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POLYMER CONCRETE-REINFORCED CONCRETE COMPOSITE
BEAMS

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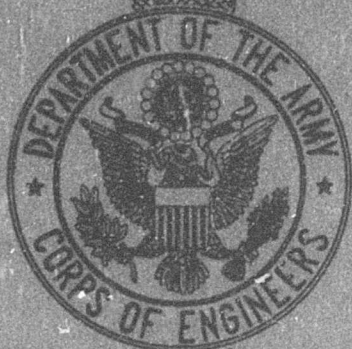
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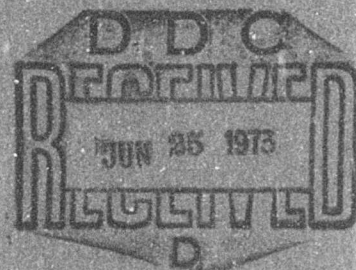


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POLYMER CONCRETE-REINFORCED CONCRETE COMPOSITE BEAMS

by
James Lott
Dan Naus
and
Paul Howdysell

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ABSTRACT

Composite beams consisting of reinforced concrete and a layer or cap of polyester-concrete in the region of high compressive stress are tested and evaluated. The creep characteristics of polyester concrete were evaluated using 1000 hour creep tests, and the effect of sustained load on the ultimate compressive strength was also determined. Composite beams which were 6 by 6.5 by 64 in. (15.2 by 16.5 by 162.6 cm) were fabricated by capping precast reinforced concrete beams with a layer of fibrous polyester concrete of various given thicknesses. The composite beams were subjected to third-point loads on a simply supported length of 57 inches (145 cm). Load-deflection behavior and ultimate strength were determined for various combinations of reinforcement and depth of fibrous polyester concrete cap. Experimental and analytical results indicate that the fibrous polyester concrete composite beams are performance and material cost effective relative to reinforced concrete beams with the same percentage of tensile reinforcement.

FOREWORD

This paper was presented by Dr. Dan Naus at the American Concrete Institute annual meeting held in Atlantic City, New Jersey, 5 - 9 March 1973. It was accepted by the American Concrete Institute for publication as part of a symposium on polymers in concrete prepared by American Concrete Institute Technical Activity Committee 548, Polymers in Concrete.

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INTRODUCTION

A composite beam consisting of reinforced concrete and a layer or cap of polyester concrete in regions of large compressive stress is an example of optimum material usage to obtain cost effectiveness and improved performance.

Polyester concretes, which consist of a polyester resin matrix and fillers such as sand, have been formulated to meet specific requirements such as the development of 20,000 psi (1410 kgf/cm²) unconfined compressive strength with a short curing period (1). Such polymer concretes tend to be brittle; however, the dispersion of fiber reinforcement would improve the toughness and tensile strength of the material. The use of fibrous polyester concrete (FPC) in the compressive region of reinforced concrete beams provides a high strength, ductile concrete at reasonable cost. The resulting composite beam utilizes steel in the region of high tensile stress, fibrous polyester concrete (FPC) with its favorable compressive behavior in the regions of high compressive stress, and portland cement concrete (PCC) in regions of relatively low flexural stress.

Polymer concretes are viscoelastic in nature (2) and are more efficient as structural materials for short-term loadings. In the composite beam the polymer concrete cap would be effective during short-term loadings even though it might relax during long-term loadings. Thus the composite beam would have a high, shortterm capacity and a lower, long-term capacity.

An experimental and analytical investigation has been conducted of the feasibility of the fibrous polyester concrete, reinforced concrete

beam. In the experimental investigation, 1000 hour compressive creep data was developed for five polyester concrete cylinders, and ten composite beams were tested to determine the effect of the depth of the FPC cap and the amount of steel reinforcement on the load-deflection behavior. In the analytical investigation, the stress-strain properties of the constitutive materials were used to determine theoretical moment-curvature relationships and ultimate moment capacities of composite beams.

EXPERIMENTAL INVESTIGATION

Creep Behavior

The creep characteristics of a polyester-concrete were determined as a part of a general evaluation of potential applications of particle-filled polyesters (1,3).

The polyester concrete consisted of a low viscosity, promoted polyester resin matrix (13% by weight) with MEK peroxide catalyst and a gap-graded quartzite aggregate (87% by weight). The polyester concrete was hand mixed until the aggregates were uniformly dispersed and all aggregates were surface wetted. The concrete was placed in 3 by 18 in. (7.6 by 45.7 cm) cylindrical steel molds in nine equal lifts with rodding after each lift. Specimens were cured for seven days in a controlled temperature ($73 \pm 4^\circ\text{F}$) and controlled relative humidity ($45 \pm 5\%$) environment. All specimens were capped with a high strength concrete capping compound of 9,000 to 10,000 psi (630 to 700 kgf/cm^2) compressive strength.

Each creep specimen was instrumented with three 1.95 in. (4.95 cm) electrical strain gages at mid-height and at 120° intervals. Mechanical

gage lengths of 10 in. (25.4 cm) were also centered at mid-height and at 120° intervals.

Loads were applied to the 3 by 18 in. (7.6 by 45.7 cm) specimens in conventional creep racks. The loads were applied in 5,000 or 10,000 lb. (2270 to 4540 kgf) steps to permit reading and recording of specimen strains after each step. No effort was made to maintain a set loading rate due to variable time involved in the readings and recordings, but approximately 10-15 min. was required to load each specimen.

Load was initially applied by hydraulic jacks through three parallel reaction springs in series with the test specimens. After approximately the first 100 hrs of loading, the jacks were removed concurrently with tightening of the rod nuts above the specimen. The nuts were tightened until the strain in the steel rods after the jacks had been removed was approximately equal to the strains before removal, thus maintaining the desired load on the test specimens. Rod strains were checked and rod nuts adjusted as required throughout the remainder of the 1000 hr load period. After completion of the load cycle, loads were removed and the specimens allowed to recover.

The strain monitoring system for the test consisted of multiple point switching units and SR-4 strain gage boxes for measuring the electrical resistance gages on the test specimens and the strain gages on the steel rods of the creep racks. A Whitmore mechanical strain gage was used to measure the relative displacement on the 10 in. (25.4 cm) gage length. The primary load monitoring devices for the initial 100 hrs of loading were pressure gages on the hydraulic jacking systems. After the hydraulic jacks were removed, strain gages on the steel rods were used to monitor

load variations. Strains registered by both electrical and mechanical gages were recorded and creep loads checked approximately every 15 min. for the first hour of loading, every hour for the next 10 hrs, every 24 hrs for the succeeding 200 hrs, and finally every 200 hrs for the remainder of the 1000 hr test. Strain recovery was monitored for the first 100 hrs after the creep loads were removed. Strains were recorded immediately before and after unloading, every 15 min. for the first hour after unloading, every hour for the next 5 hrs, and every 24 hrs for the remainder of the 100 hr period.

Load levels of 20,000, 40,000 and 60,000 lbs (9070, 18000, and 27200 kgf) were used which corresponded to stress levels of 2830 (200), 5660 (400), and 8490 (600) psi (kgf/cm^2), respectively. Two specimens were tested at each of the 20,000 and 40,000 lb (9070 and 18000 kgf) load levels, and one specimen at the 60,000 lb (27200 kgf) level. The load levels were selected based on the results of preliminary tests to assure that some specimens would creep to failure and that others would not.

For each of the five creep test specimens, five companion specimens (3 by 6 in.) (7.6 by 15.2 cm) were prepared from the same batch to determine the ultimate compressive strength of the batch. All companion specimens, after the seven-day cure, were broken on a Universal testing machine at a loading rate of 25,000 lbs/min. (11300 kgf/min.). After the first 100 hrs of recovery, all 3 by 18 in. (7.6 by 45.7 cm) creep test specimens that did not fracture during the 1000 hr loading period were sawn into 3 by 6 in. (7.6 by 15.2 cm) specimens and reloaded to failure in a Universal testing machine at a loading rate of 25,000 lbs/min. (11300 kgf/min.).

All creep tests were performed in a controlled laboratory environment of $73 \pm 4^\circ\text{F}$ and $45 \pm 5\%$ relative humidity (RH).

The results are presented in Table 1 and Figure 1. Table 1 contains a tabulation of the following test data: creep stress levels, batch and specimen strengths, Young's modulus, 1000 hr creep strains, specific creep strains (creep strain divided by creep stress), elastic recovery, and creep recovery. Figure 1 illustrates the specific creep strain-time curves. The curves were developed using least squares and regression analysis; only the electrical resistance strain gage data was used in the regression analysis.

Beam Behavior

The loaddeflection behavior of ten composite beams was evaluated by flexural tests. The parameters that were investigated were the depth of a FPC compressive layer, the percentage of tensile reinforcement, and the percentage of compressive reinforcement.

A 6000 psi (420 kgf/cm^2) nominal strength portland cement concrete was used for all ten beams. The PCC mix was one part cement to three parts sand to three parts crushed stone (1:3:3) with a water-cement ratio of 0.60. The materials were a Type III portland cement, a river sand, and a 3/4 in. (1.9 cm) max. size crushed limestone.

The fibrous polyester concrete (FPC) consisted of a low viscosity, promoted polyester resin matrix (14.4% by weight) with MEK peroxide catalyst, a gap graded, high strength quartzite aggregate (82.1% by weight), and a high strength 1 in. (2.54 cm) sheared steel fiber of 0.010 in. by 0.022 in. (.025 by .056 cm) cross section (3.5% by weight).

The tensile and compressive reinforcement was No. 4, Grade 60 reinforcing bars. A typical stress-strain curve from coupon tests is presented in Figure 2. Stirrups were fabricated of No. 2 smooth bars of approximately 40,000 psi (2810 kgf/cm^2) yield strength.

The nominal dimensions of the beam specimens were 6.0 in. wide by 6.5 in. deep by 64.0 in. long (15.2 by 16.5 by 162.6 cm), as shown in Figure 3. Five beams contained 1.0 percent tensile reinforcement, and five beams contained 1.5 percent tensile reinforcement. Three of the beams in each group had FPC compressive caps of 0.5 in. (1.3 cm), 0.75 in. (1.9 cm) or 1.0 in. (2.5 cm), and the remaining two beams in each group were companion specimens with no cap and contained either no compressive reinforcement or an amount of compressive reinforcement equal to the tensile reinforcement. The tensile reinforcement was placed so that the effective beam depth was 5.25 in. (13.3 cm). The centroid of compressive reinforcement was 1.25 in. (3.2 cm) from the compressive surface of the beam. Shear reinforcement was spaced at 2.75 in. (7 cm) along the length of the beam.

Each beam specimen is designated by a series of letters and numbers, which describes the beam. The first segment is a number and indicates the depth of the FPC compressive cap. The second segment contains the letter T and a number to indicate the percentage of tensile reinforcement. The third segment contains the letter C and a number to indicate the percentage of compressive reinforcement.

Companion compressive cylinders 3 in. by 6 in. (7.6 by 15.2 cm) were cast for each PCC and FPC batch.

The portland cement concrete (PCC) was mixed in a one-half yard turbine mixer. The PCC portions of the beams were cast in steel molds, which were filled in three equal lifts. Exposed surfaces of beams to receive a FPC cap were left rough to improve bonding. The PCC portions of the beam were moist cured in a 100 percent relative humidity for seven days and then were stored in the laboratory environment for 14 days.

When the PCC was 21 days of age, the FPC caps were fabricated. A 1.5 cubic foot (.04 cubic meter) batch of FPC was mixed in a 3 cubic foot (.08 cubic meter) drum mixer. The mixing procedure included: coating the inside of the mixer with polyester resin; adding the batch weight of polyester resin to mixer; adding an amount of catalyst equal to one percent of the resin weight and mixing resin and catalyst thoroughly; adding the aggregate and steel fibers which had previously been mixed together, to the mixer; and, mixing constituents for at least two minutes.

The FPC caps were cast in wooden molds attached to the PCC beams. A primer coat consisting of 50 percent polyester resin with catalyst and 50 percent fine sand by weight was applied to the rough, top surface of the PCC beam. The FPC was then placed in the mold and compacted with an external surface vibrator.

The FPC caps were cured in the laboratory environment for a minimum of 7 days. Thus the minimum age at time of testing was 28 days for PCC and 7 days for FPC.

A closed loop hydraulic testing machine was used for all flexure tests of the beam structural elements. The test setup is shown in Figure 4. The load was transmitted from the testing machine to the beam at

its one-third points by means of a reinforced I-beam which contained two reaction points.

The centerline deflection of the beam elements was monitored using two methods. The first method used three electro-optical auto collimators to measure differential deflections between the beam centerline and the extreme reaction points of the beam. (Changes in differential deflections were obtainable with an accuracy of ± 0.02 percent of the 6 in. (15.2 cm) full scale reading.) The second method used a linear potentiometer which was calibrated to relate resistance to centerline deflection. (Deflection changes were obtainable within an accuracy of ± 0.01 in. (.03 cm).)

The response of the two methods used to obtain centerline deflections was continuously recorded vs. load until either the composite beam failed, or the 6 in. (15.2 cm) travel of the loading machine was exceeded.

The procedure followed for testing the beam structural elements included: placing and centering the beam element in the loading frame; positioning the loading and reaction points so that the major span was 57 in. (145 cm) and the minor span 19 in. (48.3 cm), applying a pre-load of approximately 50 lb (22.7 kgf) to hold the beam in position; attaching the targets for the electro-optical auto collimator and the connection for the linear potentiometer; zeroing all deflection and load responses; and, loading the specimen to failure, or maximum deflection and system would permit, at a rate of 1000 lb/min. (454 kgf/ min.). The deflection and load responses were monitored continuously throughout the test. This procedure was repeated for all beams.

After completion of the flexure tests the companion compression cylinders were tested. The specimens were loaded to failure at a rate of approximately 10000 lb/min. (4540 kgf/min.) in a hydraulic testing machine. Load vs. deformation behavior was obtained for a few selected compression specimens. Ultimate loads were recorded for all compression specimens tested.

The mean compressive strength was 6300 psi (440 kgf/cm²) for the PCC and 11900 psi (840 kgf/cm²) for the FPC. A representative stress-strain curve for a FPC is given in Figure 5.

Beam test data are presented in Table 2, which contains maximum loads, centerline deflection at failure, and the energy requirement to failure as approximated by the area under the load-centerline deflection diagram.

The behavior of the composite beams is illustrated by the load vs. centerline deflection diagrams of Figures 6 and 7 for 1 percent and 1.5 percent tensile steel, respectively. The diagram can be divided into three regions: the initial region where a linear relationship exists between applied load and centerline deflection (cracks develop at the tension region of the beam); second region which corresponds to steel yielding and then strain hardening (the initial hairline crack widths greatly increase and the flaws propagate toward the neutral axis to produce large increases in deflection for small increments of applied load); and, the third region in which the beam fails as a result of crushing of the concrete, the concrete crushing and forcing the FPC cap to separate from the beam, or the tensile steel reinforcement fractures. The degree of development of the first and second regions is dependent on the amount of steel reinforcement and the depth of the FPC cap.

ANALYTICAL INVESTIGATION

Theoretical moment-curvature relationships and ultimate moment capacities were determined using the following procedures: strains were assumed to vary linearly across a section; the strains at the extreme compression fibers were varied from 0.001 to 0.010 in increments of 0.001; at each strain level the depth of neutral axis was determined by conducting a trial and error investigation using the stress-strain diagrams of Figure 2 for steel reinforcement and Figure 5 for FPC (a typical stress-strain diagram was assumed for PCC with crushing assumed at a strain of 0.004) and which was considered to converge when equilibrium of tensile and compressive stresses was satisfied within ± 0.5 lb; the corresponding moment and curvature were evaluated; and, procedure was repeated until the limiting strain was exceeded in either the steel reinforcement (.1), FPC (.01), or PCC (.004) materials.

Material costs of the various composite beams that were analyzed were estimated in terms of beam cost per linear foot. The cost of materials used in the analysis were \$20 per cubic yard for PCC, \$0.135 per lb of steel rebar, and \$134 per cubic yard for FPC, which reflects the actual material costs of the FPC mixes used in the experimental investigation.

The material cost, relative material cost, and the moment-curvature diagram were determined for six of the test beams. Beam geometry and costs are given in Figure 8. The moment-curvature diagrams are presented in Figure 9. (Moment-curvature was evaluated assuming the steel strain hardened.) There are two sets of numbers adjacent to each curve in Figure

9. The first number in the upper set of numbers for each beam indicates the predicted moment capacity for the beam cross-section and the second number in the upper set of numbers indicates the value of the curvature at the ultimate moment. The lower numbers in each set of numbers again represent moment and curvature values but the values are normalized so that the material cost for each element is equivalent; i.e., the moment and curvature values for each element are divided by the cost of the corresponding element relative to the conventional reinforced concrete beam. Figure 9 shows that, for the same tensile steel content and beam cross-section, an increased moment capacity of up to 80% may be obtained. Also, when compared on an equal material cost basis, the composite beam has an increased moment capacity over the conventional reinforced concrete beam. Translating this to a material cost for equal performance, the composite beam can provide equal or greater performance than the reinforced concrete beam at lower material cost.

Ultimate moment capacities were determined for a standard 12 in. (30.5 cm) wide beam as shown in Figure 10. Parameters investigated included: total depth, D , of 5 in. (12.7 cm), 10 in. (25.4 cm), 15 in. (50.8 cm), 25 in. (63.5 cm), and 30 in. (76.2 cm); tensile steel percentages of 1.0%, 1.5%, and 2.0%; and FPC compressive cap depth, H , of 1.0 in. (2.5 cm), 2.0 in. (5.1 cm), and 3.0 in. (7.6 cm).

The ultimate moment capacities of the 12 in. (30.5 cm) wide beams with 1.5% tensile reinforcement are presented in Figure 11 for various beam depths and FPC compressive cap depths. Data of Figure 11 are representative of the trends of data for other steel percentages.

A cost-performance analysis was conducted using the above capacities for the standard 12 in. (30.5 cm) width, which were scaled for beams of different widths. Limitations on element widths were established by: (1) the minimum width permissible to meet steel reinforcement cover requirements; or, (2) the maximum permissible width was arbitrarily selected as ten times the total beam depth (slab elements).

The cost-performance data for beams with 1.5% tensile reinforcement are again presented as representative. Material cost in units of \$ per linear foot for various ultimate moment capacities are given in Table 3. Moment capacities vs. material cost per linear foot of beam are presented in Figure 12.

DISCUSSION

Creep Behavior

The polyester concrete specimens with sustained stress to strength ratios of 0.478 and 0.749 failed during the loading period, while the specimens with stress to strength ratio of 0.246 had no failure tendencies during the 1000 hr loading period. The specimens which failed during the load period experienced an increased strain rate before failure. The specific creep strain rates for the specimens that did not fail were initially high, then diminished with time to a limiting rate. Elastic recovery strains and elastic strains were approximately equal, and creep recovery strains were about one half of the creep strains. The ultimate compressive strengths of the polyester-concrete specimens for the stress-strength ratio of 0.246 was not affected by the sustained loads.

Beam Behavior

Beam test data indicate that maximum load, maximum deflection, and energy capacity of reinforced concrete with only tensile reinforcement may be greatly increased by replacing part of the PCC in the compression region with FPC or by adding compression reinforcement. Figure 6 indicates that composite beams with 1.0 percent tensile reinforcement and FPC compressive caps of depths of 0.5 in. (1.3 cm), 0.75 in. (1.9 cm), and 1.0 in. (2.5 cm) had increased load capacities of 28%, 29%, and 35% and increased centerline deflections of 169%, 362%, and 231% respectively, relative to the reinforced concrete beam with no compressive strengthening. The addition of 1.0 percent of compression steel resulted in a 19% increase of load capacity and a 300% increase in centerline deflection. Figure 7 indicates that composite beams with 1.5 percent of tensile reinforcement and FPC caps of depths of 0.50 in. (1.3 cm), 0.75 in. (1.9 cm), and 1.0 in. (2.5 cm) have increased load capacities of 22%, 17%, and 30% and increased centerline deflections of 188%, 238%, and 338% respectively, relative to the reinforced concrete beam. The addition of 1.5 percent compressive reinforcement increased the load capacity by 21% and the centerline deflection by 550%.

Cost-Performance Analysis

The ultimate moment capacity data of Figure 11 indicate the effect of replacing a part of the compressive region of a reinforced concrete beam (1.5% tensile steel) with a FPC cap. The moment capacity was increased up to a maximum of 71% with a 3 in. (7.6 cm) FPC cap for a 30 in. (76 cm) deep composite beam. There was a minimum depth of composite beam for which the FPC caps are not effective for increased moment capacity.

In Figure 11, the FPC cap increased moment capacity for composite depths greater than 5.0 in. (12.7 cm). A 1.0 in. (2.5 cm) depth of FPC was as effective as a 2.0 in. (5.1 cm) depth of FPC for composite beam depth of less than 10.0 in. (25.4 cm), and a 2.0 in. (5.1 cm) cap was as effective as a 3.0 in. (7.6 cm) cap for composite beam depths of less than 15.0 in. (38.1 cm).

Cost-performance data for reinforced concrete beams and composite beams with 1.5 percent tensile reinforcement are presented in Table 3 and Figure 12. Table 3 presents costs per foot length for elements having total depths from 5 in. (12.7 cm) to 40 in. (102 cm) and have 0 in. to 3 in. (7.6 cm) of the concrete at the compression surface replaced with fibrous polyester concrete. Figure 12 presents element cost per foot length vs. moment capacity for total beam depths from 5 in. (12.7 cm) to 30 in. (76.2 cm) and depths of FPC from 0 in. to 3 in. (7.6 cm). The data indicate that composite beams with FPC compressive caps are material-cost effective compared to reinforced concrete beams of equal moment capacity and equal tensile reinforcement. Actual cost comparisons are possible only after design parameters such as beam depth have been evaluated. Figure 12 indicates the cost of various structural elements of a given capacity. An ultimate moment capacity of 5,000 kip in. (57600 m-kgf) is obtained at a cost of \$2.10 per linear foot (\$6.89 per linear meter) of beam for a 40 in. (102 cm) deep reinforced concrete beam and for a 30 in. (76.2 cm) deep composite beam with a 3 in. (7.6 cm) cap of FPC, and at a cost of \$4.90 per linear foot (\$16.07 per linear meter) of 15 in. deep (38.1 cm) deep composite beam with a 1 in. cap of FPC,

and at a cost of \$6.80 per linear foot (\$22.30 per linear meter) of 15 in. (38.1 cm) deep reinforced concrete beam. For any given depth the FPC composite beams are more economical than the reinforced concrete beams.

The increased capacities of the FPC composite beams result from the ductile behavior of the compressive caps which are compatible with the large work-hardening strains of the tensile steel.

CONCLUSIONS

Polyester concretes are viscoelastic in nature and will fail under a sustained compressive loading at stress levels greater than 50 percent of the ultimate strength. Sustained loadings at a stress level of 25 percent of ultimate strength does not reduce ultimate strength capacity for a load-period of 1000 hours. Polyester concretes should be considered for structures with a high ratio of live load to dead load and for composite structures in which the polyester-concrete may relax during long-term loadings.

The experimental and analytical investigations indicate that the FPC composite beams are performance effective relative to reinforced concrete beams of equal steel reinforcement percentages. The FPC composite beams may be used to obtain smaller and lighter weight precast elements of a given moment capacity.

The use of a FPC cap increases the ultimate moment capacity and improves beam ductility in a manner similar to compressive reinforcement. The FPC composite beams may be used to obtain improved ductility and energy adsorption capacity. The FPC composite beams are material-cost

effective. Actual costs are a function of limitations placed on design parameters such as beam depth and width. The cost-performance analysis should be extended to consider compressive reinforcement for comparison reinforced concrete beams and should be modified to incorporate total costs for given applications.

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Table 1: Summary of Creep Data*

Creep Load lbs	Creep Stress psi	Batch Strength psi	Specimen Strength psi	Young's Modulus 10^{-6} psi	Creep Strain 1000 hr in/in	Specific Creep	Elastic Recovery in/in	Creep Recovery in/in	Residual Strain in/in
20,000	2829	11,672	11,570	3.48	998	0.353	820	557	443
20,000	2829	11,672	11,530	3.37	1460	0.516	1181	641	477
40,000	5659	11,842	-	3.30	-	-	-	-	-
40,000	5659	11,842	-	3.30	-	-	-	-	-
60,000	8490	11,304	-	3.66	-	-	-	-	-

* lb x .4536 = kgf

psi x .07031 = kgf/cm²

Table 2: Summary of Beam Test Data

Beam Designation	Maximum Load Kips (kgf)	Centerline Deflection inches (cm)	Energy Requirements for Failure* in-kips (m-kgf)
0.00-T1-CO	16.19 (7344)	1.3 (3.3)	14.3 (165)
0.50-T1-CO	20.80 (9434)	3.5 (8.9)	53.7 (619)
0.75-T1-CO	20.88 (9471)	6.0 (15.2)	101.3 (1167)
1.00-T1-CO	21.81 (9893)	4.3 (10.9)	74.9 (863)
0.00-T1-C1	19.27 (8741)	5.2 (13.2)	89.2 (1030)
0.00-T1.5-CO	23.00 (10430)	0.8 (2.0)	14.3 (165)
0.50-T1.5-CO	27.98 (12690)	2.3 (5.8)	52.5 (605)
0.75-T1.5-CO	26.99 (12240)	2.7 (6.9)	59.0 (680)
1.00-T1.5-CO	29.80 (13520)	3.5 (8.9)	85.5 (985)
0.00-T1.5-C1.5	27.81 (12610)	5.2 (13.2)	130.5 (1500)

* Area under load-centerline deflection diagrams from 0 load to maximum applied load.

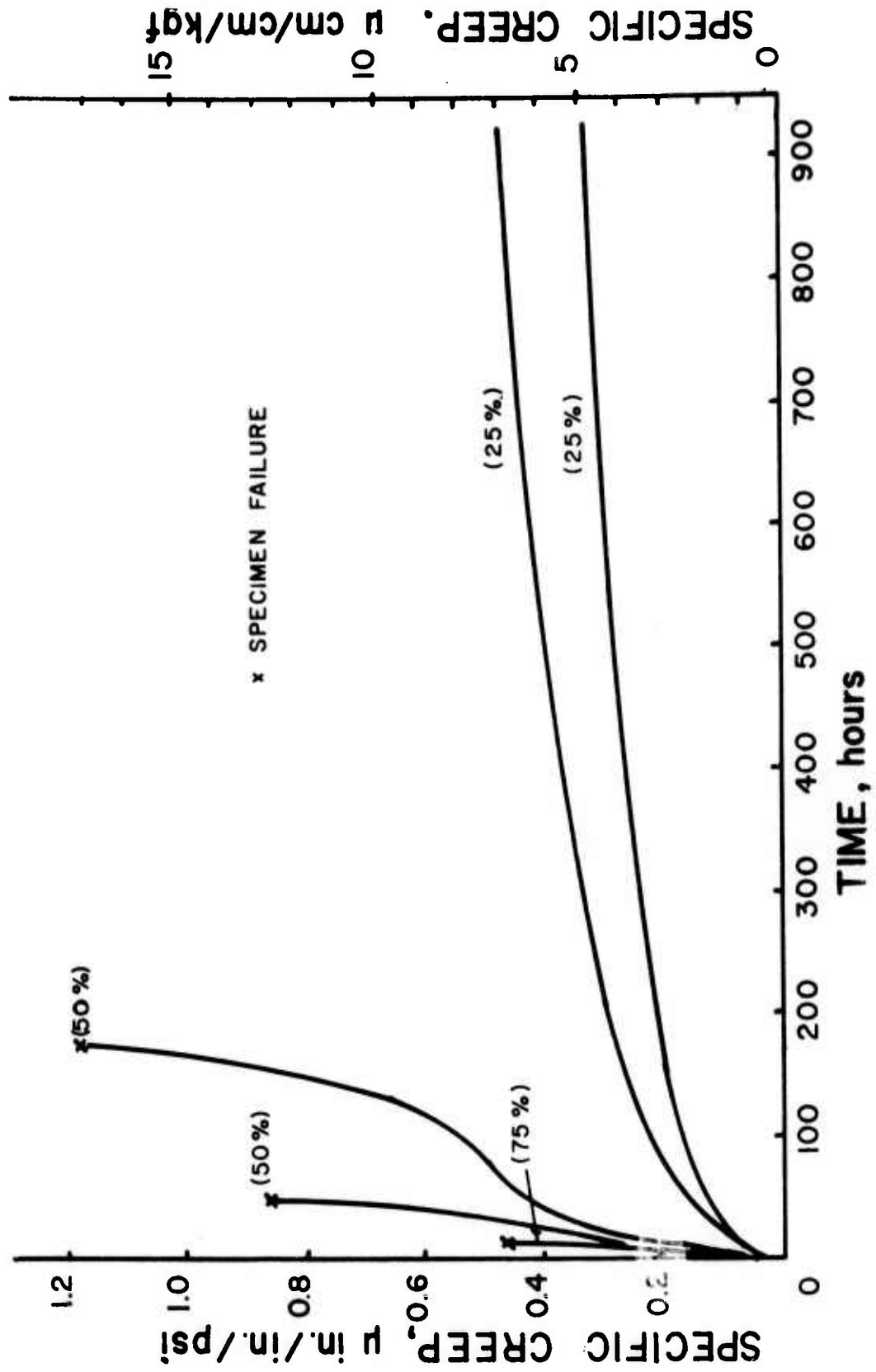


Figure 1. Specific creep vs time.

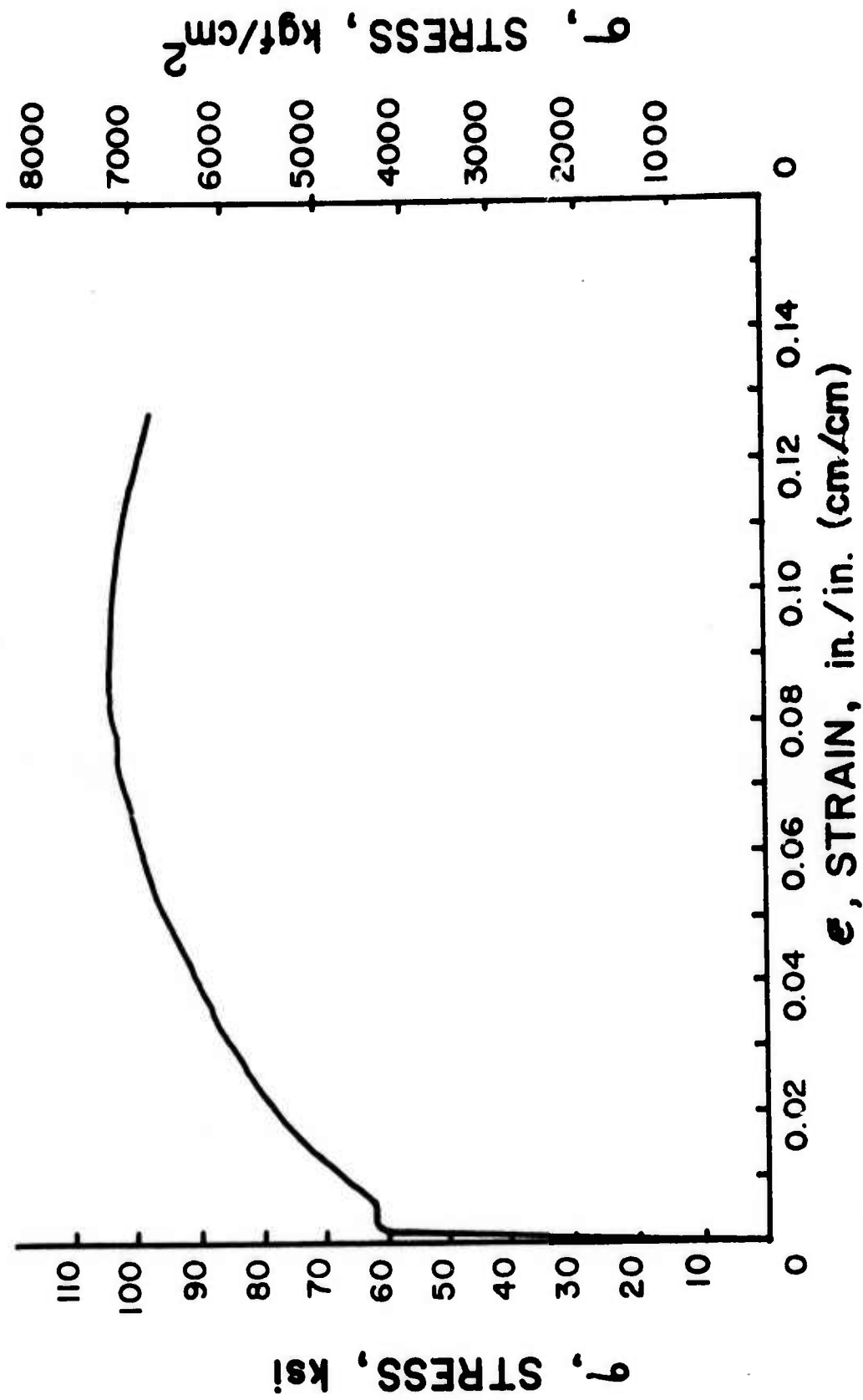


Figure 2. Steel tensile $\sigma - \epsilon$ curve.

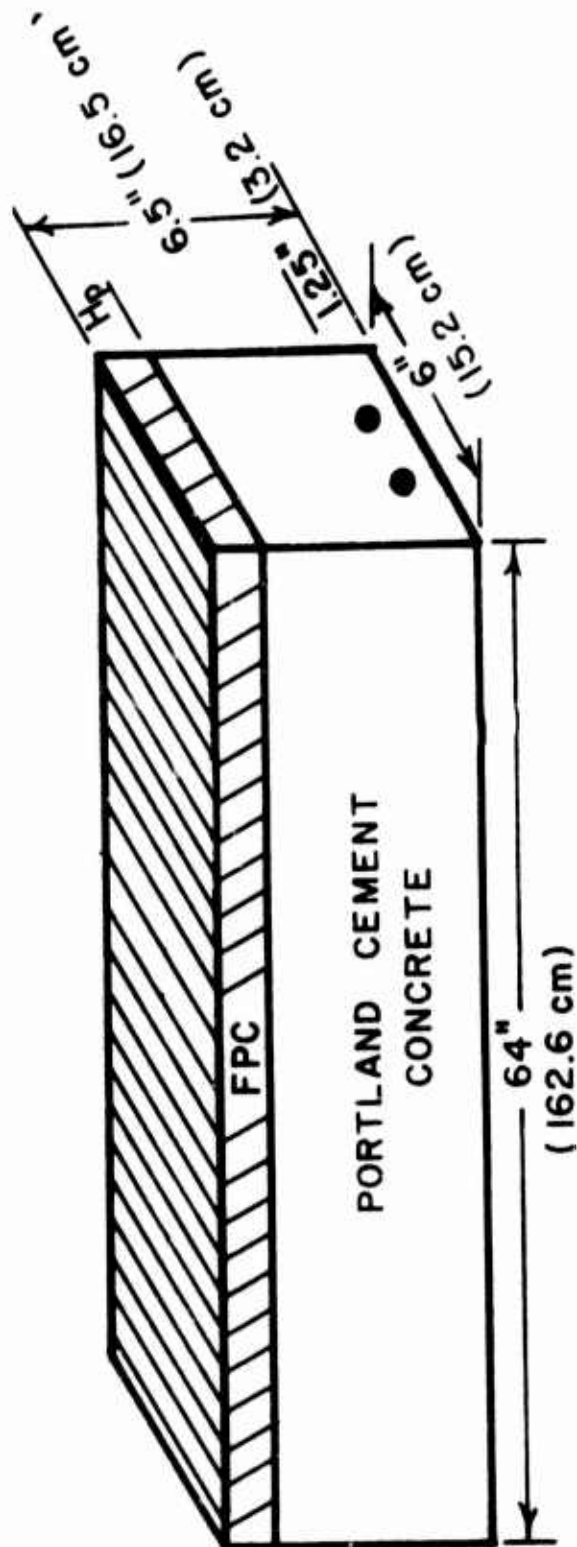


Figure 3. Beam dimensions.

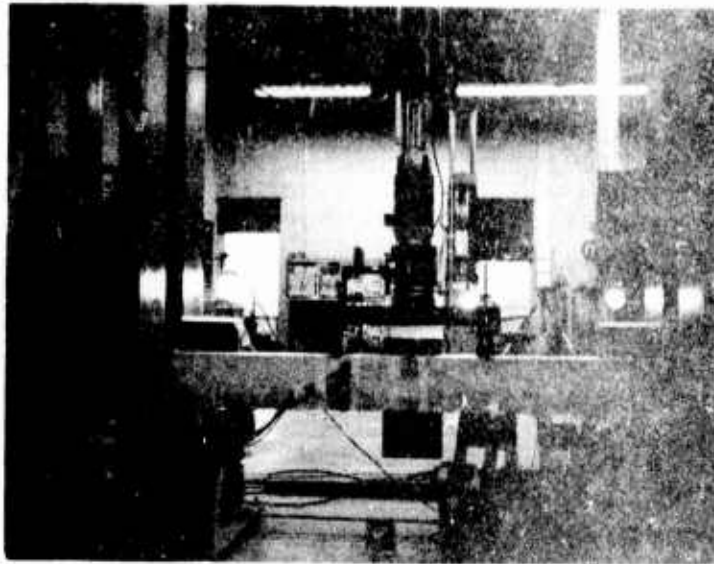


Figure 4. Test setup.

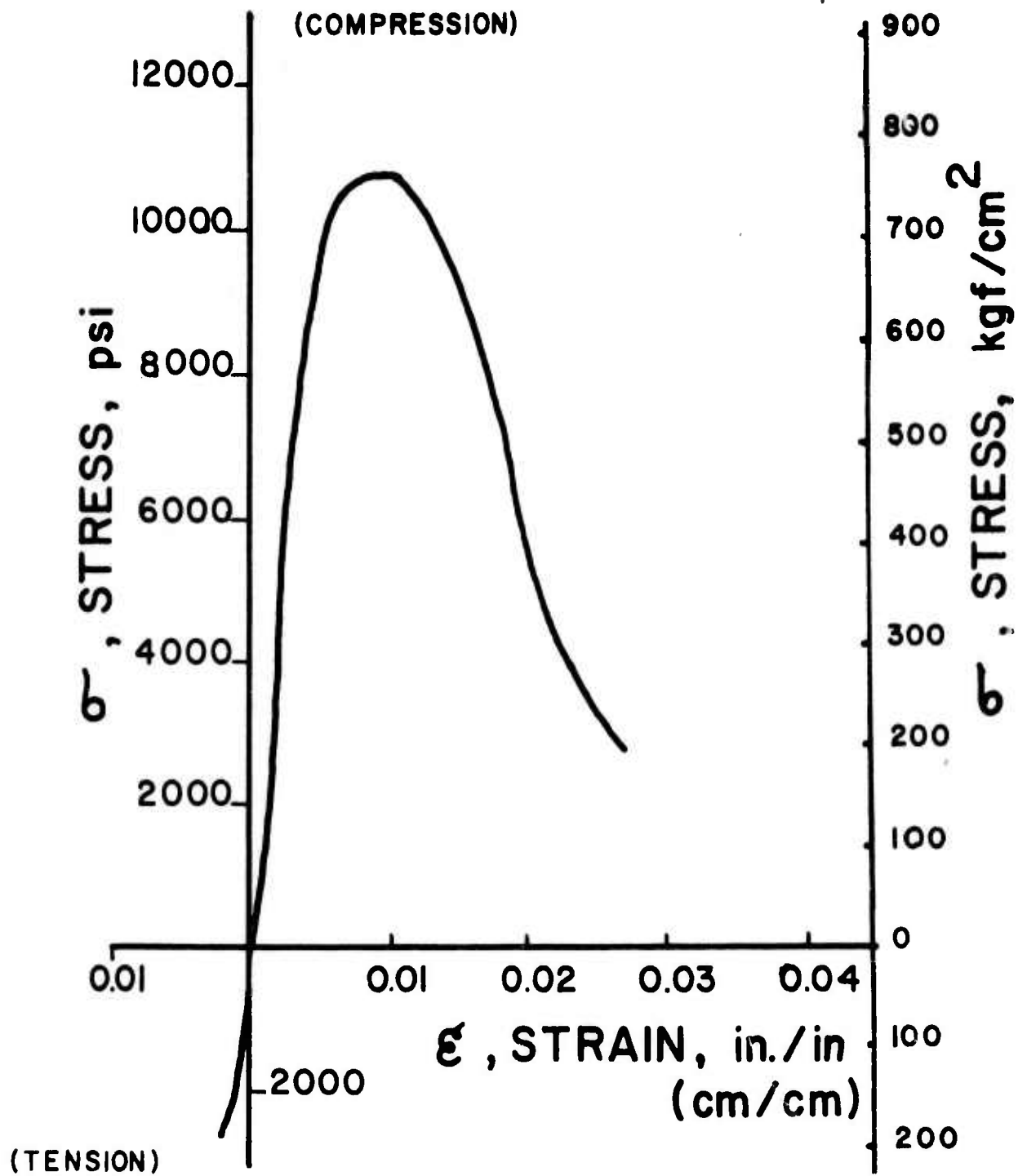


Figure 5. FPC stress-strain.

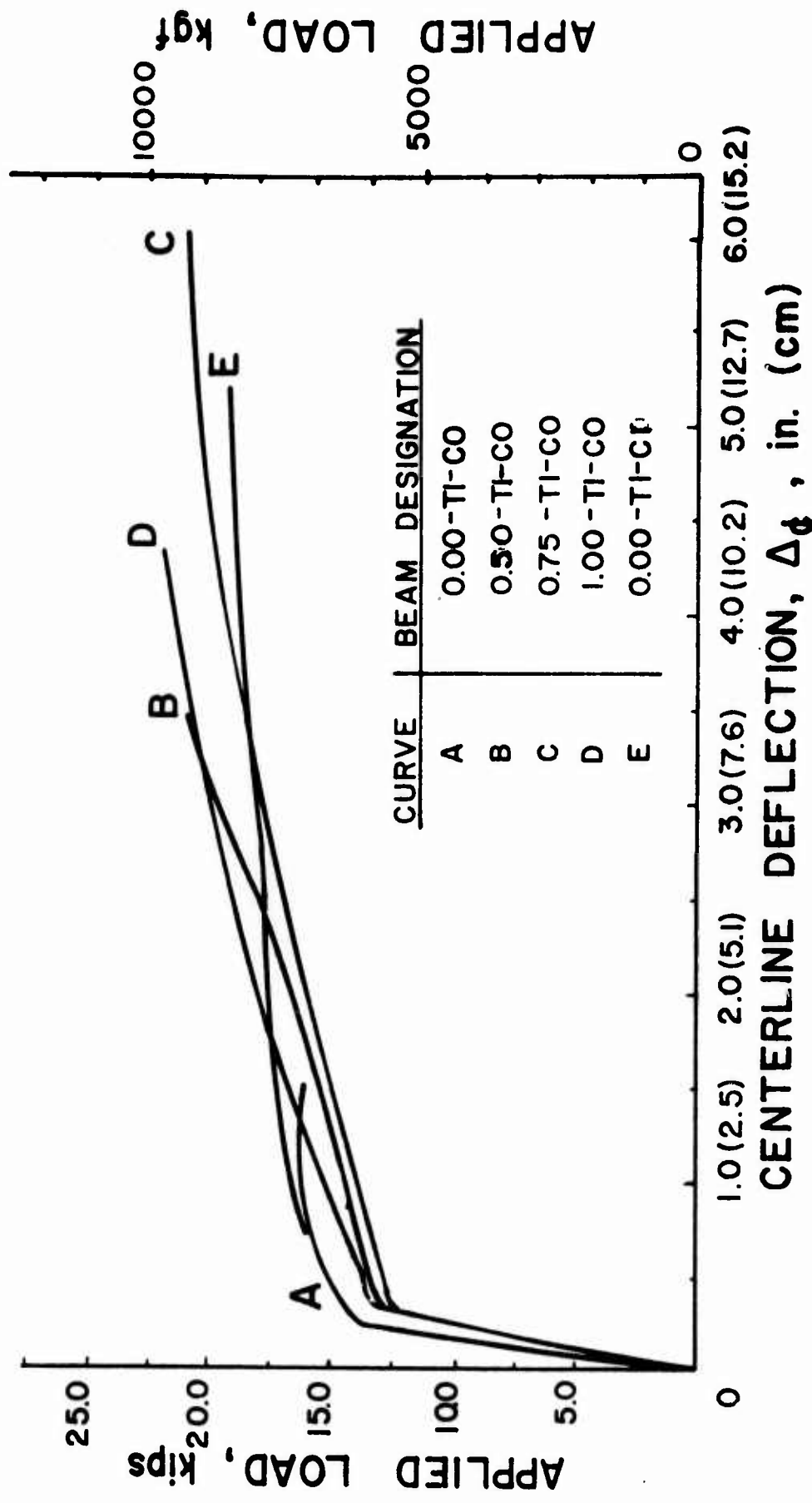


Figure 6. Load vs deflection.

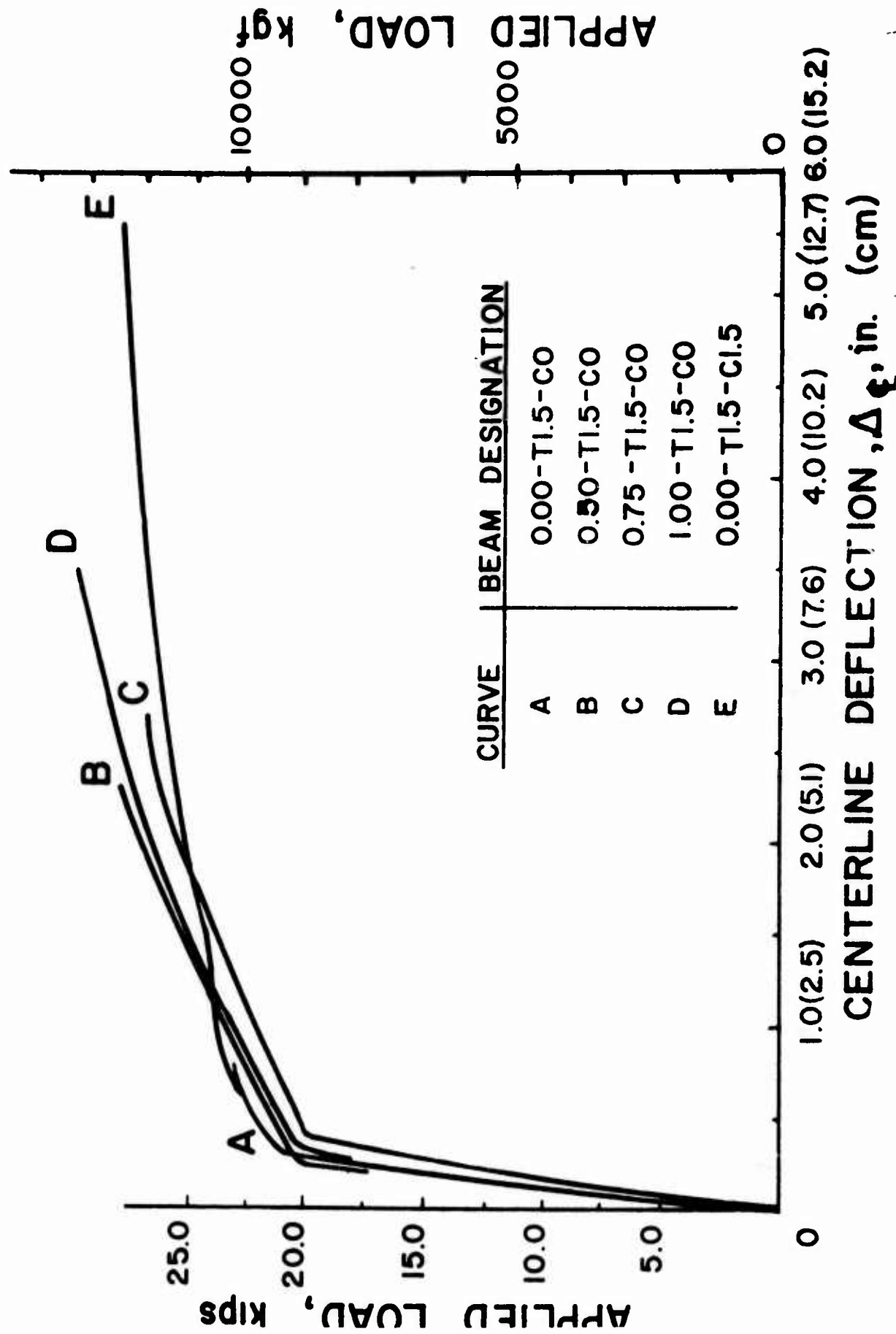


Figure 7. Load vs deflection.

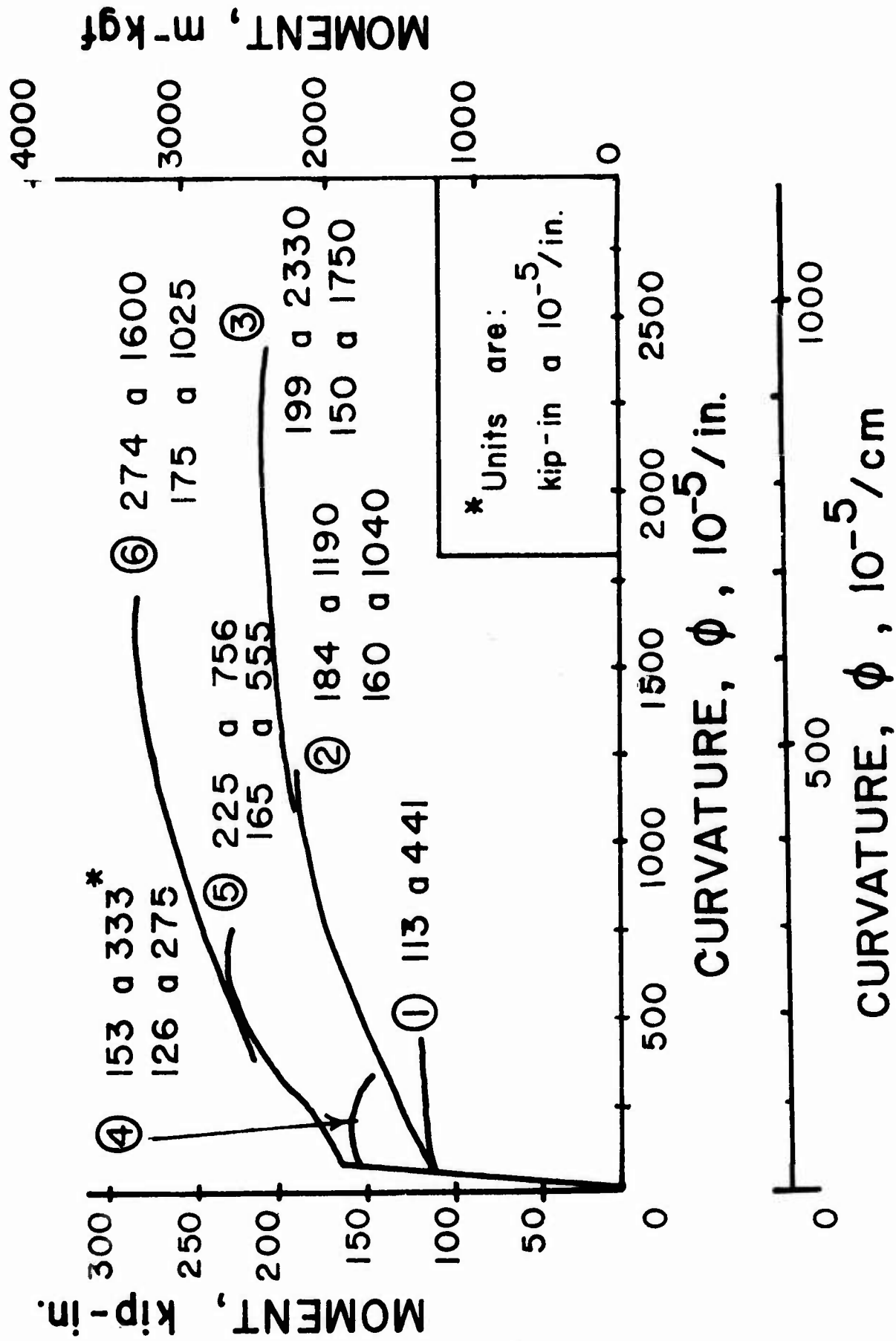


Figure 9. Moment-curvature: beam elements of figure 8.

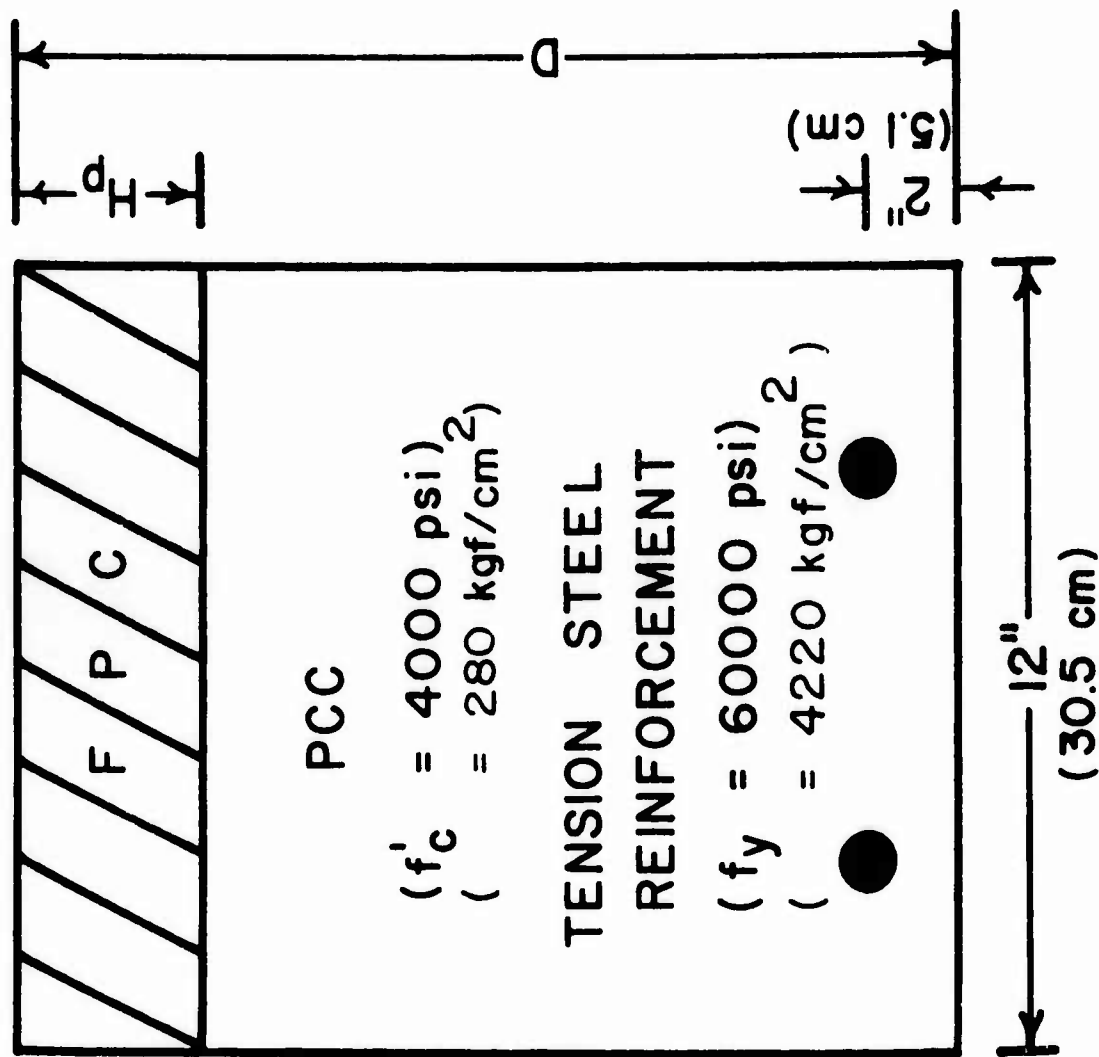


Figure 10. Beam cross-section.

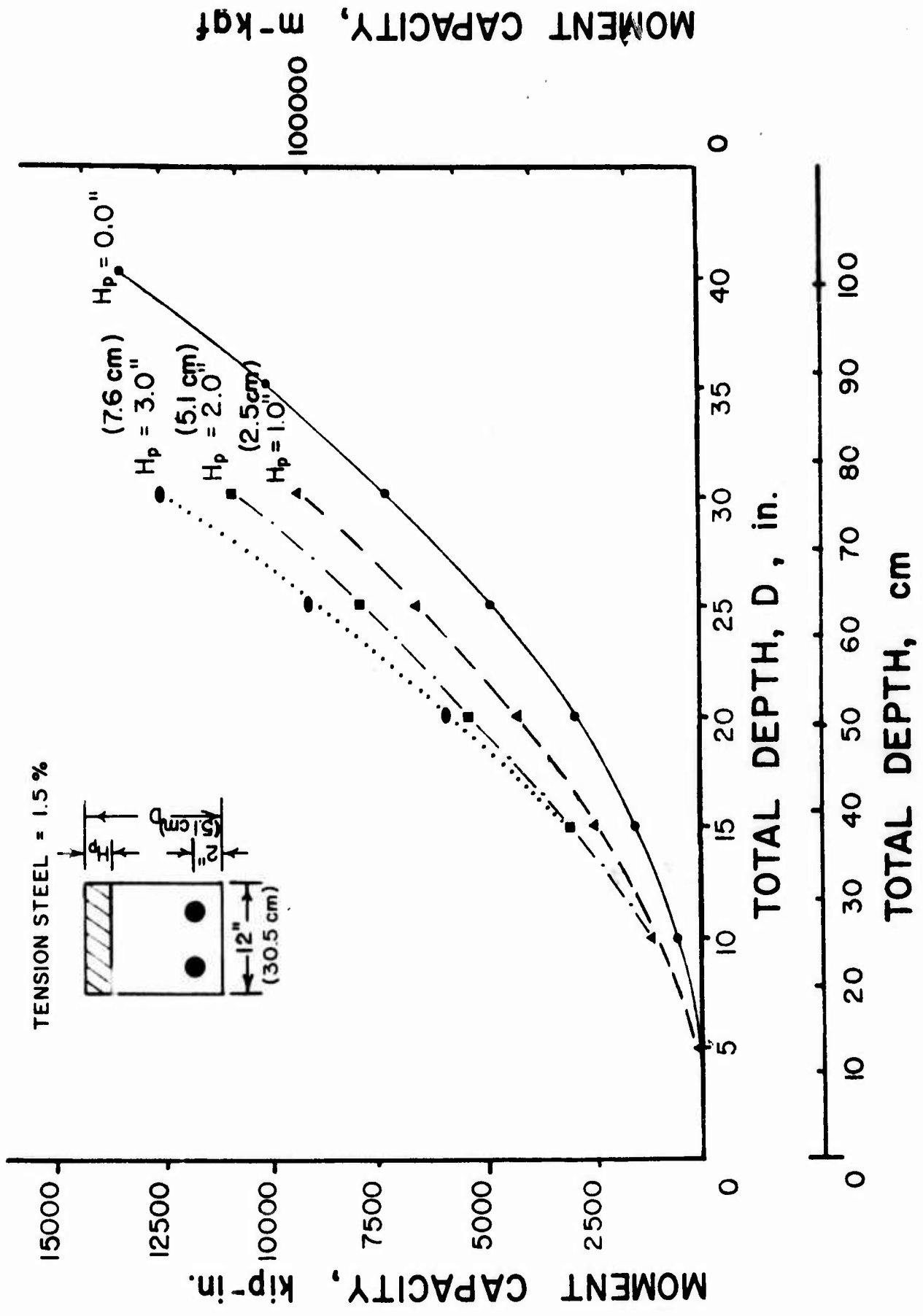


Figure 11. Moment capacity vs depth.

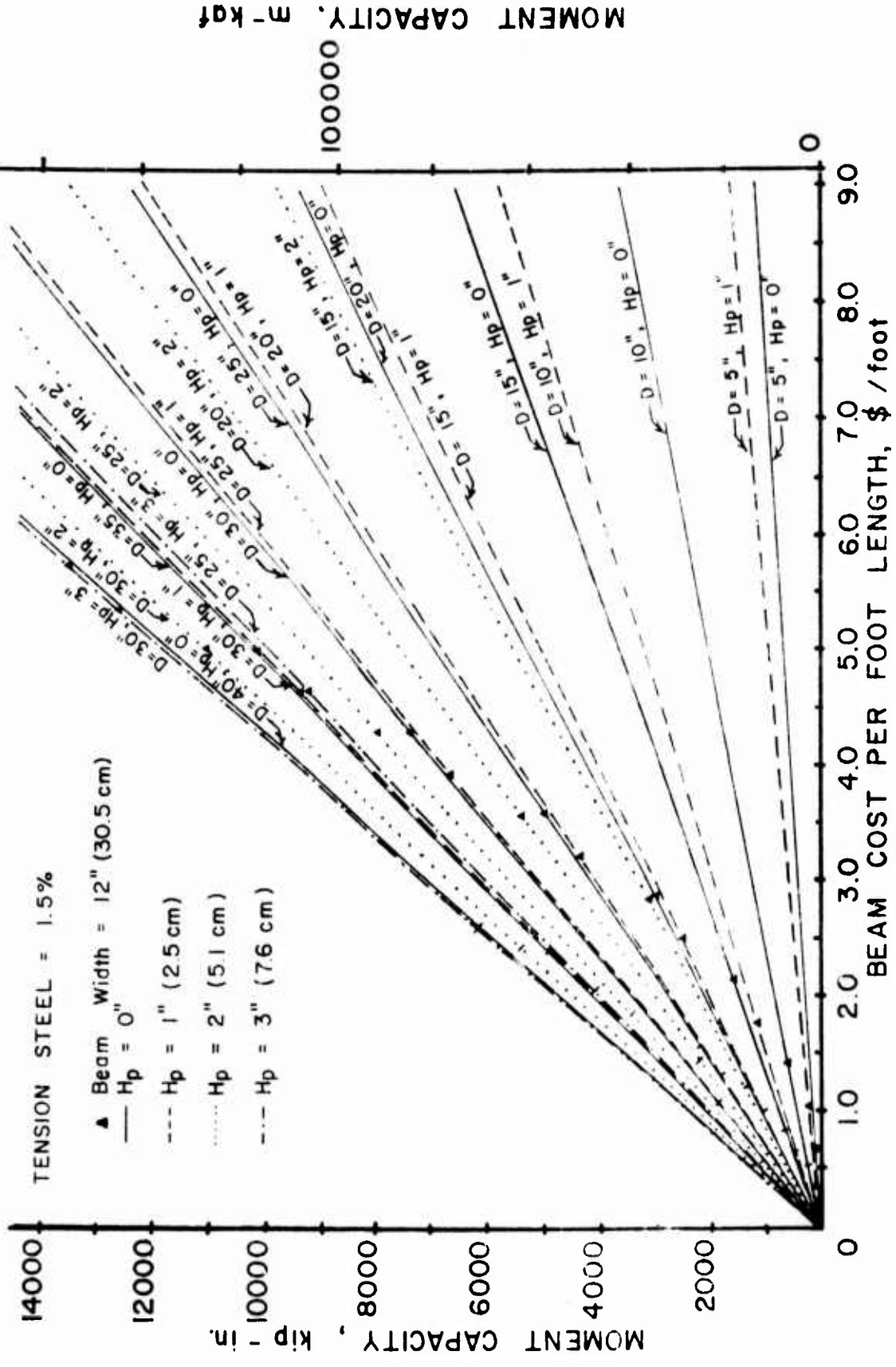


Figure 12. Moment capacity vs beam cost.