AN INVESTIGATION OF THE MECHANICAL PROPERTIES OF NAIL-GLUED WOOD-PLYWOOD TRUSSED GIRDERS

Thesis for the Degree of M. S.

MICHIGAN STATE UNIVERSITY

Richard M. Voelker

1961

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by

Richard M. Voelker

AN ABSTRACT

Submitted to the College of Agriculture
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

1961

Approved [signature]

ABSTRACT

The purpose of this study was to determine the mechanical properties of nail-glued, wood-plywood, trussed girders, and to compare these with conventional nail-glued, wood-plywood girders.

A number of model beams of each type were constructed using four section depths. The deflection of these beams was measured under static bending loads and the empirical data gathered was utilized in conventional engineering equations to calculate the strength and stiffness properties of the beams. Similar tests were conducted for both types using full-scale beams of the same section depth and span.

Stiffness factors (a function of the modulus of elasticity and section properties) were calculated for the model beams and it was found that the Type-A beams (with diagonal stiffeners) were as much as 40 per cent stiffer than the Type-B beams (with vertical stiffeners only). This superior stiffness was also reflected in the full-scale testing where the Type-A beams were found to be 16 per cent stiffer than the Type-B beams.

All the model beams failed due to horizontal shearing stresses in the plywood web. This failure occurred at a load far in excess of that producing the allowable design deflection at mid-span (generally accepted as being 1/360th of the span). The horizontal shearing stresses in the plywood web, calculated at failure, were many times greater than the allowable design horizontal shearing stresses for plywood as given by the Forest Products Laboratory and the Douglas Fir Plywood Association.

Because of this, it was concluded that the allowable design deflection of 1/360th of the span should be used as the governing design factor, and not the allowable design horizontal shear stresses recommended for the plywood web.

Buckling of the plywood web became more critical as the model beam depth increased. It was concluded that the test apparatus must be modified to prevent lateral buckling in future tests, in order to obtain valid test results.

It is recommended that strain gauges be utilized in future testing of trussed girders to determine the amount of truss action within the diagonals.

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ACKNOWLEDGMENTS

The author wishes to express his sincere thanks to Professor Byron Radcliffe, under whose inspiration, supervision and unfailing interest this study was undertaken.

The author is also greatly indebted to Dr. Alan Sliker for his guidance and assistance in editing the manuscript. Sincere thanks is also due Mr. Harry Johnston for help in fabricating and to Mr. H. F. Law for his willing assistance in fabricating and testing.

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AN INVESTIGATION OF THE MECHANICAL PROPERTIES OF NAIL-GLUED WOOD-PLYWOOD TRUSSED GIRDERS

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INTRODUCTION

History of "Built-up" Plywood I-beams

The use of the "built-up" or laminated structural wood members was first conceived in the early 1900's in Europe with Otto Hetzer of Weimar, Germany being credited as the originator.[11*] Although Hetzer construction, as it was known then, dealt mostly with laminated arches, the advantages this method offered were soon realized and adapted to use in laminated beams with rectangular, I and double-I cross-sections.

It was soon discovered by Stoy, Egner and Erdmann that the use of the I-beam section resulted in a savings of 35 per cent or more in construction materials when compared to conventional rectangular sections. In 1937 Wille published design information which enabled engineers and builders to determine the required dimensions of wooden I-beams for various support and load conditions. In most of these earlier instances the adhesive used to laminate these beams was casein glue.[11]

There was little interest in this country in built-up beams until shortly after the First World War. In 1919 the National Advisory Committee for Aeronautics sponsored a number of investigations on the use of wood. One of these projects dealt with the use of laminated beams in airplane construction. These beams were constructed by gluing pieces of wood together and then machining the assembled beams to the desired I cross-sectional shape.[11] The emphasis in this application

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*Literature cited in Bibliography

was to provide adequate strength where needed and to reduce the amount of material and weight.

The earliest structural use in a building reported in the United States for plywood I-beams was in 1942. The RCA Manufacturing Company of Camden, New Jersey had a 125,000 sq. ft. warehouse constructed which utilized 198 plywood girders 36 feet long. These girders were fabricated on the building site. Cement-coated nails were used to attach the plywood webs to the lumber flanges. When the government occupied the warehouse in 1952, after ten years of service, the beams were found to be in excellent condition, had not sagged and had not required any maintenance.[9]

With the advent of the Second World War, and the resulting shortage of solid structural materials, interest in laminated wood products was further stimulated. The Forest Products Laboratory of the United States Department of Agriculture, under the supervision of the Aeronautical Board, was charged with the responsibility of formulating design equations, substantiated by test data, that would facilitate the use of plywood and built-up sections in structural members. In 1943-1944 the Forest Products Laboratory conducted extensive studies on plywood box-beams and I-beams.[5,6,7] Based on these studies recommendations were made concerning face-ply grain orientation, stiffener spacing, buckling and cross-sectional design. It was found that significant increases in web shear resistance were obtained by reducing the spacing of the stiffeners; and it was recommended that the minimum stiffener spacing, compatible with economy, be used.[5] The conclusion was made that I-beams generally use plywood web material more efficiently than box-beams. Test results indicated that for equal panel sizes and section properties, an I-beam with a single web was usually significantly stronger in shear than a box-beam with two webs, each half as thick as the single web of the I-beam.[6] It was further demonstrated that box-beams or I-beams having the face grain of the webs at an angle of 45 degrees with the axis of the beam were more efficient than those with either 0 or 90 degree grain orientation. It made little difference whether the grain orientation was vertical or horizontal as the ultimate shear stresses were nearly equal.[7] Buckling of the plywood web proved to be a critical design factor.[7] A portion of the research done for the National Aeronautical Board concerned shear deflection. It was found that shear deflection was an important consideration in designing plywood I-beams and box-beams. The magnitude of the deflection attributable to shear was found to be inversely proportional to the unsupported length of span. Formulae were derived by which shear deflection could be calculated.[12]

After the War was concluded, further research by the U. S. Forest Products Laboratory was conducted to obtain data on the effect of variations in lamination thicknesses, joints within the laminations and the location of various size and types of defects on the strength and failure characteristics of plywood beams.[11] There was also published at this time, methods for calculating the strength and stiffness of plywood and suggested working stresses for plywood design.[10]

Based on the pioneering efforts made by the Forest Products Laboratory, other publications soon appeared from various sources.

Because of the vital interest the Douglas Fir Plywood Association has in built-up construction, it soon published a design handbook which presented to the engineer and architect useable formulae and design criteria.[2] The DFPA also published a set of design specifications embodying the latest design procedures and methods for plywood I-beams and box-beams.[3]

With the proven use and general acceptance of plywood I-beams as structural members, new methods were developed to facilitate fabrication. Nailing proved a practical means to secure adequate glue line pressure while fabricating.[14,17] With the advent of "nail-gluing", the plywood I-beam became practical enough to be used in residential construction.

While a significant amount of work has been done on built-up plywood beams, the incorporation of diagonals to produce truss action is an idea which merits further investigation. The alteration of the basic plywood I-beam design in such a manner, indeed opens a relatively new area for plywood beam research.

Purpose of the Study

The purpose of this study was to evaluate the strength and stiffness properties of plywood I-beams with diagonal stiffeners as compared to conventional plywood I-beams with vertical stiffeners.

Model beams were constructed and tested in an attempt to determine (1) if there was any increase in stiffness resulting from the use of diagonals; (2) what, if any, relationship existed between the beam depth and the strength and stiffness properties of the beams; and (3) to gather empirical information on the type of beam failures encountered, the loads at failure and the behavior of the two types of beams when loaded in excess of the proportional limit.

Full-scale beams were constructed and tested in an attempt to verify the stiffness properties observed in the model beams. Because of limitations of the testing apparatus used, no information could be gathered on the proportional limits of the full-scale beams or the type of failure.

FABRICATION

Model Beams

Sixteen, eight foot span model beams were constructed. Of these, eight were Type-A and eight were Type-B. (Fig. 1). Within each group of eight, two beams were constructed with a nominal section depth of eight inches, two with a nominal depth of nine inches, two with a nominal depth of ten inches and the remaining two were a nominal eleven inches deep.

The flange, diagonal and stiffener members were constructed of No. 1 grade Douglas-fir and western hemlock. Grade A-A exterior type, 1/4 inch, sanded Douglas-fir plywood was used for the webs. The nominal 1 x 2 inch members were cut from 2 x 6 inch stock and surfaced. All the lumber used contained the typical defects found in construction grade lumber. However, those members containing a great number of defects were restricted to use as vertical stiffeners or diagonals. The per cent moisture content of the 1 x 2 inch members was determined at the time of fabrication and was found to vary from 6% to 9%.

After the components were cut to size, they were ready for assemblance. The assembly process was essentially the same for both types of model beams. The 1 x 2 inch members were first laid-out and tacked together. Next, casein glue, meeting U. S. Specification MMM-A-125 and mixed in accordance with the manufacturer's specifications was brushed on the 1 x 2 inch members. The plywood web was then lightly tacked to this framework. The 1 x 2 inch members of the opposite side of the beam were then spread with glue and set in place.

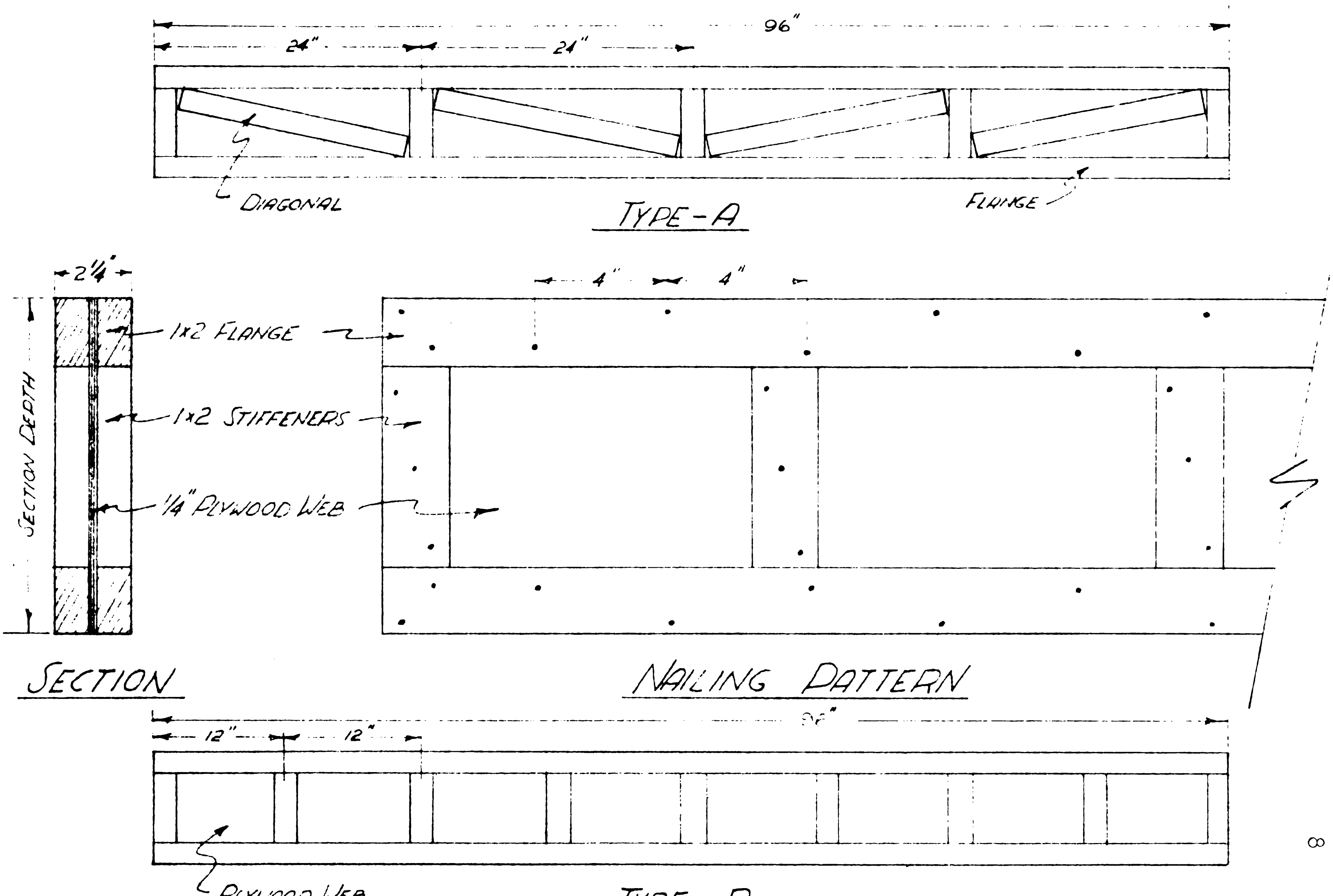


Figure 1

MODEL BEAM DETAILS

Number 4d rosin coated box-nails were used to nail the entire assembly together. The I-beams were nailed from one side only in the pattern illustrated in Figure 1. The nails were used as a means of securing glue line contact. They were driven hard in order to produce a glue "squeeze-out", which was taken as a visible indication of adequate glue line pressure.

The beams were carefully stacked after fabrication to prevent any permanent deformities while the glue was curing. The minimum curing time was 24 hours at an approximate temperature of 70 degrees Fahrenheit.

Full-Scale Beams

Ten full-scale beams 16 feet in length were constructed. Of these, five were Type-A and five were Type-B . (Figures 2, 3, and 4) All beams were 16 inches deep.

The nominal 2 x 4 inch flange, diagonal and stiffener members were of No. 1 grade ponderosa pine, while the plywood web was of C-C grade, exterior type, 3/8 inch Douglas-fir. The 2 x 4's were used as delivered without further surfacing. There was no attempt made at selecting members, with the exception that those pieces containing a greater number of defects were restricted to use as stiffeners or diagonals. The moisture per cent of the 2 x 4 inch stock was found to vary from 9% to 10% at the time of fabrication.

The fabrication process for the large beams was similar to that of the model beams with the exception that the 3/8 inch plywood webs were fabricated in three pieces; an eight foot section and two four foot pieces.

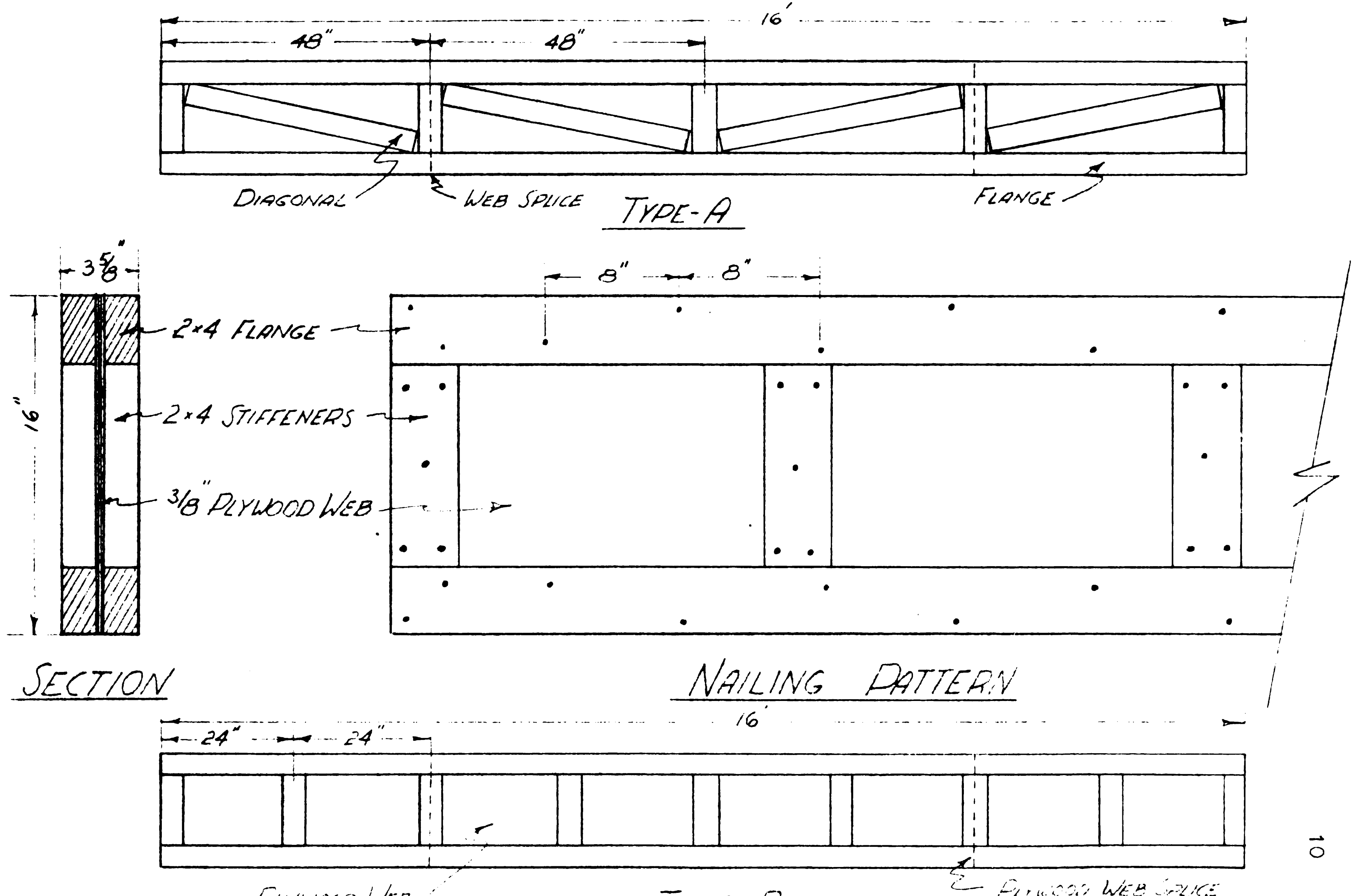


Figure 2

FULL-SCALE BEAM DETAILS

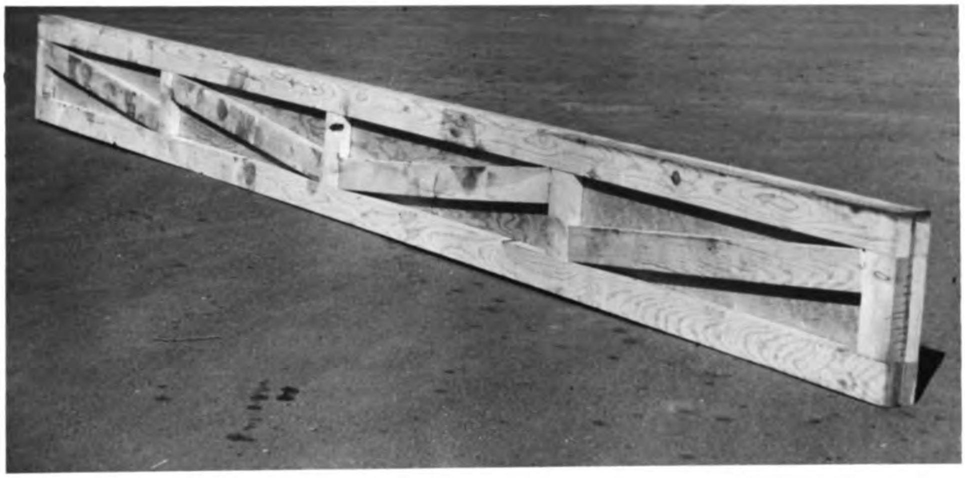


Figure 3
Full-Scale Type-A Beam

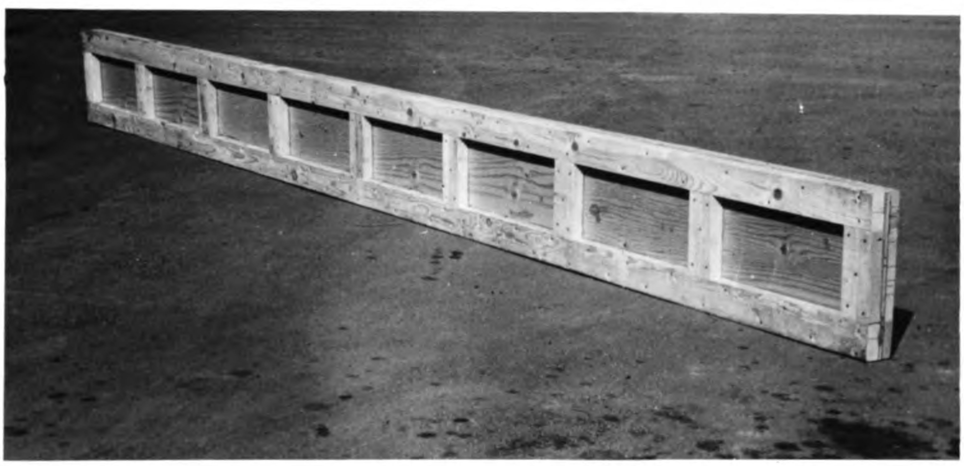


Figure 4
Full-Scale Type-B Beam

These were assembled in such a manner as to result in having two web joints, each being four feet from either end of the beam, with the respective 2 x 4 inch vertical stiffeners serving as splice plates, as illustrated in Figure 2. Because of the increased thickness of the section, 16 d common nails were used for nailing.

After fabrication, the beams were carefully stacked and allowed to cure for a period of 24 to 36 hours before testing.

TESTING

Model Beams

The model beams were tested in a 100,000 lb. Reihle Universal Testing Machine. The load was applied at two points, each being two feet from the supported ends of the beam, at a constant deflection rate of 1/16 inch per minute. The method of testing and the apparatus used is illustrated in Figures 5 and 6.

Deflection readings were taken at the neutral axis at mid-span. Readings were taken to the nearest .001 inch, using an Ames dial gauge, for every 200 lbs. of increased load up to the point of beam failure. The load at failure was noted, as was the type of failure; and any non-conformities resulting from the applied load, such as buckling or twisting, was also noted. Figure 7 is an illustration of buckling found to be typical in the beams of deeper sections.

Upon the completion of the static bending tests, the beams were disassembled and eight inch compression specimens were taken from the upper or compression flange and from the lower or tension flange. (Fig. 8) These specimens were tested in compression parallel to the grain in accordance with ASTM standards.[1] An empirical modulus of elasticity of the flanges was calculated using the load vs. compression data collected. The formulae used and the results obtained are covered in detail in the analysis of data.

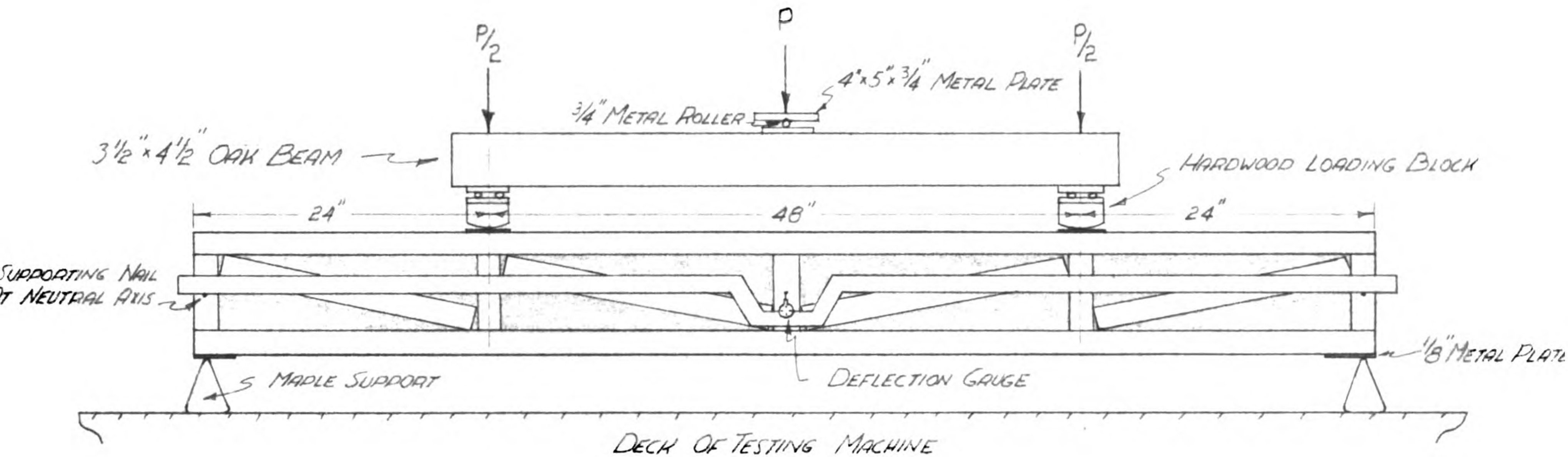


Figure 5

MODEL BEAM TEST & APPARATUS

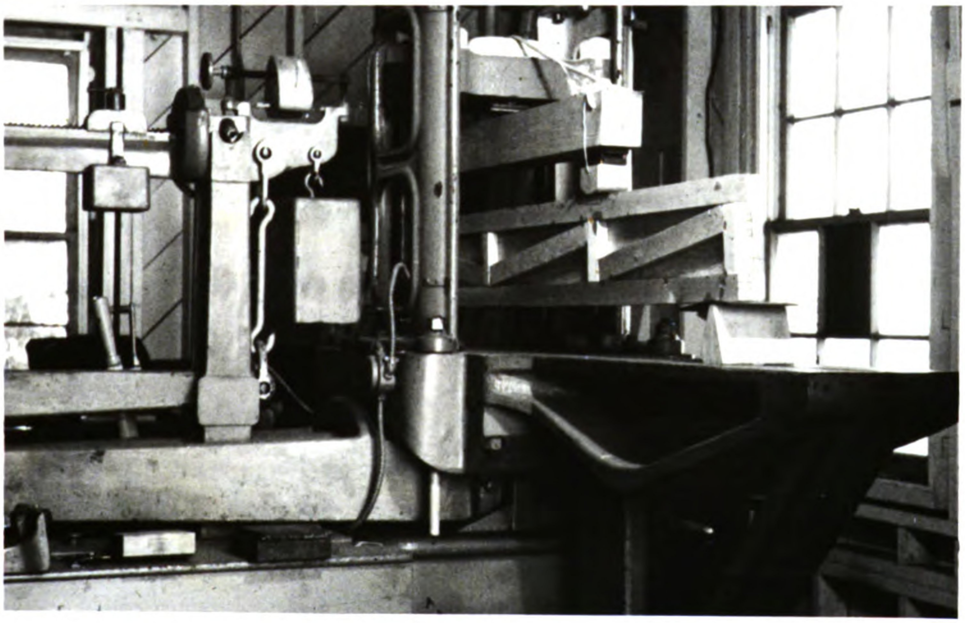


Figure 6

Model Beam Testing

Figure 7

Model Beam Buckling Under Load

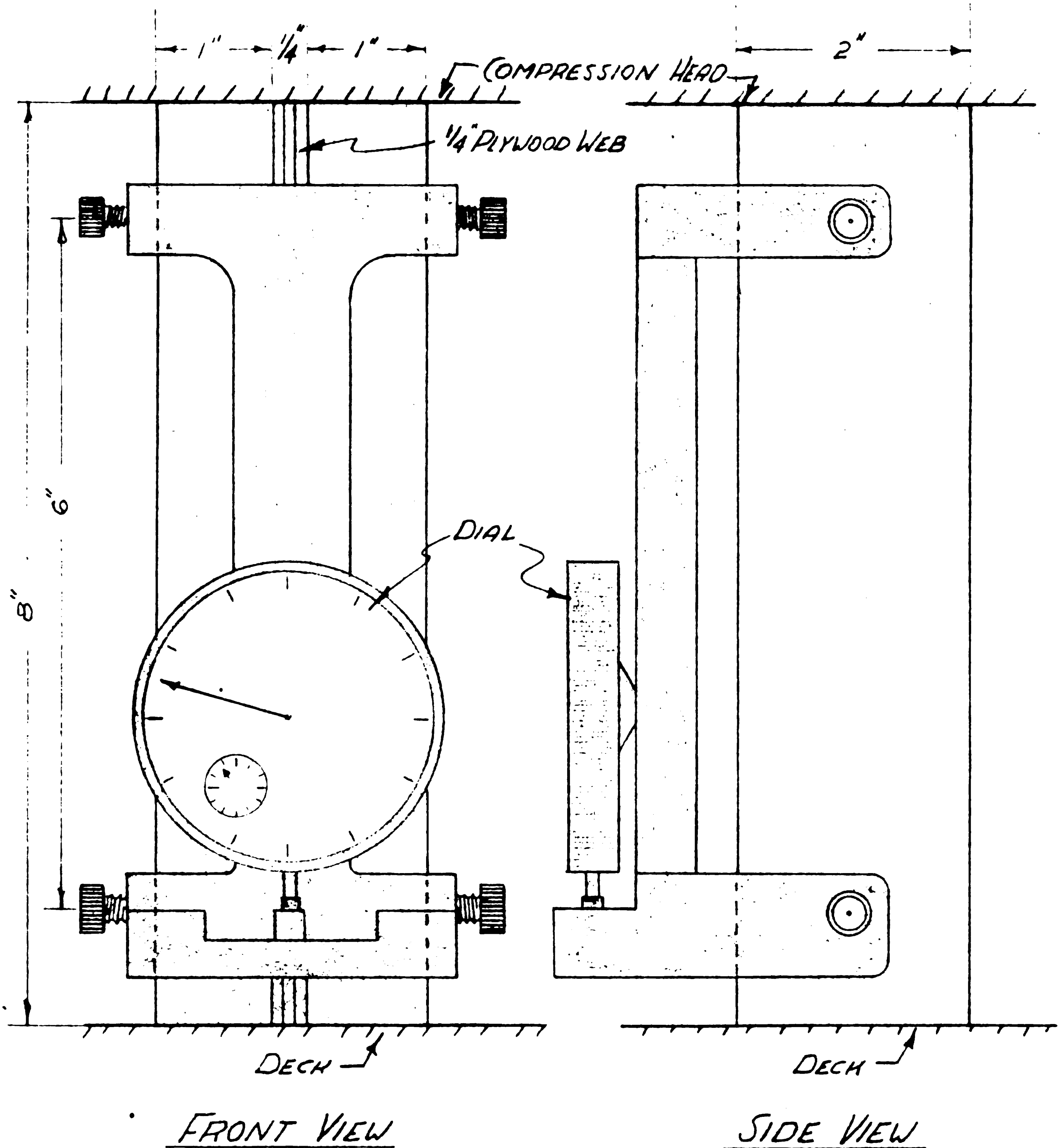


Figure 8

8" FLANGE SAMPLE COMPRESSION TEST

Full-Scale Beams

The ten full-scale 16 foot beams were tested using a hydraulic testing floor. This is shown in Figure 9 and is graphically illustrated in Figure 10. A hydraulic cylinder was located every two feet along the length of the beams with the load being applied directly over the stiffeners. There was no load applied over the two end supports. The cylinders were connected in series to assure uniformity of pressure. The load was applied in increments of 25 p.s.i. up to 400 p.s.i. of cylinder ram, the maximum output of the testing machine. The piston head area was 2.94 in.^{2}, thus, each p.s.i. represented 2.94 lbs. per cylinder.

Deflection readings at mid-span were taken for each increase in load of 25 p.s.i. An Ames dial deflection gauge was placed against the lower flange and the readings taken to the nearest .001 inch directly as the beam deflected under load.

The beams were restrained from lateral buckling by means of metal straps secured to the test floor. Steel rollers and blocking were used to assure freedom of movement of the beams as they deflected. (Fig.11)

Because the capacity of the hydraulic system was limited to 400 p.s.i. of ram per cylinder, it was impossible to gain information or data as to the proportional limit of the beams or the type of failure that would have occurred. However, the allowable deflection (1/360th of the beam span) of .533 inches was exceeded in each instance and the data obtained gave a good relationship between load applied and deflection measured. This was sufficient to make a stiffness comparison between the two types of beams.

Figure 9

Hydraulic Testing Floor

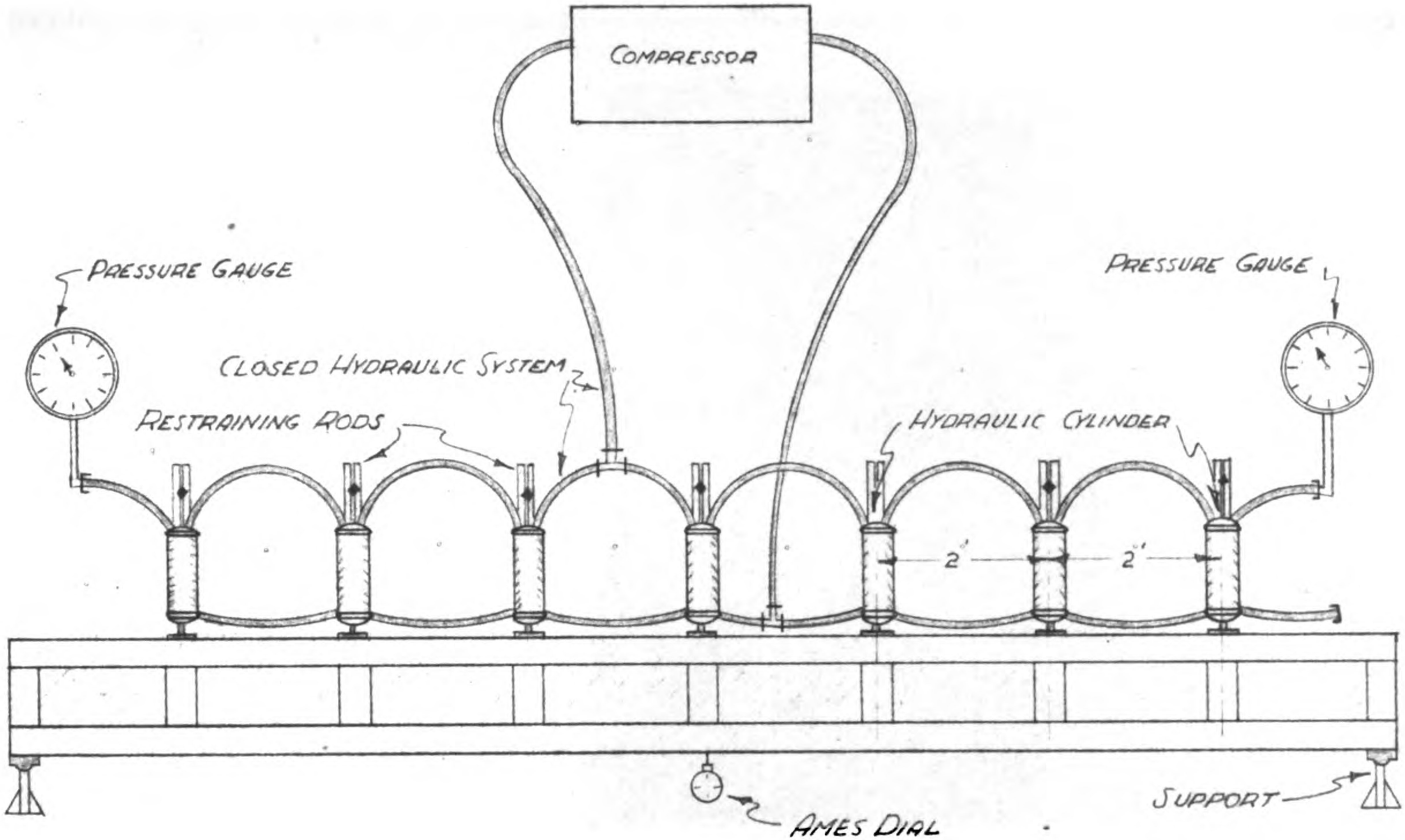


Figure 10

FULL-SCALE BEAM TEST & APPARATUS

Figure 11

Metal Rods and Blocking

Restraining Buckling

ANALYSIS OF DATA

Model Beams

Beam Test Performance

The data obtained from the static bending tests was used to plot a load vs. mid-span deflection curve for each of the 16 model beams tested. A composite plot is shown in Figure 12 with the average curve for each beam type superimposed.

Loads producing the allowable deflection (1/360th of the span), the proportional limit and failure in each beam are recorded in Table I. The type of failure occurring for each beam was also noted.

Shear Deflection

Because a gross deflection was measured at the time of testing, it was necessary to compute the amount of deflection attributable to shear in order that the amount due to bending alone could be found.

The formula used to compute shear deflection is that advocated by the Douglas Fir Plywood Association[3] and verified experimentally by the U. S. Forest Products Laboratory and is expressed as:

$$d_s = \frac{PlKh^2C}{GI} \qquad (1)$$

where:

d_s = shear deflection, in.

P = total load on beam, lbs.

l = span length, in.

K = a factor determined by the beam cross-section

h = depth of beam, in.

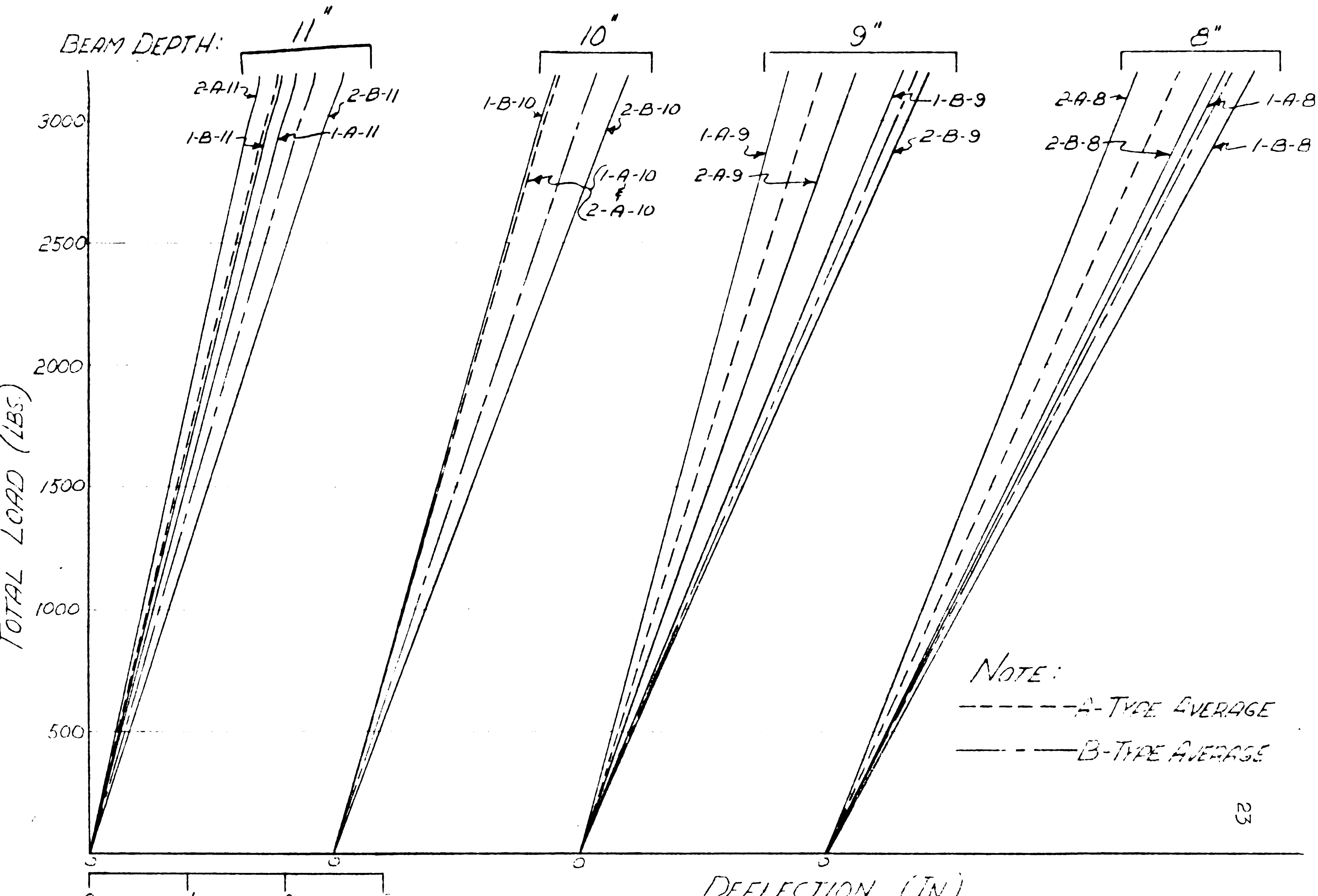


Figure 12

LOAD-DEFLECTION CURVES FOR MODEL BEAMS

MODEL BEAM PERFORMANCE

Beam No.	Load at Allowable Deflection (Lbs.)	Load at Proportional Limit (Lbs.)	Load at Failure (Lbs.)	Type of Failure
1-A-8	2100	2400	3300	Horizontal Shear
2-A-8	2700	2900	3700	" "
1-B-8	1970	2400	3700	" "
2-B-8	2190	2700	4100	" "
1-A-9	4060	4300	5200	Horiz.Shear & Buckling
2-A-9	3020	3900	----	Buckled*
1-B-9	2590	3400	4500	Horizontal Shear
2-B-9	2400	3600	4900	" "
1-A-10	3740	3500	4300	Horiz.Shear & Buckling
2-A-10	3680	3000	4200	" " "
1-B-10	3840	----	----	Buckled*
2-B-10	2820	3000	5300	Horiz.Shear & Buckling
1-A-11	4010	2800	----	Buckled*
2-A-11	4810	3400	6000	Horizontal Shear
1-B-11	4250	3800	4900	Horiz.Shear & Buckling
2-B-11	3300	3800	4500	Buckled

*Extreme buckling caused beam to "spring out" of testing machine

Table I

C = a coefficient depending on the manner of loading

G = the shearing modulus of the Douglas-fir plywood webs, determined empirically to be 117,000 p.s.i. if the face grain of the plywood is either parallel or perpendicular to the span at 15% moisture content,[3] p.s.i.

I = gross moment of inertia of the section, $in.^4$.

The calculated amounts of deflection attributed to shear corresponding to a given load for the various section depths are recorded in Table II. These shear deflection values were subtracted from the gross observed deflection corresponding to the total load on the beam (P) in order to arrive at the deflection due to bending alone, for that load (P). These corrected bending deflection values were used to calculate the stiffness factors (EI) for the beams.

Modulus of Elasticity

Using the data obtained from the compression tests of the flange sections, as previously described, the modulus of elasticity was computed using the equation:

$$E = \frac{P6}{A\Delta} \qquad (2)$$

Derived as follows:

$$\text{Modulus of elasticity (E)} = \frac{\text{stress } (\sigma)}{\text{strain } (\varepsilon)}$$

and:

$$\text{stress } (\sigma) = \frac{\text{load (P)}}{\text{area (A)}} \qquad \text{Modulus of elasticity (E)} = \frac{\text{deflection } (\Delta)}{6}$$

SHEAR DEFLECTION COMPONENT OF TOTAL

MEASURED DEFLECTION (MODEL BEAMS)

Nominal Depth (h) in.	Actual Depth (h) in.	K	Gross* I in.^{4}	Shear Deflection (d_s) in.
8	7.88	1.36	77.73	.089
9	8.88	1.37	105.86	.084
10	9.88	1.37	138.78	.079
11	10.88	1.34	176.61	.074

*Includes total thickness of plywood web

Table II

therefore:

$$E = \frac{P6}{A\triangle}$$

where:

E = modulus of elasticity, lbs. per $in.^2$

P = total load, lbs.

A = cross-sectional area, $in.^2$

$\triangle$ = deflection, in.

The empirical E-values calculated by using the above formula are reported in Table III (Appendix). These values are later used to calculate the stiffness factors (EI) of the beams.

Moment of Inertia

A theoretical moment of inertia was then calculated for the four beam sections using the formula:

$$I = \frac{t_1 d_1^3}{12} - \frac{t_2 d_2^3}{12} \qquad (3)$$

where:

I = moment of inertia, $in.^4$

t_1 = total flange thickness including two plys of the 1/4 inch plywood web. in.

d_1 = total depth of the beam section, in.

t_2 = total flange thickness, not including the plywood web, in.

d_2 = total depth of the beam section, minus twice the depth of the flanges, in.

These calculated I-values are recorded in Table IV and are used in the calculation of the theoretical stiffness factors.

THEORETICAL MOMENTS OF INERTIA

(Model Beams)

Nominal Section Depth (In.)	Actual Section Depth (In.)	I Values* ($In.^4$)
8	7.88	74.31
9	8.88	100.96
10	9.88	132.04
11	10.88	167.61

* Includes only two plys of the plywood web

Table IV

Stiffness Factor

Three comparative stiffness factors were next computed for each beam using the previously calculated data.

The first of these is termed the theoretical stiffness factor (EI) and was calculated by using the theoretical moment of inertia values recorded in Table IV. These I-values were multiplied by 1.95×10^6, the average modulus of elasticity for coastal type Douglas-fir at a moisture content of 12 per cent.[4] These theoretical EI-values are recorded in Table V.

The second group of stiffness factors or EI-values might best be termed the actual EI-values. They were calculated using the actual deflection recorded at the time of testing and they reflect the true or actual performance of the beams as loaded.

These values were computed from the equation:

$$EI = \frac{\sum A\bar{x}}{\Delta - \Delta_s} \qquad (4)$$

where:

E = modulus of elasticity, p.s.i.

I = moment of inertia, in.^4

$\sum A\bar{x}$ = the sum of 1/2 the individual areas located within the bending moment diagram shown in Figure 13, multiplied by the distances ($\bar{x}$) of the centroids of these areas from the left edge of the diagram, in.^3 (Sometimes referred to as the second area moment theorem)

Δ = total deflection measured at mid-span, in.

Δ_s = calculated shear deflection, in.

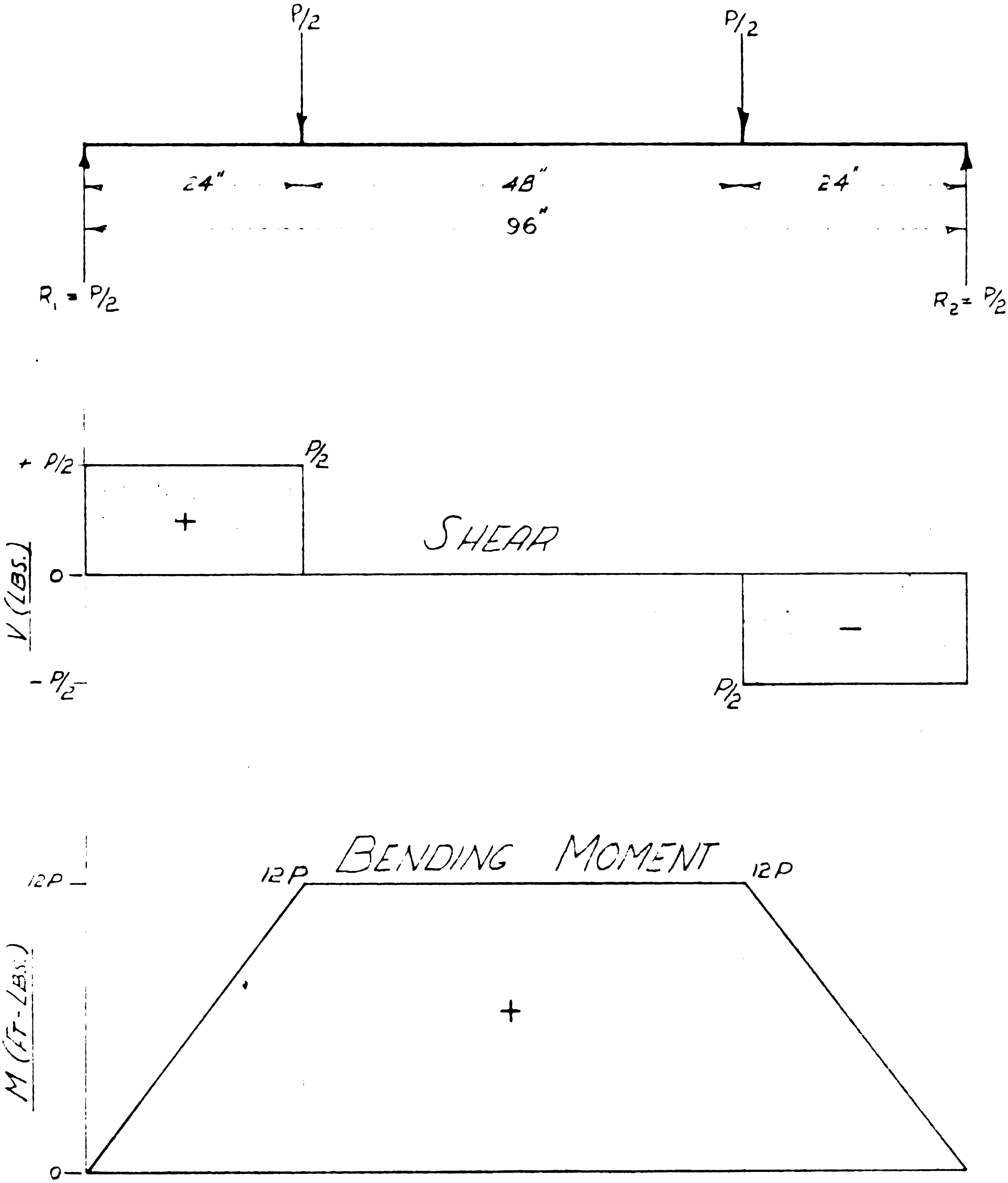


Figure 13

SHEAR & BENDING MOMENT DIAGRAMS FOR 1/4 POINT LOADING OF MODEL BEAMS

These EI-values are recorded in Table V.

The third and final group of stiffness factors calculated are based on the moduli of elasticity determined earlier from the eight inch compression samples taken from the beam flanges.

Because of the variability of these E-values, adjusted moments of inertia (I-values) were next calculated for each beam. These adjusted I-values would theoretically compensate for the variances encountered within each beam between the modulus of elasticity of the compression flange and that of the tension flange. This is generally referred to as the method of equivalent sections.

An equivalent thickness for the tension flange was computed based on the modulus of elasticity of the compression flange. This was accomplished by using the formula: (See Fig. 14)

$$b_2 = \frac{E_c\ b}{E_t} \qquad (5)$$

where:

b_2 = equivalent thickness of the tension flange member, not including the plywood web, in.

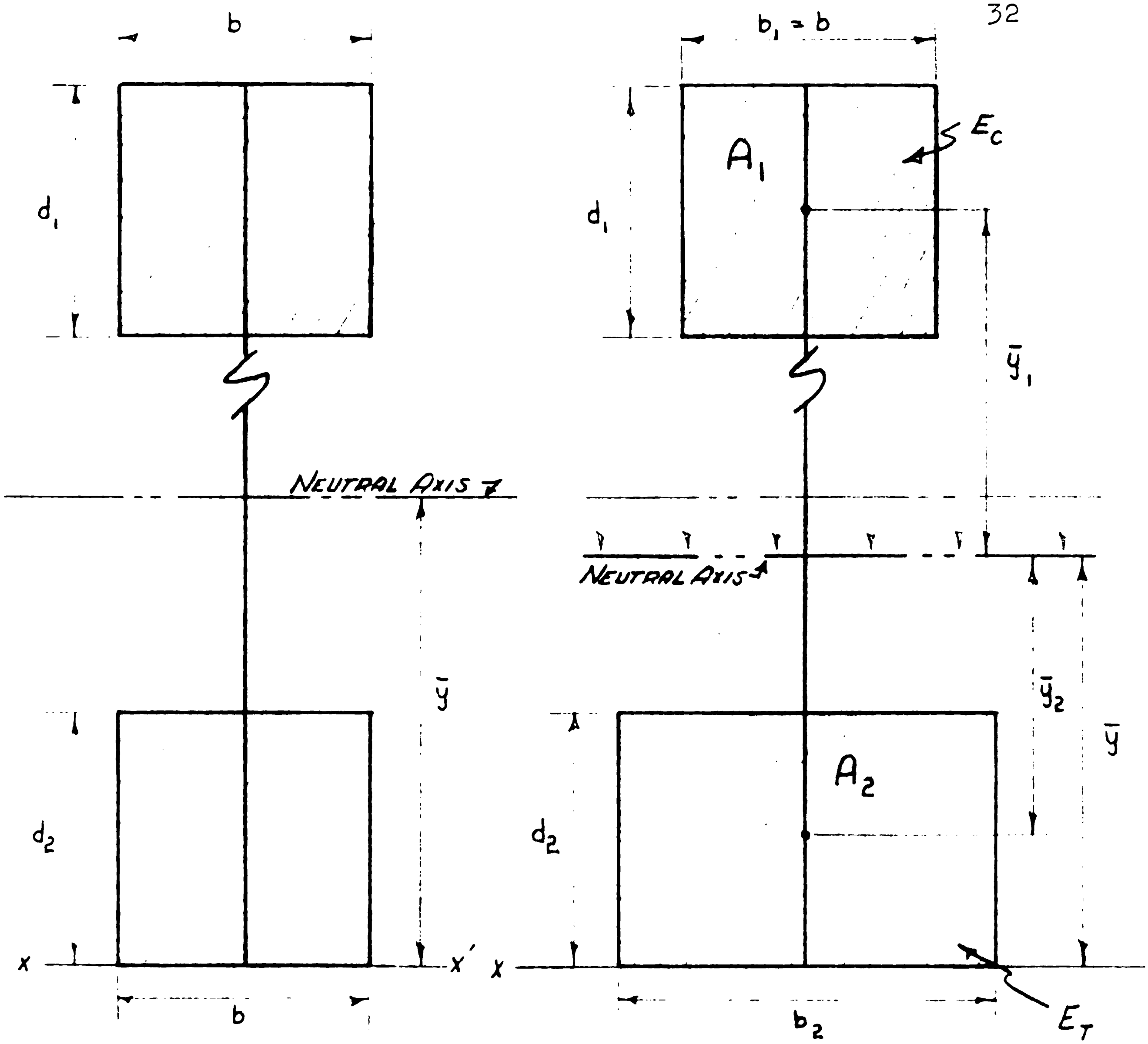
E_c = the modulus of elasticity of the compression flange, p.s.i.

E_t = the modulus of elasticity of the tension flange, p.s.i.

b = the actual thickness of the compression flange member, not including the plywood web, in.

Because of the change in section-geometry and the resulting shift of the neutral axis, (Fig. 14) a new theoretical location of the neutral axis had to be computed using the formula;

$$\bar{y} = \frac{\sum A\bar{y}}{\sum A} \qquad (6)$$



ACTUAL X-SECTION OF MODEL BEAM

THEORETICAL EQUIVALENT X-SECTION OF MODEL BEAM, ILLUSTRATING THE NON-SYMETRICAL GEOMETRY AND THE RESULTING SHIFT OF THE NEUTRAL AXIS

Figure 14

THEORETICAL EQUIVALENT X- SECTION

where:

y = distance to the neutral axis from the x-axis, in.

ΣAy = the sum of the areas of the flanges multiplied by the distance of their respective centroids from the x-axis, in.^{3}

ΣA = the sum of the cross-sectional areas of the actual compression and theoretically equivalent tension flange members, in.^{2}

Using the equivalent thickness calculated for the tension flange and the new location of the neutral axis, the adjusted equivalent I-values were next computed for each beam, ignoring the plywood web, using the formula:

$$I = \left[\frac{b_1(d_1)^3}{12} + A_1(\bar{y}_1)^2\right] + \left[\frac{b_2(d_2)^3}{12} + A_2(\bar{y}_2)^2\right] \quad (7)$$

where:

I = moment of inertia, in.^{4}

b_1 = thickness of the compression flange, in.

d_1 = depth of the compression flange, in.

A_1 = x-sectional area of the compression flange, in.^{2}

$\bar{y}_1$ = distance to the neutral axis from the centroid of the compression flange, in.

b_2 = equivalent thickness of the tension flange, in.

d_2 = depth of the tension flange, in.

A_2 = x-sectional area of the tension flange, in.^{2}

$\bar{y}_2$ = distance to the neutral axis from the centroid of the tension flange, in.

These calculated adjusted equivalent I-values were then multiplied by the moduli of elasticity of the compression flange, as determined in the ASTM compression tests, to compute the adjusted equivalent EI-values of each beam. These are recorded in Table V. A graphical comparison is made between the theoretical stiffness factors and the actual stiffness factors in Figure 15.

Extreme Fiber Stress in the Flanges

To determine the stresses developed in the extreme fibers of the flanges, when the beams were loaded to the allowable deflection at mid-span (1/360th of the span or .267 inches), the flexure formula was used.

$$\sigma = \frac{Mc}{I} \qquad (8)$$

where:

σ = extreme fiber stress, p.s.i.

M = bending moment at allowable deflection, in.-lbs.

c = distance from the neutral axis to the extreme fiber, in.

I = the adjusted equivalent moment of inertia, as previously computed, $in.^4$

These computed fiber stress values are recorded in Table VI.

COMPARISON OF STIFFNESS FACTORS

(Model Beams)

Beam No.	EI Values (Lbs.-In.^{2} x 10^6) Theoretical		Actual		Adj. Equiv.	
1-A-8	144.90		264.94		142.33	
2-A-8	144.90		232.51		118.47	
Average:		144.90		248.73		130.40
1-B-8	144.90		139.25		132.56	
2-B-8	144.90		162.46		156.29	
Average:		144.90		150.86		144.43
1-A-9	196.87		491.89		203.23	
2-A-9	196.87		272.52		188.15	
Average:		196.87		382.21		195.69
1-B-9	196.87		206.05		192.26	
2-B-9	196.87		183.65		202.21	
Average:		196.87		194.85		197.24
1-A-10	257.48		389.91		244.14	
2-A-10	257.48		389.91		257.16	
Average:		257.48		389.91		250.65
1-B-10	257.48		415.48		211.67	
2-B-10	257.48		230.40		267.53	
Average:		257.48		322.94		239.60
1-A-11	326.84		436.97		314.41	
2-A-11	326.84		704.00		399.18	
Average:		326.84		570.49		356.80
1-B-11	326.84		496.94		348.65	
2-B-11	326.84		288.00		263.83	
Average:		326.84		392.47		306.24

Table V.

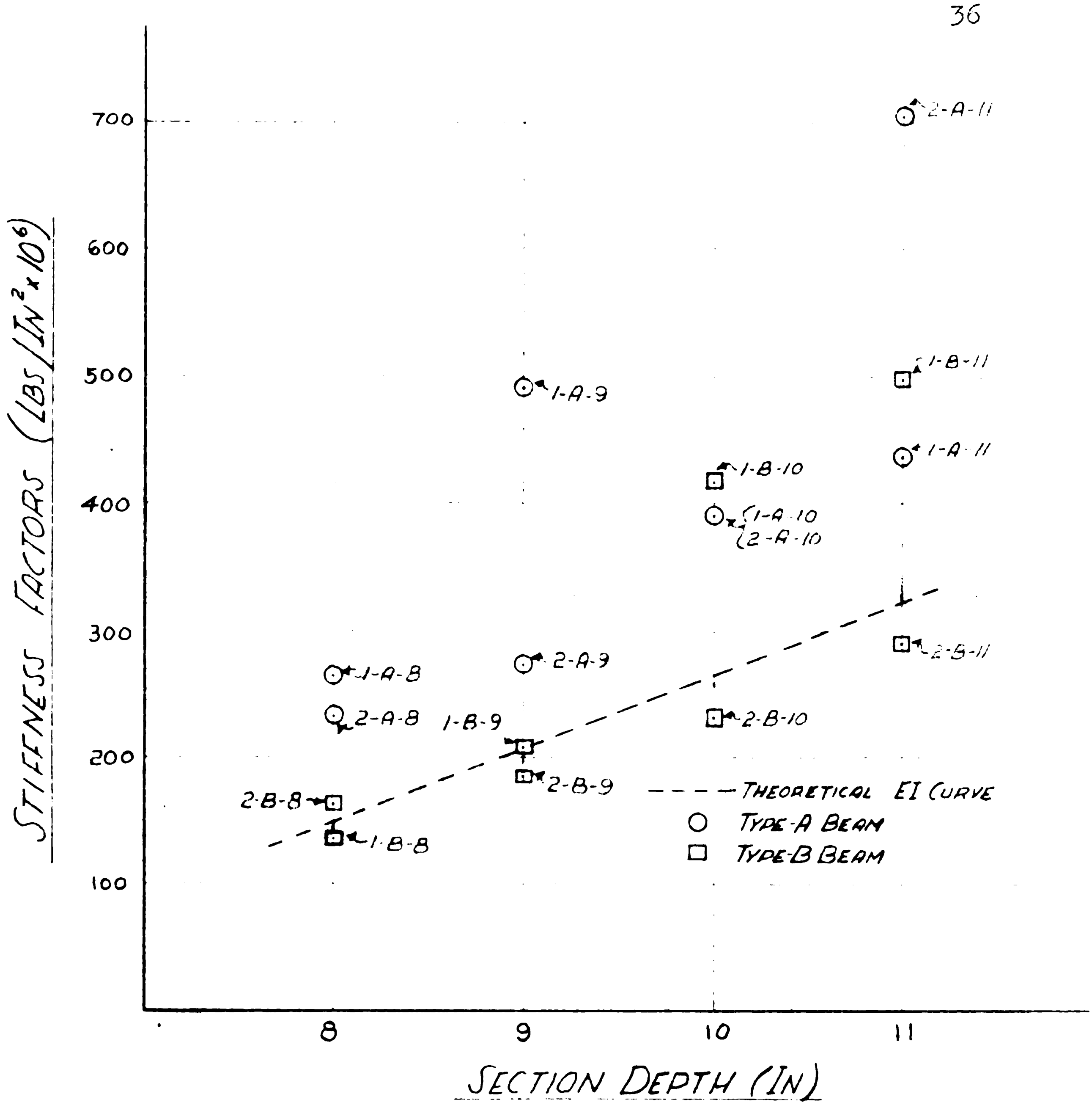


Figure 15

MODEL BEAM
STIFFNESS FACTORS

EXTREME FIBER STRESS IN FLANGES AT

THE ALLOWABLE DEFLECTION (1/360th) of the span)

(Model Beams)

Beam No.	Load (Lbs.)	Flexure (P.s.i.)
1-A-8	2100	2083
2-A-8	2700	1841
1-B-8	1970	1405
2-B-8	2190	1397
1-A-9	4060	2152
2-A-9	3020	1540
1-B-9	2590	1536
2-B-9	2400	1284
1-A-10	3740	2023
2-A-10	3680	1884
1-B-10	3840	1613
2-B-10	2820	1461
1-A-11	4010	1375
2-A-11	4810	2145
1-B-11	4250	1568
2-B-11	3300	1640

Table VI

Horizontal Shear Stresses

The horizontal shear stresses occurring in the plywood web when the beams were loaded to the allowable deflection and failure were next computed using the formula:

$$\tau = \frac{VQ}{Ib} \qquad (9)$$

where:

τ = horizontal shear stress, p.s.i.

V = vertical shear, lbs.

Q = the statical moment about the neutral axis, $in.^3$

I = the adjusted equivalent moment of inertia, as used previously, $in.^4$

b = the plywood web thickness, in.

A comparison is made of the calculated shear stress values of the plywood webs of each beam and is presented in Table VII.

Full-Scale Beams

Beam Test Performance

A load vs. mid-span deflection curve was plotted for each of the ten 16-foot beams tested. A composite plot is shown in Figure 16 with the average curve for each beam type superimposed.

Shear Deflection

Because a gross deflection was observed at mid-span at the time of testing, equation (1) was again used to calculate that portion of the measured deflection attributable to shear. This value (d_s) was found to be .149 inches for the 16 inch deep beams.

HORIZONTAL SHEAR STRESSES IN THE PLYWOOD WEB

(Model Beams)

Beam No.	Shear Stress at Allowable Deflection* (p.s.i.)	Shear Stress at Failure (p.s.i.)
1-A-8	809	1271
2-A-8	954	1307
1-B-8	728	1367
2-B-8	724	1355
1-A-9	1164	1491
2-A-9	833	-----*
1-B-9	831	1444
2-B-9	695	1419
1-A-10	1138	1308
2-A-10	1058	1208
1-B-10	906	----**
2-B-10	821	1543
1-A-11	796	----**
2-A-11	1245	1553
1-B-11	909	1048
2-B-11	951	1297

* 1/360th of the span

** Extreme buckling caused test beam to "spring out" of the testing machine.

Table VII

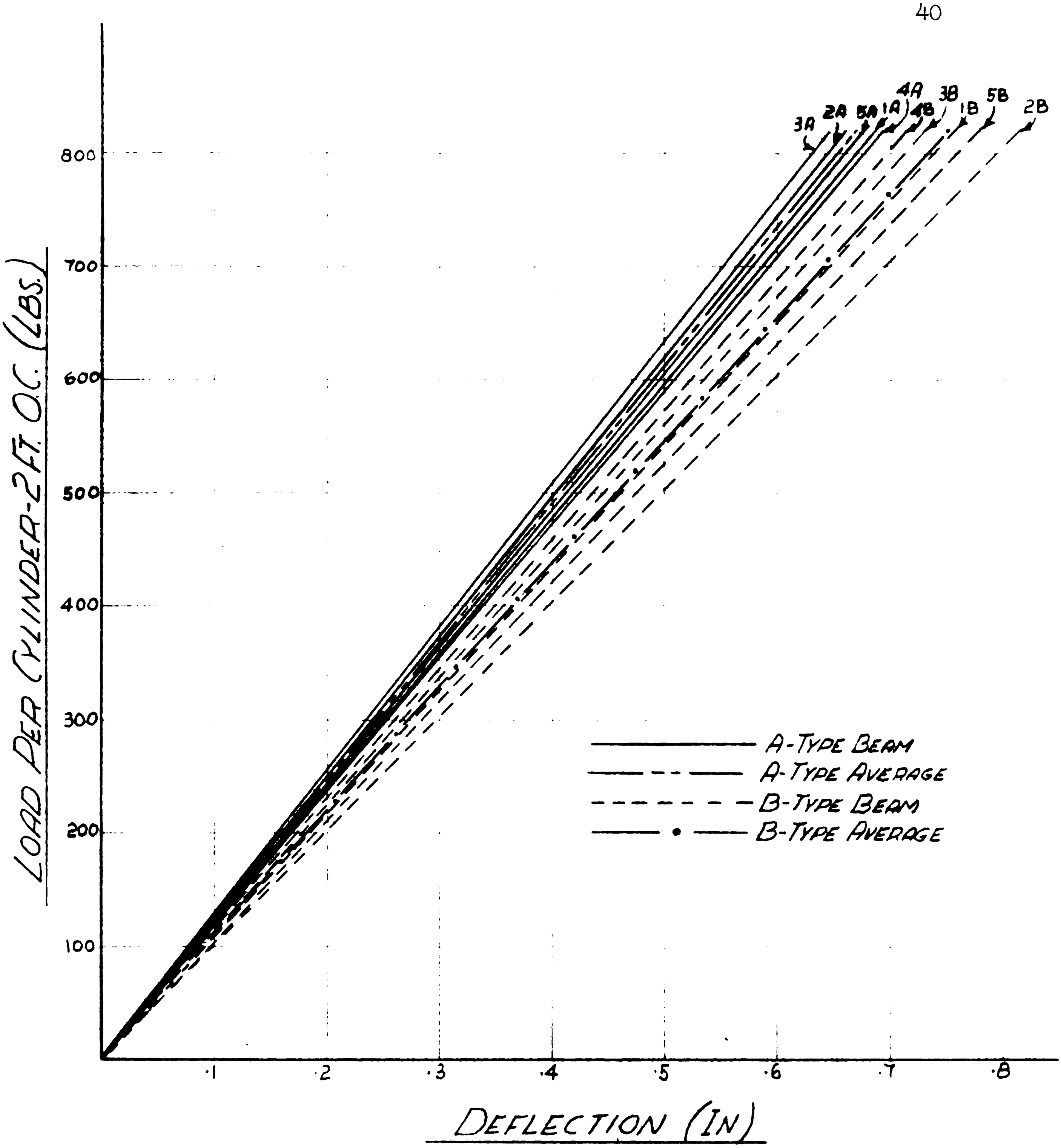


Figure 16

LOAD DEFLECTION CURVES FOR FULL-SCALE BEAMS

Modulus of Elasticity

A theoretical modulus of elasticity was next calculated. This calculation was made using formula (4) and using the bending moment diagram as given in Figure 17. The deflection due to bending was calculated by subtracting the above calculated shear deflection from the average deflection observed for the five B-Type beams at a load of 500 lbs. per two feet. The deflection due to bending was found to be .312 inches. The Type-A beams were not included in this calculation because of the presence and uncertain influence of truss action within the diagonals.

The theoretical modulus of elasticity calculated as described was found to be 1.15×10^6 lbs. per in.^{2} This compared favorably with the average E-value for ponderosa pine of 1.26×10^6 lbs. per in.^{2} (4)

Moment of Inertia

The theoretical moment of inertia was next computed using formula (3). As with the model beams, only the plys of the plywood web parallel to the span were included in this calculation. The theoretical I-value calculated was 1017 in.^{4}

Stiffness Factor

Two stiffness factors were calculated, the theoretical and the actual.

The theoretical EI-value was found by first computing the theoretical I-value using the formula:

$$I = \frac{b_1 d_1^3}{12} - \frac{b_2 d_2^3}{12} \qquad (10)$$

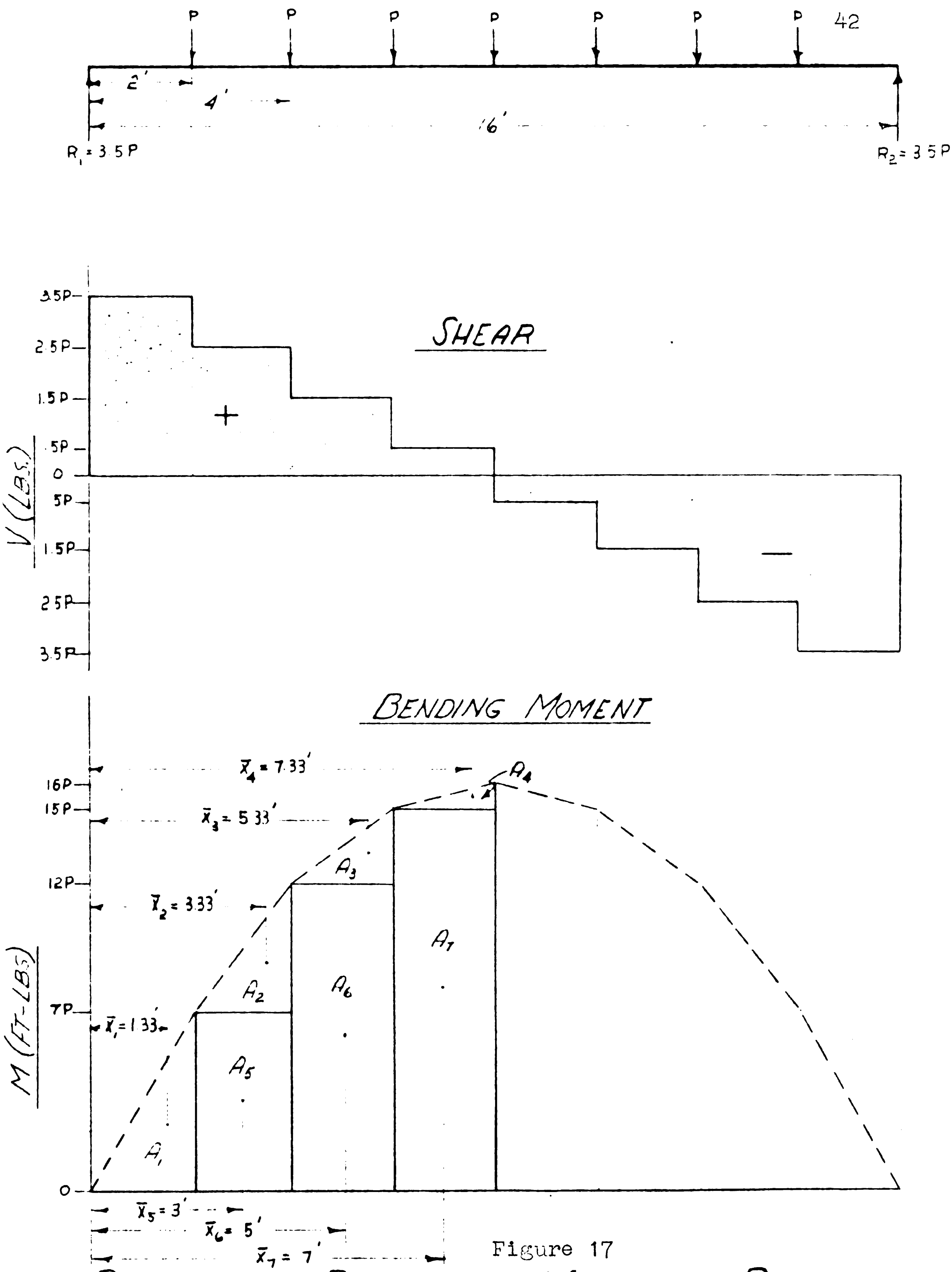


Figure 17

SHEAR & BENDING MOMENT DIAGRAMS FOR FULL-SCALE BEAMS where:

I = the theoretical moment of inertia, in.^{4}.

b_1= the total thickness of the flanges, including two plys of the 3/8 inch plywood web, in.

d_1= total depth of the beam, in.

b_2= total thickness of the flanges, excluding the plywood web, in.

d_2= total depth of the beam minus twice the depth of a flange, in.

However, because the flange members were of ponderosa pine and the plywood was Douglas-fir, an equivalent thickness of ponderosa pine was calculated to compensate for the difference in E-values of the two species. This was accomplished by dividing the E-value for Douglas-fir by the E-value of ponderosa pine (4) and multiplying this calculated equivalent section factor times the thickness of two plys of the Douglas-fir plywood web (.25 in.). The equivalent thickness of ponderosa pine calculated was.388 inches. This value was then used in formula (10) to compute the theoretical I-value of the beam. The theoretical I-value calculated was 1047 in.^{4}.

The theoretical EI-value was found by multiplying the above theoretical I-value times the average modulus of elasticity of ponderosa pine. (4) This stiffness factor or theoretical EI-value was 1318.89 x 10^6 lbs. per in.^{2}.

The second set of stiffness factors, or the actual EI-values, was calculated using formula (4). The bending moment diagram (Fig. 17) was used to compute $\Sigma A\bar{x}$ in this formula. The actual EI-values calculated may be found in Table VIII, which also includes the average EI-values by beam types.

Extreme Fiber Stresses in the Flanges

The extreme fiber stresses in the flanges occurring at mid-span corresponding to the load producing the allowable deflection of 1/360th of the span or .533 inches, was calculated using the flexure formula (8). These values are recorded for each beam in Table IX.

Horizontal Shear Stresses

The horizontal shear stresses occurring in the plywood web when the beams were loaded to the allowable deflection were again computed using formula (9). The statical moment used in this formula was calculated using the equivalent thickness of ponderosa pine rather than the thickness of Douglas-fir plywood actually tested. This equivalent thickness, as reported earlier, was .388 inches and the resulting statical moment about the neutral axis was 85.67 $in.^3$.

The horizontal shear stresses occurring in the plywood web are given in Table X.

COMPARISON OF STIFFNESS FACTORS

(Full-Scale Beams)

Beam No.	Theoretical EI-Values Lbs.-In.^{2} x 10^6	Actual EI-Values Lbs.-In.^{2} x 10^6	
		Type - A	Type - B
1	1318.89	1571.94	1358.97
2	"	1665.14	1217.57
3	"	1726.56	1423.24
4	"	1548.82	1478.18
5	"	1607.94	1300.25
Average:		1624.08	1355.64

Table VIII

EXTREME FIBER STRESSES IN THE FLANGES

AT THE ALLOWABLE DEFLECTION (1/360th of

the span) (Full-Scale Beams)

Beam No.	Flexure in Flanges (p.s.i.)	
	Type-A	Type-B
1	936	851
2	973	789
3	998	878
4	929	902
5	924	828

Table IX

HORIZONTAL SHEAR STRESSES IN THE PLYWOOD WEB AT THE ALLOWABLE DEFLECTION (1/360th of the span) (Full-Scale Beams)

Beam No.	Horizontal Shear Stress (p.s.i.)	
	Type-A	Type B
1	487	443
2	506	411
3	519	457
4	484	470
5	497	431
Average:	499	442

Table X

DISCUSSION OF RESULTS

Model Beams

Figure 12 shows graphically that the Type-A beams (with diagonals) were stiffer than the Type-B beams (without diagonals). The load vs. deflection curves for each beam tested are plotted and the average curve for each beam type is superimposed. It can be noted that the stiffness of the beams increases with the section depth. Also, within each depth category, the average curve of the two Type-A beams is stiffer, in all instances, than the average curve of the two B-Type beams.

By comparing the EI-values, or stiffness factors, (Fig. 15 and Table V)it can be readily observed that the Type-A model beams were significantly stiffer than the Type-B model beams. The Type-A beams having a section depth of eight inches are approximately 40 per cent stiffer than the Type-B beams of the same depth. This stiffness trend is also apparent in the nine, ten and eleven inch deep beams; however, as the section depth increases, the per cent increase in stiffness decreases.

Because the Type-A beams were considerably stiffer in all instances than the expected or theoretical EI-values (Table V), it was assumed that the diagonal stiffeners impart truss action to the beam, thus resulting in a more rigid construction.

The deviation of some of the points plotted in Figure 15 (EI-values vs. section depth) from the apparent average curves can be explained by considering the buckling and twisting observed in the beams having section depths greater than eight inches. Because of the relatively long laterally unsupported span, there was a decided tendency for the beams to buckle under load. This buckling and twisting was noted at the time of testing and was reflected in the plotted EI-values of Figure 15. Erroneous deflection readings undoubtedly resulted as the beams twisted about the neutral axis and the deflection-recording nail at the neutral axis rotated in the direction of twist. Figure 7 is a photograph taken of a representative beam twisting during testing.

It was found that the adjusted equivalent EI-values (Table V), computed using the eight inch compression sample data (Table III in Appendix), agreed favorably with the computed theoretical EI-values. The close agreement of the actual EI-values computed for the eight and nine inch deep B-Type beams to the theoretical EI-values for the same beam depths, indicated that conventional design equations can be used to predict the stiffness properties of this type of beam. However, this relationship did not exist for the A-Type beams. The expected or theoretical EI-values computed were lower in all instances than the actual EI-values. This reflected the inability of conventional design formulae to evaluate the effect of truss action imparted to the A-Type beams by the diagonals. Because of this, stiffness predictions and calculations for beams with diagonals should be made only if based on actual empirical results.

The calculated extreme fiber stress values of the flanges at a deflection equal to 1/360th of the span for the B-Type beams (Table VI) compared favorably with the allowable design unit stress values.[4] The fiber stress values calculated for the A-Type beams were, in most instances, higher than those calculated for the Type-B beams. This was another indication of the superior stiffness of the Type-A beams. The loads observed at the allowable deflection were greater; therefore, the computed fiber stresses were also higher than those of the B-Type beams.

With the exception of those few model beams that failed by buckling only, all the model beams failed due to horizontal shearing within the plywood web. The shear stresses calculated at a deflection equal to 1/360th of the span (Table VII) are above those allowable design shear stresses listed for Douglas-fir plywood.[2,3] The shear stresses calculated at failure (Table VII) ranged from 1048 p.s.i. to 1553 p.s.i. and are many times higher than the allowable design shear stresses mentioned above. This agrees favorably with the range of shear stresses at failure reported by Robbins[19] of 1028 p.s.i. to 1532 p.s.i.

It is apparent that the allowable design shear stresses were exceeded prior to reaching the allowable deflection of 1/360th of the span. However, shear failures did not occur till the beams were loaded far in excess of this allowable deflection. When the beams did fail, the shear stresses at failure were many times higher than the previously mentioned allowable design shear stresses. In this instance, it is apparent that the allowable deflection of the beam should be the governing design factor, and not the shear stresses in the plywood web.

The shear failures encountered while testing were limited to areas of the web between the ends of the beam and the points of loading, as shown in Figures 18 and 19. The shear failures were first evidenced by minute cracks opening horizontally in the face plys of the web. As the load was increased and the proportional limit exceeded, the cracks enlarged until the beam failed. In many instances the face plys of the plywood separated at the glue line from the inner ply.

It is quite evident that a definite relationship exists between the presence of diagonal members in the Type-A beams and the superior stiffness exhibited by this type of beam. However, because only a relatively few comparative tests were conducted, it is difficult to draw definite conclusions as to the magnitude of this stiffness. It is recommended that further study be done in this area to a degree large enough to warrant a thorough statistical analysis of the data.

As mentioned previously, there was a decided tendency for the beams to twist and buckle. The beam failures observed in Table I were often a combination of shearing and buckling. This repeated occurrance of buckling further limits the formulation of definite conclusions. It is recommended that in future tests of this nature, the beams be laterally supported. Only then will the test data reflect the actual mechanical properties of the tested beams.

It is also recommended that strain gauges be used in any future testing of Type-A beams to determine the amount of truss action within the diagonals and the effect of various beam depths on this action.

Full-Scale Beams

In comparing the stiffness factors or EI-values of the larger, full-scale beams (Table VIII), it was apparent that there was close agreement between the theoretical EI-value and the actual or tested EI-values for the Type-B beams (without diagonals). It can again be concluded, as with the B-Type model beams, that the normal design formulae are applicable to this type of beam as long as shear deflection

Figure 18

Shear Failure of Model Beam

Figure 19

Model Beam Failure

is compensated for. The small variations that exist between the EI-values for these beams could best be attributed to the expected variations of the moduli of elasticity of the flanges.

In this instance, it was found that the Type-A beams (with diagonals) were 16 per cent stiffer than the B-Type beams. This increase in stiffness again reflects the truss action of the diagonals, as demonstrated by the model beams.

Ponderosa pine is not generally used for structural purposes and for this reason recommended design stresses were not available to compare with the extreme fiber stresses, as calculated for the flange members and reported in Table IX. As with the model beams, the extreme fiber stresses displayed by the A-Type beams are again higher than those of the B-Type beams. As mentioned before, this reflects the superior stiffness of the beams having diagonals.

The calculated horizontal shear stresses in the plywood webs at a deflection equal to 1/360th of the span (Table X) were considerably lower than those of the model beams. This was as expected and reflects only the difference in loading methods used between the model beams and the full-scale beams. Although the shear stresses are less than those of the model beams, they are again considerably higher than the allowable design shear stresses for Douglas-fir.[2,3] Again, as with the model beams, the shear stresses calculated for tye A-Type beams are higher than those of the Type-B beams at a given deflection equal to 1/360th of the span.

As previously mentioned, because the full-scale beam tests were limited by the small maximum load output of the hydraulic testing floor, no information was obtained pertaining to the proportional limot of these beams or the type of failure that could be expected.

It is felt that the uniformity in testing results displayed by the full-scale beams reflects a better method of testing. The use of metal rods and rollers to restrain any buckling, as illustrated in Figure 11, undoubtedly resulted in a more accurate illustration of beam performance.

CONCLUSIONS AND RECOMMENDATIONS

This paper presents the results of a limited number of tests to determine the mechanical properties of nail-glued, wood-plywood, trussed girders. While the tests reported herein were of an exploratory nature and were too few in number to establish design criteria, the results show that:

1. Type-A beams (with diagonal stiffeners) are significantly stiffer than Type-B beams (with vertical stiffeners only). Full-scale Type-A beams were approximately 16 per cent stiffer than the full-scale Type-B beams. Type-A model beams were stiffer than the Type-B model beams; however, the per cent increase in stiffness lessened as the section depth increased.
2. Conventional design formulae can be utilized with reasonable accuracy to predict the stiffness properties of conventional nail-glued plywood I-beams without diagonal stiffeners (Type-B); however, they are not applicable to trussed I-beams (Type-A) and the use of these formulae yield stiffness values consistently below empirical results.
3. Buckling of the plywood web becomes critical as the span/depth ratio of the beam decreases. Valid test results can be obtained only if this buckling is controlled or restrained.
4. Calculated horizontal shear stresses in the plywood web were very high; however, because the shear failures occurred at loads in excess of both the allowable design deflection at mid-span (1/360th of the span) and the proportional limit of

the beams, it is recommended that the governing design criteria be the allowable design deflection at mid-span and not the allowable design horizontal shear stresses in the plywood web.

5. Shear deflection in a wood-plywood girder is an important component of the total beam deflection and cannot be ignored in stiffness calculations.

6. Strain gauges should be used in any future testing of the Type-A beams in order to determine (1) the amount of truss action within the diagonals, and (2) the effect of various beam depths on this action.

7. Nail-gluing with casein glue (U. S. Spec. MMM-A-125) provided an adequate method of bonding the beams; however, precautions must be taken to protect the beams from any prolonged exposure to moisture.

8. It is recommended that further study be done in this area to a degree large enough to warrant a thorough statistical analysis of the data.

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A P P E N D I X

MODULI OF ELASTICITY OF FLANGE COMPRESSION

SAMPLES (Model Beams)

Flange Sample No.	X-Section Area Sq. In.	Moisture Content %	Modulus of Elasticity (Lbs/In.^{2} x 10^6
1-A-8-C*	4.19	7.5	2.24
1-A-8-T**	4.23	7.5	2.00
2-A-8-C	4.20	9.0	1.71
2-A-8-T	4.25	7.5	1.81
1-B-8-C	4.12	7.5	2.00
1-B-8-T	4.06	7.0	1.94
2-B-8-C	4.17	8.0	2.11
2-B-8-T	4.08	7.5	2.58
1-A-9-C	4.13	8.5	2.02
1-A-9-T	4.03	9.5	2.50
2-A-9-C	4.06	7.0	1.80
2-A-9-T	4.13	7.0	2.42
1-B-9-C	4.13	9.0	2.14
1-B-9-T	4.20	7.0	2.07
2-B-9-C	4.15	10.5	2.03
2-B-9-T	4.09	9.0	2.44
1-A-10-C	4.02	7.0	2.23
1-A-10-T	4.12	7.0	1.92
2-A-10-C	3.94	8.0	2.22
2-A-10-T	4.02	7.5	2.14
1-B-10-C	4.10	6.5	1.50
1-B-10-T	4.04	7.0	2.20
2-B-10-C	4.20	7.0	2.34
2-B-10-T	4.10	7.5	2.18
1-A-11-C	4.22	8.5	1.65
1-A-11-T	4.15	7.5	2.88
2-A-11-C	4.02	7.0	2.73
2-A-11-T	3.95	8.5	2.61
1-B-11-C	4.07	6.5	1.97
1-B-11-T	4.12	7.0	2.86
2-B-11-C	4.14	7.5	2.01
2-B-11-T	4.18	7.0	1.56
Average:	4.11	7.7	2.15

* = Compression Flange
**= Tension Flange

Table III