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ABSTRACT

A series of sixteen under-reinforced beams was tested to study the effectiveness of external strengthening using fiber-reinforced plastic (FRP) plates. FRP is attractive for this application due to its good tensile strength, low weight, and resistance to corrosion. Thin plates of glass, carbon, and Kevlar™ fibers were bonded to the tension side of the beams using a two-part epoxy. Unplated and steel-plated beams were included in the series for comparison.

Increases in stiffness (over the working load range) from 17-99% and increases in strength (ultimate) from 40-97% were achieved for the beams with FRP plates, compared with the unplated beams. Experimental failure generally occurred in the beams at the ends of the FRP plates, although two failures occurred in the maximum moment region. Attempts to shift failure to the maximum moment region by using end anchorages were only partially successful.

An iterative analytical method was developed to predict the stiffness and maximum strength in bending of the plated beams. Predicted and actual load-deflection curves showed fairly good agreement, although generally the theoretical curves predicted greater stiffness. The ultimate strength of the beams that did fail in the maximum moment region were within about 5% of predicted values.

The results of this series of tests suggest that further evaluations of external FRP reinforcement should be conducted.

INTRODUCTION

Traditionally, the strengthening or upgrading of steel beams has been relatively easy. Through the use of welding or mechanical connections, additional steel can be added to increase the load-carrying capacity of a beam. This is not so easily accomplished with concrete. Until recently, there has not been a reliable and economical method of upgrading reinforced concrete beams, short of demolition and replacement. The emergence of high strength epoxies and other structural adhesives has changed this situation. By using adhesives, additional material can be bonded to a concrete beam, increasing its strength and stiffness, in much the same manner as for steel.

One method which can be used involves the bonding of thin plates, which are strong in tension, to critical areas on the concrete beam which are under tension. This will increase the capacity of the beam, while only minimally altering its dimensions. While much research has been performed on the bonding of steel plates to concrete members,[1-14] other materials with good tensile properties could also be effective. One such class of material is fiber-reinforced plastics (FRP). Some exploratory work has been done on the use of FRP to strengthen concrete structures.[15-18] Additional work in this area is in progress.[19,20]

Bonding of steel plates to concrete has been shown to be an effective upgrading method when three important practices are followed. First, the surfaces to be bonded must be clean. Sand blasting the steel and concrete surfaces is preferred, although other methods have been used effectively. Second, the epoxy bonding agent should have a strength of at least that of the concrete (failure should occur in the concrete). The epoxy should also be usable under the prevailing environmental conditions. Third, plates must be long and thin in order to avoid an undesirable brittle plate separation failure, although additional anchorage at the ends of the plate can also be used to avoid this type of failure.[8] By following these guidelines, steel plates have been used effectively and economically to improve the strength and serviceability of existing reinforced concrete structures.[4] It is safe to assume that similar procedures, as well as additional considerations, must be applied to plates of other materials used for external reinforcement, such as fiber-reinforced plastics.

Plates with the three most common reinforcing fibers, glass, carbon, and Kevlar™, were bonded to the tension side of a concrete beam. Since some of the plates were off-the-shelf products and others were not, the plates did not always have the optimal properties required, but a good cross-section of materials and properties was investigated.

The primary area of interest was the magnitude of increases of strength and stiffness of the beam provided by the bonded plates, and the effect that the differing strength and elastic modulus of the plates had on these increases. Other interests included theoretical analysis for prediction of strength and stiffness, and investigation of the failure modes. Accurate predictions are needed in order to develop suitable procedures for strengthening.

While this study focused on the flexural behavior, the possibility of using these plates for shear reinforcement is recognized. On some beams where side plates were used for end anchorage (and not considered in shear), they did contribute to the shear strength. Additional work could be performed in this area, to determine the feasibility of using FRP plates for shear strength upgrade.

DESIGN OF TESTS

DESIGN OF CONCRETE BEAM

The design of the concrete beam was carried out according to the specifications of the ACI code.[21] The steel reinforcement was chosen to approach the lower limit for an under-reinforced beam (minimum reinforcement ratio - A_{st}/bd - is 0.0033, maximum ratio - 75% of balanced - is 0.0309, actual ratio used was 0.0067). This was done so that additional reinforcement could be added without over-reinforcing the beam (which would lead to premature brittle failure of the concrete), and attempts were made to stay within this range (although some plates exceeded the limit, no failures occurred). The dimensions of the beam were 6 inches wide, by 12 inches deep, by 9 feet long (see Fig. 1). The span of the beam was limited by the length of the available FRP plates, since it was considered undesirable to try to splice the plates at this stage of the investigation.

The internal flexural reinforcement consisted of two #4 steel reinforcing bars (1/2 inch diameter, 60 ksi minimum yield stress - see Fig. 1). Tensile tests were performed on the reinforcing bars and the values used for theoretical predictions. The flexural strength of the unplated beams according to ACI can be found in Table 1.

Shear reinforcement consisted of D2.5 deformed steel bars every 4 inches (see Fig. 1). The beam was overdesigned in shear (>100%) in order to avoid a brittle shear failure due to the increased shear load on the strengthened beam. Although in many cases the load on the strengthened beams exceeded twice the design ultimate strength, none of the beams failed in shear through the original cross-section.

CASTING OF THE BEAMS

Sixteen beams were cast for the program on April 12, 1988. Wooden forms with the top of the beams exposed were used with a wax form release agent. A standard ready-mix concrete was ordered, with a maximum aggregate size of 3/4 inch, and a minimum compressive strength of 4500 psi. After eight beams were poured (batch 1 - beams A,E,G,H,K,M,O,P), some water was added to the mix to improve workability (batch 2 -beams B,C,D,F,I,J,L,N). Six 6x12 inch concrete test cylinders were cast for each batch. The strength of the cylinders in unconfined compression is shown in Table 2 (for theoretical predictions, concrete compressive strength for each beam was interpolated from cylinders tested before and after all beams were tested). As can be seen, the actual strength greatly exceeded the specified. This problem will be discussed later. There was some variation in the cross-sections of the beams, but overall the dimensions were fairly accurate.

PLATES USED

The plates used in tests were as follows (beam designations in parentheses):

1. Morrison Molded Fiber Glass standard pultruded fiberglass sheet (C,D,E,F).
2. 0°/90° glass fiber-reinforced plastic (G).
3. Morrison Molded Fiber Glass standard pultruded fiberglass channel 8" x 2 3/16" x 3/8" (split into angles) (H).
4. 0°/90° 65% glass/35% carbon fiber-reinforced plastic (I).
5. 3M Scotchply 1002 spring orientation glass fiber-reinforced plastic (J,K).
6. 0°/±60° carbon fiber-reinforced plastic (L).
7. 0°/90° carbon fiber-reinforced plastic (M).
8. Unidirectional Kevlar™ fiber-reinforced plastic (N).
9. Mild steel plate (O,P).

All the plates were subjected to longitudinal tensile tests (according to ASTM D3039-76) to determine elastic modulus and ultimate strength. Dimensions and properties of the bonded plates are given in Table 3.

ADHESIVE USED

Several different adhesives, including epoxies and acrylics, were explored for use on this project. It was determined that a two-part, rubber-toughened epoxy would be the most appropriate. An epoxy produced by the Lord Corporation, Fusor 320/322, was selected since it could be cured at room temperature and was tolerant of variations in mix proportions. Preliminary pull-off tests using steel bolts bonded to concrete coupons showed that it satisfied the requirement of failure in the concrete (conformed to ASTM standard C881-78). This preliminary finding was borne out in the subsequent tests.

BONDING OF THE PLATES

After the plates were cut to size, the bonding surfaces of the FRP plates were sanded to remove the shine, and the steel plates were sandblasted to remove mill scale. The concrete beams were turned upside down, and the form release and other loose debris was removed using an abrasive stone with water. The beams were then rinsed and allowed to dry. The two parts of the epoxy were then weighed to produce the right proportions. The epoxy was hand mixed and hand applied at an approximate thickness of 1/16 inch to both parts to be bonded using a metal spatula. After the plate was positioned, it was held down with steel weights during cure. Bond thickness was not specifically controlled, but excess epoxy was squeezed out along the edges of the plate, assuring complete epoxy coverage. The epoxy was allowed to cure a minimum of 24 hours before the beam was tested.

INSTRUMENTATION AND TEST PROCEDURE

Electrical resistance strain gages were affixed to five locations along the plate and to three locations along the centerline of the beam. The plate gages were at centerline, 18 inches outside the load points, and 2 inches from the end of the plate. The beam gages were in two locations in the compression zone of the concrete, and one location internally on the reinforcing steel (see Fig. 1).

The beams were tested in four-point bending over an 8 foot simple span (the span was increased 2 inches for beams with 8 foot plates, due to plate interference with supports), with the load points 1 foot either side of center (see Fig. 1). Centerline deflection measurements were made using a dial gage. Clip gages were positioned between the concrete and the FRP at the ends of the partial length plates to measure any slip that occurred during loading. It was found that any movement prior to vertical cracking of the concrete at the end of the plate was negligible.

The beams were tested to ultimate load. Two beams without plates were tested as control specimens. The control beams were statically loaded to failure in one cycle, while the beams with external reinforcement were cycled up and down several times to determine permanent displacements. The loading was force controlled (each load step corresponding to a specified increase in force) as long as some of the tensile reinforcement remained elastic. Deflection control (each load step corresponding to an incremental increase in deflection) was used when the beam entered the plastic range. This occurred only in the control beams and those with steel plates.

MODIFICATIONS AS THE TESTS PROGRESSED

The initial tests showed that with bonded plates the increase in strength was indeed substantial. The beams, however, failed not in the maximum moment region but at the end of the partial length plates (see Fig. 2a). In order to try to shift the location and mode of failure, as well as increase the ultimate strength of the beams, four modifications were tried.

The first consisted of anchoring the ends of the plate using an unequal leg fiberglass angle (Fig. 3b) similar to the method used by Jones, Swamy, and Charif in their steel plate tests.[8] This led to a higher load capacity, but the failure mode was not altered.

The second method consisted of bonding full height FRP plates to the sides of the beam at the plate ends and then connecting these to the plate using bonded fiberglass angles (see Fig. 3c). This method also led to a higher load capacity, and successfully shifted the failure mode on one beam (Beam E). On the others (Beams J,N), however, the connection between the side plate and lower plate failed and the failure mode was not altered.

A third method tried was replacing the plate with a pair of angles bonded along the underside of the beam (Fig. 2d). Since the beams were failing along the level of the longitudinal reinforcing steel, it was thought that having the angles extend above the steel would be beneficial. This was indeed the case, but the angles did not extend high enough above the steel level to shift the failure mode.

The final method used was the extension of the plates right up to the supports (Fig. 3a). This method was very successful in increasing the load capacity and in shifting the failure mode of one beam (Beam L). This method would be more effective in cases where the shear to moment ratio was lower (for instance on longer spans).

THEORETICAL ANALYSIS

OVERVIEW OF METHOD

The ACI method for determining ultimate strength is based on the ability of the reinforcement to deform plastically. Obviously, this is incompatible with fiber-reinforced plastics, since they have no yield plateau. An alternative analysis technique must be used to deal with this condition.

The technique chosen to predict the strength and stiffness of these beams was an iterative analysis technique developed by Geymayer in his study of reinforced concrete beams with unconventional reinforcement.[22] This technique is only practicable by computer.

ASSUMPTIONS

Several assumptions commonly made in reinforced concrete theory are used including:

1. Plane sections remain plane.
2. No slip between **any** longitudinal reinforcement and concrete.
3. Tensile strength of concrete is zero.
4. Stress-strain relationships of materials as determined by standard uni-axial tests are representative of their behavior as part of the beam.

Based on these assumptions, a computer program was developed to analyze each beam reinforcement configuration.

DATA REQUIRED

First of all, all dimensions of the beam (height, width, depth to steel, external plate dimensions) must be known. The span and the external load points are required to determine the stresses and internal forces of the beam. The entire stress-strain relationships of the steel, the external plate, and the concrete (which was approximated with a tri-linear curve in this program, although a more precise curve could be used), must also be known.

ITERATIVE COMPUTER METHOD

Once all data are input into the computer program, the load and deflection are determined using strain compatibility (refer to Fig. 4). First, a top fiber concrete strain and a neutral axis depth are assigned. The depth between the top compression fiber and the neutral axis is divided into ten slices. Using the average strain for each slice (made using the assumption of piecewise linear fiber strain), the compression stress can be found using the concrete stress-strain curve. Multiplying this by the area of the slice gives the compressive force. A similar method is used to determine the two tensile forces of the reinforcing steel and external plate. The neutral axis is then adjusted until the sum of the ten compressive forces equals the sum of the tensile forces (equilibrium).

When this is achieved, the moment is determined by summing the ten compressive forces and two tensile forces times their moment arms about a single point. The curvature is determined from the top fiber strain and the neutral axis depth. Using a rather coarse finite difference model (half of beam cut into four sections), the slope and deflection of the beam are found using the moment-area method. The maximum strength of the beam is determined when either the moment reduces for an increase in the top fiber strain, or the reinforcement fractures. This analysis method differs from the ACI code in that the concrete compressive strain is permitted to exceed 0.003 in/in. The strength is not limited by that parameter but by the measured material properties directly.

INFORMATION PROVIDED BY COMPUTER PROGRAM

Early versions of the program were used as a guide in designing the experiment, including selecting the dimensions of the beam, selecting the concrete strength and steel reinforcement, and sizing and selecting several of the plates that were used. Thicknesses of the plates were selected to try to keep the improved strength below a 100% increase over the design strength. This was not altogether successful, due to both the concrete and certain plates being stronger than anticipated, but no major problems resulted.

All of the beams in the test program were analyzed using this program. The program was mainly developed to predict ultimate strength in bending, but it also predicted the load-deflection characteristics (stiffness) of the beam. For the beams that failed in shear at the plate end, the program could not predict ultimate strength, but it did provide the load-deflection relationship up until failure.

RESULTS AND DISCUSSION

Experimental results for all the beams tested are summarized in Tables 4-7 and Fig. 5. The development of crack patterns for the unplated control beams and plated beams is shown schematically in Figs. 6 and 7. Finally, examples of additional test results are shown graphically in Figs. 8 and 9. Details of the remaining tests are available in Reference 23.

Although a wide variety of plates was used, with a wide range of properties, the following behavior was characteristic of most beams tested.

CRACK PATTERNS

The crack patterns developed similarly for all the plated beams. First cracking usually occurred at a slightly higher load than in the unplated beams. Initially, the cracks were vertical, as would be expected for flexural cracks, but later they would bend over in the shear regions. Generally, there were more cracks, more closely spaced, more uniformly distributed, and narrower on the plated beams than on the unplated beams (see Figs. 6 and 7).

STRESS AND STRAIN DISTRIBUTIONS

The strain in the compression portion of the concrete never reached the crushing stage for any beam with an FRP plate bonded to it. Part of the reason for this is that the concrete was stronger than intended, and it was able to withstand a higher compression force than originally anticipated. This higher strength probably also helped postpone failure at the plate ends, however, due to its higher tensile strength.

When the internal steel bars yielded, on most beams a marked stiffness decrease occurred (seen as a "bend" in the moment-deflection curves - most pronounced in the beams with low modulus glass FRP plates, Fig. 5, at an approximate moment of 300 k-in). This was attributed to the yield strain of the internal steel being much lower than the ultimate strain of the external plate. On the steel plated beams the external plate yielded first since its yield stress (and corresponding strain) was lower than that of the internal steel. In the Kevlar™ plated beam (Beam N) and the beam with bonded angles (Beam H), failure occurred before the internal steel yielded due to the high strength and stiffness of the external reinforcement.

A typical moment-plate stress diagram is shown in Fig. 8. The stresses are determined from the strain gages located on the plate as shown in Fig. 1. As can be seen, the plate does not reach ultimate strength in this case.

Failure modes of the beams are summarized in Table 4. Many plated beams failed through the concrete, with cracking initiating from the end of the plate (see Fig. 2a). Notice in Fig. 8 that the stress at the end of the plate decreased as failure initiated there, before failure could occur in the constant moment region. Attempts to reinforce the plate ends were only partially successful. Stresses in the plates did reach ultimate on beams E and L, which had end reinforcement and a full-length plate, respectively.

STRENGTH/STIFFNESS INCREASE

All plated beams showed at least a modest increase in stiffness, and some showed a substantial increase. The increases ranged from 17 to 99% over the working load range (see Table 5). The stiffness values were calculated by taking a line from the origin through the point on the load-deflection curve where the deflection equaled span/480 (ACI limit for roof or floor construction attached to nonstructural elements likely to be damaged by large deflections).

Strength increases were similarly substantial, ranging from 19 to 99% over the working load range (see Table 6), and from 28 to 97% at ultimate (see Table 7), despite full flexural capacity not being reached on many beams. Percentage increases were based on the actual strength values (not design values) for the control beams.

Ultimate loads (ultimate stress x area) for the bonded FRP plates ranged from about 26 kips to about 77 kips (omitting the Kevlar™ plate which was much thicker and stronger). As mentioned previously, these strengths led to an undesirable brittle shear failure mode. By using a full length plate, one of the plates with 26 kip strength was fractured in the constant moment region. This suggests that the limiting average bond stress between the plate and the concrete for these beams is about 120 psi. As the average bond stress becomes higher, anchorage of the plates becomes necessary.

The relationship between force that must be transferred and bond area required should be determined for each individual case, since it is dependent on the concrete strength and applied loading. Lowering the average anchorage stresses is preferable to trying to anchor the ends of the plate, since extending the plate is easier than other end anchorage schemes.

The maximum strength attainable for these beams using attached plates was not determined in this study, since the concrete was not crushed in the compression region for any of the beams with FRP plates, and plenty of reserve concrete strength was available. To determine this capacity, better end anchorage or longer specimens would be required.

DUCTILITY OF PLATED BEAMS

Although all of the FRP plates used were brittle, and did not demonstrate the yield plateau associated with steel, deflections of many of the beams exceeded an inch at failure (see Fig. 5 and Table 7). Heavily reinforced standard concrete beams probably would not be any more ductile than some of these beams were. Despite their brittleness, through proper design fiber-reinforced plastics can develop enough ductility to be utilized as effective concrete reinforcement.

PERFORMANCE OF EPOXY

The epoxy used, Lord Fusor 320/322, caused no problems at all during testing. The fairly wide range of mix ratios for which properties remained consistant was helpful, as slight errors could be tolerated. Despite several incompletely bonded plates (voids present) early in the program, failure never occurred through the epoxy layer, although sometimes the failure was in the first layer of concrete next to the epoxy plane.

COMPARISON WITH THEORETICAL MODEL

The mathematical model was not verified completely due to the lack of failures through the constant moment region, but the two beams (E and L) with FRP plates which failed in bending were within about 5% of the predicted strength (see Table 7). The strength of beams with steel reinforcement was underestimated since the effect of strain-hardening was not taken into account. Load-deflection curve prediction was rather crude, but although stiffness was almost always overestimated, in most cases the theoretical curves were very close to the experimental curves (see Fig. 9 for a typical example). By means of better approximations of stress-strain relationships and a more complex computer model, better agreement with tests could probably be achieved.

CONCLUSIONS

The results of these experiments are very encouraging. It has been demonstrated that bonded plates of fiber-reinforced plastic are indeed a feasible method of upgrading the strength and stiffness of a reinforced concrete beam. Although the material costs of such a system are higher than steel, the light weight may be advantageous for installation and the corrosion resistance should be useful under harsh environmental conditions.

Although it was demonstrated that the strength of concrete beams can be increased through the bonding of external plates, the mathematical model was only partially verified. Not enough beams failed in the constant moment region to give statistically meaningful results, although those that did were within about 5% of the predicted load. In order to validate the model, additional tests should be run, preferably with a longer span, in which the shear to moment ratio is lower, such that the failures will occur in bending in the constant moment region.

The beams with externally bonded plates also exhibited another desirable phenomenon in that the crack patterns shifted from several widely spaced and large width cracks to many more closely spaced narrower cracks. This could be advantageous in the serviceability of the structure, as it is always better to have smaller, less noticeable cracks.

The stress concentration and end anchorage problem need additional study in order to avoid the prevalent failure mode at the plate ends. Additional work also needs to be done to develop design rules to deal with the brittle nature of these materials. Finally, if this method is applied to an actual structure, the adhesive performance should be tested for creep, fatigue, and environmental stability.

Fiber-reinforced plastics are very versatile. They can be made into any shape, and the properties can be varied widely with differing fibers and orientations. Their high strength and light weight make them attractive structural components. Work should continue in this field with the goal of making fiber-reinforced plastics structurally and economically appealing for construction.

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Table 1 - Control beam results

METHOD USED TO DETERMINE	BEAM	YIELD LOAD (KIPS)	ULTIMATE LOAD (KIPS)
ACI CODE	A[a]	12.04	12.70
	B[b]	12.04	12.66
ITERATIVE ANALYSIS	A	12.02	12.76
	B	11.80	12.54
EXPERIMENT	A	12.50	16.40
	B	12.00	16.30

[a] Concrete batch 1
[b] Concrete batch 2

Table 2 - Concrete compression tests
6" X 12" cylinder

DATE TESTED	CONCRETE BATCH	ULTIMATE STRENGTH (KSI)	MODULUS OF ELASTICITY (KSI)
4-25-88	1	5.20	N/A
4-25-88	2	4.80	N/A
5-27-88	1	6.23	3700
5-27-88	2	5.77	3300
10-17-88	1	7.39	N/A
10-17-88	2	6.85	N/A

N/A - Not Available

Table 3 - External reinforcing plate properties.

BEAM	TYPE OF PLATE	FIBER ORIENT. (DEG)	MODULUS OF ELAST. (KSI)	ULTIMATE STRENGTH (KSI)	PLATE DIMENSIONS WIDTH (IN)	THICKNESS (IN)	LENGTH (IN)
A	CONTROL BEAM - NO PLATE						
B	CONTROL BEAM - NO PLATE						
C	G	0-R	1700	23.3	6.00	.1875	80
D	G	0-R	1700	23.3	5.95	.1875	80
E	G	0-R	1700	23.3	6.05	.1875	78
F	G	0-R	1700	23.3	6.05	.375[c]	78,54
G	G	0-90	1500	26.7	6.00	.165	96
H	G	0-R	3000	35.0	6.00	.3646[d]	80
I	C-G	0-90	4000	46.3	5.90	.16	80
J	G	0-90	4400	85.6	6.00	.126	70
K	G	0-90	4400	85.6	6.00	.126	70
L	C	0-60	7900	89.0	6.00	.05	96
M	C	0-90	17,100	216.0	6.00	.05	96
N	K	0	10,500	170.0	6.05	.25	72
O	STEEL	N/A	29,000	30.0[e]	6.05	.102	80
P	STEEL	N/A	29,000	30.0[e]	5.90	.100	96

G-Glass, C-Carbon, K-Kevlar[tm], R-Random, N/A-Not Applicable

c - Two .1875 inch thick plates, 78 and 54 inches long.

d - Two angles, values given are effective plate width and thickness.

e - Yield point for steel, not ultimate strength.

Table 4 - Failure mode for each beam

BEAM	END ANCHOR.	FAILURE MODE	ILLUST. IN
A		STANDARD REINFORCED CONCRETE BEAM FAILURE STEEL YIELD, THEN CONCRETE CRUSHING	
B		" "	
C		INTERNAL STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
D		" "	FIG. 3A
E	FIG. 4C	INTERNAL STEEL YIELD, PLATE FRACTURE IN CONSTANT MOMENT REGION	FIG. 3C
F		INTERNAL STEEL YIELD, PLATE FRACTURE AT END OF COVER PLATE THRU LONGER PLATE	FIG. 3A
G	FIG. 4A	INTERNAL STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
H		NO STEEL YIELD, SHEAR FAILURE THRU CONCR. & ALONG REBAR, CONCR. SPLIT BETW. ANGLES	FIG. 3D
I		INTERNAL STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
J	FIG. 4C	" "	FIG. 3A
K	FIG. 4B	" "	FIG. 3A
L	FIG. 4A	INTERNAL STEEL YIELD, PLATE FRACTURE IN CONSTANT MOMENT REGION	FIG. 3C
M	FIG. 4A	INTERNAL STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
N	FIG. 4C	NO STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
O		EXTERNAL STEEL YIELD, SHEAR FAILURE THRU CONCRETE & ALONG REBAR AT PLATE END	FIG. 3A
P	FIG. 4A	EXTERNAL & INTERNAL STEEL YIELD, LARGE DEFLECTION, CONCRETE CRUSHING	FIG. 3B

Table 5 - Stiffness increase in working load range (at deflection = span/480)

BEAM	DEFLECTION SPAN/480 (IN)	CONTROL BEAM STIFFNESS (KIP-IN/IN)	EXPERIMENTAL BEAM STIFFNESS (KIP-IN/IN)	% INCREASE
A	.200	801.0	801.0	0.0
B	.200	774.0	774.0	0.0
C	.200	"	918.0	+18.6
D	.200	"	945.0	+22.1
E	.200	801.0	990.0	+23.6
F	.200	774.0	1062.0	+37.2
G	.204	801.0	934.3	+16.6
H	.204	"	1152.0	+43.8
I	.200	774.0	1080.0	+39.5
J	.200	"	1044.0	+34.9
K	.200	801.0	1035.0	+29.2
L	.204	774.0	916.2	+18.4
M	.204	801.0	1079.4	+34.8
N	.200	774.0	1539.0	+98.8
O	.200	801.0	1350.0	+68.5
P	.204	"	1414.7	+76.6

Table 6 - Strength increase in working load range
(at deflection = span/480)

BEAM	DEFLECTION SPAN/480 (IN)	CONTROL BEAM LOAD (KIP-IN)	EXPERIMENTAL BEAM LOAD (KIP-IN)	% INCREASE
A	.200	160.2	160.2	0.0
B	.200	154.8	154.8	0.0
C	.200	"	183.6	+18.6
D	.200	"	189.0	+22.1
E	.200	160.2	198.0	+23.6
F	.200	154.8	212.4	+37.2
G	.204	160.2	190.6	+18.9
H	.204	"	235.0	+46.7
I	.200	154.8	216.0	+39.5
J	.200	"	208.8	+34.9
K	.200	160.2	207.0	+29.2
L	.204	154.8	186.9	+20.7
M	.204	160.2	220.2	+37.4
N	.200	154.8	307.8	+98.8
O	.200	160.2	270.0	+68.5
P	.204	"	288.6	+80.1

Table 7 - Ultimate strength increase for each beam.

BEAM	THEORETICAL STRENGTH (KIP-IN)	EXPERIMENTAL STRENGTH (KIP-IN)	% DIFF.	CONTROL BEAM STRENGTH (KIP-IN)	% INCREASE (EXPER.)	MAX. DEFL. (IN)
A	229.5	295.2	+22.2	295.2	0.0	2.621
B	225.7	293.4	+23.1	293.4	0.0	2.835
C	518.9	448.2	N/A	"	+52.8	1.094
D	519.5	482.4	N/A	"	+64.4	1.210
E	529.3	504.0	-5.0	295.2	+70.4	1.350
F	758.7	538.2	N/A	293.4	+83.4	1.030
G	524.8	508.8	N/A	295.2	+72.4	1.635
H	740.6	450.9	N/A	"	+52.7	0.590
I	708.6	409.5	N/A	293.4	+39.6	0.479
J	658.6	497.7	N/A	"	+69.6	0.795
K	648.5	486.0	N/A	295.2	+64.6	0.906
L	525.7	496.7	-5.8	293.4	+69.0	1.124
M	923.5	582.8	N/A	295.2	+97.4	0.821
N	700.0	441.0	N/A	293.4	+50.3	0.433
O	437.6	378.0	N/A	295.2	+28.0	0.412
P	434.7	511.5	+15.0	"	+73.3	1.730

N/A - Failure occurred outside constant moment region

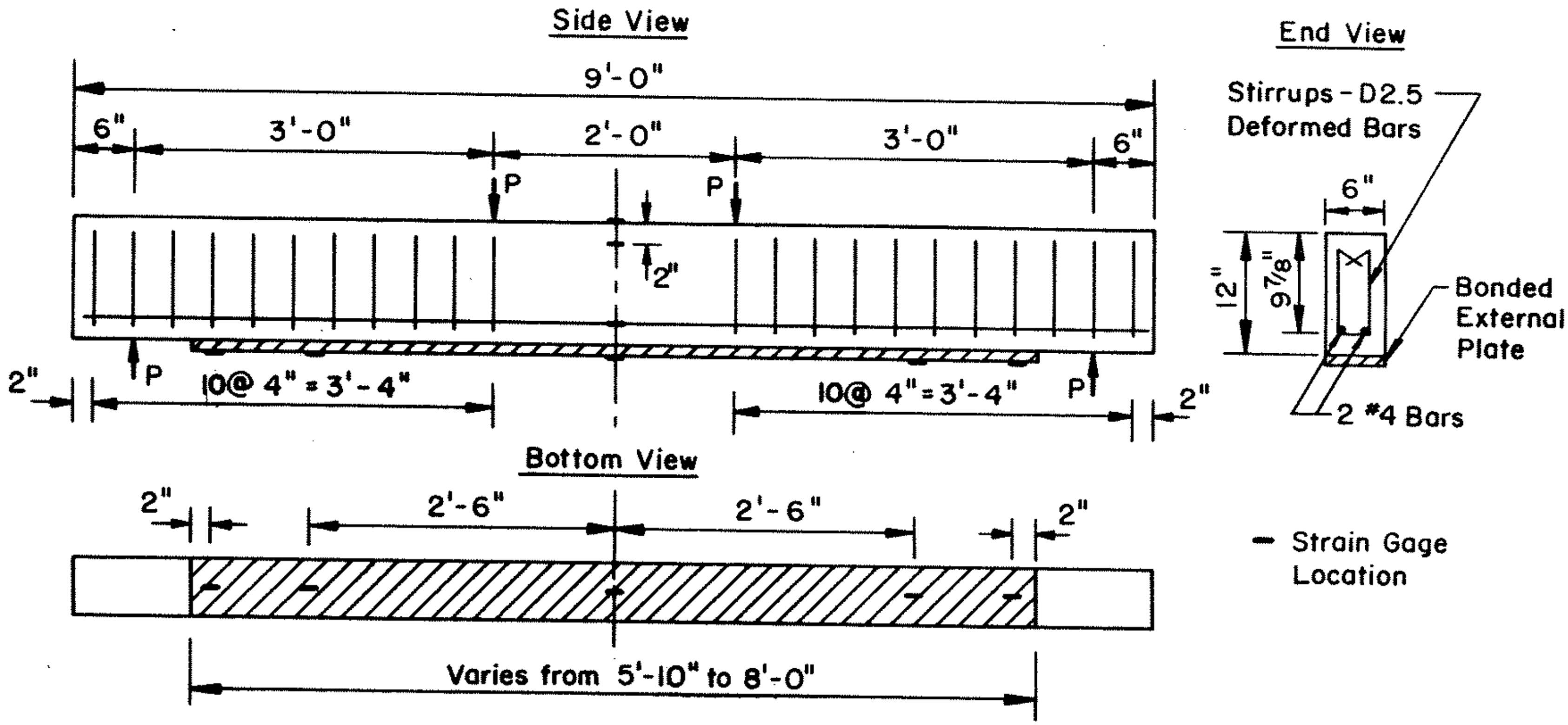

Figure 1 - Beam dimensions, internal reinforcing, strain gage locations, and load configuration

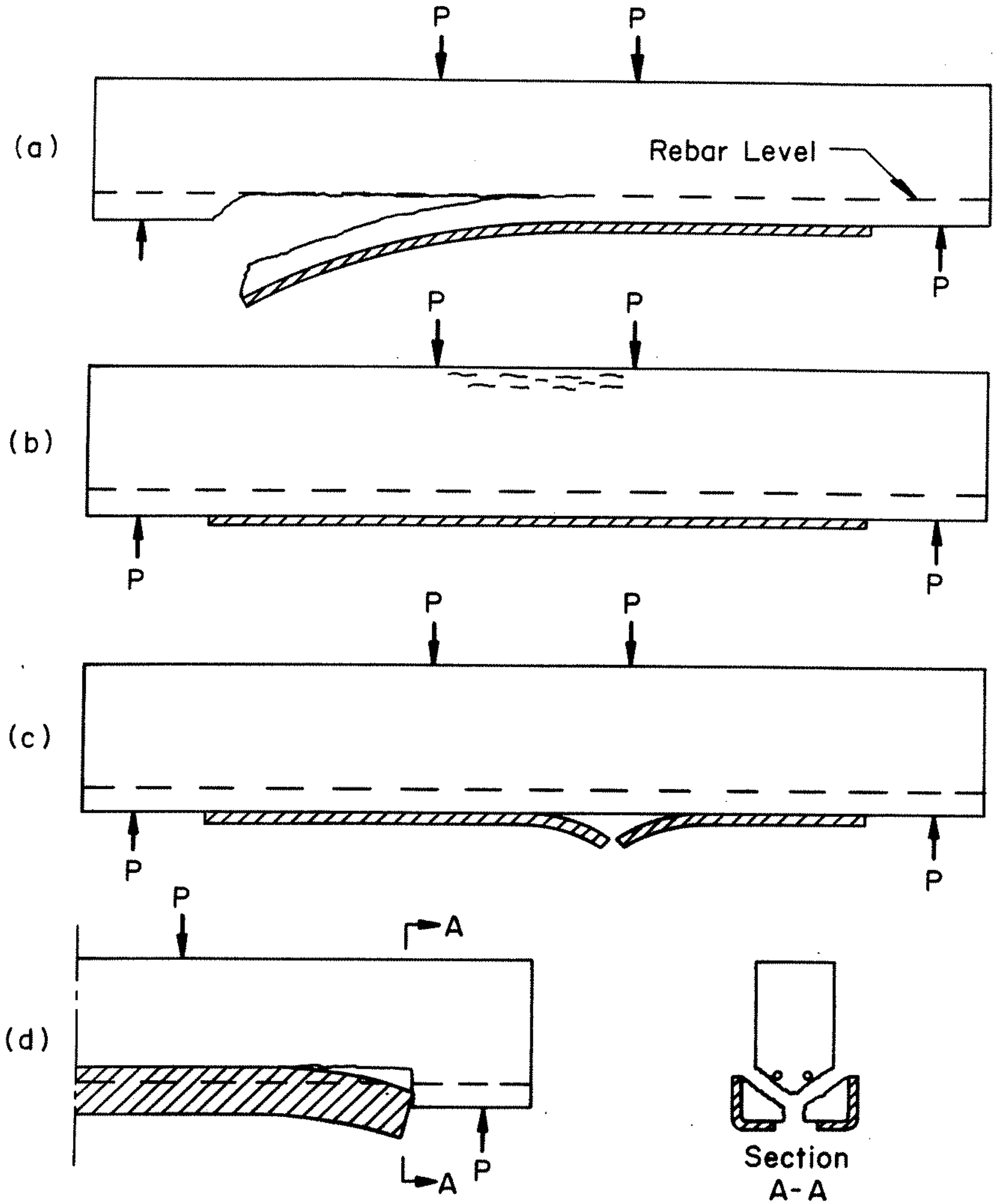

Figure 2 - Failure Patterns:
a) Typical end-of-plate failure through concrete
b) Concrete crushing in constant moment region
c) External plate failure
d) End of double angle failure through concrete

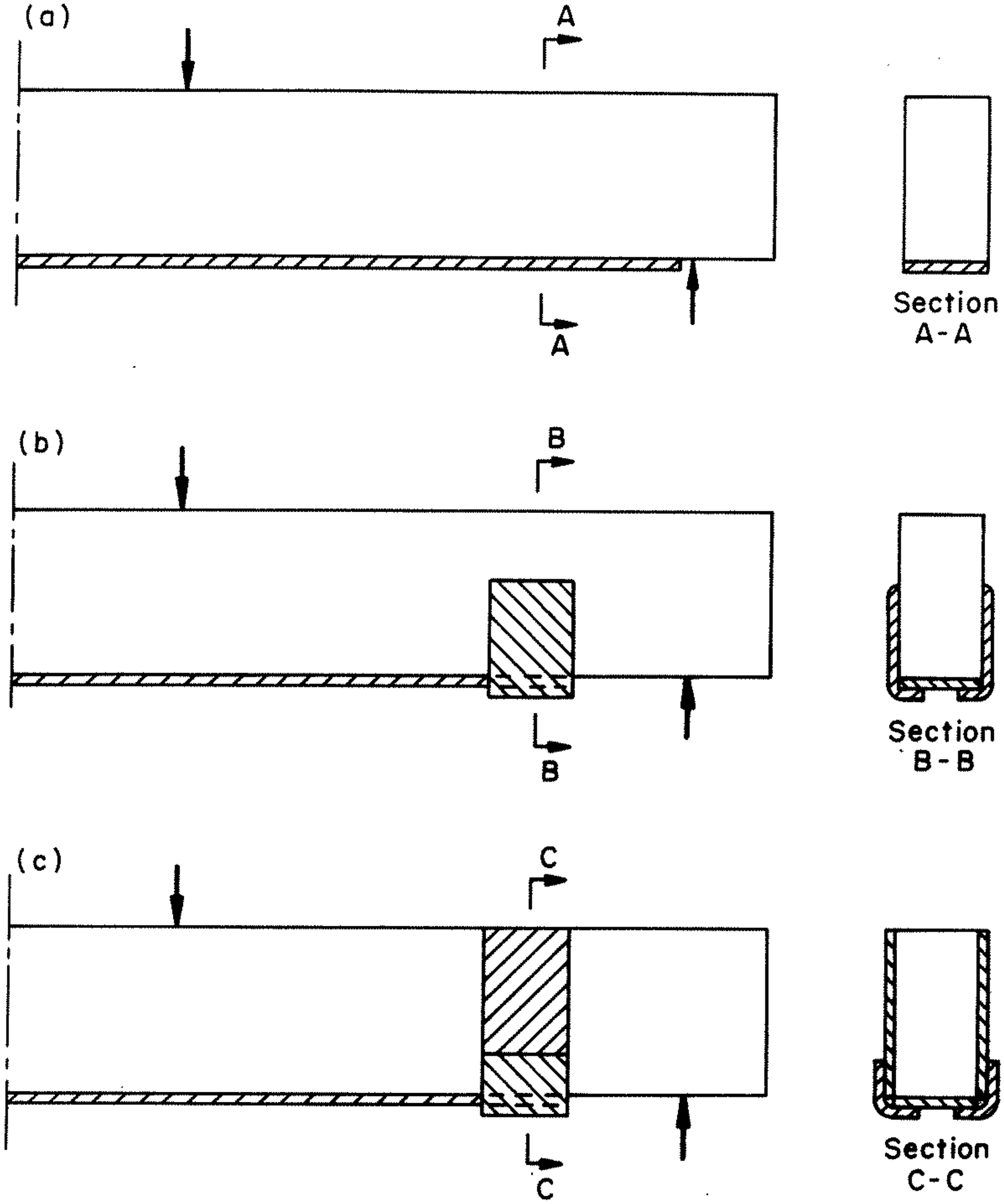

Figure 3 - End anchorages:
a) Plate extended up to (but not under) support
b) Partial height bonded angles
c) Full height bonded plates connected by bonded angles

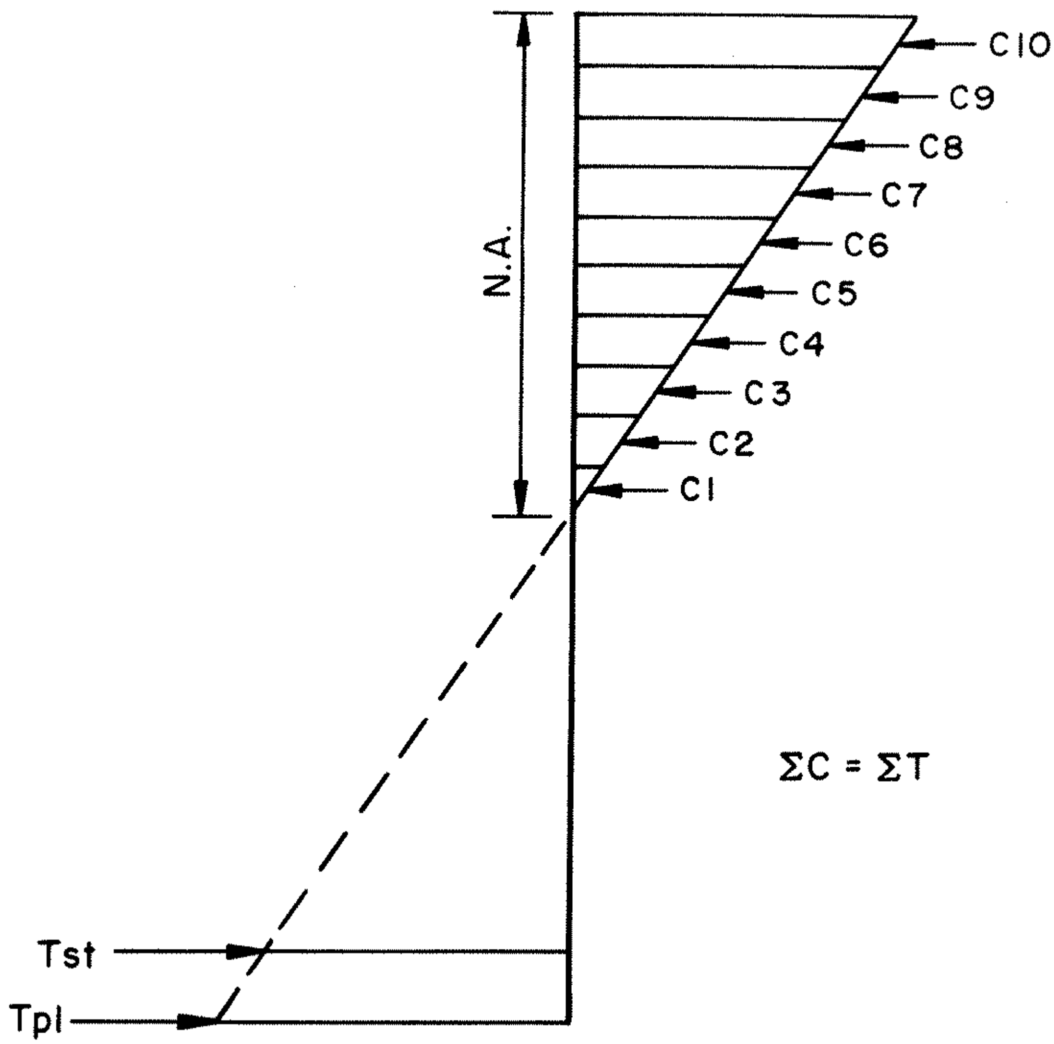

Figure 4 - Strain compatibility diagram for theoretical analysis

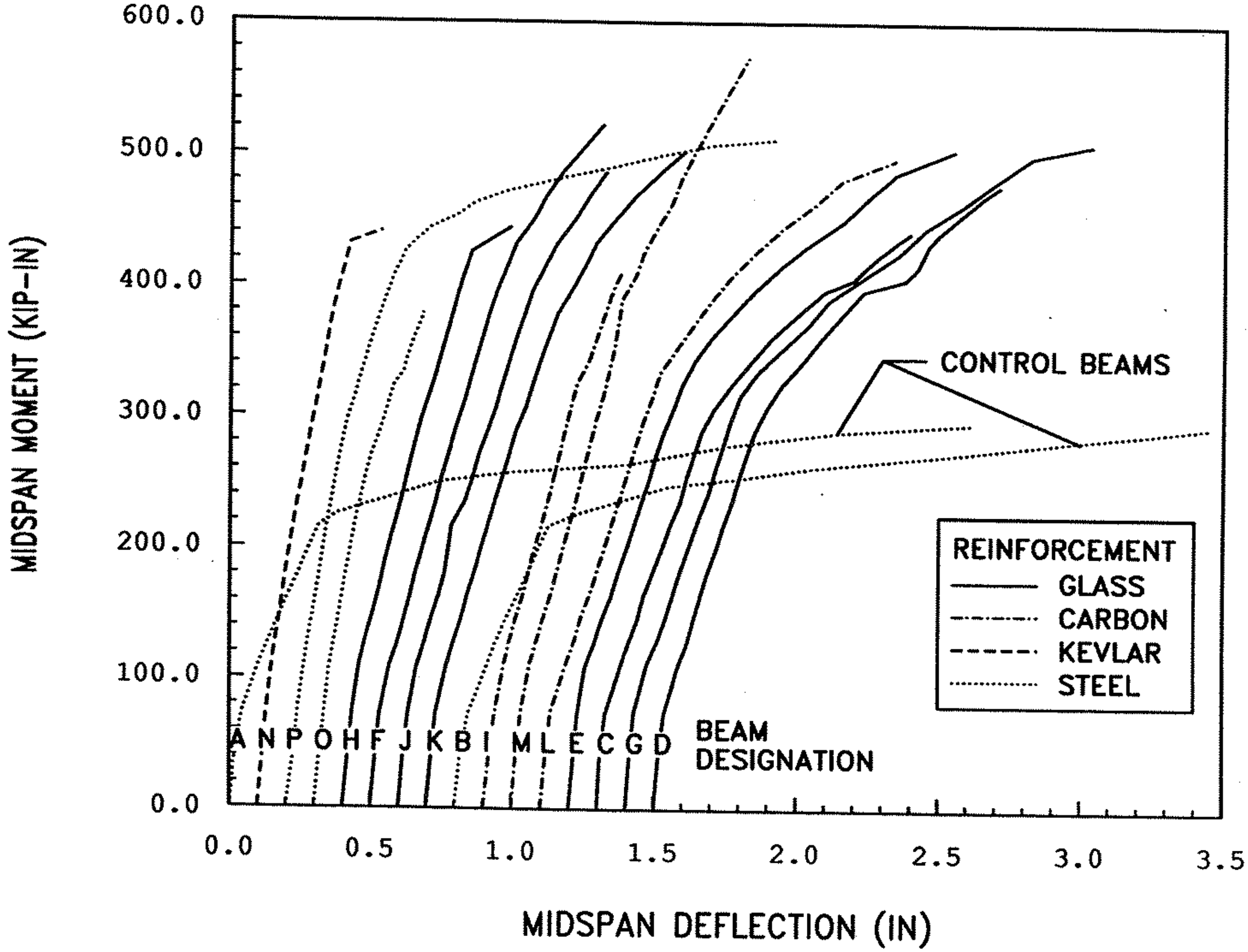

Figure 5 - Midspan moment vs deflection diagram for all beams (unloading cycles omitted for clarity)

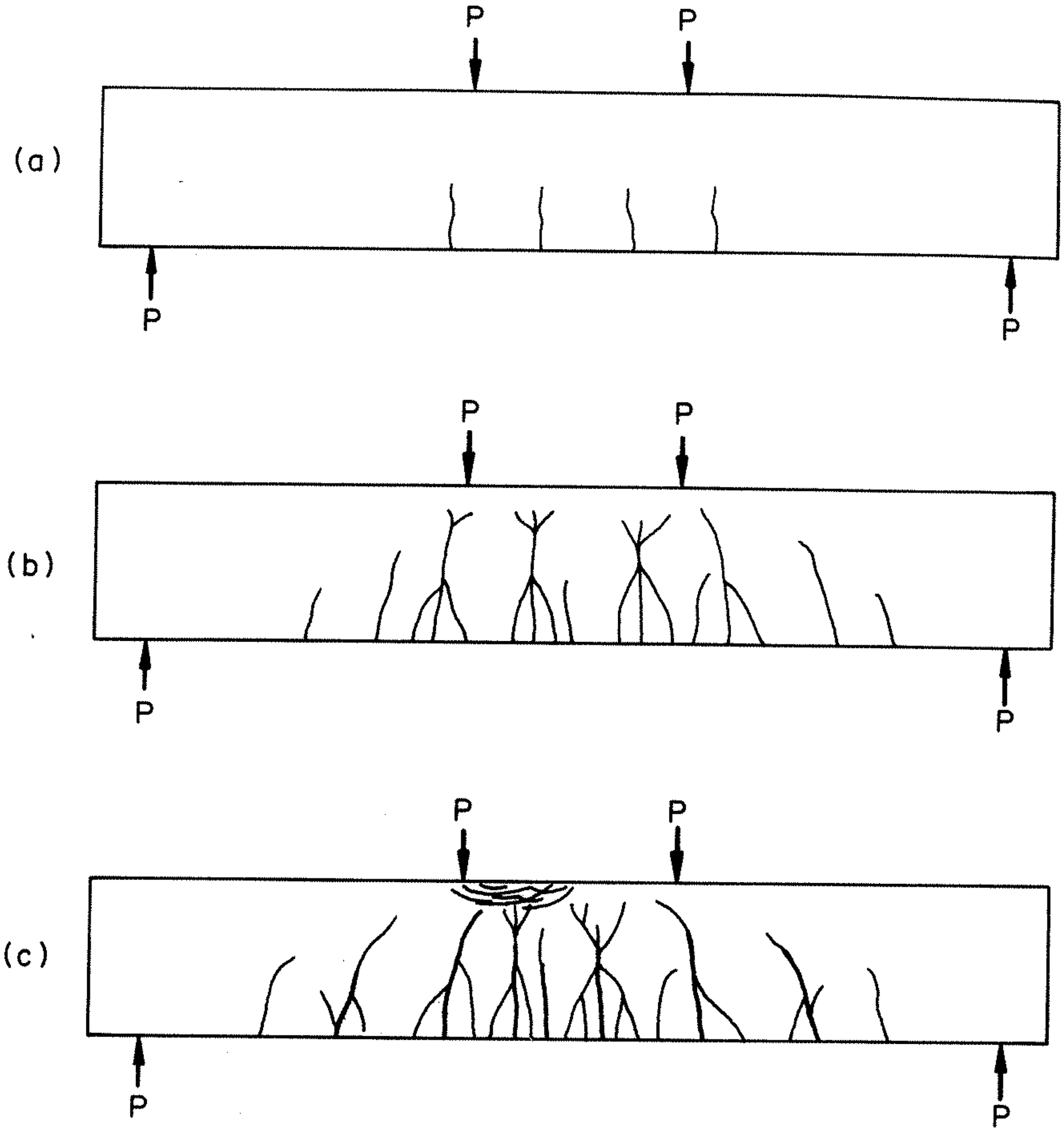

Figure 6 - Unplated beam typical crack patterns:
a) Initial cracking
b) Intermediate cracking
c) Ultimate cracking

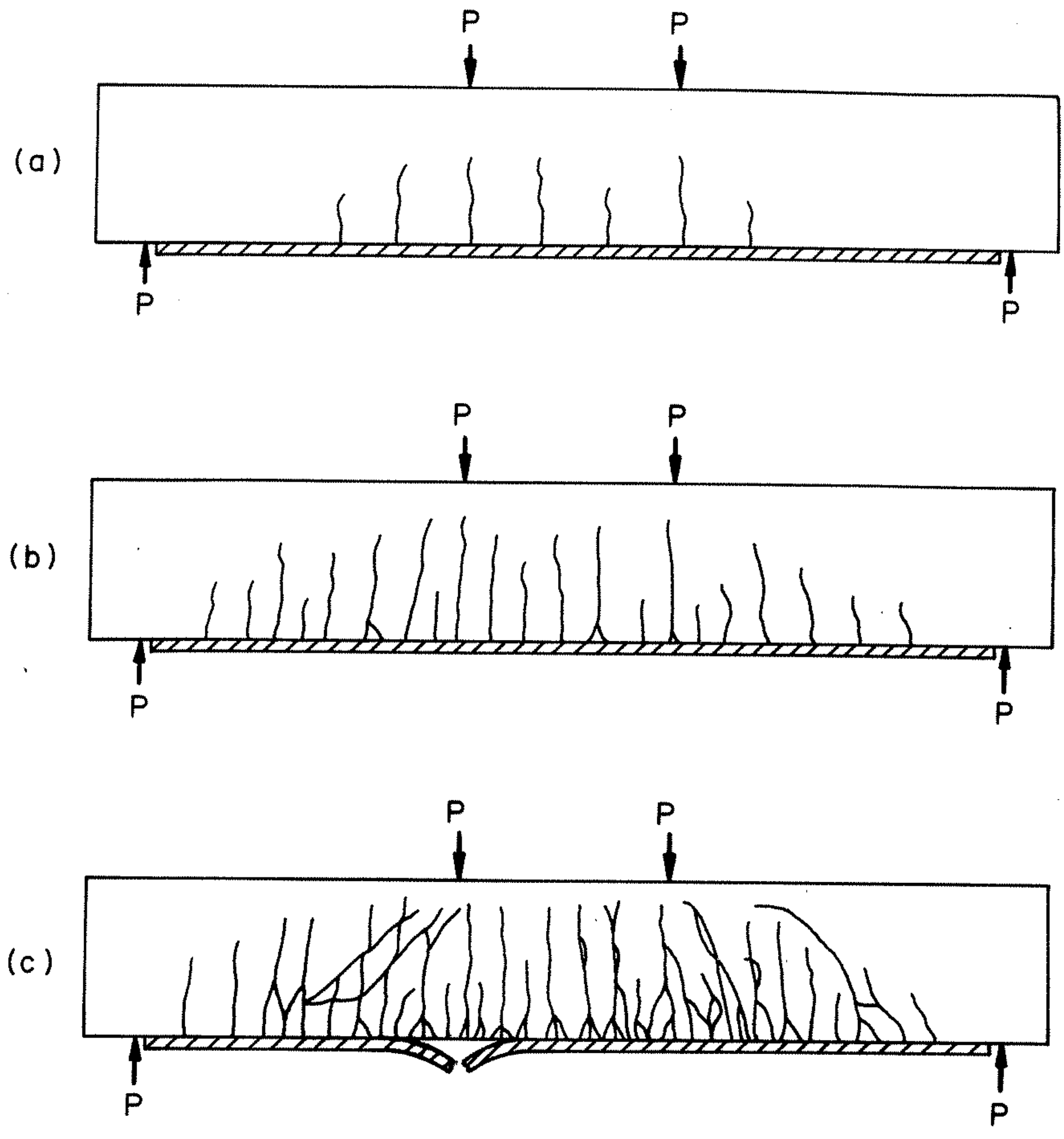

Figure 7 - Plated beam typical crack patterns:
a) Initial cracking
b) Intermediate cracking
c) Ultimate cracking

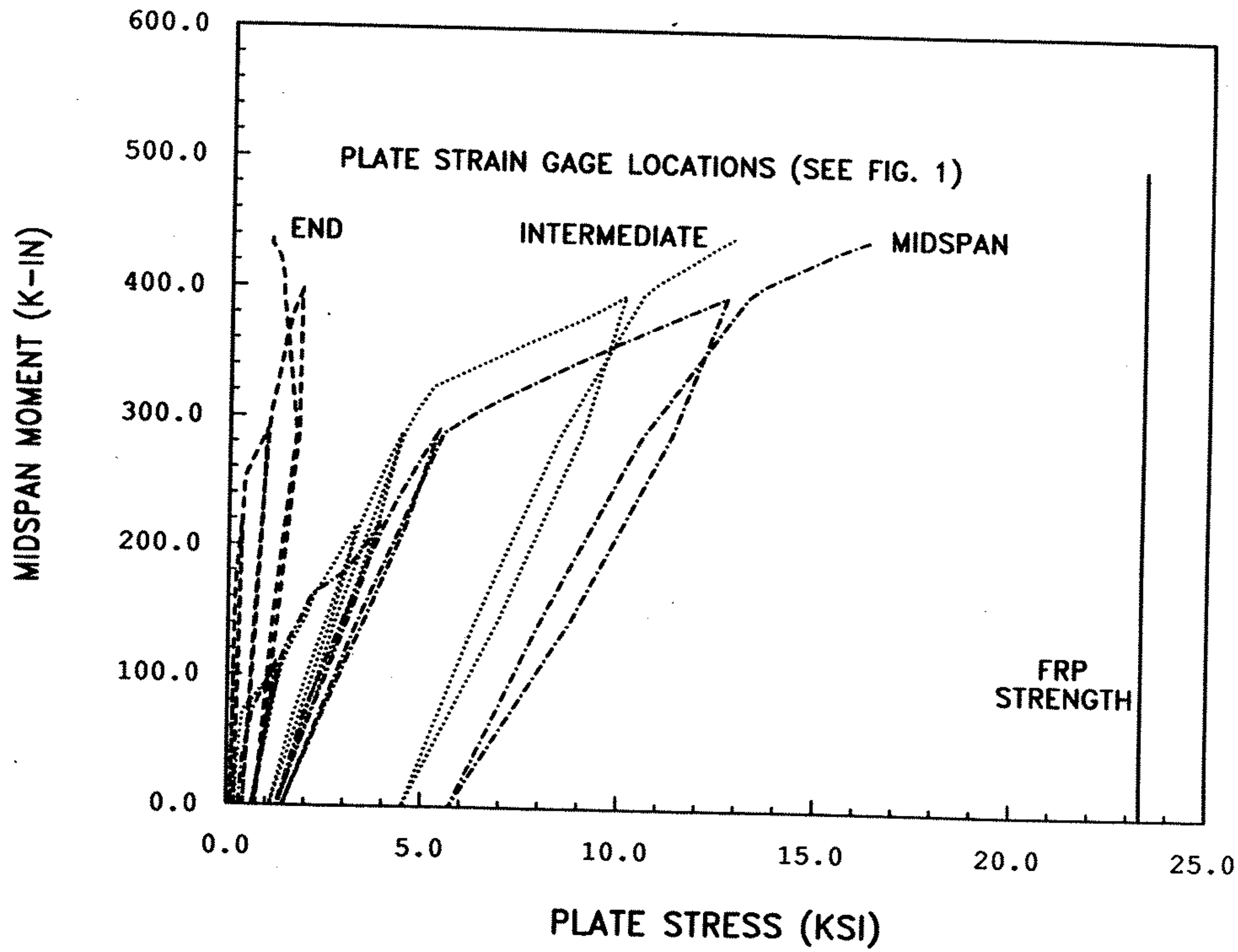

Figure 8 - Midspan moment vs plate stress diagram for beam C, with four load cycles. Ultimate strength of fiberglass-reinforced plastic plate is 23.3 ksi.

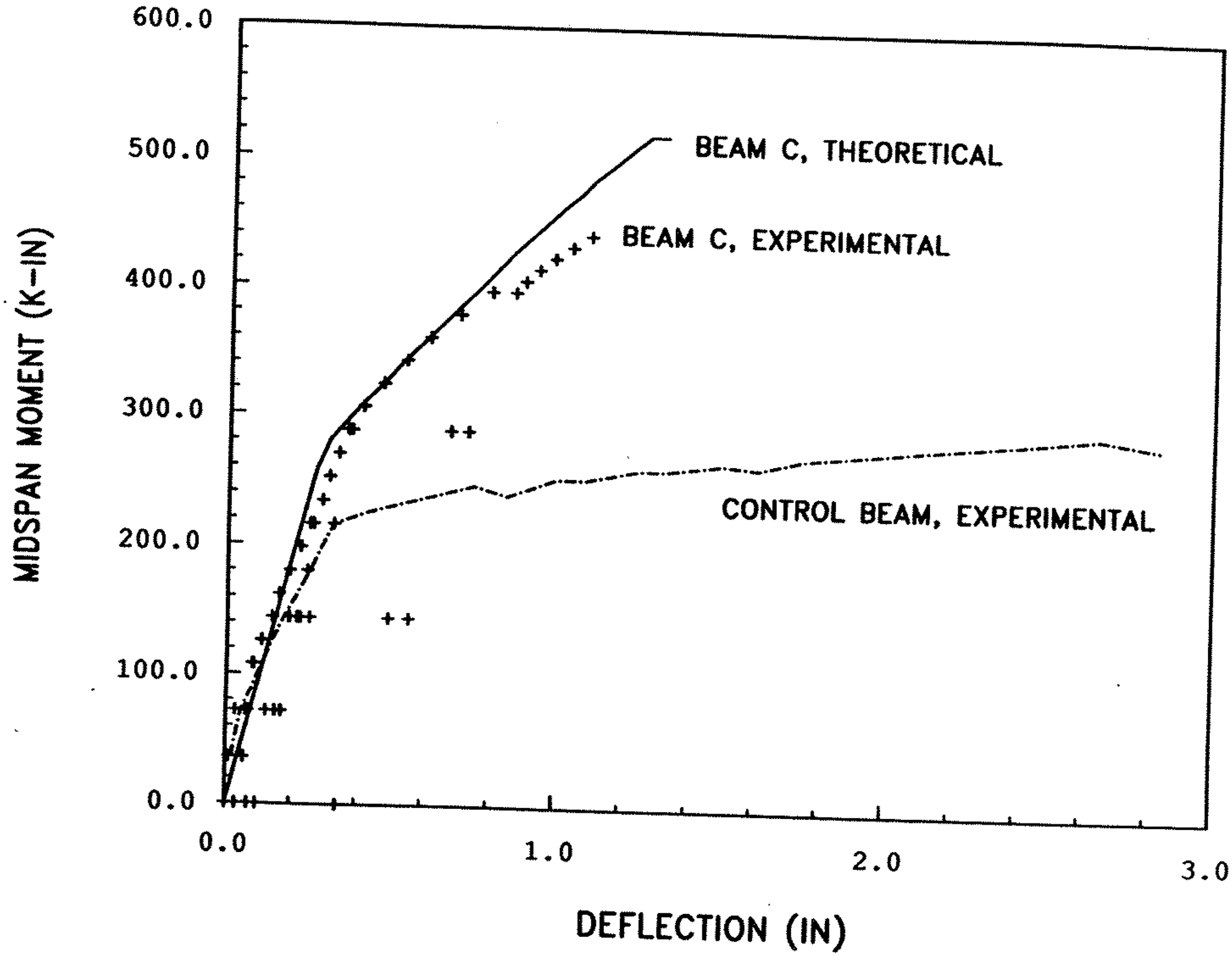

Figure 9 - Midspan moment vs deflection diagram for beam C and control beam