obtained in the tests of the model frames with theoretical values calculated on the basis of the conventional plastic-hinge theory leads to the following conclusions:

1. If the apparent instantaneous collapse load is defined as the load under which the deflections begin to increase very rapidly, the agreement between experimental and computed values is very good. If, however, it is defined as the maximum value that the frame is capable of carrying, then the theoretical value is conservative.

2. Agreement between experimental and estimated values of horizontal displacement seems to be fairly good, for all levels of the load. Theoretical and experimental values of vertical displacement seem to agree only just at the load where the last plastic hinge required to convert the structure into a mechanism forms. For loads below this stage, the estimated vertical deflections are larger than the observed deflections.

VI - ACKNOWLEDGMENT

The research reported in this paper forms part of a general investigation on expansive cement and expansive concrete that is conducted at the Engineering Materials Laboratory of the University of California, Berkeley. This investigation is carried out under the sponsorship of the National Science Foundation.

Acknowledgment is due to the staff of the Engineering Materials Laboratory and in particular to Professor Joe W. Kelly for editorial cooperation.

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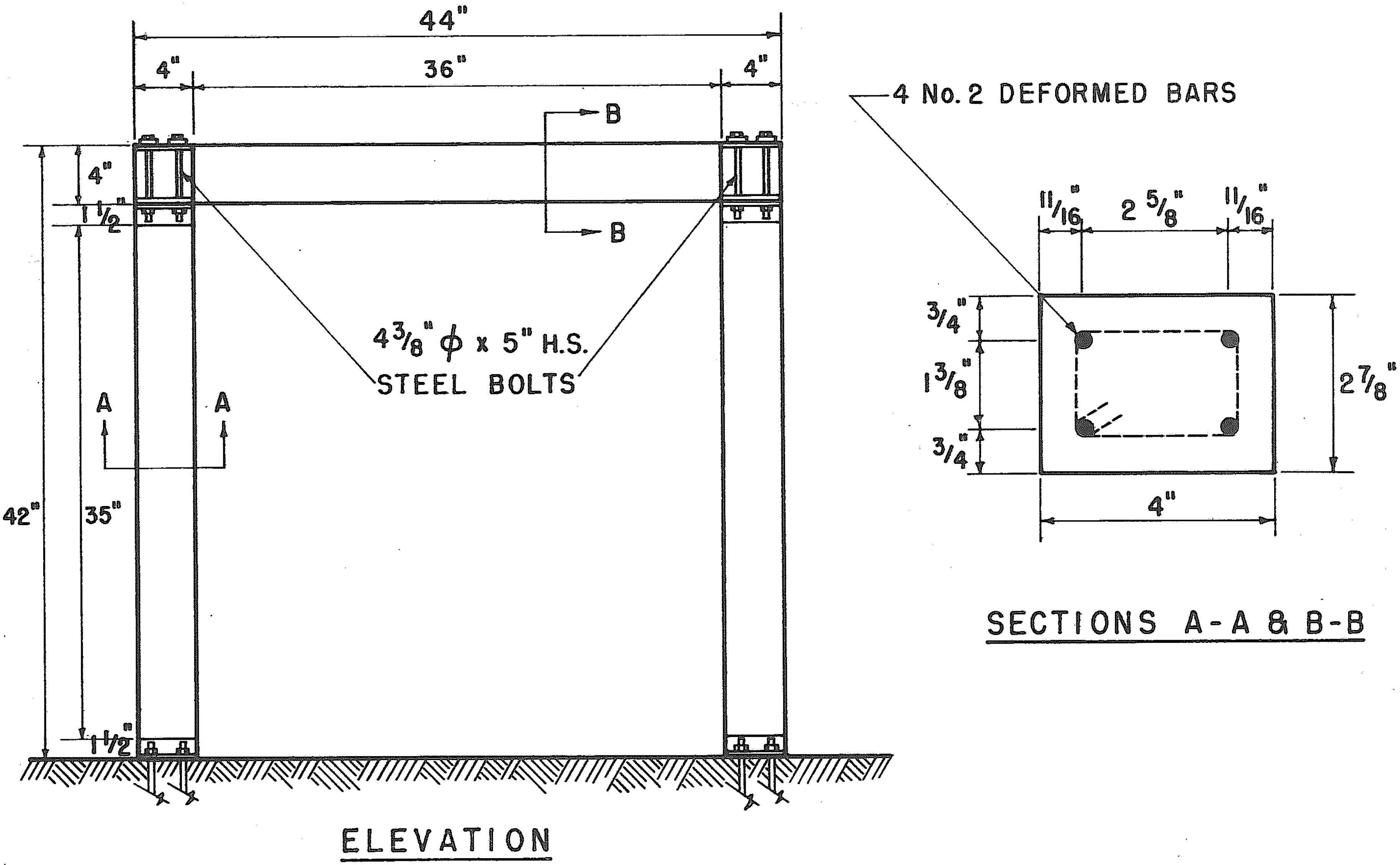


FIG. 1 NOMINAL DIMENSIONS OF MODEL FRAMES

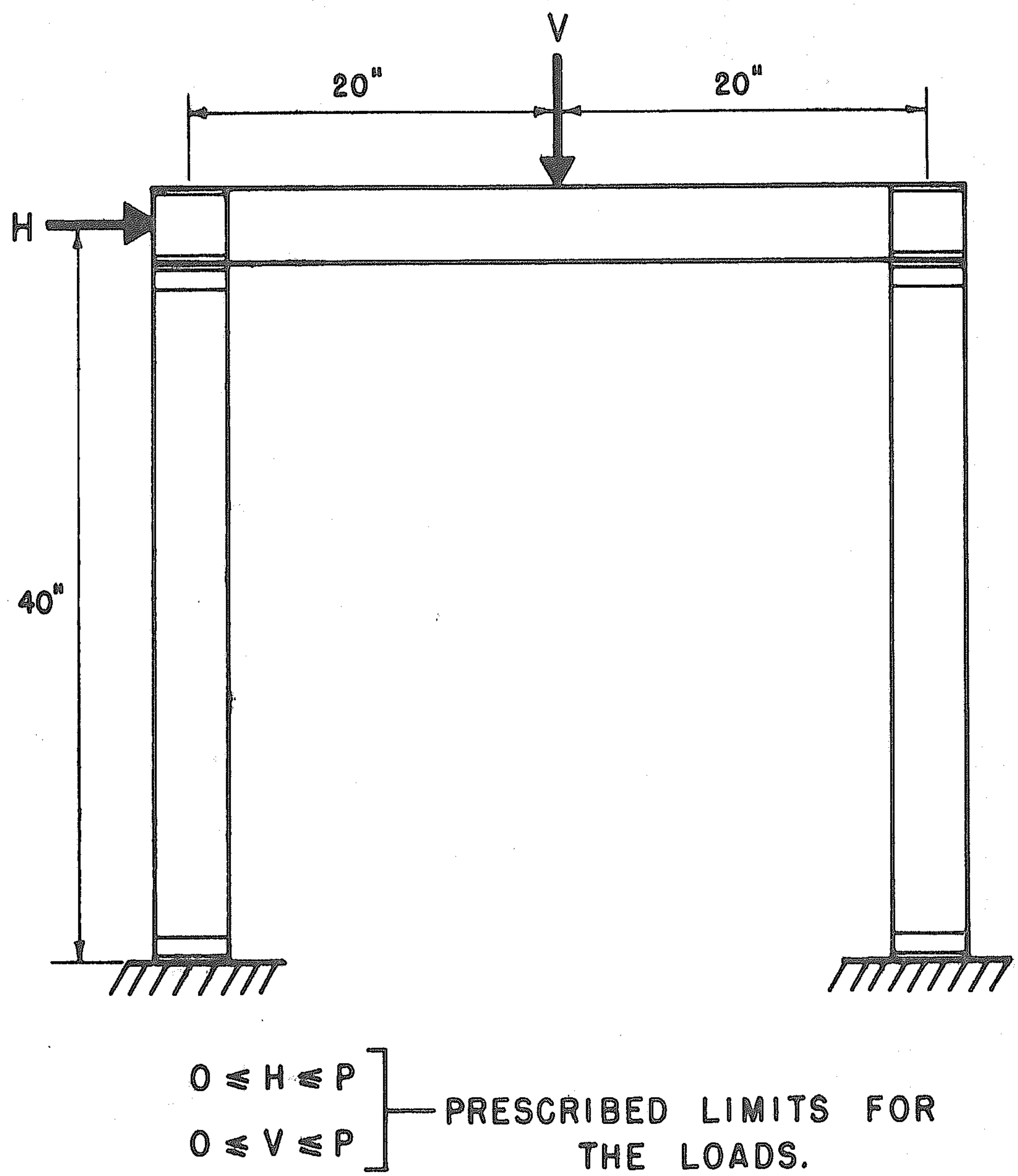
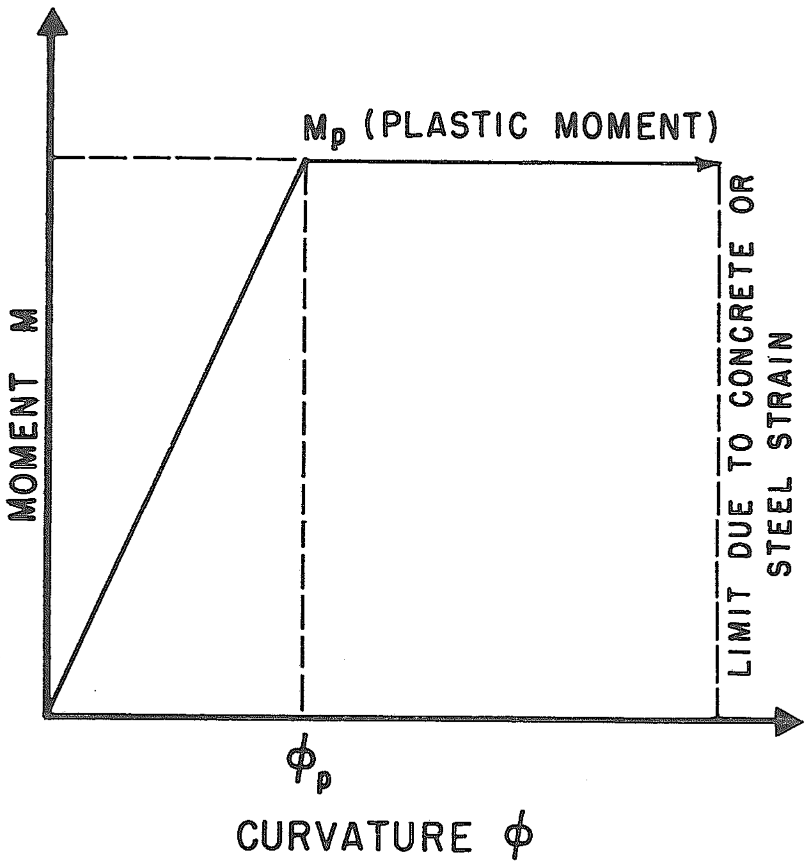
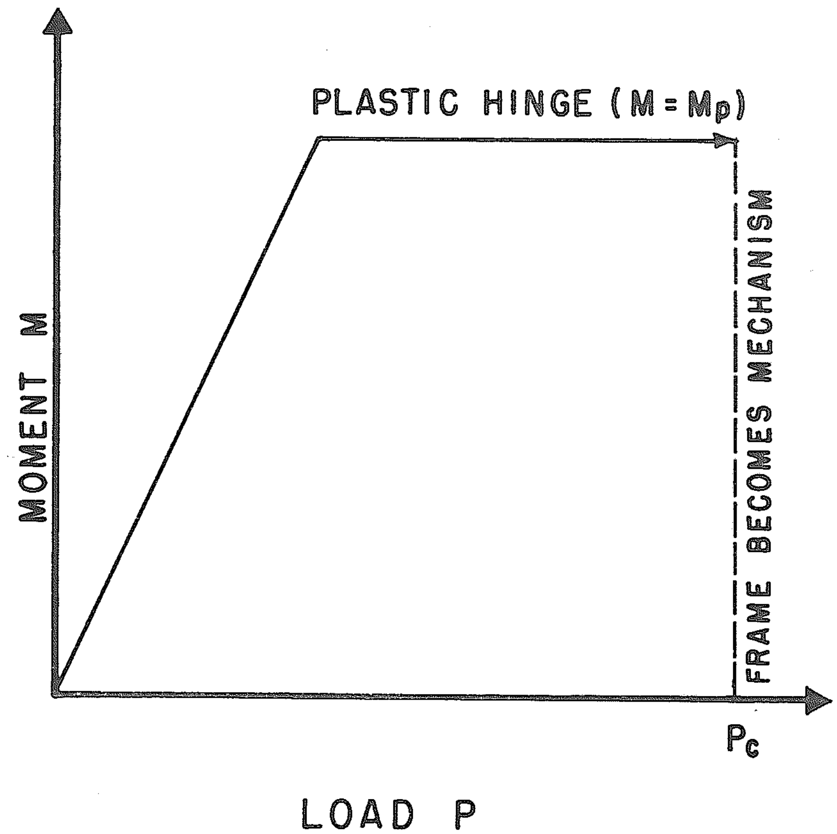


FIG. 2 TEST LOADING CONDITIONS



(a) MOMENT - CURVATURE



(b) MOMENT - LOAD

FIG. 3 IDEALIZED RELATIONSHIPS

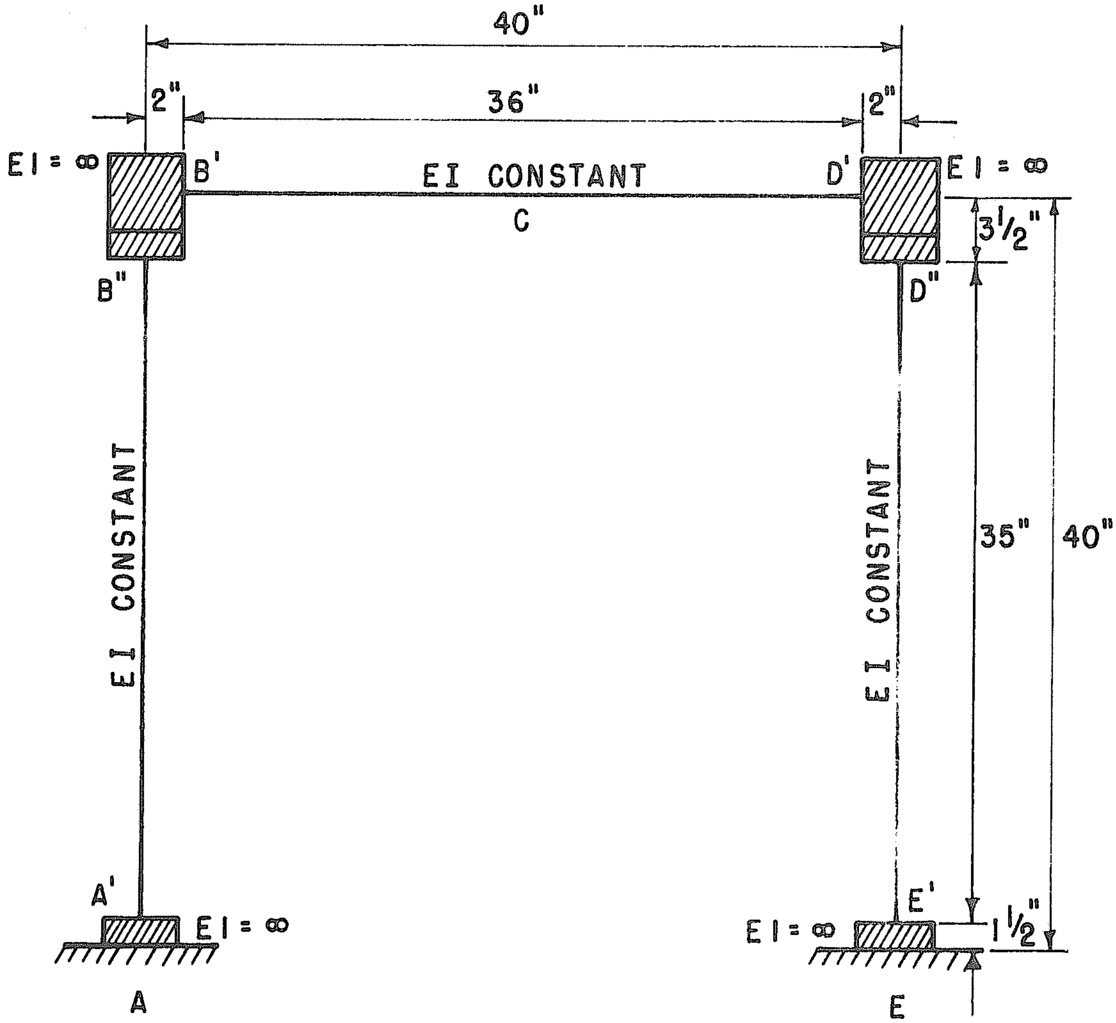
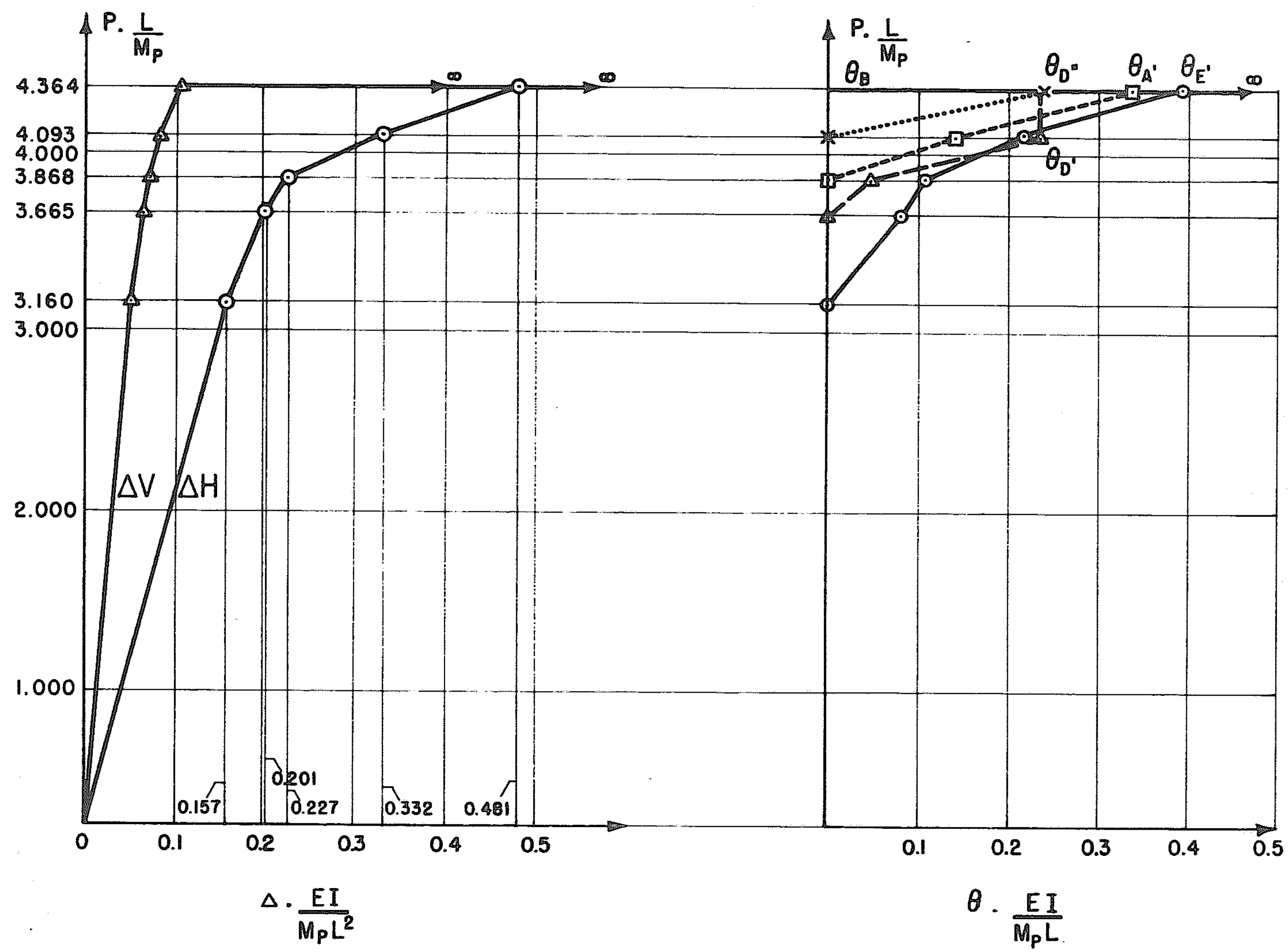


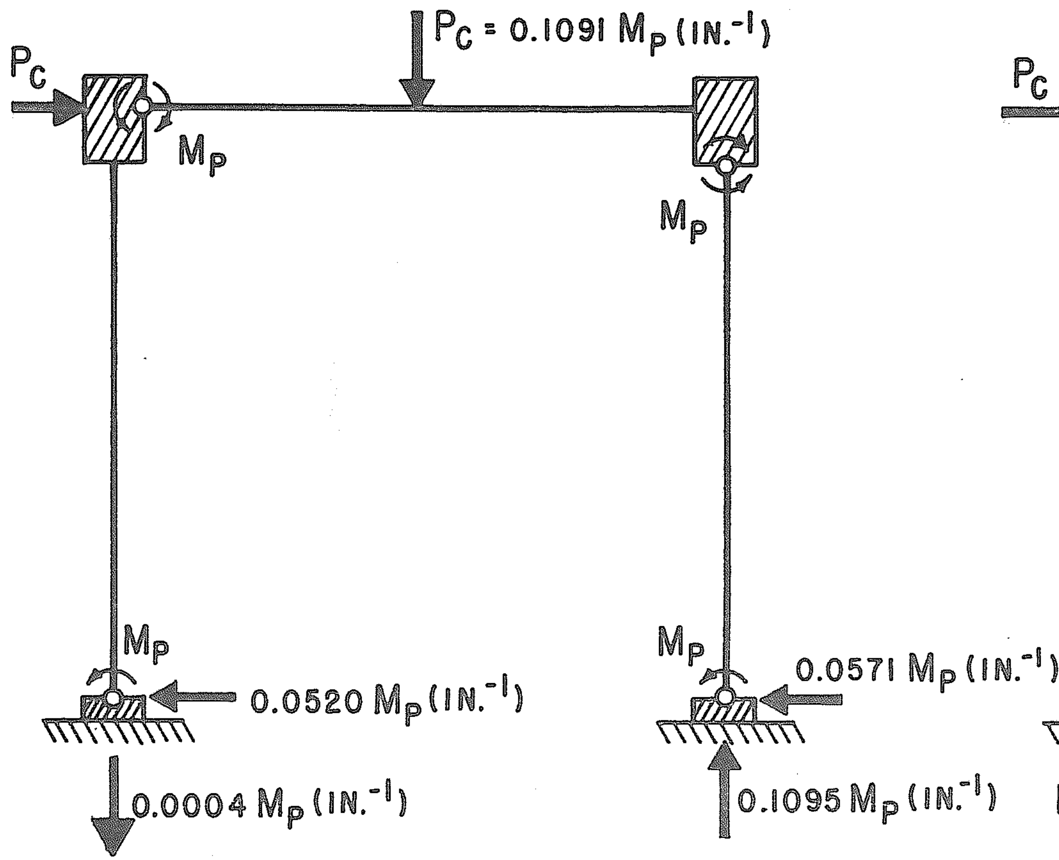
FIG. 4 IDEALIZED FRAME USED IN THE STEP BY STEP CALCULATION.



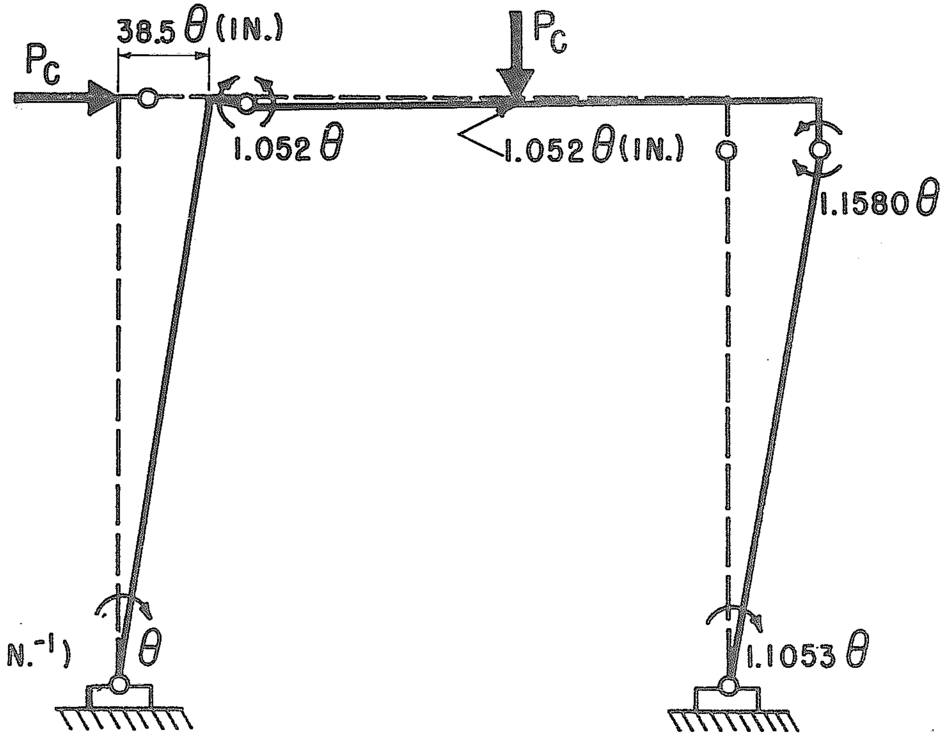
(a) DEFLECTIONS

(b) PLASTIC HINGE ROTATIONS

FIG. 5 IDEALIZED LOAD-DEFLECTIONS AND LOAD-PLASTIC HINGE ROTATIONS RELATIONS



(a) COLLAPSE MECHANISM RESULTING FROM THE STEP BY STEP CALCULATION.

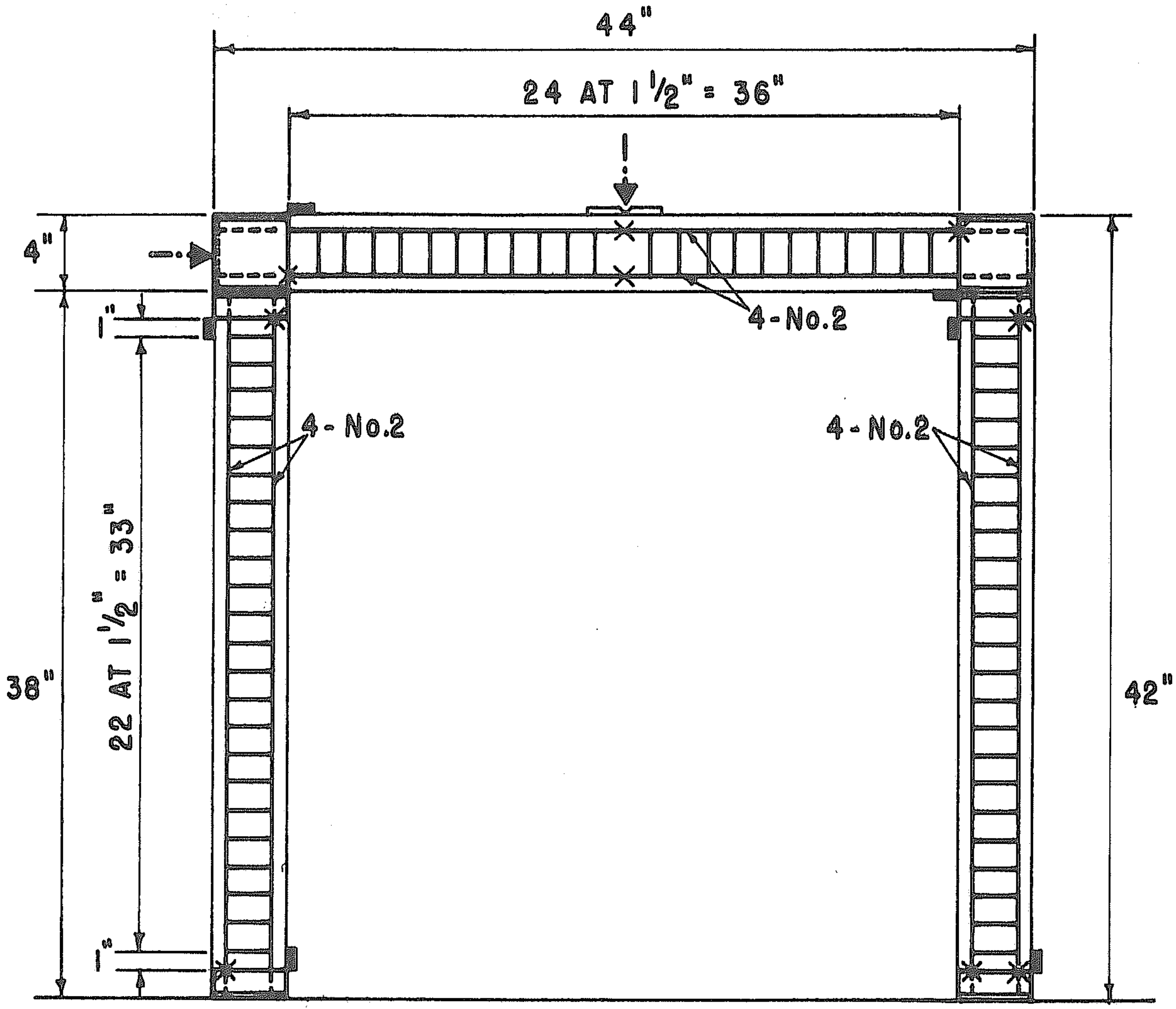


$$P_C\,(38.5 + 1.052)\,\theta\,(\text{IN.}) = M_P\,(1 + 1.052 + 1.1580 + 1.1053)\,\theta$$

$$P_C = 0.1091\,(\text{IN.}^{-1})$$

(b) CHECK OF COLLAPSE LOAD USING ENERGY - MECHANISM APPROACH.

FIG. 6 COLLAPSE MECHANISM AND INSTANTANEOUS COLLAPSE LOAD.



MAIN REINFORCEMENT: No.2 DEFORMED BARS
STIRRUPS: 12 Ga. WIRE
LOCATION OF STRAIN GAGES IS REPRESENTED BY:

× ON STEEL
▮ ON CONCRETE

FIG.7 DISPOSITION OF REINFORCEMENT STEEL IN THE FRAME, SPACING OF STIRRUPS AND LOCATION OF STRAIN GAGES.

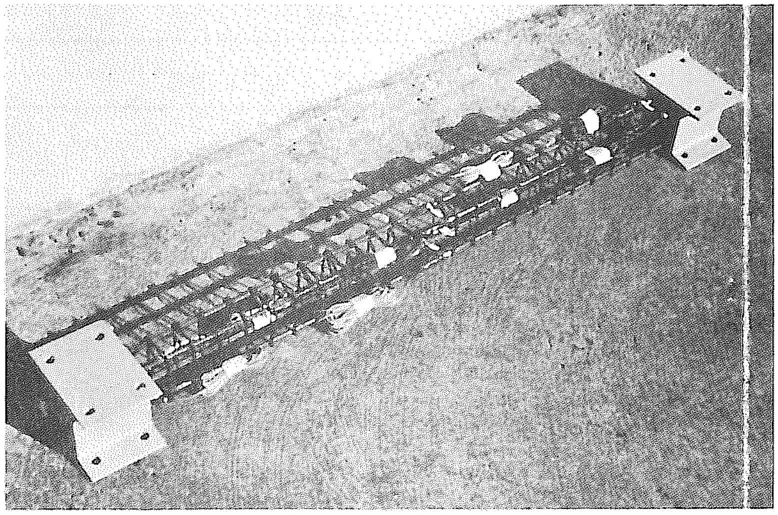


FIG. 8 - REINFORCEMENT OF THE STRUCTURAL ELEMENTS USED AS BEAMS.

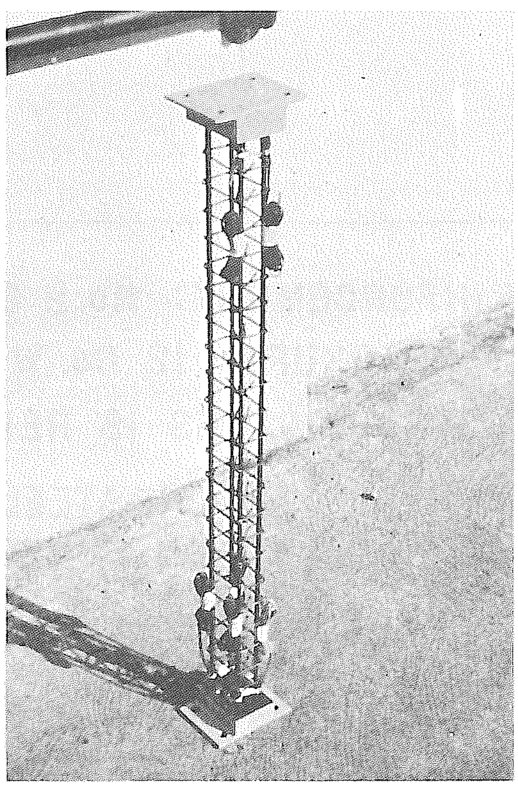


FIG. 9 - REINFORCEMENT OF THE STRUCTURAL ELEMENTS USED AS COLUMNS.

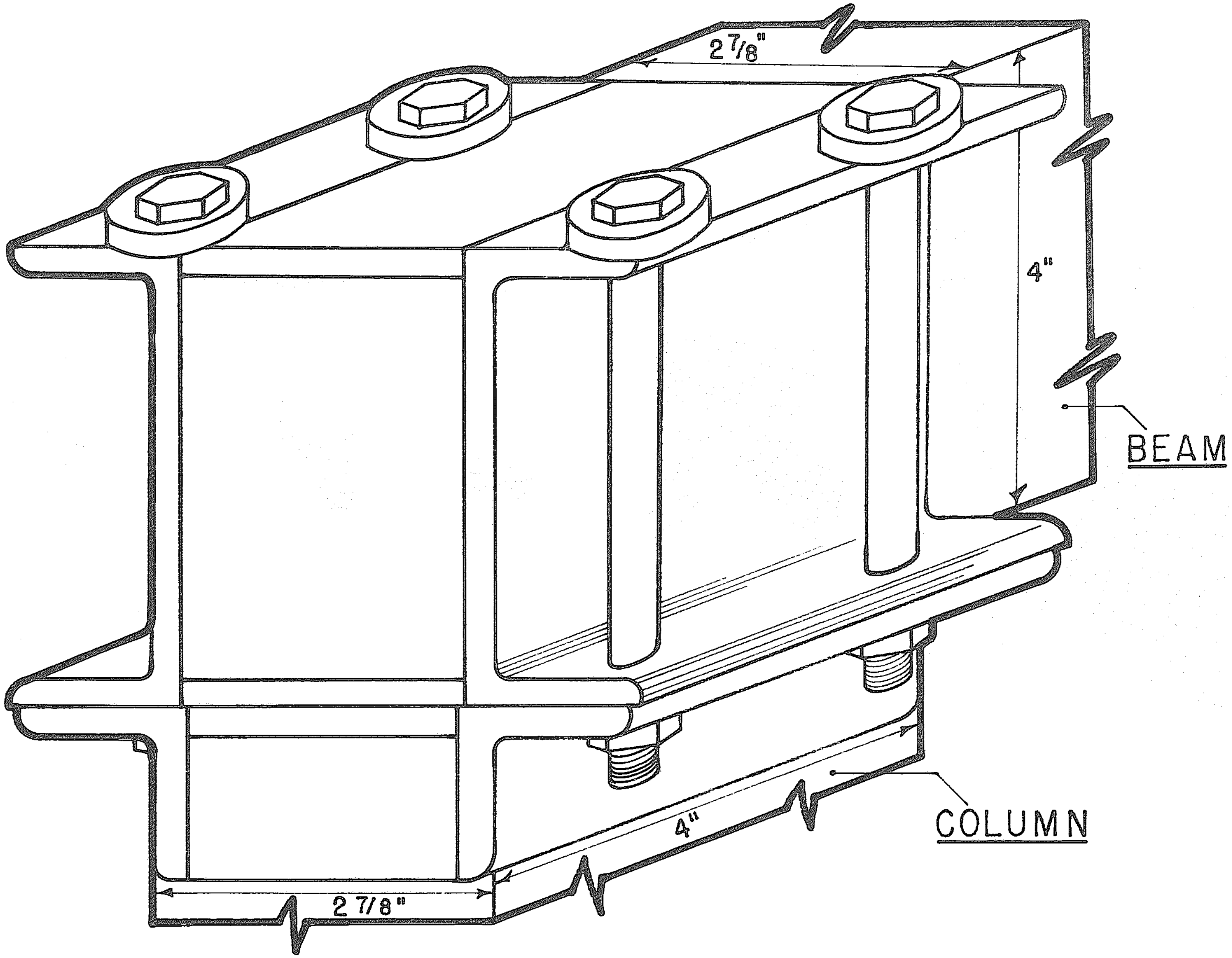
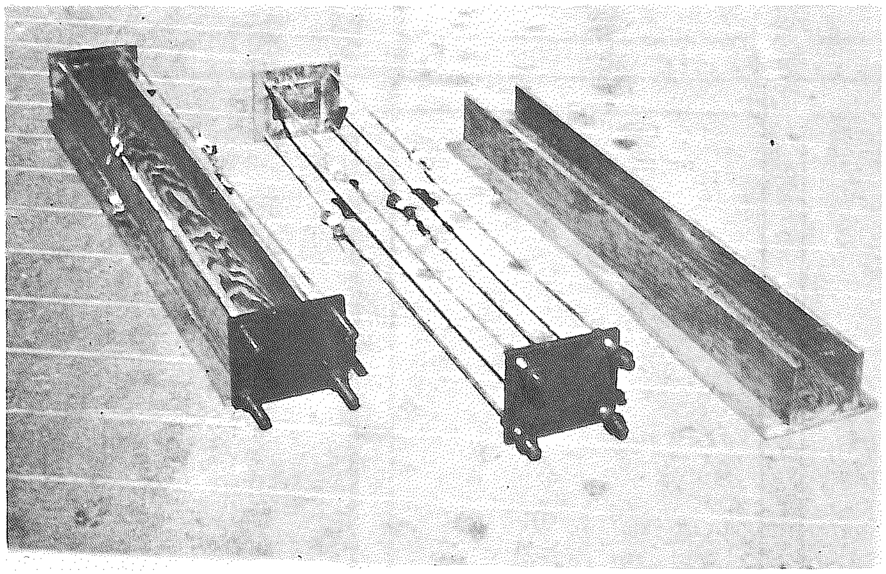
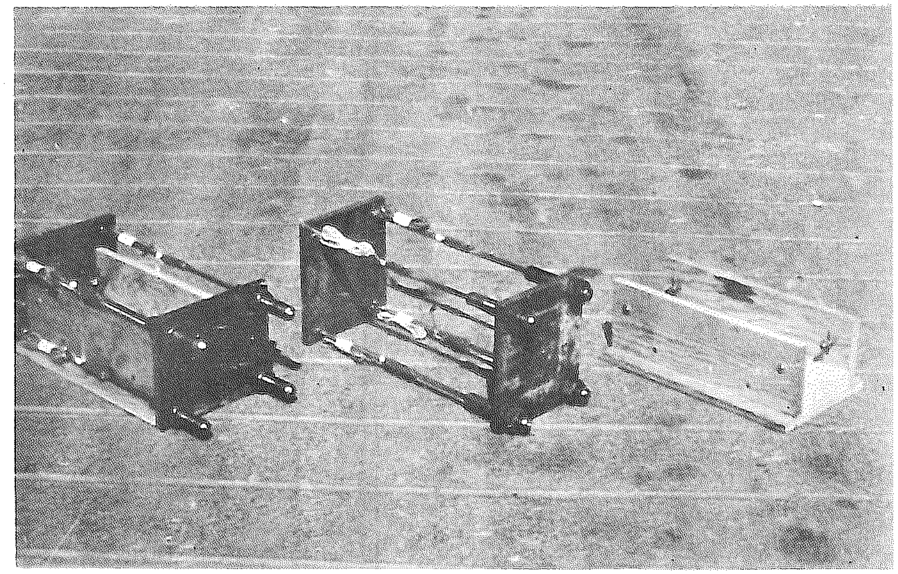


FIG. 10 TYPICAL CONNECTION OF A BEAM TO COLUMN



a. For Modulus of Rupture



b. For Compression Test

FIG. 11 - DETAILS OF CONTROL SPECIMENS

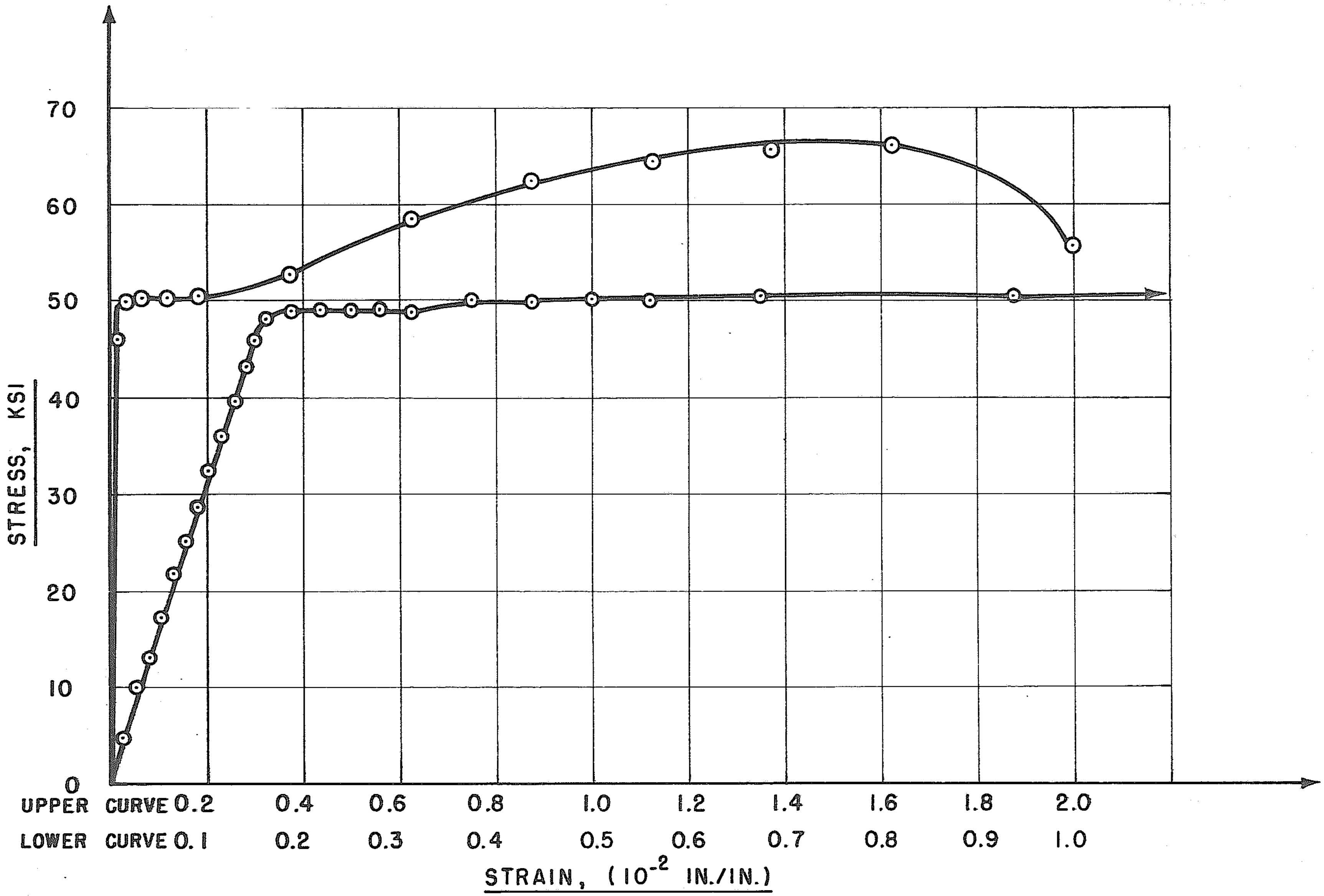


FIG. 12 TYPICAL TENSILE STRESS STRAIN CURVE FOR REINFORCEMENT.

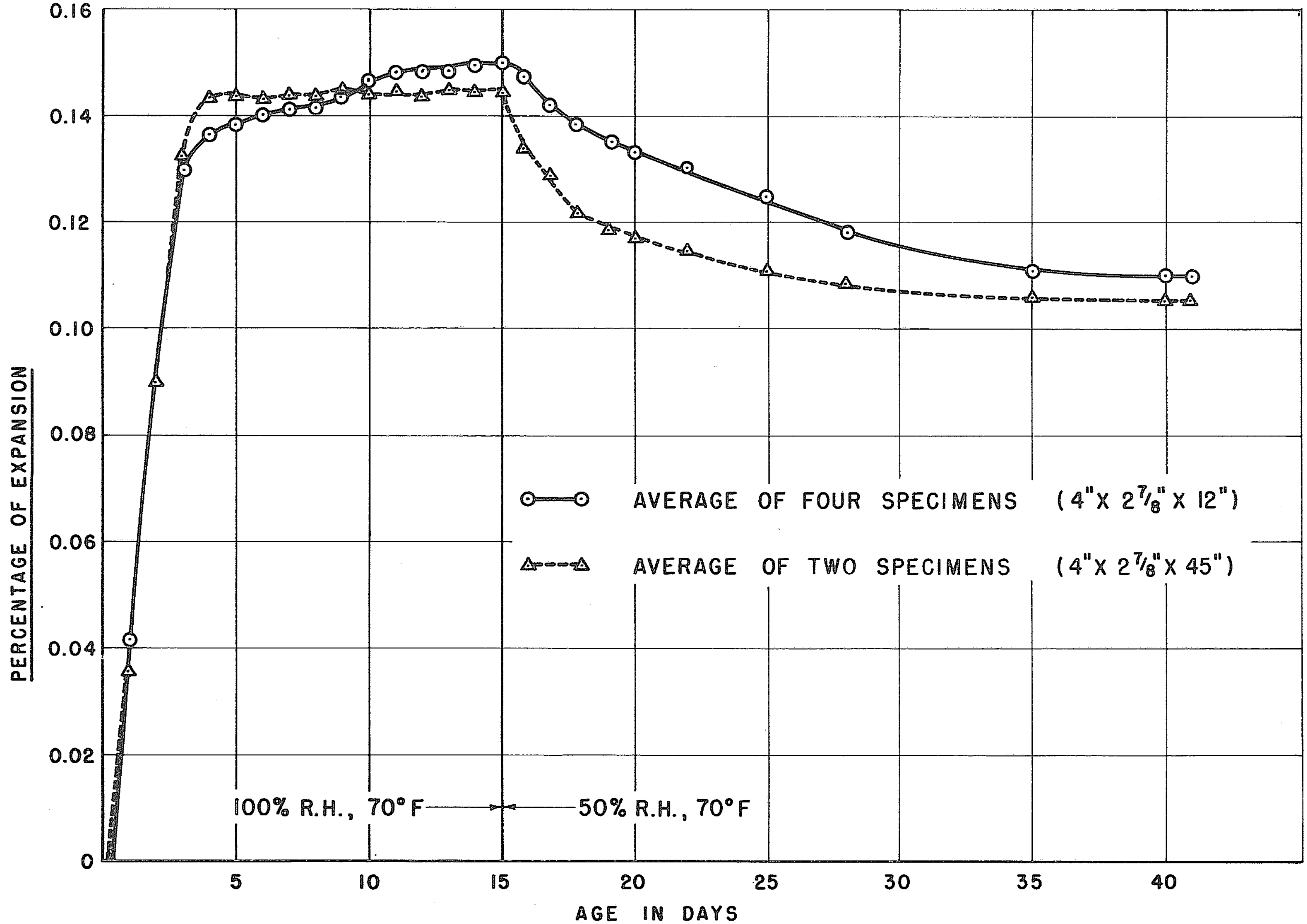


FIG. 13 EXPANSION - TIME CURVES FOR CONTROL SPECIMENS

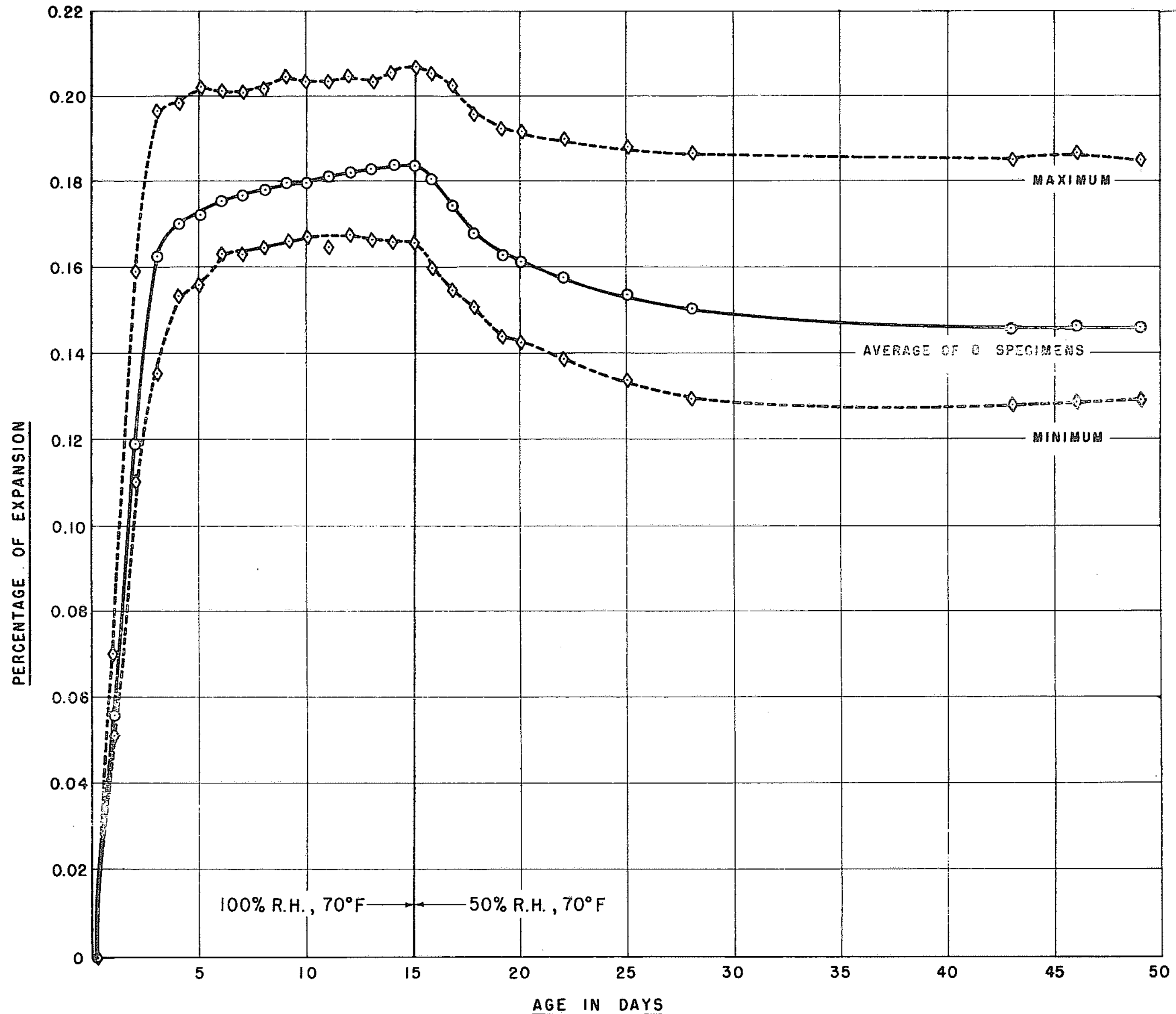


FIG. 14 EXPANSION - TIME CURVES FOR STRUCTURAL ELEMENTS

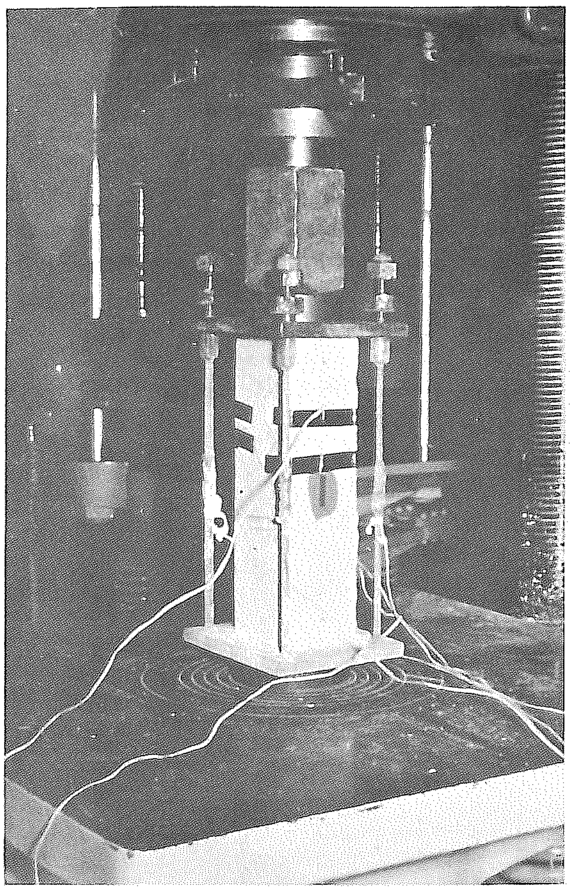


FIG. 15 - COMPRESSION TEST SPECIMEN IN POSITION FOR TEST.

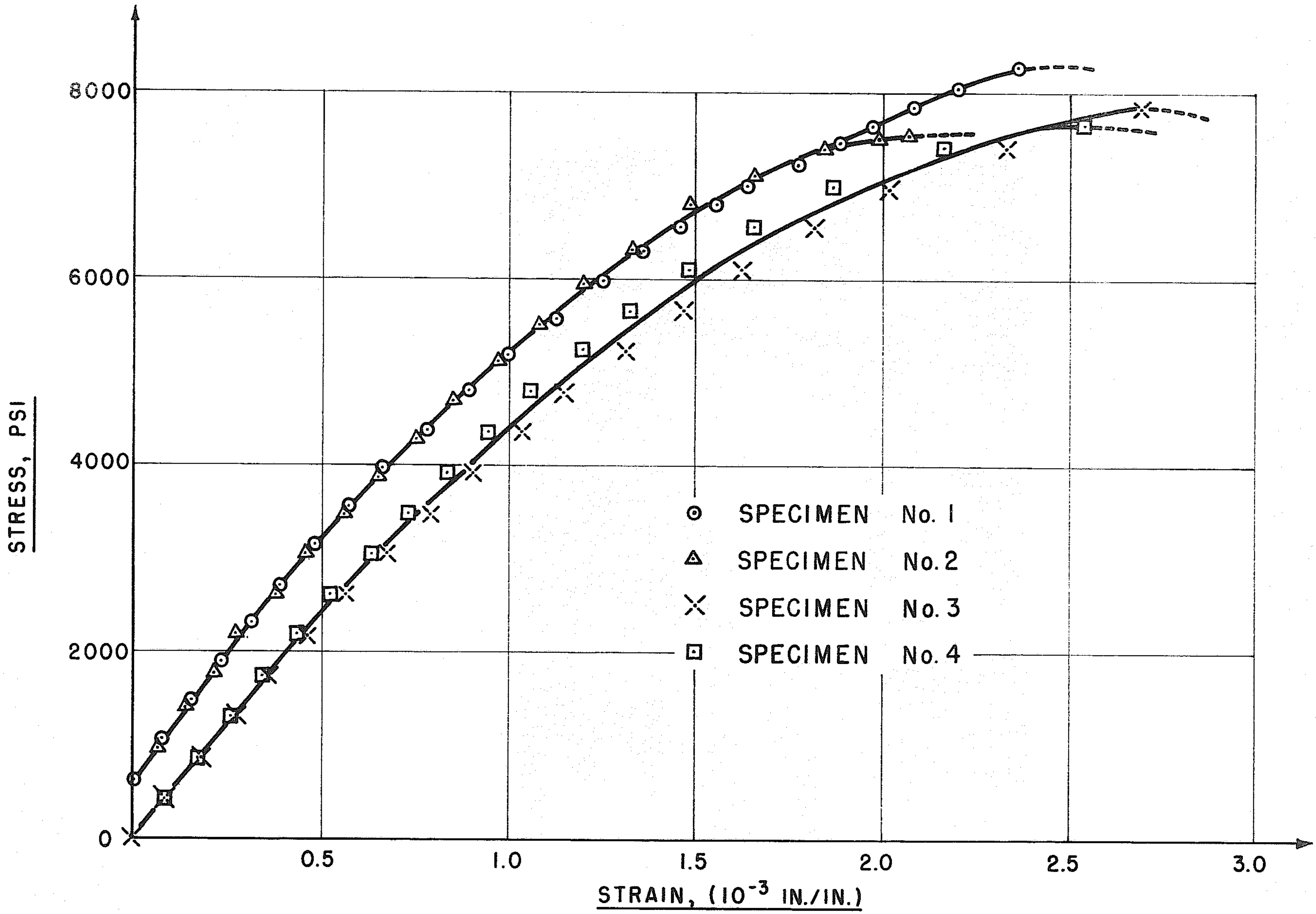


FIG. 16 STRESS STRAIN CURVE FROM COMPRESSION TEST OF RESTRAINED EXPANSIVE CONCRETE PRISMS.

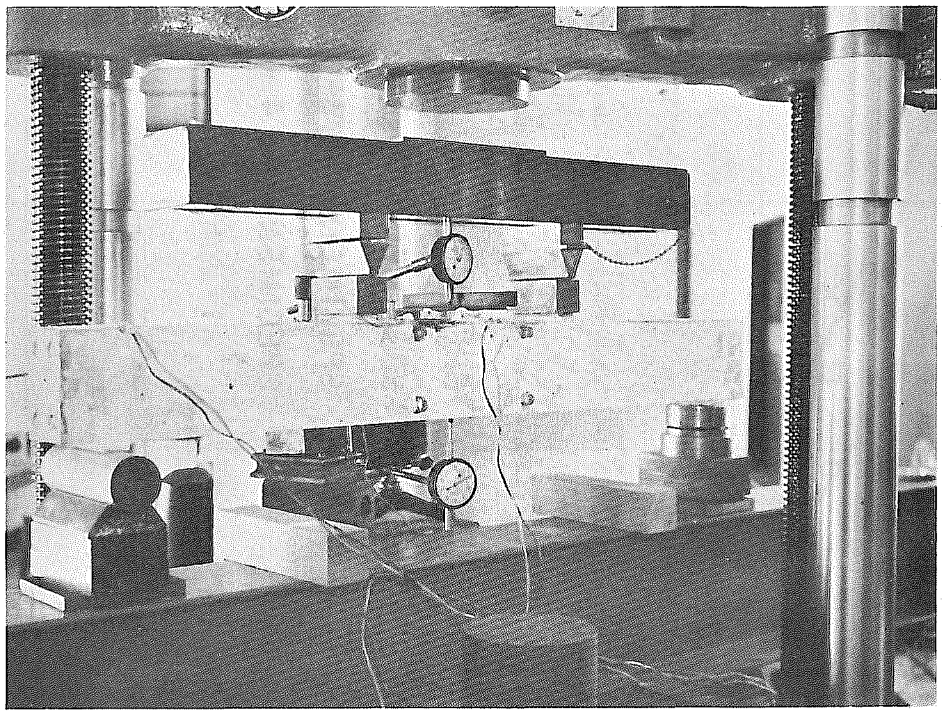


FIG. 17 - STRUCTURAL ELEMENT IN POSITION FOR TEST.

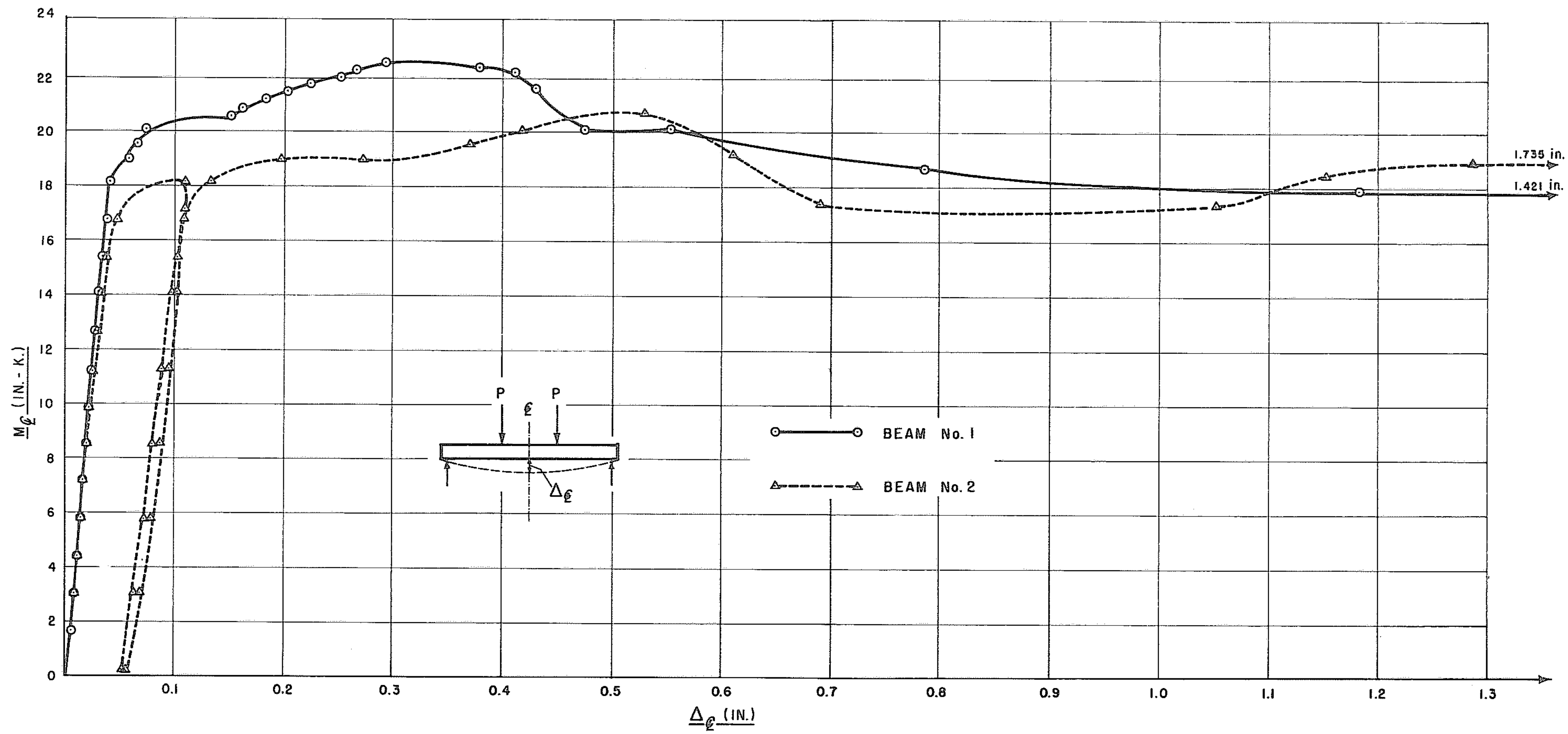


FIG. 18 MOMENT - DEFLECTION CURVE FOR STRUCTURAL ELEMENTS

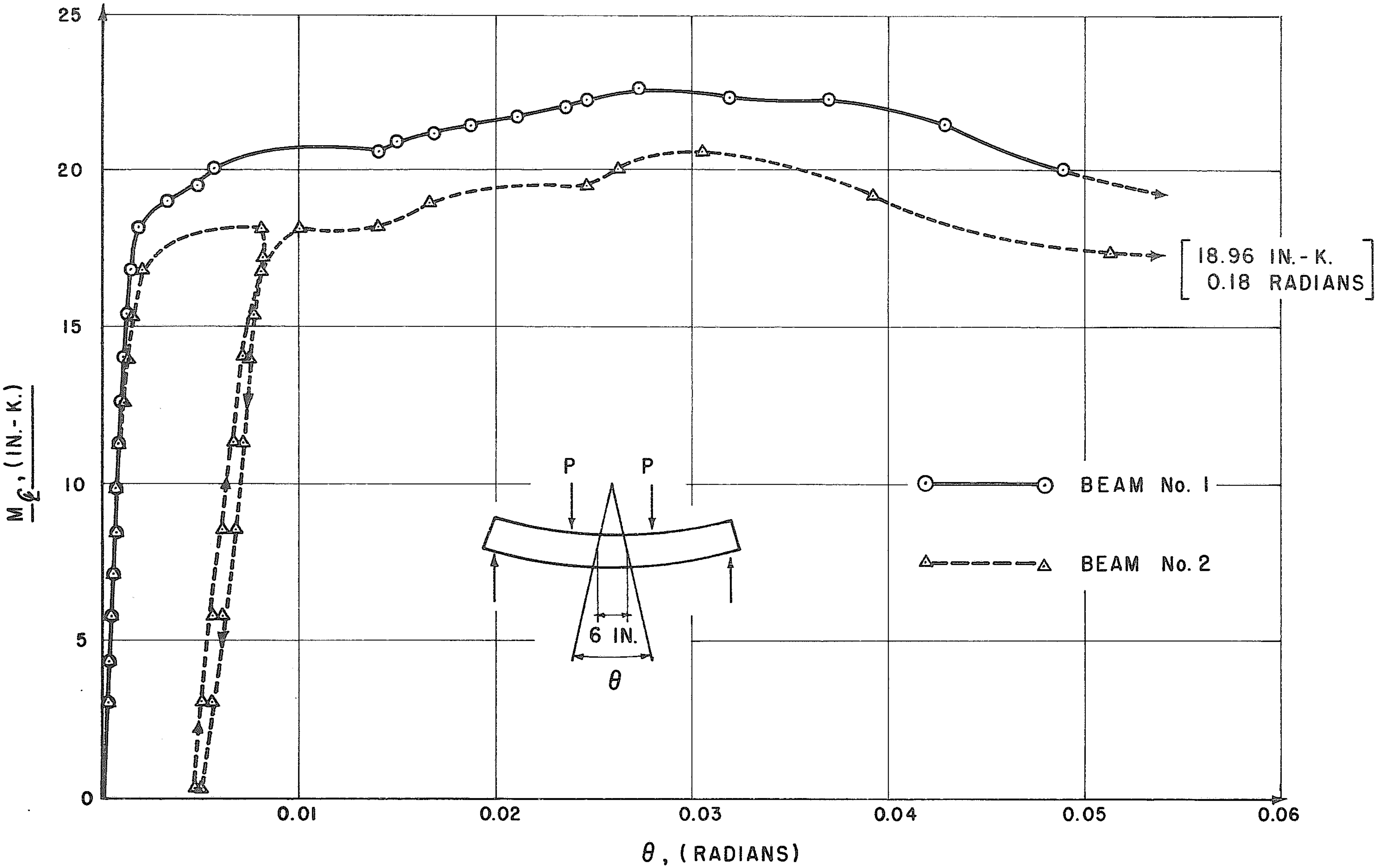


FIG. 19 MOMENT ROTATION CURVES FOR STRUCTURAL ELEMENTS

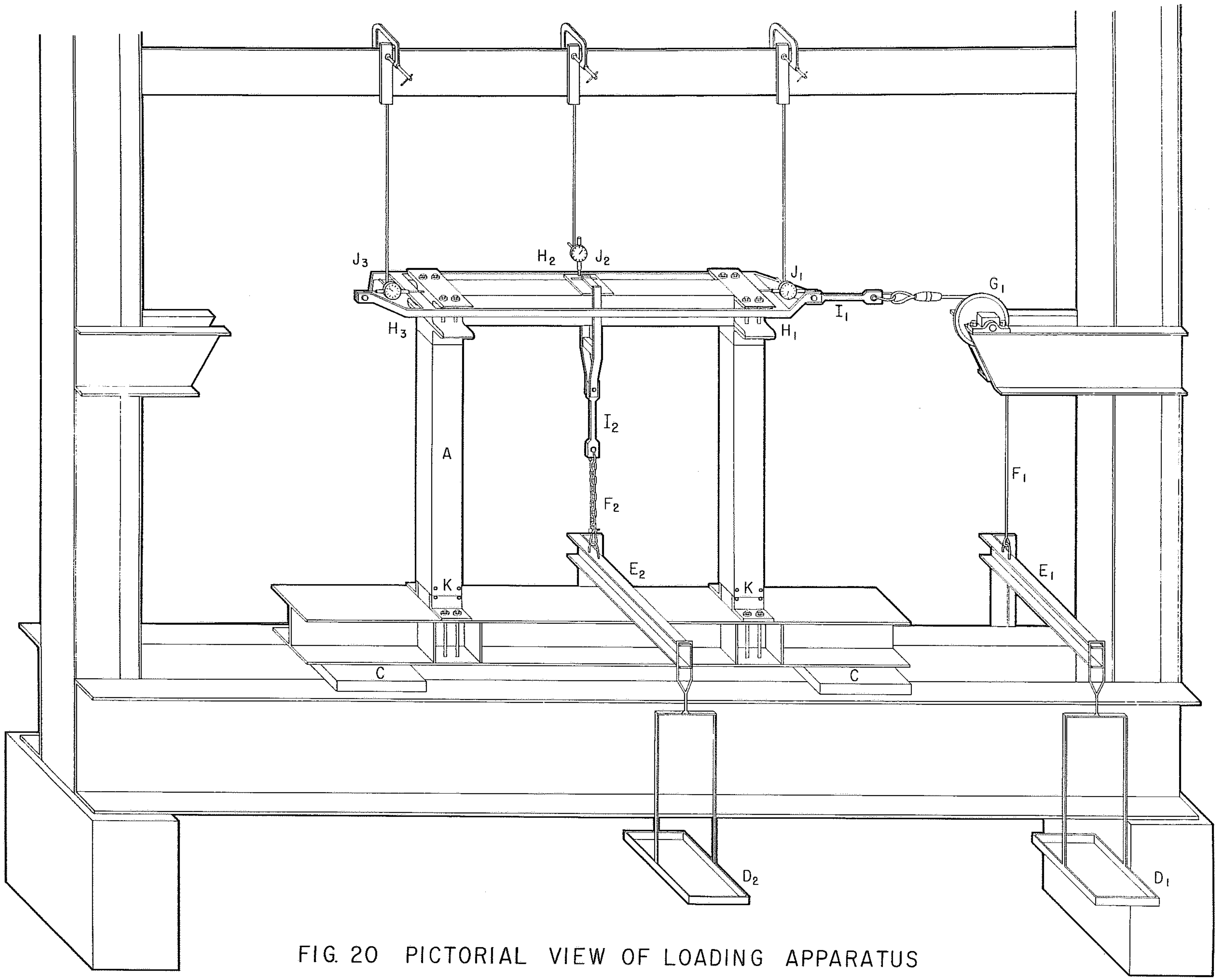


FIG. 20 PICTORIAL VIEW OF LOADING APPARATUS

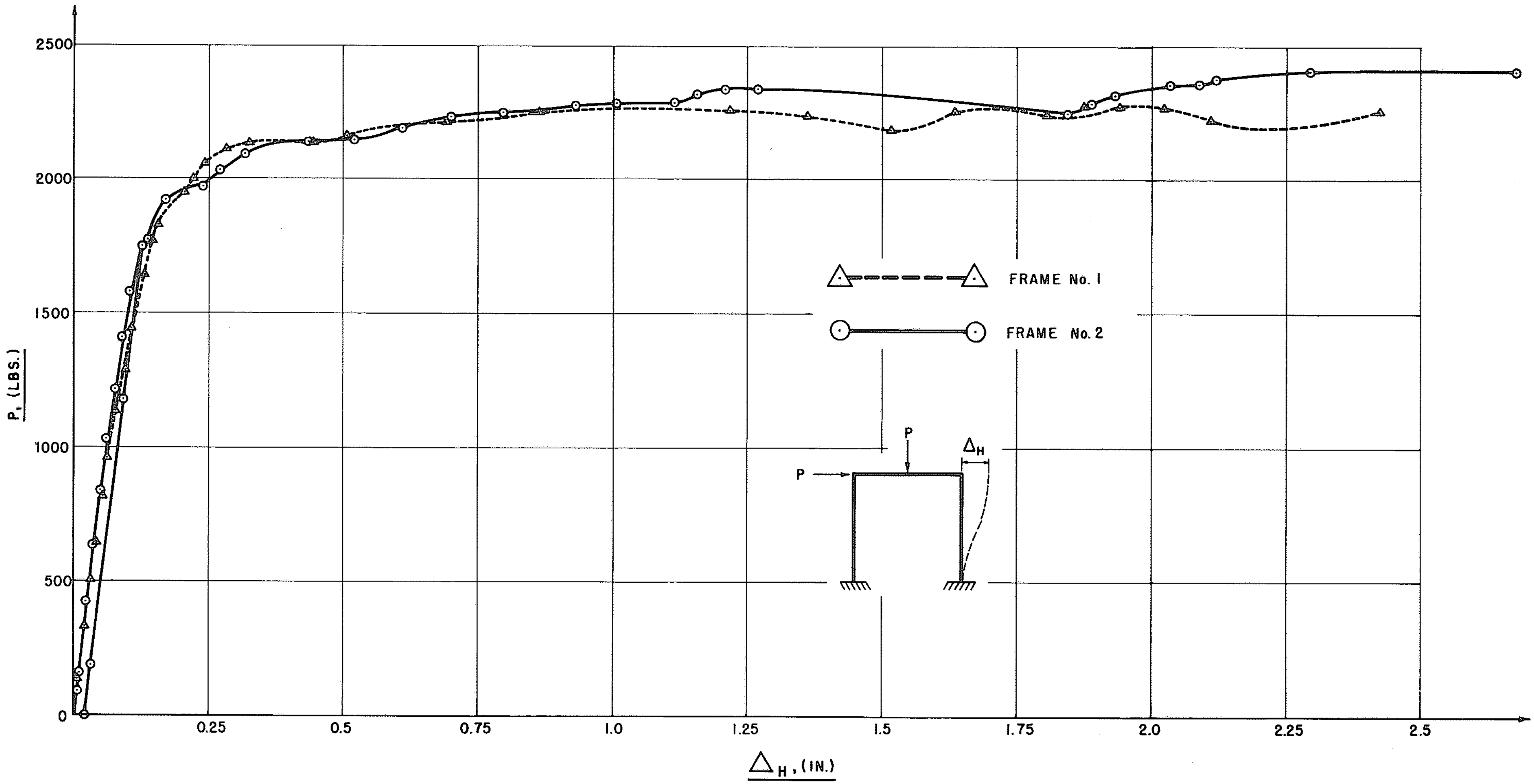
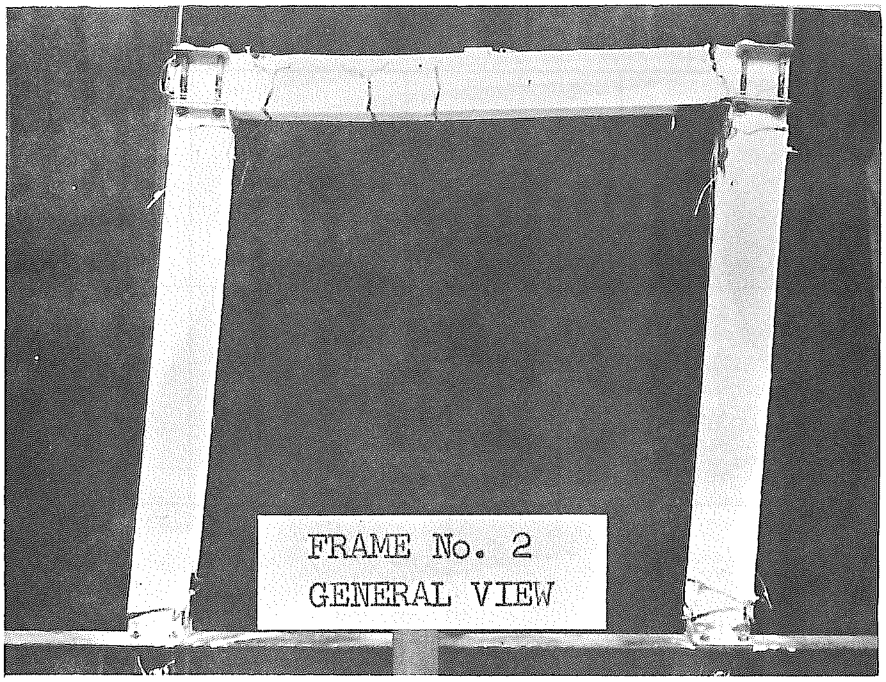
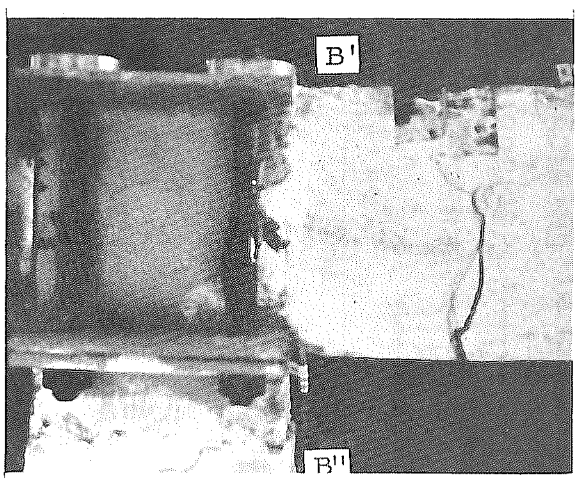
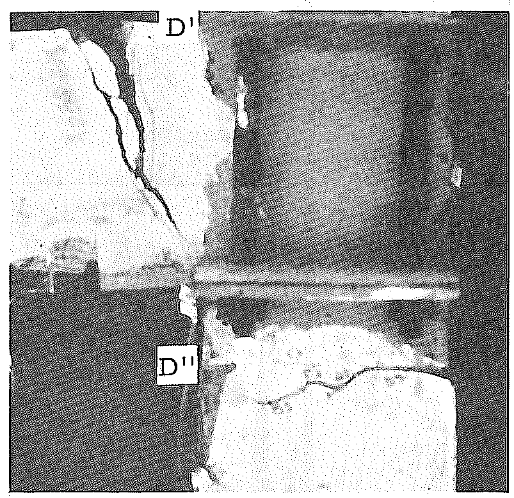
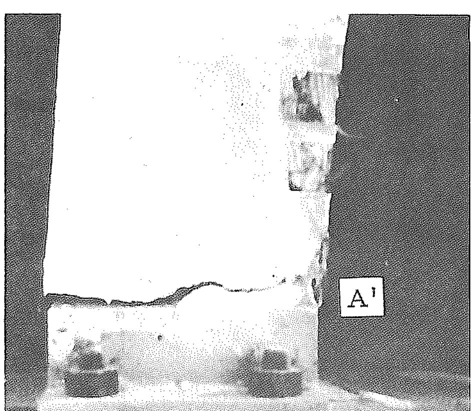


FIG. 21 LOAD-HORIZONTAL DEFLECTION CURVES.









FRAME No. 2

CRACKING AT PLASTIC HINGE LOCATIONS

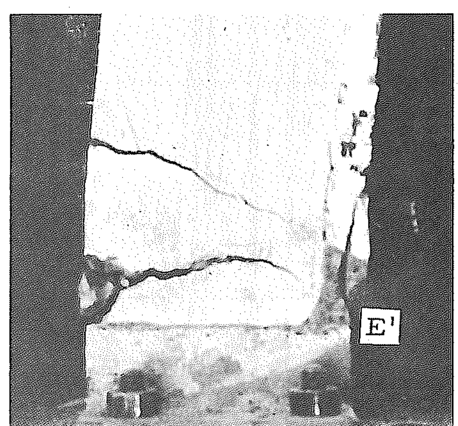


FIG. 22 FRAME No. 2 AFTER APPARENT COLLAPSE

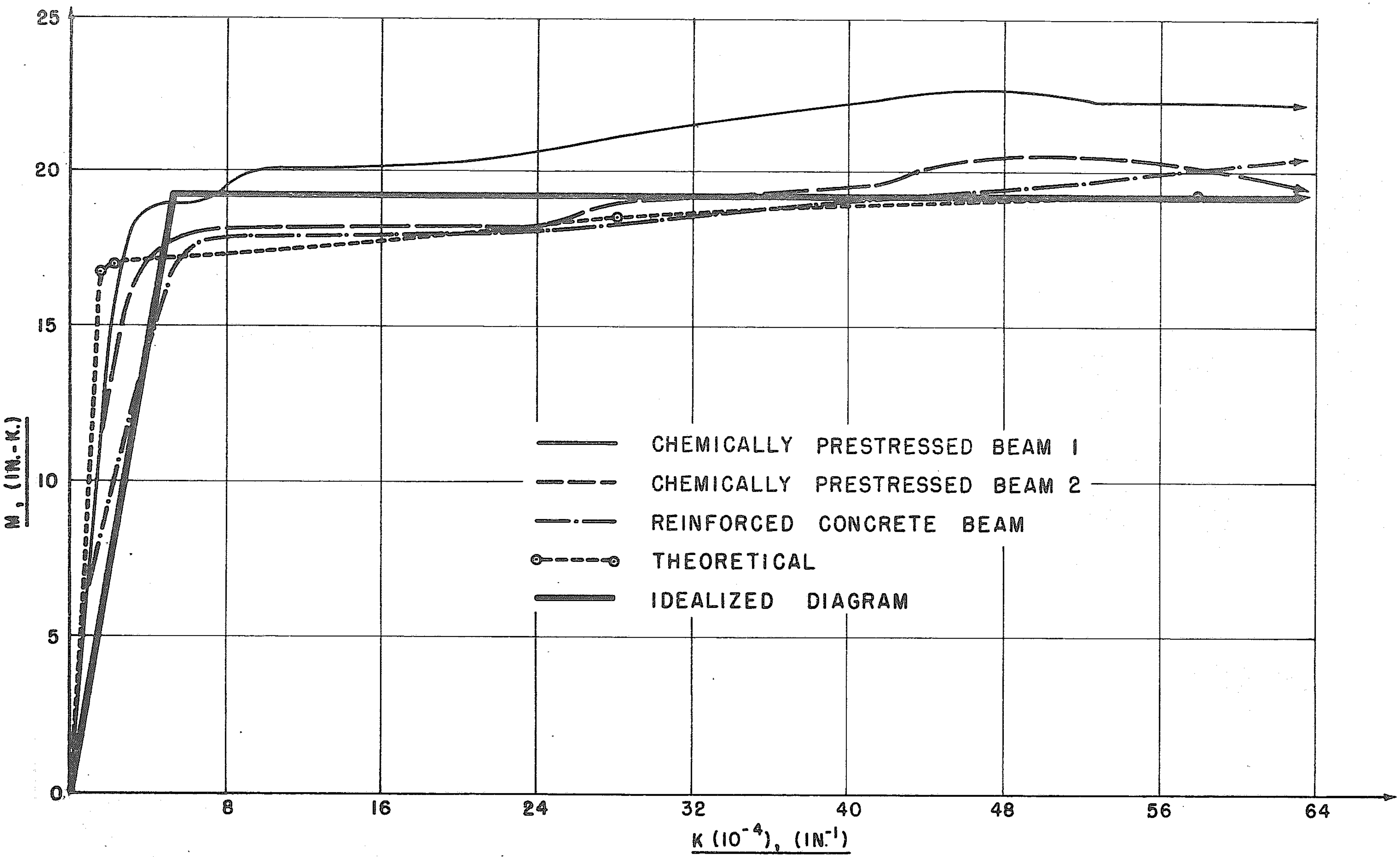


FIG. 23 COMPARISON OF MOMENT-CURVATURE CURVES

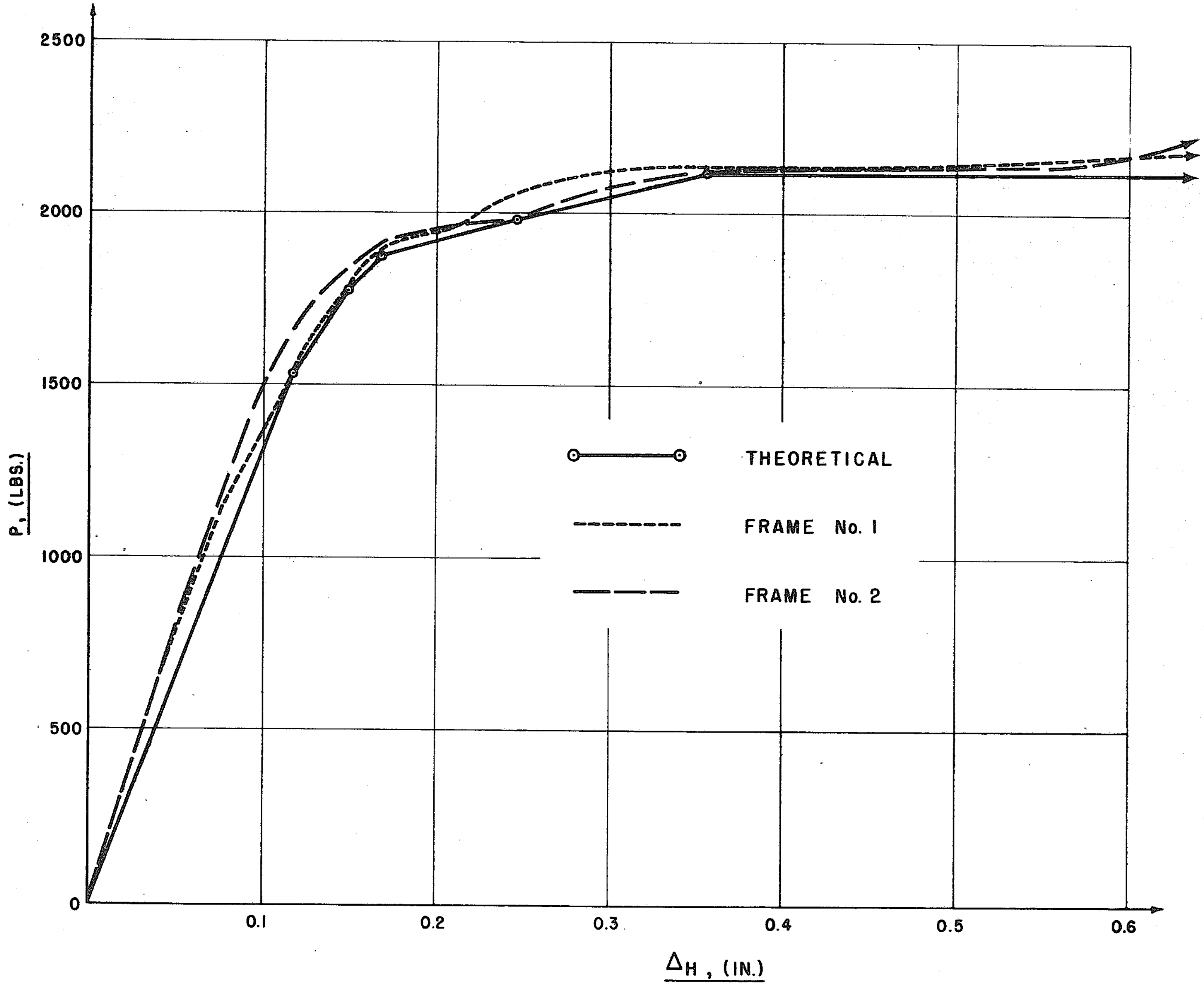


FIG. 24 COMPARISON OF LOAD - HORIZONTAL DEFLECTION CURVES.