STRESS AND STIFFNESS PROPERTIES OF REINFORCED WOOD BEAMS

Thesis for the Degree of M. S.

MICHIGAN STATE UNIVERSITY

Jordan A. Tsolakides

1962

THESIS

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OF REINFORCED WOOD BEAMS

By

JORDAN A. TSOLAKIDES

AN ABSTRACT

Submitted to the School of Graduate Studies of Michigan State University of Agriculture and Applied Science in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Department of Forest Products

1962

Approved Alan Sliker

ABSTRACT

This study describes the manufacturing and testing of twelve reinforced wood beams with cross section of 1.5 by 4.5 inches. Nine of the beams were 96 inches long and three 45 inches long. The beams were divided into three groups according to the board from which they were cut. Each group was constisted of four beams, each one of different type. Three beams of each group were reinforced with steel strips. One was reinforced in the sides (Type A), one on top and bottom (Type B), one the same as Type B but of 45 inches length (Type b), and one left unreinforced(Type C).

The purpose of this problem was to evaluate and compare stress and stiffness properties of the beams when subjected to static bending. Also, an attempt was made to obtain certain information about the behavior of the reinforced beam within the two different types of reinforcement used in this study.

Stiffness properties (a function of the modulus of elasticity and section properties) calculated for all beams were found to be 2 to 3 times greater for the reinforced beams as compared to the unreinforced ones. Among the two types (A and B) Type B indicated 10 percent higher values.

The load carrying capacity of the reinforced beams was also increased. Calculations at proportional limit showed an increase between 35 and 50 percent. At maximum load the increase was much greater.

Eleven of the beams failed in tension and one (reinforced) in compression with buckling in the compression side.

Theoretical stiffness values were also calculated. A mathematical model of the beams' cross section was built, using the weighed ratio of the two moduli of elasticity of the composite beam. This gave an I form cross section in which the neutral axis was located and the inertia of the whole transformed cross section was calculated with the parallel axis theorem.

In general, a close agreement was observed between theoretical and practical stiffness values within groups; but a small variation was indicated among the groups.

The grooves used for the placement of the steel strips proved to be critical for the whole construction, and further study is needed toward this direction.

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1. INTRODUCTION

1.1 Definition of Reinforced wood beams:

A reinforced wood beam is a wood beam assembled in combination with some other material in order to improve a desirable strength property.

In the present problem the term "reinforced" is used to describe a wooden beam with steel strips inserted in milled grooves in the span direction.

1.2 Historical Background:

Although a number of papers have been published on the general subject of wood reinforcement very few were close to the subject of this particular problem. One of the first ideas for reinforcing wood was presented by C. Volk in Germany in 1907.[6] His construction was of a very primitive nature, consisting of wooden boards placed on top of each other, and tied together with metal strips (see Fig. 1).

In 1921 J. B. Aatila from Chicago, Illinois, registered a type of hollow rectangular wooden beam comprising solid upper and lower flanges that were thick and slab-like, and thin parallel connecting webs of flat laminated wood veneer [13] (see Fig. 2).

Later in 1926 a German patent (D. R. P. 547576) was filed by A. Fischer. In this patent another type of solid timber, reinforced with steel rods, was introduced. Steel rods were fastened to the timber with a type of elastic cement (Fig. 3).

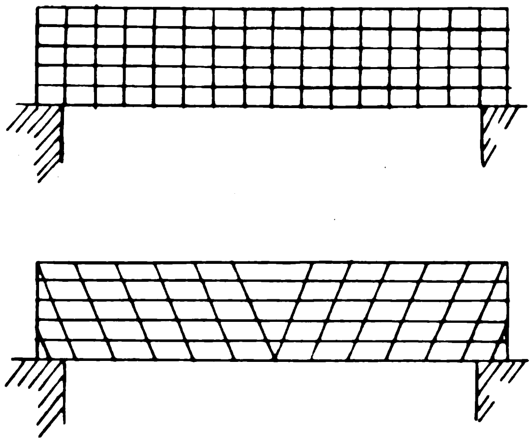


Fig. 1. Steel reinforced wood beams according to a German patent No 233658 (1907). Beam consists of a number of boards, which are connected by crosswise glued reinforcements.

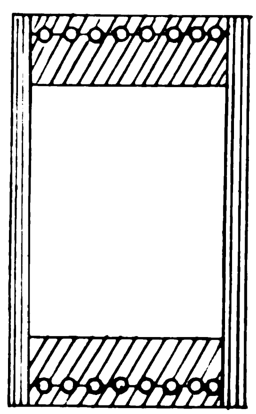


Fig. 2. Reinforced wooden beam, with longitutinal reinforcement in both tension and compression sides.

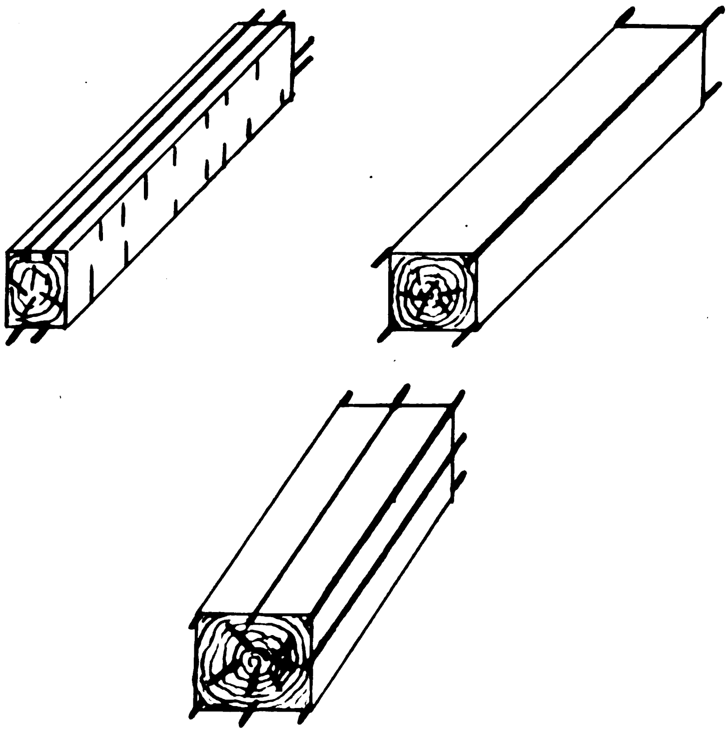


Fig. 3. Various types of reinforced solid timbers. Fastening of metal to wood was done with an elastic cement.

Fischer's results were doubtful because of the glue used. The concept of combining solid wood with steel is particularly important today, due to the fact that adhesives have been improved tremendously.

J. F. Seiler, in 1932, introduced an original work of outstanding value to this field; the development of a procedure for the structural assembly of composite laminated wood-concrete construction.[4] This combination has become a rather common practice today, for bridges, pier decks and miscelaneous heavy duty services.

Trilaminated wood-steel beams are sometimes fabricated.[2] In such a construction the core lamination is of wood and the flanges are of steel. Another type consists of a wooden core encased by a welded steel shell. Fastening of the metal to the wood (in both cases) is usually done by mechanical fasterners, such as bolts or pins, which secure the wood core to the flanges or to the metal shell.

The subject of reinforcement with metal strips of various cross sectional areas, placed in milled grooves and glued with wood, was experimented by H. Granholm in 1954.[6]

Today many workers are engaged in studying the technical possibilities of combining timber and metal of various forms in order to obtain beams that could be used in practical engineering construction.

Wood-aluminum beams were investigated by R. Mark.[4] Also, an experiment of laminated wood beams reinforced with aluminum was recently done by A. Sliker.[11]

A number of other workers have been engaged in research of I beams, reinforced with steel bars or trellis beams with braces, built of wood members, and sheet steel strips.[10]

All the above mentioned experiments by various people present a historical picture of the progress in reinforcement work to date.

1.3 Testing of Reinforced Beams and their Importance:

Wood as a structural material has good strength properties, it is easy to work, is light in weight, has good insulating values, and is moderate in cost. Metals on the other hand are characterized by high strength, are heavy, and when in thin sheets lacking of stiffness. A reinforced wood beam combines the good qualities of both, and compensates for the less desirable qualities of each.

Where a strength/weight ratio is of great importance, the combination of wood with metal can be of maximum value. Improvements in strength, stiffness, dimensional stability, and other advantages, such as light weight compared with metals or concrete, more fire resistance, and improved resistance to weather and decay, make the wood-metal combination an important product. Adding to this the decreasing amount of first grade timber, and the competition that wood faces in the market today, a broad field of applications would be available for wood derived products.

1.4 Purpose of the Study:

The object of this study was to evaluate and compare the stiffness and strength properties of two types of reinforced wood beams and one of the conventional type (unreinforced).

The beams were constructed in an attempt to determine:

1) Whether or not there was any increase in stiffness by changing the orientation of the stiffening material.

2) The differences between theoretical and experimental stiffness values.

3) The load at failure, the type of beam failure, and the behavior of reinforced beam as compared to unreinforced.

2. PROCEDURE

2.1 Description of Test Beams:

A total of twelve beams with like cross sectional dimensions (1.5 by 4.5 inches), were prepared in this study. Nine of them were 96 inches long, and three were 45 inches long. Typed as follows, the separate groupings included:

Type A: Three beams, each reinforced on two opposite vertical sides.

Type B: Three beams reinforced top and bottom.

Type b: Three beams reinforced as Type B, but of shorter length.

Type C: Three beams with no reinforcement.

The above specimens were divided into three groups. The members of each given group were cut from a single board and consisted of four beams, one beam of each type.

For the sake of simplicity each group was given a number which together with the letter characterizing the beam type would be the code number in the following pages (see Fig. 4).

2.2 Materials Used:

Defect free and kiln dry Redwood (Sequoia sempervirens) was used in this study. The lumber was flatgrain.

Three pieces of nominal 2 by 12 inches and 16 feet long, boards were used. In selecting the lumber, attention was paid as to the slope of grain and growth rings, in order to eliminate variations between the three pieces as much as possible.

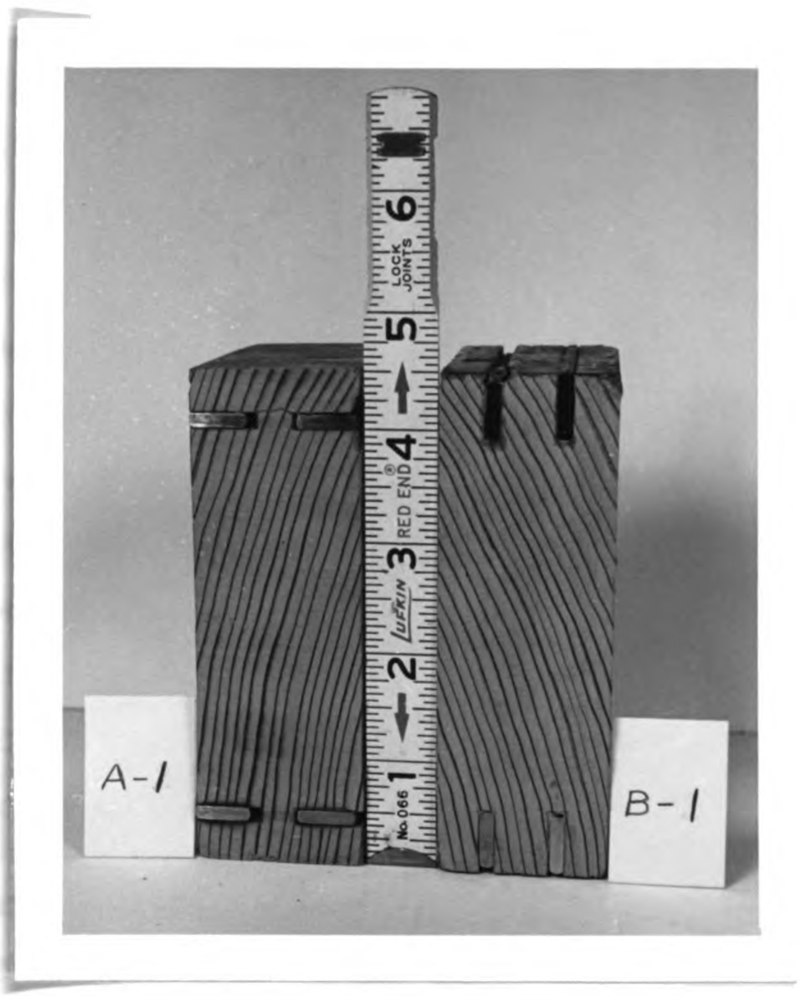


Fig. 4. Cross sections of reinforced beam types. Both beams have been cut from the same board.

Their moisture content, tested by a resistance type moisture meter, was found to be approximately nine percent.

As a stiffening material a mild hot rolled steel was used. Data given by the manufacturers, indicate an average modulus of elasticity for tension and compression of 30 times 10^6 pounds per square inch, a yield point of 70,000 pounds per square inch, and hardness 163, under the Bernoll Scale. The nominal dimension of the individual pieces was 1/8 by 1/2 inch.

Because of the nature of beam types (milled grooves) which did not permit application of sufficient pressure when steel strips were placed in the milled grooves, it was important that a gap filling glue be used. The epoxy resins were best suited for the purpose of the project, since they exhibit little or no shrinkage from loss of solvent. A commercial epoxy-resin adhesive, Hysol 2030, and catalyst C-1 was utilized for wood to metal bondings.

2.3 Lumber Preparations:

Each 16 foot board was sawed lengthwise into two halves and each half was cross-sawn into two pieces. In this manner four beams, one of each type, were obtained from each board. Then each beam was planed into final dimension of 1.5 by 4.5 inches, and the grooves corresponding to the orientation of reinforcement in three of the beams were made with a circular saw.

In Type A beams, the size of the grooves was 1/8 by 1/2 of an inch, and in Type B, 1/8 by 9/16 of an inch.

An essential condition in designing the beams, was to safeguard the reinforcement against buckling. This was done by placing the milled grooves in Type A one half of an inch (including the grooves) inward from the top and bottom of the beam, and for the Type B, one half of an inch inward of the sides of the beam (see Fig. 5). In Type B beams, the depth of the groove was 1/16 of an inch larger than that of Type A. The reason for this was that it was not desirable to apply the load directly against the steel, during the testing procedure.

Table 1 gives the actual size of specimens and the location of the steel strips in the grooves.

2.4 Steel Preparations:

The surface of the steel was prepared for bonding, by sanding it with a No. 50 Aluminum Oxide abrasive. During this process the surface of the metal was slightly scratched and some metal was removed.

2.5 Adhesive Preparations:

The adhesive was mixed at a weight ratio of 100 parts of epoxy (Hysol 2030) to 8.8 parts of catalyst (C-1). The catalyst was poured into the epoxy and the mixture was stirred for five minutes.

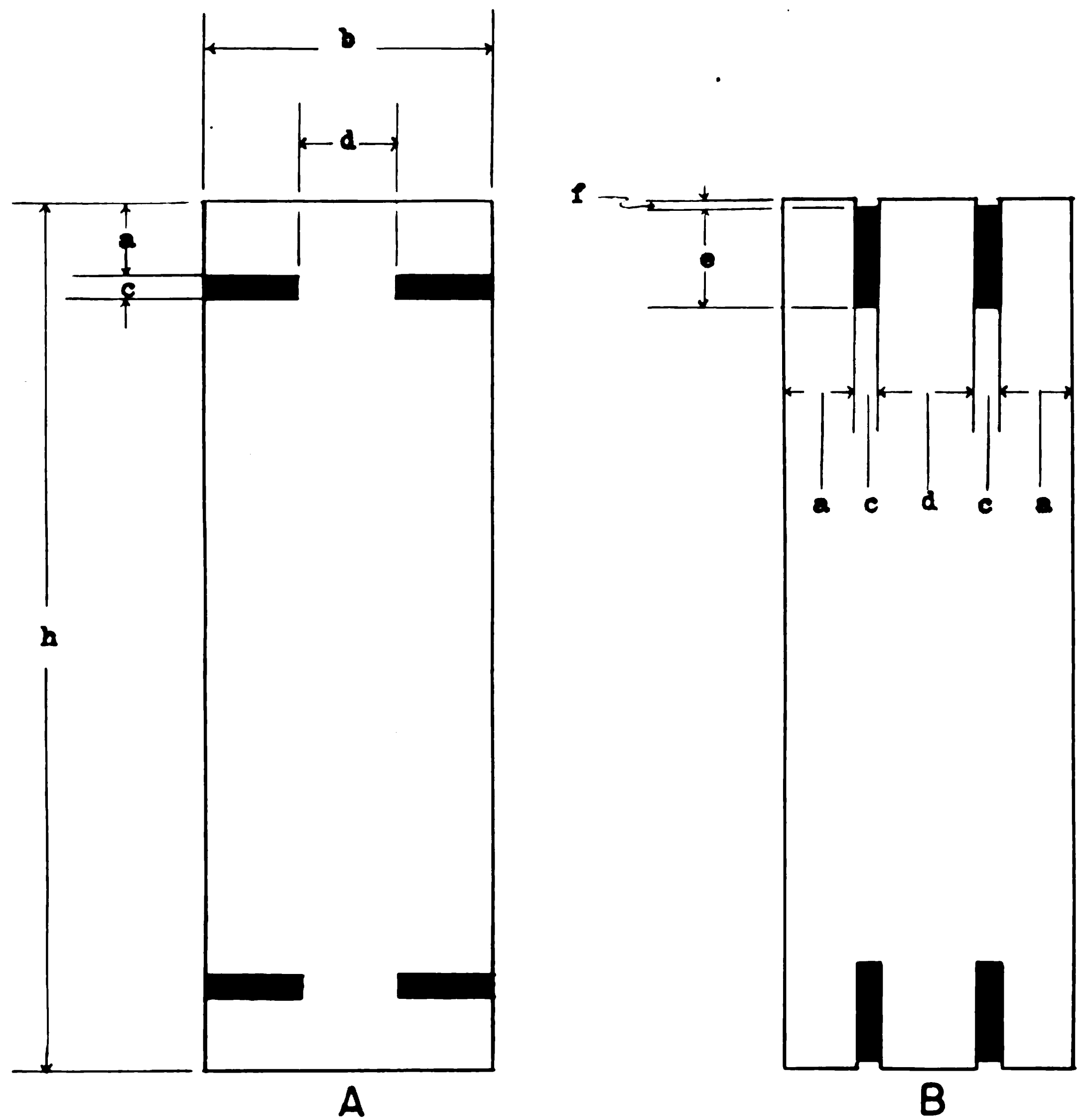


Fig. 5. The two types of reinforced beams used in the study

Table 1 - Actual Size of Specimens

Beam Number	Wood Width b-in.	Wood Height h-in.	Steel Width c-in.	Steel Height e-in.	Location of Reinforcements a	d	f	Beam Length in.
A - 1*	1.500	4.512	.124	.511	.376	.478	-	96
B - 1	1.500	4.500	.121	.508	.379	.500	.037	96
C - 1	1.505	4.500	-	-	-	-	-	96
A - 2	1.505	4.510	.125	.516	.375	.473	-	96
B - 2	1.500	4.490	.125	.519	.380	.490	.049	96
C - 2	1.505	4.510	-	-	-	-	-	96
A - 3	1.495	4.495	.124	.507	.375	.481	-	96
B - 3	1.495	4.495	.126	.507	.375	.493	.027	96
C - 3	1.505	4.495	-	-	-	-	-	96
b - 1	1.505	4.505	.125	.508	.360	.535	.032	45
b - 2	1.505	4.500	.125	.506	.354	.547	.036	45
b - 3	1.500	4.505	.124	.506	.350	.552	.036	45

*Letters refer to the type of beam. Number refer to board from which each beam was made.

2.6 Assembly Procedures:

Immediately after the adhesive was mixed, it was applied into the grooves with a small brush and the steel strip was imbedded. When all four strips were placed, wooden clamps were used to secure the strips in the grooves and to apply a small pressure. Then the beams were left for 24 hours to cure at room temperature and were removed to a conditioning room at 70°F. temperature and 60 percent relative humidity, for a week.

2.7 Strain Gauges:

Wire, electrical resistance strain gauges were placed on the top and bottom of each beam. The gauges were made from 120 ohm lengths of 1 mil constantan wire which were formed into the shape of a hairpin when bonded to the test members.

3. TESTING PROCEDURES

3.1 Static Bending:

In general the testing procedures prescribed by the American Society for Testing Materials (Designation D 198-27) were used.[1]

The test was performed in a 100,000 pound Reihle Universal Testing Machine. According to A. S. T. M., the specimens were measured to a nearest of 0.001 inch ± 0.003 inches for the cross section and the results were recorded (see Table 1).

The beams were loaded at third point and the load was applied at a rate of 1/16 inch per minute. This speed is not the one recommended by A. S. T. M. but it was used in order to facilitate strain gauges reading at each deflection interval measured. The span of the long beams was 90 inches and of the short ones 39 inches. The method of testing and the apparatus used are illustrated in figure 6.

Loads were recorded to the nearest 10 pounds at 0.025-inch intervals of mid-span deflection with an Ames dial gauge reading to 0.001 inch. This gauge was held by a yoke, supported at the beams' end and was mounted on the neutral plane of the specimen.

Strain readings were also recorded at the same intervals (0.025 inch) of the deflection gauge, with two strain gauges mounted on the mid span top and bottom surface of the beam. These gauges were connected with SR-4 Baldwin Strain indicator, measuring microinches per inch (see Fig. 7).

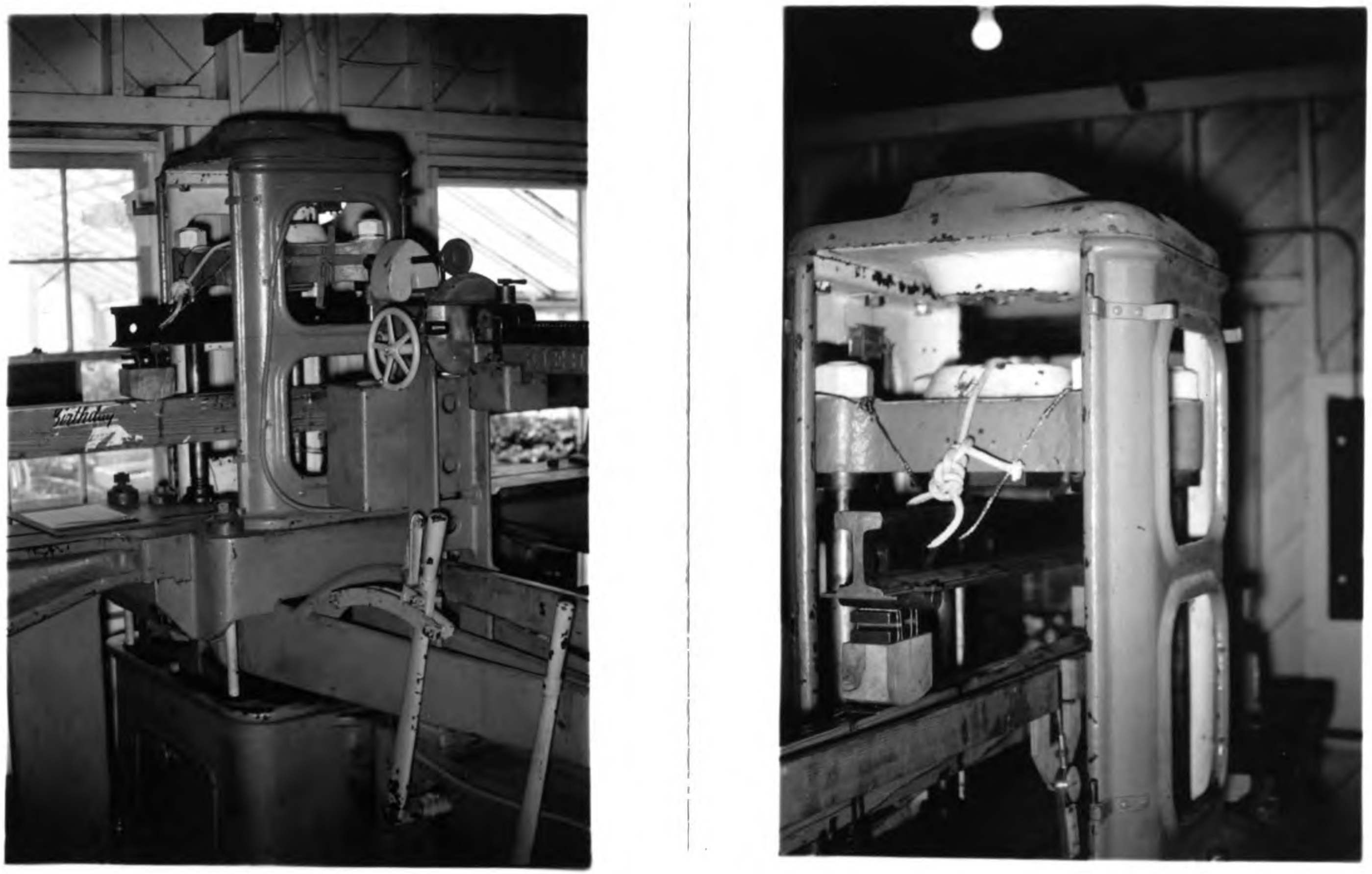


Fig. 6. Left picture shows testing of Type B beam, and right picture shows a Type A beam. Note the method of load application, and the location of the dial gauge for deflection measurements.

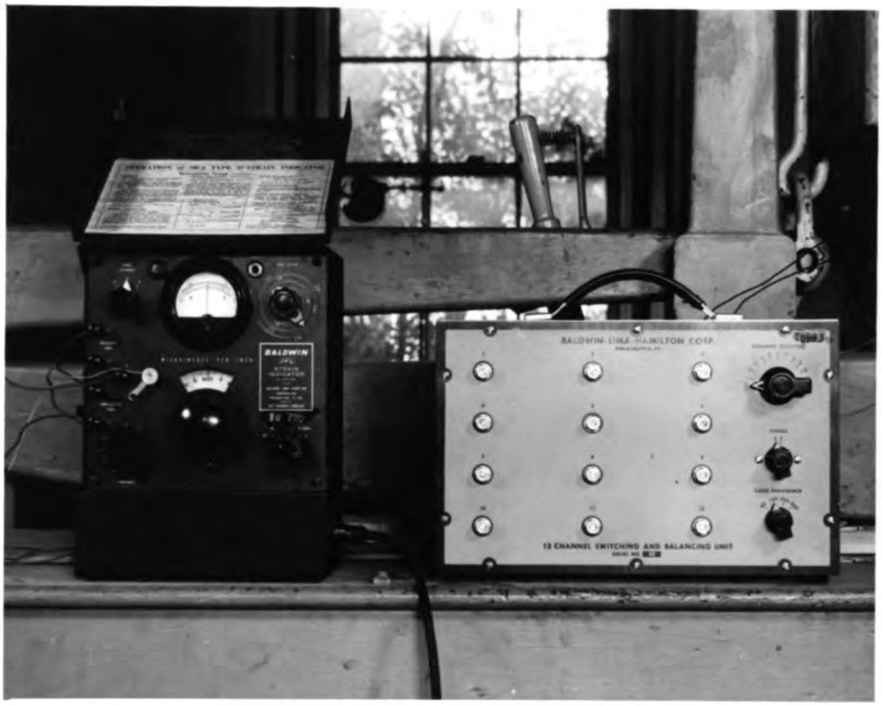


Fig. 7. Baldwin Type M, SR-4, static strain indicator for making strain measurements.

3.2 Compression Parallel to Grain Tests:

After each beam was tested in bending, two pieces of wood were cut from between reinforcements, close to the failure and laminated together to form a standard A. S. T. M. compression parallel to grain specimen. The procedures described by the A. S. T. M. (Designation 143-52) were followed in this test. The specimens were weighed and measured to an accuracy of 0.001 inch for the cross section, and 0.01 inch for the length and the results were recorded.

A Reihle 50,000 pounds testing machine was used, for the compression testing. Following the test, the specimens were placed in an oven at a 103 $\pm$ 2°C. for moisture content, and specific gravity calculations.

This test was done in order to find out the modulus of elasticity of the wooden part of each beam. The results of this test are illustrated in Table 2.

Table 2 - Moduli of Elasticity in Compression parallel to grain as determined from 2" x 2" x 8" test specimens

Beam Number	Moisture Content Percent	Specific Gravity *	Modulus of Elasticity 1000 p.s.i.	Remarks
A - 1	8.6	.28	908.2	
B - 1	8.5	.29	1,052.6	
C - 1	8.8	.29	1,004.9	
A - 2	8.8	.40	1,595.5	
B - 2	9.3	.42	1,726.9	
C - 2	8.9	.40	1,503.5	
A - 3	8.5	.34	1,051.5	
B - 3	9.9	.39	1,291.1	
C - 3	9.3	.39	1,382.0	
b - 1	-	-	1,010.0	Average of
b - 2	-	-	1,570.0	the group values
b - 3	-	-	1,240.0	were used.

*Based on oven dry weight and volume at test.

4. ANALYSIS OF DATA

4.1 General Considerations:

The two material beams have different moduli of elasticity for their components. When these beams are subjected to bending within the elastic range of each material, the following assumptions of the flexure theory are valid:

a) Plane sections at right angle to the axis of a beam remain plane. Therefore the strain must vary linearly from the neutral axis.

b) The neutral axis passes through the center of gravity of the section.

c) The modulus of elasticity of each component is the same in tension and compression.

Then, since the elastic case is considered, stress is proportional to strain, and the stress distribution follows a pattern depending upon the position of the reinforcement on the cross section of the beam.

Because M.E. of steel is greater than that of wood, stresses are greater in the stiffer material, at the same distance from the neutral axis.

A common technique for calculating moment of inertia and stresses in composite members is used, as described in Popov,[9] by constructing an equivalent cross section of one material. For the reinforced test beams, an I shape results. This I form cross section is termed, transformed cross sectional area. The two material beam, when transformed in I cross section beam, is considered as a one material beam and the usual Flexure Formula applies. In order to achieve this I form cross section, Popov utilizes the E steel/ E wood ratio. Through the use of this ratio, multiplied by the width of the steel strip, the breadth of one of the flanges of the I form cross section was found (in terms of wood).

$$b_1 = b \frac{E_{st}}{E_w}$$

where

b_1 = the breadth of one of the flanges in inches.

b = the width of the steel strip in inches.

E_{st} = modulus of elasticity of steel in p.s.i.

E_w = modulus of elasticity of wood in p. s. i.

The modulus of elasticity for steel was known and modulus of elasticity of wood was calculated in a test with compression parallel to grain. Then the b_1 value multiplied by two (in this problem) plus the width of the intermediate wood (web) gave the total width of either upper or lower flange of the transformed I section.

The stresses and strains of the I form beam vary linearly from its neutral axis, and the stresses for the material of which the transformed section was made can be calculated from the conventional stress formula.

$$\sigma_w = \frac{Mc}{I}$$

For the steel the following formula has been adopted for the purpose of this problem.

$$\sigma_{st} = \frac{Mc}{I} \times \frac{E_{st}}{E_w} \times \frac{y}{c}$$

which simplified gives

$$\sigma_{st} = \frac{My}{I} \times n$$

where

σ = stress in extreme fiber (subscripts refer to wood or steel) p.s.i.

M = maximum bending moment, pound - inches.

c = distance from neutral axis to the furthest point of the wood in inches.

I = moment of inertia of the transformed cross section in $inches^4$.

y = distance from the neutral axis to the furthest point of the steel in inches.

n = ratio of the elastic moduli E_{st} / E_w.

The reinforcement of the beams was symmetrical; therefore, the neutral axis was located at the center of the cross sectional area.

The moment of inertia I, of plane area with respect to an axis in its plane is given by:

$$I_x = \int_A y^2 \, dA$$

In a composite beam as in our case, the cross sectional area was broken in small components and its component's moment of inertia was determined by the parallel axis theorem:

$$I = \frac{1}{12} ab^3 + (ab)\ c^2$$

where

a = width of the area in inches.

b = height of the area in inches.

c = distance from neutral axis of the beam to the centroid of the area in inches.

Then the total moment of inertia around the neutral axis was found as the sum of the inertias of the components.

$$I_T = I_1 + I_2 \ldots\ldots I_n = \quad I_n$$

4.2 Test Data:

From the data recorded for each beam, loads were plotted versus mid-span deflection, and curves were drawn (see Fig. 8, 9, 10, 11).

The usual deflection formula[7] for a third point loading was used to calculate the practical stiffness values:

$$EI = \frac{23PL^3}{648y}$$

where

$\frac{P}{y}$ = slope of load deflection curve in pounds / inch.

P = one of two equal concentrated third point loads in pounds.

L = beam span in inches.

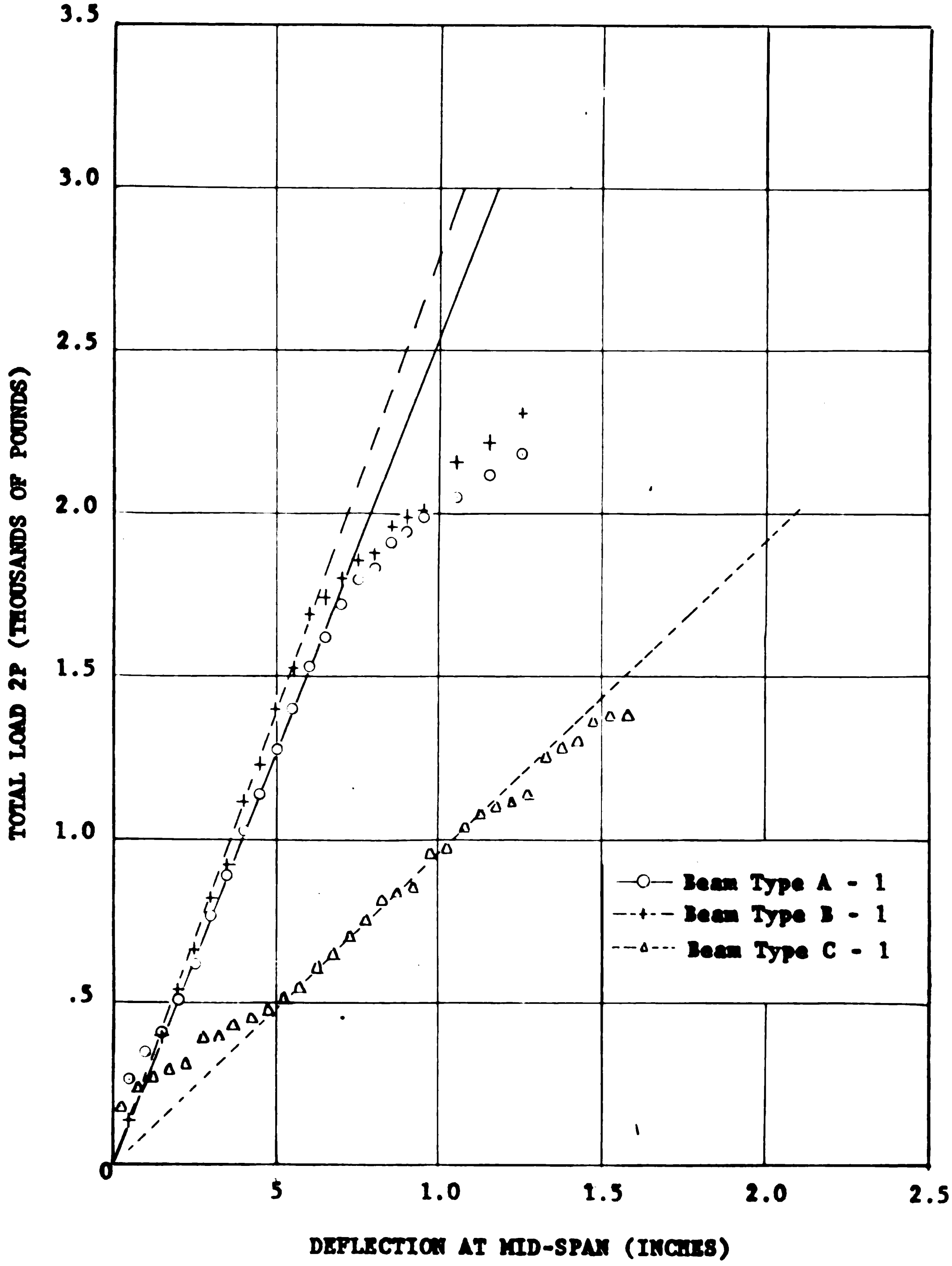


Fig. 8. Load deflection curves for long beams

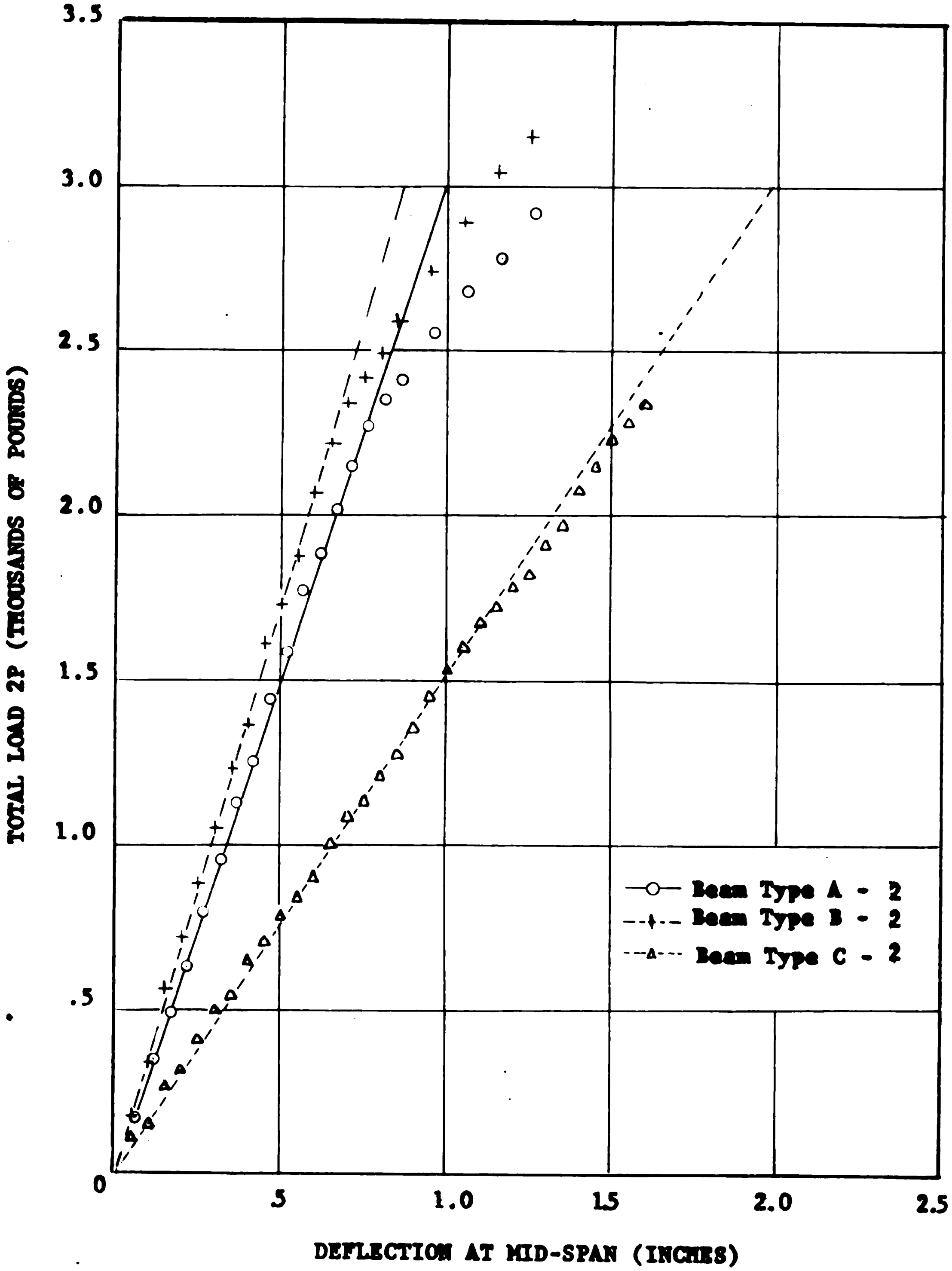


Fig. 9. Load deflection curves for long beams

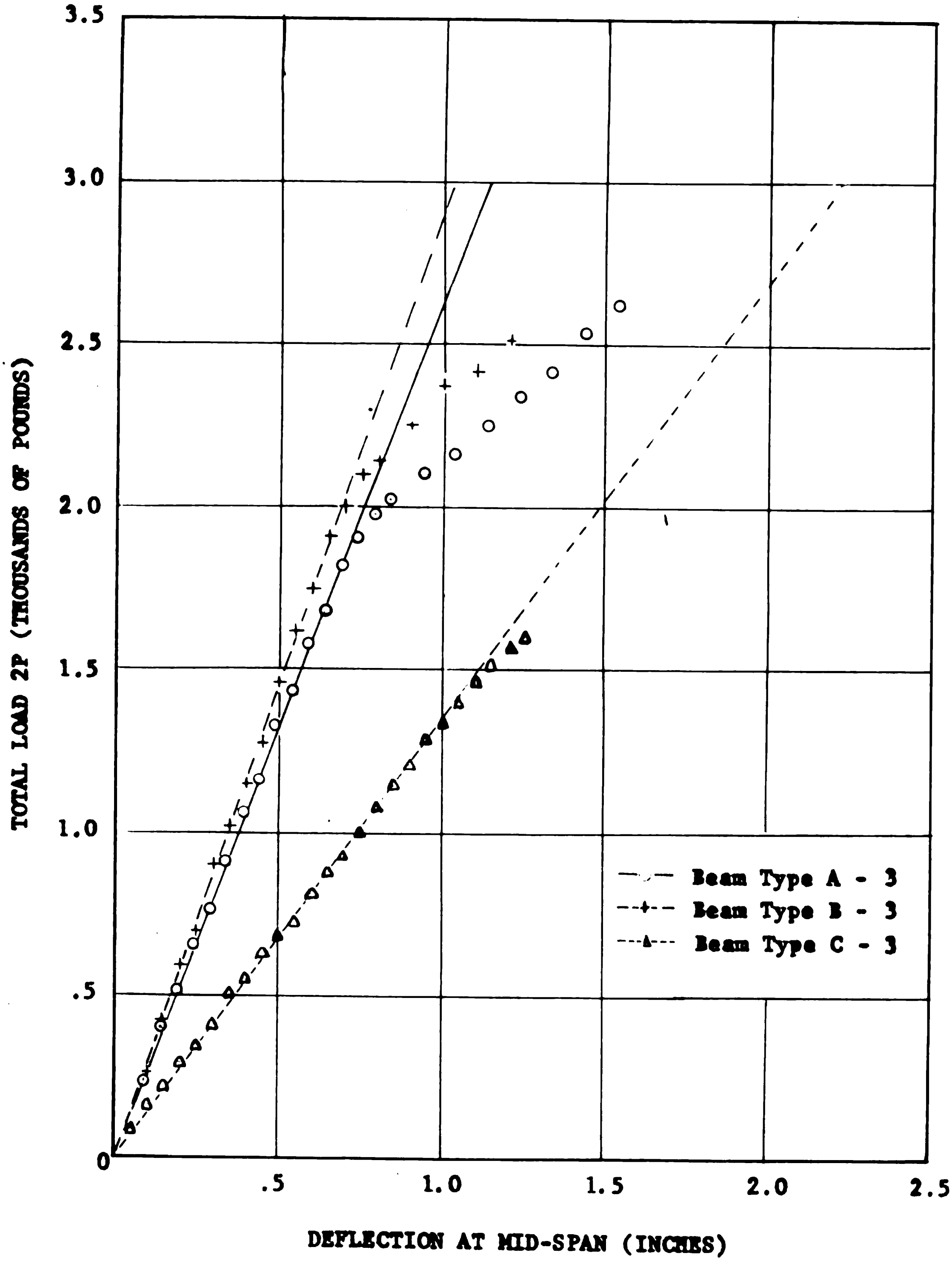


Fig. 10. Load deflection curves for long beams

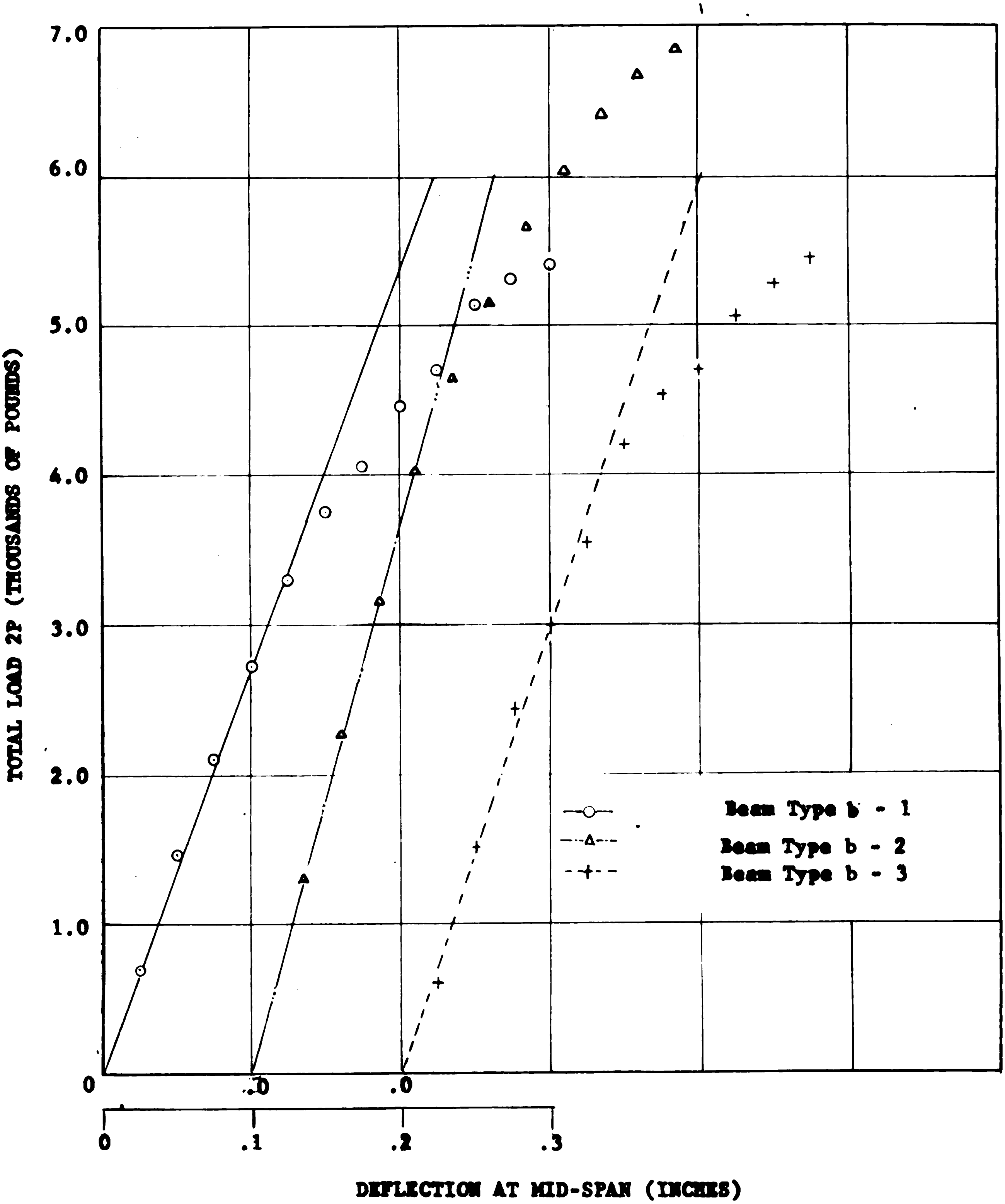


Fig. 11. Load deflection curves for short beams

The bending stress calculations for steel were done by the formula

$$\sigma = \frac{Mc}{I} \times \frac{E_{st}}{E_w} \times \frac{y}{c}$$

where I is the inertia of the transformed cross sectional area. The weighed ratio of the two different moduli of elasticity becomes equal to one as far as wood is concerned.

For wood the following formula was used:

$$\sigma = \frac{Mc}{I} \times \frac{y}{c}$$

This formula gives the stress at any infinitesimal area of the cross section at distance y from the neutral axis. When y equals c, the distance from the neutral axis to the extreme fiber, σ, represents the maximum stress (σ max.).

By transforming the cross sectional area of a two material beam, we obtain a new cross sectional area consisting only of wood and then can use the regular shear stress equation.

$$T = \frac{VQ}{It}$$

where

T = shear stress in p.s.i.

V = total shearing force at a section in pounds.

I = moment of inertia of the whole transformed cross section in inches^4.

Q = statical moment of the transformed cross sectional area above the shear plane around the neutral axis in inches cubed.

t = width of the beam of the shear plane in inches.

For T max., Q obtains its largest value by considering the shear plane at the neutral axis.

A comparison of load carrying capacity (apparent bending stress) of the reinforced beams at a maximum load and at proportional limit was evaluated by dividing the corresponding bending moment by the quantity $bh^2/6$.

$$\frac{P \times L/3}{bh^2/6}$$

where

P = load at maximum carrying capacity or at proportional limit in pounds. (One of two equal third point loads.)

L = span in inches.

b = actual beam width in inches.

h = actual beam depth in inches.

Strain development informations were provided by strain gauges mounted on tops and bottoms of the beams. Strain readings were plotted versus deflection, and the proportional limits in the compression and tension surfaces were observed (see Fig. 12, 13, 14, 15).

All data obtained for the various evaluations are shown in tables 3, 4, and 5.

4.3 Theoretical Data:

Theoretical EI values were also determined. Earlier mention was made of the transformed cross sectional area. The

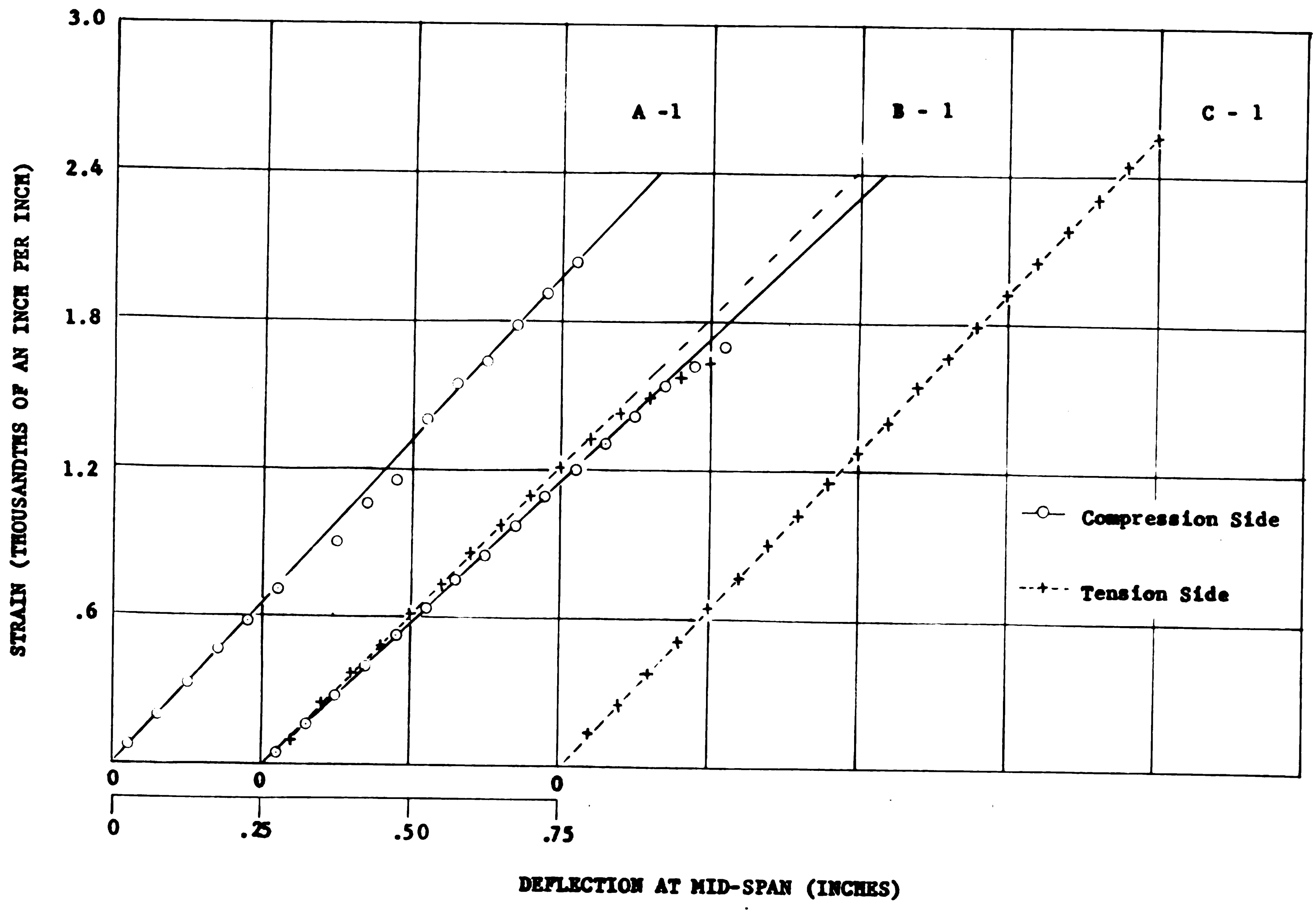


Fig. 12. Strain measurements recorded on tension and compression surfaces at mid-span of long beams

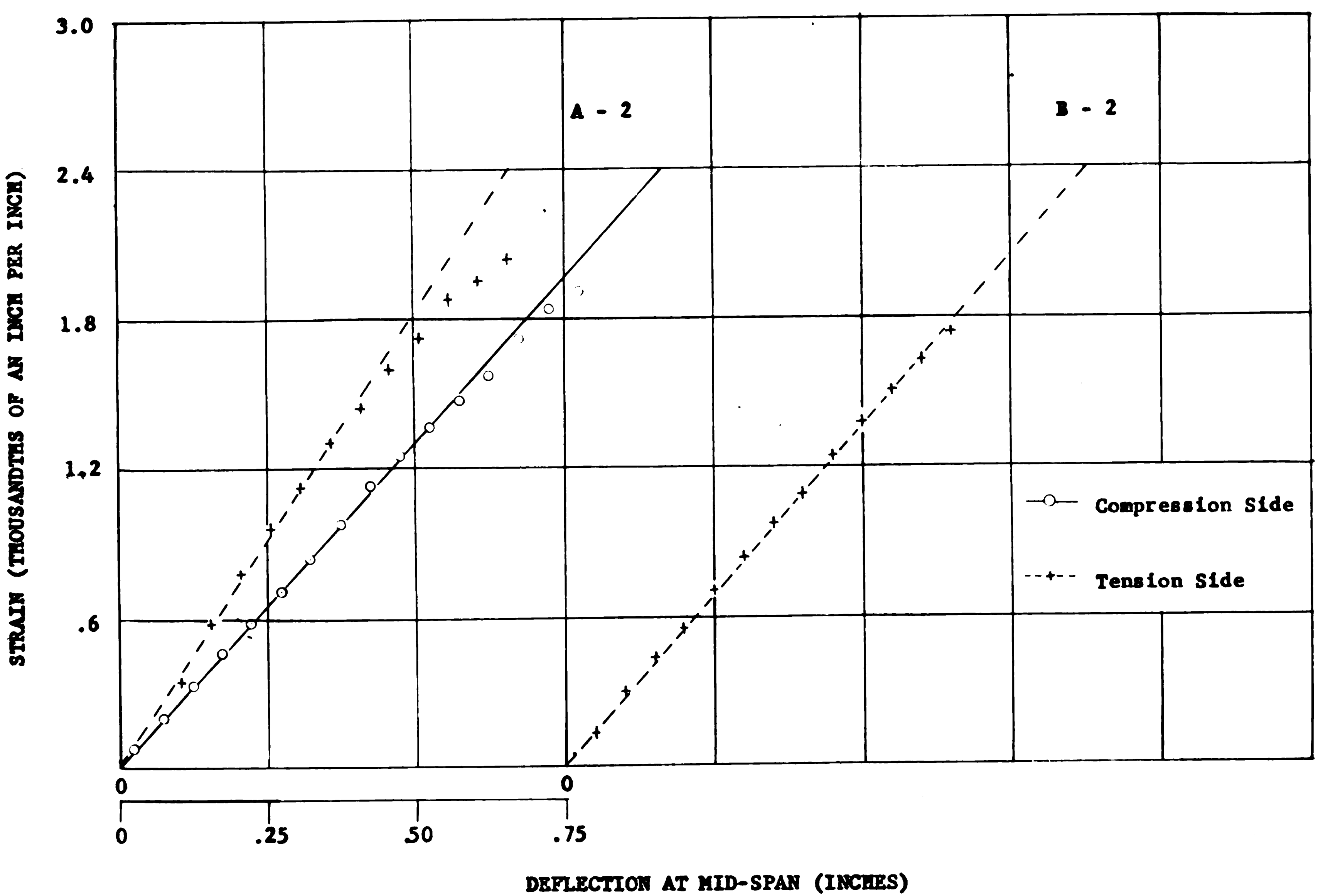


Fig. 13. Strain measurements recorded on tension and compression surfaces at mid-span of long beams

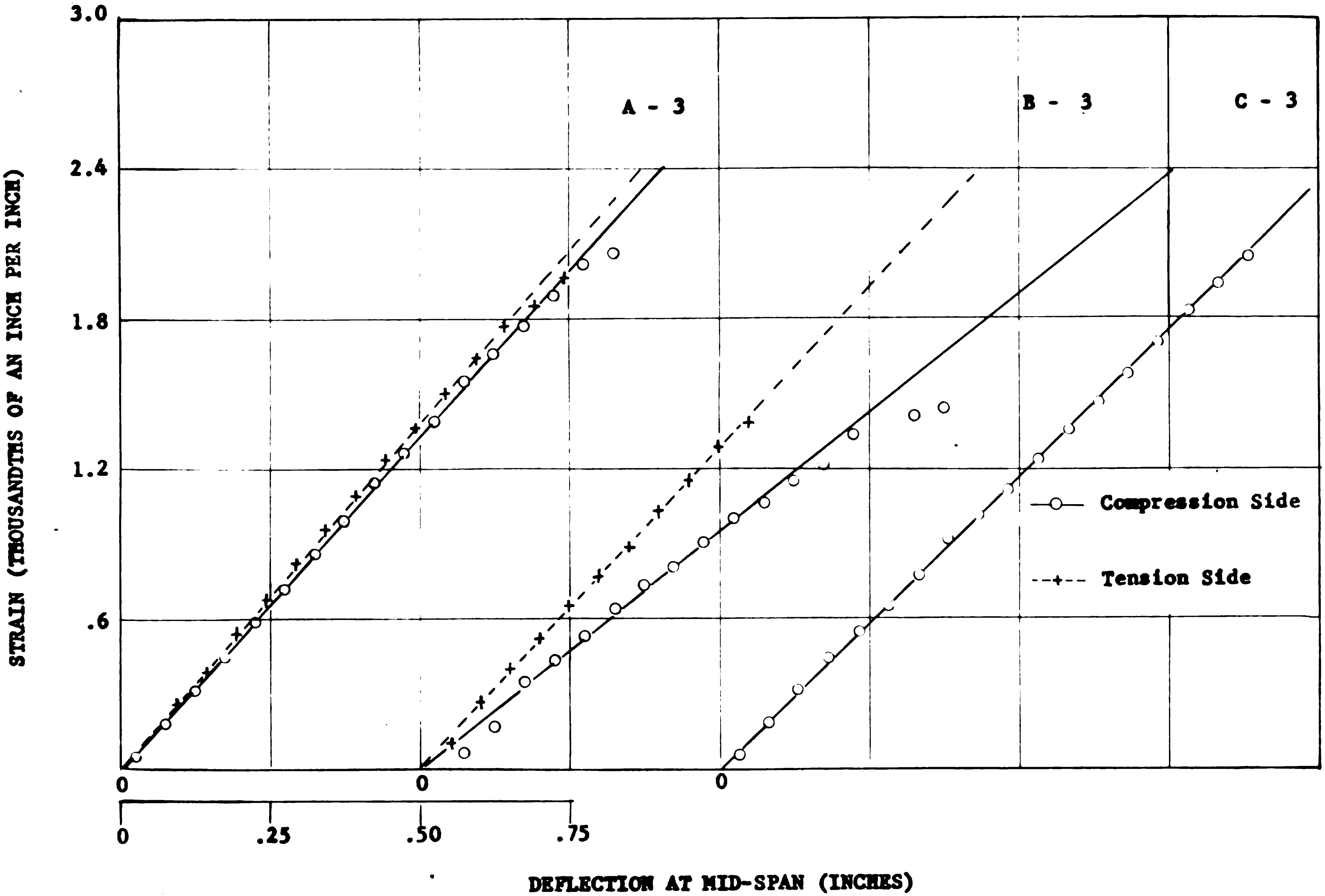


Fig. 14. Strain measurements recorded on tension and compression surfaces at mid span of long beams

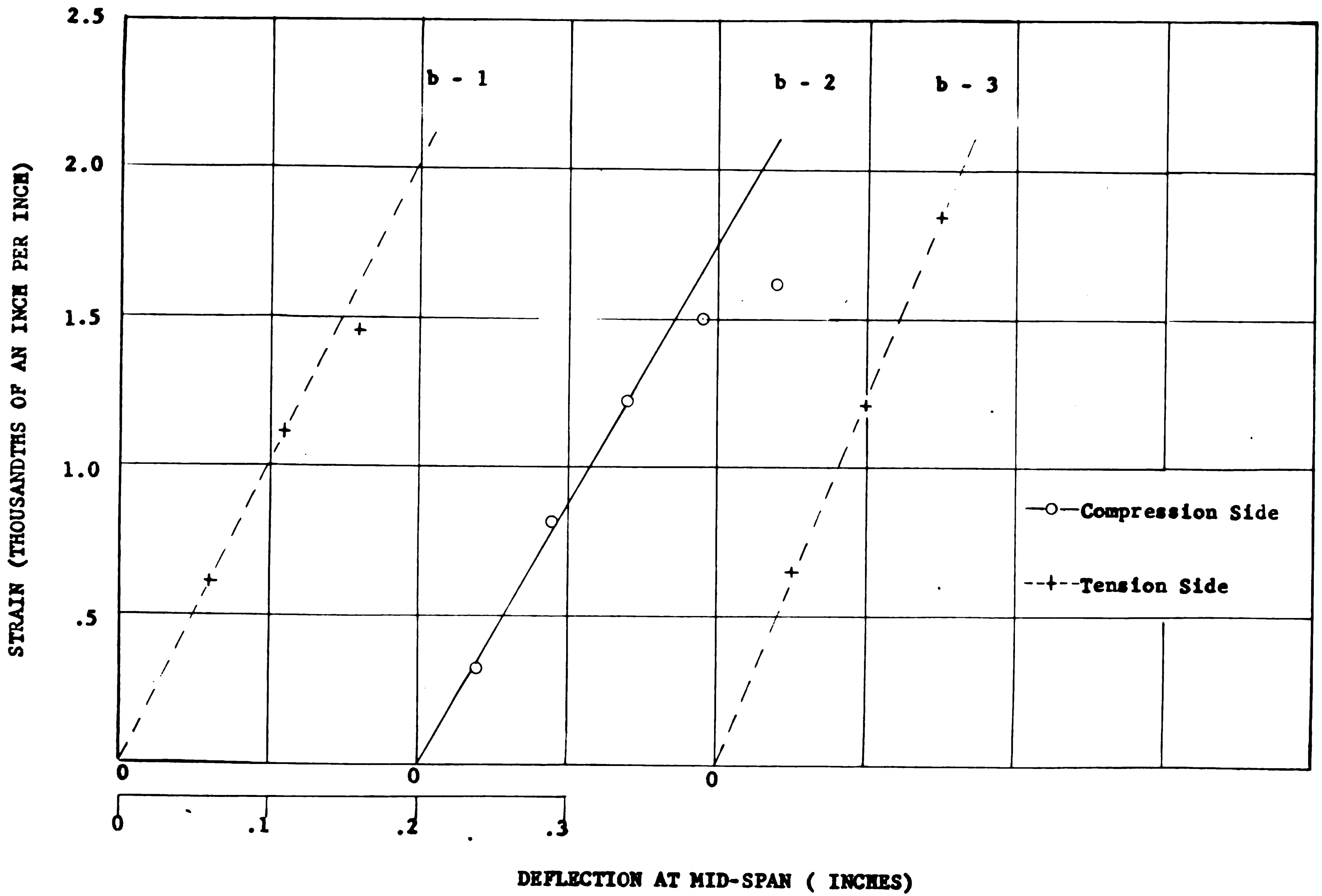


Fig. 15. Strain measurements recorded on tension and compression surfaces at mid-span of short beams

neutral axis because of symmetry passes through the centroid of the cross sectional area of the beam ($\bar{y} = o$). The area was broken into simple shapes, and the inertia for any area composed of a simple shape was found with the parallel axis theorem. Then the total inertia around the neutral axis was found as the sum of the inertia of the parts.

$$I_T = \sum_{i = n}^{i = n} I_i$$

The theoretical EI values were determined by multiplying E of wood by the inertia of the whole transformed cross section.

$$EI = E_w I_T = \Sigma E_i I_i$$

where

E_w = modulus of elasticity of wood in p.s.i.

I_T = total inertia of the transformed cross section in inches^4.

In calculating experimental EI values, shear deflection was also involved. Because theoretical EI values based on bending stress alone were larger than the test values, therefore, a correction of theoretical EI values was done. This was made by using the formula:[11]

$$EI = \Sigma E_i I_i^{*} \div \left[1 + \left(3/4 \frac{E_1}{G} - 3/10 - \frac{3}{4} U \right) 2/3 \frac{h^2}{L^2} \right]$$

---

* $\Sigma E_i I_i$ stands for theoretical stiffness.

where

$G =$ shear modulus of elasticity for wood in p.s.i.

$E_1 =$ apparent Young's modulus for the beam obtained by dividing $\Sigma E_i I_i$ for the beam by I based on gross dimension in p.s.i.

$U =$ Poisson's ratio.

$h =$ beam depth in inches.

$L =$ beam span in inches.

The shear deflection was also determined as percent of the bending deflection for each beam. This was done as follows:

$$\frac{\Sigma E_i I_i - \Sigma EI}{\Sigma E_i I_i} \times 100$$

where

$\Sigma E_i I_i =$ theoretical stiffness values.

$\Sigma E I =$ corrected theoretical stiffness values.

Table 3 - Experimental and Theoretical Stiffness Values for Test Beams

Beam Number	Moisture Content at Test Percent	Cross section Dimensions Width x Depth Inches	Maximum Load Pounds	Experimental EI from Load Deflection Data Pounds Inches^2 x 10^6	Theoretical $EI = \sum E_i I_i$ Pounds Inches^2 x 10^6	Shear Deflection as % of Bending Deflection	Theoretical EI Corrected for Shear Deflection Pounds Inches^2 x 10^6	Corrected Theoretical EI ÷ Experimental EI values	Experimental EI of Reinforced ÷ EI Unreinforced Percent	Experimental EI of Type B ÷ EI of Type A Percent	Experimental EI of Type B Long Beam ÷ Type b Short Percent
A - 1	8.7	1,500 x 4,512	2,780	32,775	34,837	3.28	33,691	1.027	266.6	110.5	
B - 1	8.9	1.500 x 4.500	3,050	36.225	39.356	3.66	37.915	1.046	294.7		-
C - 1	8.5	1.505 x 4.500	1,370	12.290	11.484	.99 1	11.380	.925	100.0		
A - 2	8.5	1.505 x 4.510	4,100	39.807	42.521	3.66	40.965	1.029	205.1	111.8	
B - 2	9.5	1.500 x 4.490	4,480	44.505	46.755	4.03	44.870	1.008	229.3		-
C - 2	8.3	1.505 x 4.510	3,480	19.406	17.297	1.47	17.041	.878	100.0		
A - 3	8.0	1.495 x 4.495	2,810	34.500	35.734	3.10	34.685	1.005	197.5	111.2	
B - 3	9.5	1.495 x 4.495	3,100	38.381	43.050	4.21	41.236	1.074	219.7		-
C - 3	8.8	1.505 x 4.495	1,620	17.465	15.741	1.28	15.539	.889	100.0		
b - 1	8.6	1.505 x 4.505	6,010	28.435	40.060	16.73	33.356	1.173	-	-	127.3
b - 2	8.4	1.505 x 4.500	7,340	38.600	45.573	17.62	37.540	.972	-	-	115.3
b - 3	8.1	1.500 x 4.505	6,030	31.055	42.009	16.60	35.037	1.128	-	-	123.6

Table 4 - Bending and Shear Stress for Test Beams

Beam Number	Moisture Content Percent	Specific Gravity	Bending Stress at Maximum Load: Wood Pounds/Inches^2	Bending Stress at Maximum Load: Steel Pounds/Inches^2	Shear Stress in Wood at Maximum Load Pounds/Inches^2	$M/bh^2/6 \div EI$: M at Proportional Limit 10^{-4}	$M/bh^2/6 \div EI$: M at Maximum Load 10^{-4}
A - 1	8.7	.28	2,434	67,020	268	139	248
B - 1	8.9	.29	2,753	77,170	312	139	249
C - 1	8.5	.29	4,050	-	152	276	329
A - 2	8.5	.40	5,203	81,570	408	158	303
B - 2	9.5	.42	5,572	95,110	434	147	300
C - 2	8.3	.40	10,230	-	384	250	527
A - 3	8.0	.34	2,782	66,130	277	157	243
B - 3	9.5	.39	3,134	71,930	290	151	241
C - 3	8.8	.39	4,800	-	185	228	274
b - 1	8.6	.28	2,218	64,940	551	134	269
b - 2	8.4	.40	3,702	69,620	787	133	244
b - 3	8.1	.39	2,605	62,020	620	124	244

Table 5 - Strength and Strain Data

Beam Number	Wood Moisture Content at Test Percent	Specific Gravity	Apparent Bending Moment At Proportional Limit - $bh^2/6$ Pounds/Inches^2	Apparent Bending Moment At Maximum Load - $bh^2/6$ Pounds/Inches^2	Proportional Limit Deflection (inches) Load Deflection Curves	Proportional Limit Deflection (inches) Strain - Deflection Curves Compr.	Proportional Limit Deflection (inches) Strain - Deflection Curves Ten.	Slope of Strain - Deflection Curves 10^{-3}/inches Compr. Side	Slope of Strain - Deflection Curves 10^{-3}/inches Ten. Side	Type of Failure
A - 1	8.7	.28	4,568	8,137	.62	none	-	2.64	-	Tension
B - 1	8.9	.29	5,037	9,037	.60	.70	.60	2.30	2.42	Tension (Cross-Grain)
C - 1	8.5	.29	3,396	4,046	1.20	-	none	-	2.56	Tension
A - 2	8.5	.40	6,321	12,054	.70	.52	.40	2.61	3.69	Buckling-Compression
B - 2	9.5	.42	6,547	13,333	.65	-	.60	-	2.72	Tension (Splintering)
C - 2	8.3	.40	4,851	10,231	1.10	-	-	-	-	Tension (Splintering)
A - 3	8.0	.34	5,437	8,372	.68	.67	.65	2.64	2.40	Tension
B - 3	9.5	.39	5,810	9,236	.66	.55	.55	1.90	2.56	Tension
C - 3	8.8	.39	3,995	4,795	1.00	none	.70	2.33	-	Tension
b - 1	8.6	.28	3,830	7,674	.11	-	.11	10.00	10.00	Tension
b - 2	8.4	.40	5,135	9,424	.11	.14	-	8.75	-	Tension (Splintering)
b - 3	8.1	.39	3,843	7,725	.10	-	none	-	12.10	Tension

5. RESULTS AND DISCUSSION

Variables used in the reinforced beams were orientation of steel strips, and beam length. Beam length variation was performed with beams of Type B cross section, and it was introduced in order to find out how the beam reacts when the span depth ratio (L/h) is decreased (h constant).

In nine beams the span was kept constant at 90 inches and the depth at 4.5 inches (L/h = 20) while the orientation of reinforcement was parallel to the neutral plane of the beam or at an angle of 90° with it (Types A and B). In the remaining three beams the orientation of the steel was kept constant in the form of Type B and the span changed to 39 inches (L/h = 8.66).

5.1 Stiffness and Load Carrying Capacities of Long Beams:

As noted on table 3 the stiffness values calculated from load-deflection curves, show Type A beams to be 2 to 2.7 times as stiff as Type C and Type B beams to be 2.2 to 2.9 times as stiff as Type C ones.

The higher values were observed in group No. 1. This group had a smaller modulus of elasticity for wood and the effect of the reinforcement on a percent type basis, was greater due to this fact (see Fig. 16). As indicated in Figure 16 and Table 3, there was up to 90 percent variation in increased stiffness values among the three groups; proving that no constant ratio exists between reinforced and unreinforced beams. However, a constant ratio does exist between Type A and

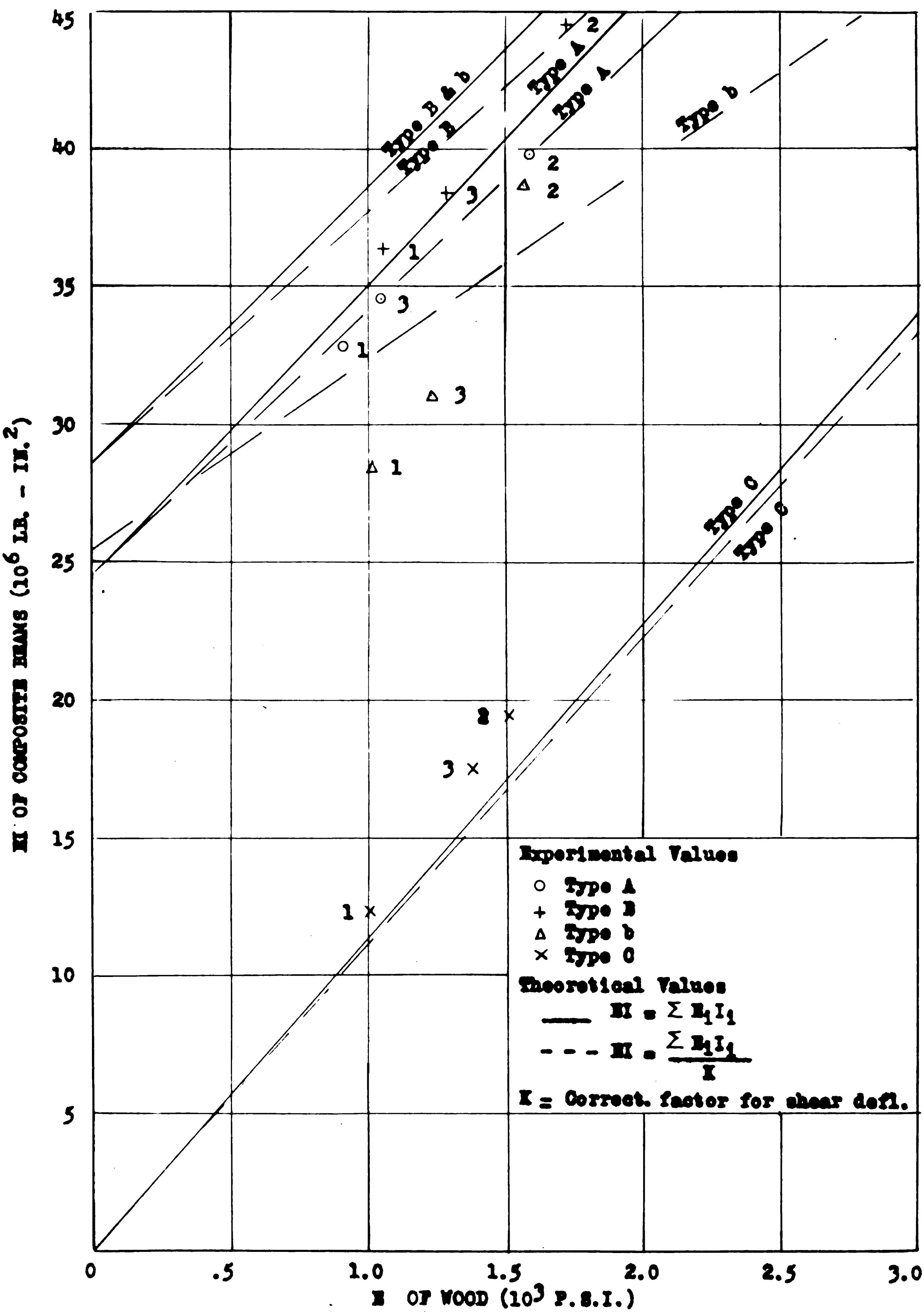


Fig. 16. Experimental and theoretical stiffness values of all beams

Type B. Type B values in all cases were 11 percent above Type A values. This 11 percent increase was very nearly constant in all three groups, with a negligible range of 10.5 to 11.8 percent.

Theoretical EI values of reinforced beams indicate a difference of 0.5 up to 4 percent as compared with experimental EI values, with one exception the B-3 beam which showed 7 percent difference. The unreinforced beams showed larger differences.

One factor that might have contributed to the differences can be the variation in ME values throughout the entire beam length. Modulus of elasticity from compression samples may not be representative of the entire beam. If B-1 and B-3 were closer in E values to A-1 and A-3 respectively, they would have fit their theoretical curves better. Constant ratios between stiffness values for Type A and B seems to support the idea that they were closer in the E value of the wood from which they were made than is indicated.

Also the placement of the steel with respect to the neutral axis of the beam, is less certain with Type B than with Type A; therefore, larger deviations from theoretical values would be expected with Type B. A change in steel location with respect to the beam centroid of 1/16 inch would cause a large change in actual EI.

Another factor might be a possible error in the formula used to adjust theoretical EI values for shear deflection.

The values taken for shear modulus of elasticity and Poisson's ratio were for Sitka spruce. In using these values, adjustment was made for moisture content and specific gravity by interpolation which might have introduced some error.

Shear deflection as percent of bending deflection indicates an almost constant relationship between Types A and B. This is obvious in all three groups with values ranging from 3.2 to 4.2 percent. In the unreinforced beams shear deflection is lower between 1 and 1.5 percent and there is a close agreement in the three groups. Even if in both cases, shear deflection was very small, as compared to total deflection, an increase from 1.5 to 4.2 percent between reinforced and unreinforced represent a 100 to 200 increase which is a significant factor.

Bending moments divided by a geometrical assigned section modulus ($bh^2/6$), calculated at proportional limit and at maximum load, showed an increase for the reinforced beams, over the unreinforced in all three groups. The increase in proportional limit was between 35 - 48 percent. Between A and B, Type B had 3 - 10 percent higher values. This was something that was expected to happen, because of the steel being further from the neutral axis. Calculations at maximum load showed an increase from 75 to 120 percent for the reinforced beams as compared with the unreinforced. The only exception was group No. 2 which showed a 20 - 30 percent increase. The unreinforced beam of group 2 when tested, to the point of failure, reached a deflection of almost 3.5 inches, and very high maximum load. Due to this fact, the calculated value for this beam was much higher. As a result, the difference between reinforced and unreinforced was smaller. Since this group had a high ME value for the wood, it seems that the higher the ME value the less the effect of the reinforcement on ultimate strength.

The percentage of increase in the apparent bending moment between proportional limit and at maximum load was almost constant for Type A and B of the same group. Group No. 1 showed 78 percent increase for A and 79 percent for B, while group No. 2, 90 percent for A and 100 percent for B, and group No. 3 54 percent for A and 59 percent for B. The unreinforced beam of group No. 1 and 3 showed an increase of 20 percent and in group No. 2 a 110 percent increase. This increase in the last group was probably partly due to the above explained reasons.

Predictability of ultimate beam strengths in reinforced members is indicated by the constant values of $M/bh^2/6/EI$ (see Table 4) for the reinforced beams cut from a single board. That a variation of these constants occurs between boards suggests that a wood property such as E is also an important factor. It might be possible with more testing to define ultimate strength of reinforced beams as a function of beam stiffness and wood stiffness.

Proportional limits determined from load-deflection curves demonstrated clearly the effect of reinforcing in the beams. The load at proportional limit was increased while the deflection at which the proportional limit took place decreased. As the strain-deflection curves indicate, the proportional limit was reached sooner in the tension side. For some reasons, strain gauges did not work as they were expected during the tests. In some cases gauges on the tension sides indicated compression and vice versa. For the gauges which worked satisfactorily, a trend was observed after they reached the proportional limit. This was illustrated as a jump upward, and then continuation in almost normal way for a certain deflection interval, with the same phenomenon appearing again. It seemed that after the proportional limit was reached, the curve was reversed with the concave upward, It is not easy to explain why this happened. Rather a stress concentration could cause this phenomenon but it is more probable that the center part of the beam (between loading points) did not follow the bending in the same proportion, and a buckling occured, transfering stress from steel to wood. Later in describing the failure of the beams, we give some reasons which probably are connected with this phenomenon.

5.2 Stiffness and Load Carrying Capacity of Short Beams:

So far the discussion was concerned with the nine long beams. As it was mentioned in previous pages three beams of shorter length were tested. These beams were manufactured according to Type B, but of shorter span. The other dimensions were kept constant so that the effect of the reduced span over height ratio, can be demonstrated. The following discussion concerns the two categories of beam B and b (b = short span beams). Data obtained for these beams are shown along with the other beams on tables 3, 4, and 5.

The short beams were 15 - 27 percent less stiff than the long beams, as a result of greater shear deflection in the wood.

Theoretical EI values showed a deviation from 12 - 17 percent over practical, in beams of No. 1 and 3 with a difference of 3 percent under practical, indicated in No. 2 beam.

Several factors may have contributed to these differences. One of the more important of these was an inexact knowledge of the shear modulus of elasticity for each test member. Also, the validity of the shear deflection correction factor which was used has not been adequately determined for beams with small span-depth ratios. In addition, stress concentration factors were more critical in the short span beams, than in the long span ones because of the closer arrangement of the loading points in the short span beams. Another factor would have been the modulus of elasticity used. Average ME values in compression parallel to the grain were used for the short beams in calculating theoretical stiffness values, with a tendency of taking a value closer to that of the neighboring beam. Finally, location of the centroid of the steel reinforcement with respect to the centroid of the beam cross-section varied more in Types B and b than in Type A, because of the orientation of the slots for receiving the steel and the fact that sometimes the steel did not fit evenly along the bottom of the slots. Differences in distances between centroids of a few hundredths of an inch change EI values very markedly.

The shear deflection (as a percent of bending deflection) increased up to 17 percent in the short beams, with a very close agreement among the three beams.

Apparent bending moment at both proportional limit and maximum load were less for the short beams than the long ones, but the proportional limit was reached at a lower amount of deflection in the short beams.

5.3 Stresses in Beams:

Comparing stresses, the maximum bending stress was lower in the short beams in both wood and steel parts.

As it was expected shear stresses were much more higher than in the long beams. They were approximately 1.5 to 2 times greater and apparently a significant factor in ultimate failure.

The weighed ratio of load carrying capacity over experimental stiffness values showed that a constant relation existed between the two types of reinforced beams. This suggests that the increase in the load carrying capacity, both at proportional limit and at maximum load, is directly proportional to their stiffness values.

5.4 Failure

One thing that was a point of consideration in the designing of the beams proved to be a critical point. This was the placement of the steel strips in grooved lines and the resulting stress concentrations at these locations when the beams were loaded. Almost in all cases the failure started from the region of the steel placement. In the Type B beams, long and short, the failure was very similar in all beams. They failed in tension with a starting point at the outer part of wood in the tension side. This resulted in the loosening of the steel part in the wood, and consequently failure in the rest of the wooden part.

Type A-1 and A-3 failed in tension with the starting point in the bottom wooden surface below the steel strip. But as soon as this part failed, the break continued in an almost horizontal line toward both ends of the beams. (Shear or tension perpendicular to the grain.) Only in A-2 beam a buckling of the steel on the compression side was noticed and the beam failed in compression. This buckling failure was the only one in the entire test.

According to the calculations of stress in the wood of the reinforced and the unreinforced beams, the wood in the reinforced beams failed at a much lower stress value than did that in the unreinforced ones. One way to account for this apparent anomaly is that stress concentrations were introduced at the edges of the slots made for the reinforcement, and these contributed to wood failure at comparatively low stresses. Consideration must also be given to the fact that stress distributions are different in reinforced wooden beams after a certain point of loading has been reached: the reinforcement increases the proportional limit stress of the wood on the compression side over the value that would be found in an unreinforced wood beam.[11] The failure of the beams at close to the calculated yield stress of the steel may also have played an important role in ultimate strength of the reinforced beams.

Another explanation of the failure can be that initial failure was in the adhesive. Load at critical cross section was transfered to wood and the wood failed from the increased load.

As the data in table 4 show, the steel reached its maximum stress value at a level equal to its yield point stress. It proved that for steel thickness of 1/8 inch when combined with wood, a 4.5 inch beam depth is sufficient enough to withstand failure prior the yield point of steel is reached.

6. CONCLUSION AND SUMMARY

Reinforced wood beams showed a greater stiffness and load carrying capacity than the unreinforced beams. Greater increase was shown in Type B beams, where the steel reinforcement was located further from the neutral axis.

Stiffness values increased more than 100 percent and a close agreement was noted between experimental and theoretical values, except when shear deflection was a large part of total deflection.

For the span depth ratio (L/h) of 20, shear deflection was 4 percent, or less, of total deflection. For the L/h of 8.7, shear deflection was a considerable part of the total deflection ranging from 16.6 to 17.7 percent.

In general, stiffness factors for the reinforced test beams with a span-depth ratio of 20 were more predictable than were those of the unreinforced beams or of reinforced beams with a span-depth ratio of 8.7. In the longer reinforced beams a major part of the stiffness was determined by the steel properties. In the other beams wood properties were a major determining factor of stiffness.

Load carrying capacity at proportional limit was increased, but due to the effect of reinforcement it was increased more at maximum load.

The proportional limit was reached at a smaller amount of deflection in all reinforced beams, and there was a constant relationship between reinforced and unreinforced in the amount of the decreased deflection at proportional limit.

A relationship was evident between load carrying capacity, beam stiffness, and ME of wood for each beam. Load carrying capacity was apparently a function of beam stiffness and E of the wood. Beam stiffness was a function of E of the wood, and also shear deflection of the wood.

Further observation showed that the percentage increase in stiffness, accomplished by steel reinforcement, decreases as the E of the wood increases.

A strong bond of steel to wood was made for the reinforced beams. There were no delaminations during the testing until the failures of the beams, which occured at the yield-point of the steel, or slightly above this point.

In general a number of advantages were gained by reinforcing the beams:

a) Increase in stiffness.

b) Increase in load carrying capacity at proportional limit and maximum load.

c) Reduction in the variability of wood from place to place and among beams of the same species with greater predictability of load carrying capacities.

These first two advantages were better attained with Type B beams. In these the reinforcement was placed closer to the top and bottom of the beam surface, with consequent increase of the strength and stiffness. In Type A beam the strength and stiffness properties were lower as compared to Type B, but they were easier to predict than in the Type B beams.

There were also some disadvantages in reinforcing the beams. One was that reinforcement increased the amount of shear deflection, and two, the wood of the beam was not utilized to its maximum strength due to the nature of reinforcing material and the stress concentrations around it. Different geometries of reinforcement and methods of inserting it might alleviate these problems.

In summary, both Types A and B had some advantages and disadvantages. Type A was easier to manufacture, the strength properties were more predictable, and shear deflection was lower. The disadvantages were: less efficient use of reinforcement, and lower strength and stiffness properties. Type B showed higher strength and stiffness values, and the reinforcement was utilized more efficiently. Its disadvantages were: it was not easy to place the reinforcement at the slots as they were designed, strength properties were less predictable, and the percentage of shear deflection was higher.

Some problems to be faced in future work of this type are:

1) Relationship between specific gravity and increase in stiffness and other properties.

2) Placement of the reinforcement in regard to the distance from top and bottom or sides of the wood beam.

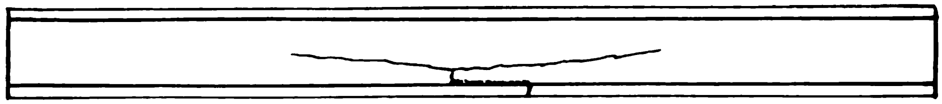
3) The cross section of the steel.

4) Attaching of reinforcement to wood.

5) Predicting ultimate strength of beams.

The economical aspect of the problem for the part of the reinforcing material is not as serious as it appears to be. The amount of wood necessary to achieve the same strength values, cost more than the steel necessary to substitute it, besides the other advantages gained from wood-metal combination. The stiffness of a wood beam can be increased considerably by the addition of a relatively small amount of reinforcing material.

APPENDIX



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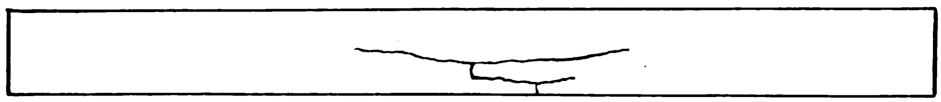

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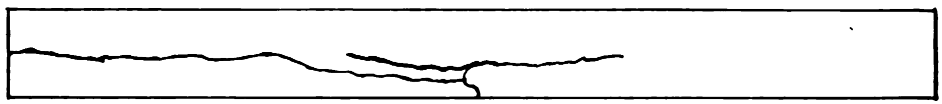
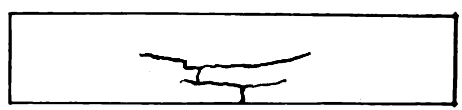
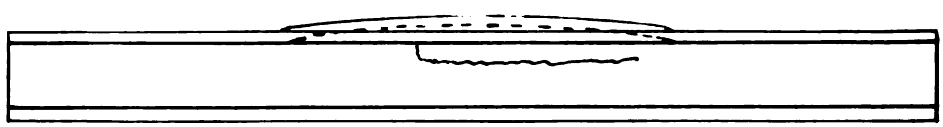


C - 1

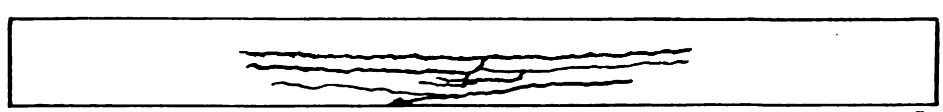


b - 1

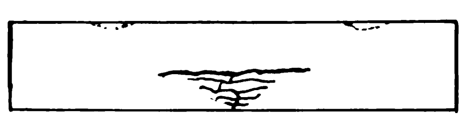


A - 2

B - 2



C - 2



b - 2

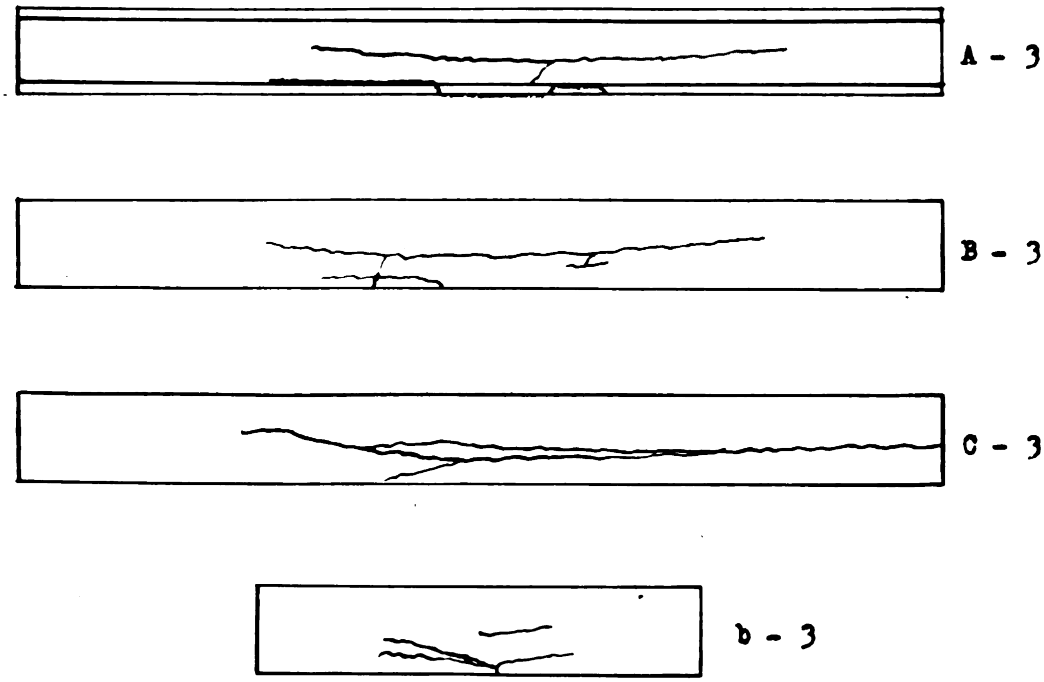


Fig. 17. Types of Failures in Static Bending

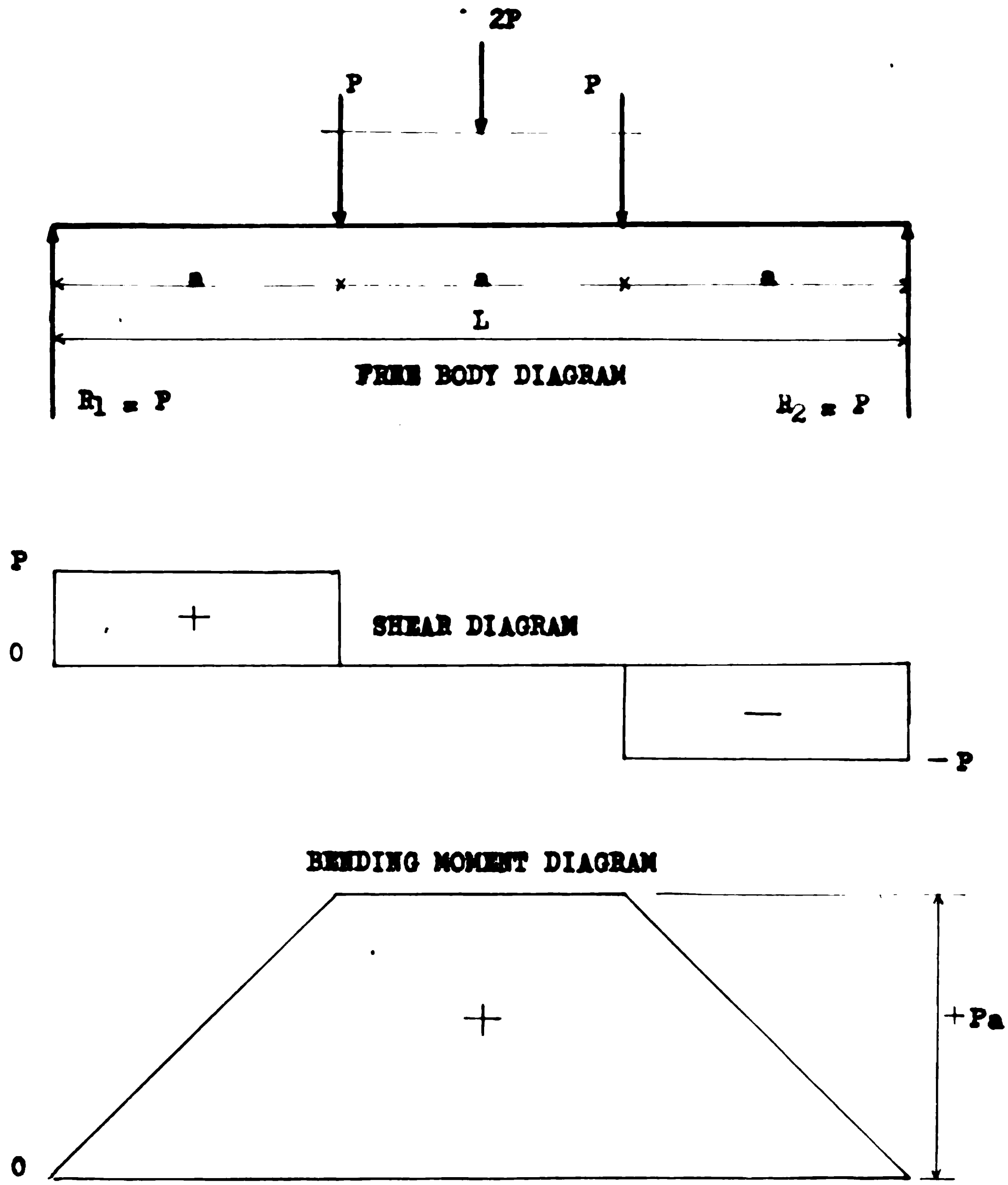


Fig. 18. Shear and Bending Moment Diagrams

Theoretical EI values for constant cross section (1.5 by 4.5 inches) with E of wood as variable (see Fig. 16).

Type A: $EI = E_w I_w + E_{st} I_{st}$

$= 10.57E_w + 30 \times 10^6 \times 0.82$

$= 10.57E_w + 24.6 \times 10^6$

Type B: $EI = 10.451E_w + 28.2 \times 10^6$

Type b: $EI = 10.451E_w + 28.2 \times 10^6$

Type C: $EI = 11.39E_w$

Correction factor K for Shear Deflection.

Type A: $K = 1 + 11 \times 10^{-9} E_1$ *

Type B: $K = 1 + 10 \times 10^{-9} E_1$

Type b: $K = 0.995 \times 60 \times 10^{-9} E_1$.

Type C: $K = 1 + 10 \times 10^{-9} E_1$

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* $E_1 = \dfrac{E_i I_i}{I}$ where I = Inertia of Gross Dimension.

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