A STUDY OF THE STIFFNESS PROPERTIES OF WOOD FLANGE-STEEL WEB I-BEAMS

Thesis for the Degree of M. S.

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Herbert F. Law

1961

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OF WOOD FLANGE-STEEL WEB I-BEAMS

By

HERBERT F. LAW

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 Byron M Radcliffe

ABSTRACT

This study was undertaken to investigate the possibilities of using thin sheet steel as the web material for built-up I-beams instead of plywood and thus substantially reduce shear deflection. This is a serious limitation for the case of wood-plywood I-beams.

Twenty scale model test beams with a span of eight feet were fabricated in depths of eight, nine, ten and eleven inches. Two full scale beams, sixteen inches in depth and sixteen feet long, were built and tested in an attempt to correlate results of model beam tests. The method of fastening flanges and stiffners to the web was by nails only. No adhesive was used.

All model beams were tested to failure but full scale beams were not due to the limited capacity of the test machine used.

Theoretical and actual stiffness graphs were plotted for all test beam and full scale beam test data. These results were compared. Comparison was also made with a nail-glued wood-plywood I-beam of similar section. The effect of nailing pattern on stiffness was also studied.

It was found that wood flange-steel web I-beams, using either medium or heavy nailing, are superior in resistance to deflection to a comparable nail-glued I-beam. It was also observed that shear deflection in the steel web was less than 2% and thus could be neglected in design calculations for beams of this type.

Lateral instability, which gives rise to buckling

of the web, is a major problem in beams of this type. However, buckling is not critical until well above the design deflection criteria which is normally recognized to be 1/360 of the span.

Further research is recommended in this field with emphasis on spacing of stiffners, different gages of steel and with an attempt to control lateral stability. In any event, a much larger sample should be used so that the results would be more conclusive statistically.

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INTRODUCTION

History of " Built-up " I-Beams

The use of laminated structural wood members or " Built-up " beams, as they are often called, dates back to the early 1900's. This process was first used in Europe. It dealt mostly with laminated beams but the advantages this method offered were soon adapted to use in laminated beams with rectangular, I and double I cross sections.

Most of the development of structural uses of plywood has taken place since the second world war. Previous to that time, little such development had taken place, due to the shortage of plywood and the lack of suitable adhesives. With the advent of the second world war, a search for structural members, other than steel, took place.The necessity of conserving materials was responsible for this search.

One of the earliest intensive uses of plywood I-beams in this country was a 125,000 sq. ft. warehouse built in 1942 for the RCA Manufacturing Co. at Camden, New Jersey[6]. A total of 198 identical plywood girders, 36 feet long, were job-fabricated using webs that were nailed only to the lumber flanges with 8d cement-coated nails. After ten years of service, the warehouse was taken over by the government. The beams were found to be in excellent condition at that time; they had not sagged and had required no maintenance.

The first extensive research and experimentation on the strength and stiffness properties of plywood web I and box type beams was done by the Forest Products Laboratory. This work was done for the U. S. Government to determine the feasibility of using such sections as structural members for aircraft. This work was subsequently revised and adopted by the Douglas Fir Plywood Association for design and use in building construction[2]. Methods of construction of plywood web I-beams have been much the same for many years and revised design and fabrication specifications have been published just recently[3]. They applied conventional engineering calculations using allowable design values of wood and made recommendations concerning the webs, flanges and stiffners of box beams. They were largely concerned with buckling of the web[20], and horizontal shear stresses[21]. The Forest Products Laboratory found that it made little difference whether the face grain of the plywood webs was horizontal or vertical. Plywood webs oriented at 45 degrees, however, were found to be substantially more efficient[22]. They also found that for thin beam webs significant increases in web shear resistance could be secured by reducing stiffner spacing[20].

David Countryman, in full scale tests of plywood beams in 1944, found that nail-gluing was an effective method of fabrication[23]. Butt-joining plywood web splices was also determined to be adequate to develop the full beam strength in both bending and shear. Countryman's tests showed no buckling in the webs nor were any beam failures caused by horizontal shear faults, even though this was the limiting design stress in many of the beams. He concluded that a better balanced beam might have resulted had the allowable horizontal shear stresses for the plywood been higher.

Based on the pioneering efforts made by the Forest Products Laboratory, other publications soon appeared from various sources. Because of the interest of the Douglas Fir Plywood Association 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].

The present published reports on the strength properties of plywood I-beams indicate that the design strength may be predicted by existing engineering equations. Recent experimentation at Michigan State University[6,14,18] and previous work done by Radcliffe[24] at Purdue University suggest that the recommended working stress for horizontal shear of plywood is conservative and places plywood web structural constructions, such as I and box beams, at a definite design disadvantage.

It is important to note that deflection in I-beams has two main components, flexural deflection and shear deflection. Flexural deflection is caused by the lengthening of tension fibers and shortening of compression fibers and is generally considered the major component of total deflection. Shear deflection, resulting from horizontal shearing distortion of fibers, is of considerable importance in I-beams due to the small cross section of the web. Shear deflection is present in all wooden beams, but the sectional characteristics of an I-beam serve to amplify this deflection. Since shear deflection has proven to be a significant part of total deflection, there has been, in recent months, an attempt to substitute the plywood web with a web of thin sheet steel. It was felt that since the shear modulus for steel is much higher than that of wood, that the use of steel would greatly reduce deflection caused by shear. Research has been in progress in Washington, D. C., on an I-beam which incorporates two webs of steel[8]. This thesis is the result of research into the advisability of substituting one thin sheet of steel for the plywood web which has been used in the nail-glued I-beam. Results obtained in this research indicated that the use of steel practically eliminates shear deflection and merits further study in this direction.

Purpose of the Study

The purpose of this study was to evaluate the stiffness and strength properties of wood flange-steel web I-beams in terms of existing theory and engineering equations.

The flexural behavior of half scale wood flange-steel web I-beams was to be compared with theoretically predicted behavior, not only with actual results obtained for these beams but with plywood I-beams of similar section. This was to be done by comparing the theoretical and actual stiffness factors of the beams being compared.

Similar tests were to be conducted on two full scale wood flange-steel web I-beams and to correlate these results with those obtained from model beam tests.

Finally, an attempt was made to determine the effect on flexural behavior of the beams tested in relation to the number of nails used in fabrication.

FABRICATION OF BEAMS

A. Model Beams

Twenty half scale model beams of I section were constructed in eight foot lengths and in depths of eight, nine, ten and eleven inches. There were five beams of each depth. All beams had 1 x 2 inch wood flange members and stiffners applied to both sides of the web.

Flange and stiffner material was cut from No. 1 structural grade 2 x 6 inch Douglas Fir and surfaced four sides to a nominal 1 x 2 inch dimension. Material showing serious defects was eliminated in order that actual full size beam fabrication would be duplicated as closely as possible.

Web material was 26 gage cold-rolled galvanized sheet steel having an actual thickness of 0.0184 inches.

Method of fastening the flanges and stiffners to web was by No. 6d coated box nails. Glue was not used in fabrication. In practical applications to actual beams gluing would add materially to the cost inasmuch as only certain expensive adhesives could be used.

Beams were assembled using three different nail spacing arrangements to determine if a correlation existed between nail spacing and stiffness. One beam of each depth was nailed every two inches on both sides. Four inch spacing, on both sides, was used on two beams of each depth. On the remaining two beams of each depth nails were spaced at four inch intervals on one side only. All nails were staggered to effectively distribute the holding power. For nailing pattern refer to figure 2.

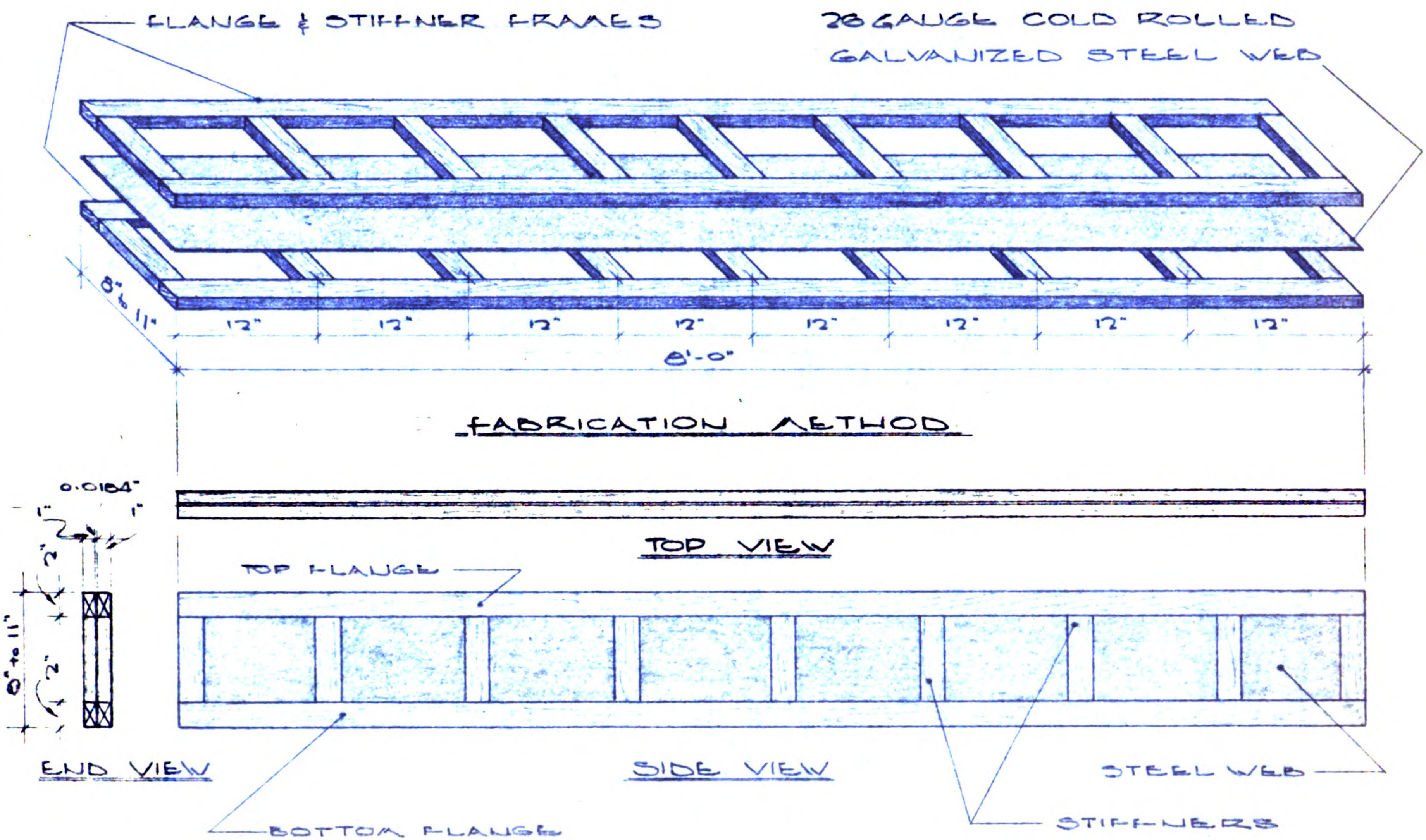

FIGURE : 1 FABRICATION of MODEL BEAMS

Model beams were constructed by fabricating two identical frames of upper and lower flanges and vertical stiffners spaced twelve inches on center. The sheet steel web was then inserted between these frames and nailed securely as shown in figure 1.

Forty small bending samples, thirty inches in length and nominally 1 x 2 inches in section, taken at random from the same stock of material used to construct model beams were tested with a Reitle Universal testing machine using testing procedure as outlined in the ASTM, to establish an average modulus of elasticity.

Moisture content readings were taken for each standard bending specimen and also for each model beam at the time of test. An electrical resistance type moisture meter was used in these determinations.

B. Full Scale Beams

Two full size test beams were constructed for the purpose of correlating results of model beams. Both beams were 16 feet in length and 16 inches in depth. The method of fabrication was identical to that used for model beams.

2 x 4 inch flange and stiffner material used was a mixture of structural grade Douglas Fir and Western Hemlock. It was used as recieved from the lumber yard. Web material was 14 gage, cold-rolled sheet steel having an actual thickness of 0.077 inches. No. 12d hardened steel nails were used to fasten flanges and stiffners to the

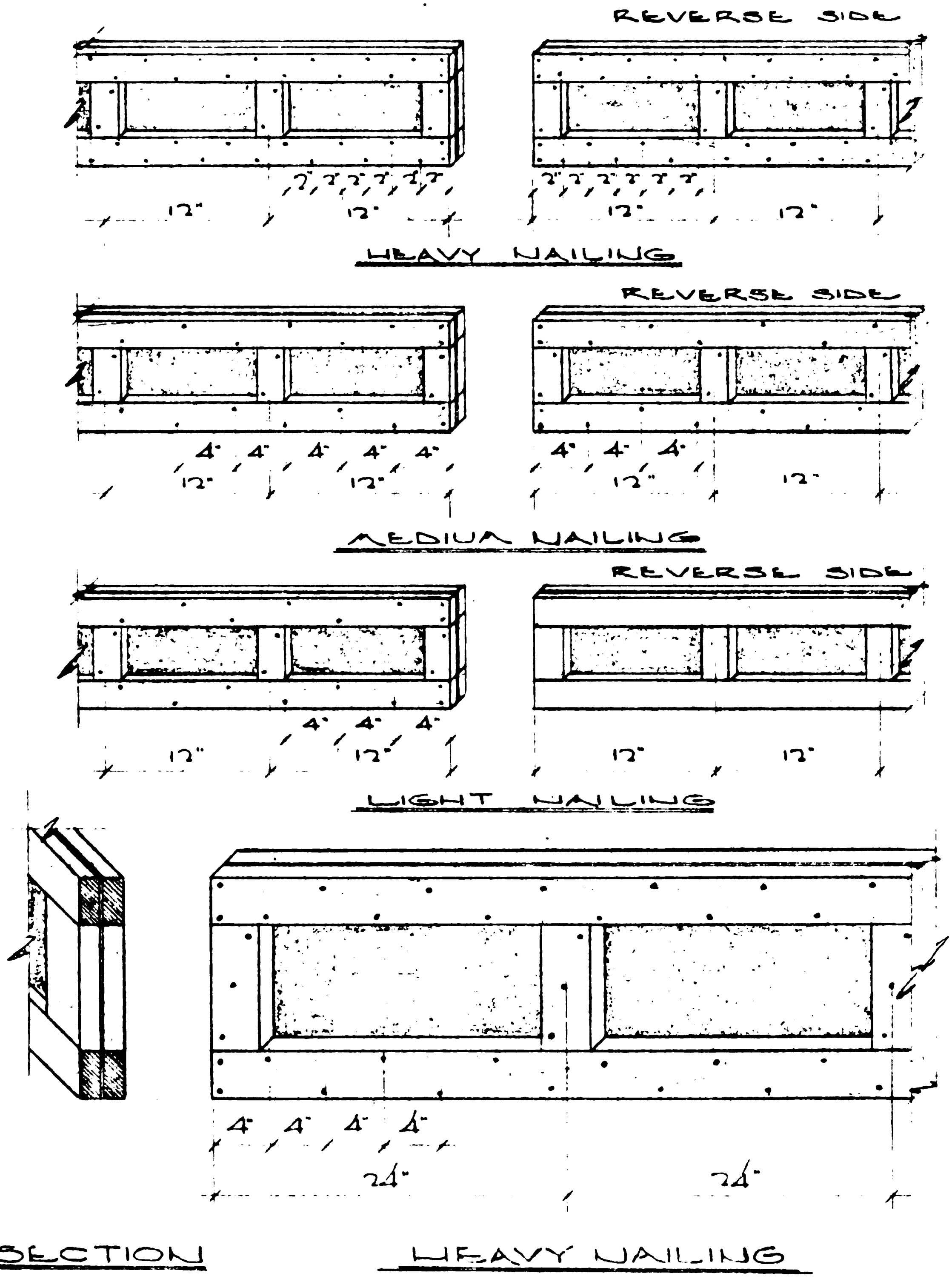

FIGURE: 2

NAILING SCHEDULE

web. Stiffners on both sides were spaced at 24 inches on center. See figure 2.

Eight 2 x 2 inch small bending samples, 30 inches in length were cut from the same flange material used in fabricating the full scale beams and tested in bending to establish an average modulus of elasticity.

TESTING PROCEDURE

A. Model Beams

Model beams were tested in static bending in accordance with ASTM standards. The load was applied in a Universal Testing machine at a constant rate of 1/16" per minute mid span deflection. Beams were supported at each end by means of maple blocks and load was applied at two points along the 96 inch span. Loads were applied at 24 inches from each end of span. Refer to figure 3, for loading arrangement.

Deflection was measured through the use of a deflection yoke, supported at the neutral axis of the beam over the bearing points. Deflection at the neutral axis at the center of the beam span was determined by an Ames dial as shown in figure 3. Deflection readings were taken for every 200 pounds force to failure. Loading was at 1/4 points.

B. Full Scale Beams

Two full scale beams were tested in static bending on a hydraulic floor type testing machine. Refer to figure 7 for graphic illustration of testing set-up.

The beams were strapped to the concrete floor in four places by means of angle iron and metal rollers and plates were used at every point of friction to provide freedom of movement. See figure 7. This was done to restrict buckling and assimilate practical use situations. Buckling had been the cause of failure in all model beams and every attempt was made to restrict it in these tests by using the metal straps as referred to above.

Load was applied by means of seven hydraulic cylinders spaced two feet on center. Beams were supported at each end of the span. There was no load applied at either end which resulted in a load being applied at each stiffner excepting those located at each end of the beam. The area of each cylinder was 2.94 inches and therefore the indicated loads were multiplied by the area of each cylinder to give the actual load applied in p. s. i. The seven cylinders were connected hydraulically with gages at each end of the hydraulic system. Loads were applied in increments of 25 p. s. i. These, as mentioned above, were the indicated readings and were adjusted by the multiplying factor, 2.94, as discussed above.

An Ames dial deflection gage was placed at mid span and readings were taken at every 25 p. s. i. to the closest 0.001". Readings were taken at the bottom flange rather than the neutral axis as in the case of the model beams, due to the type of test set-up used. Refer to figure 7.

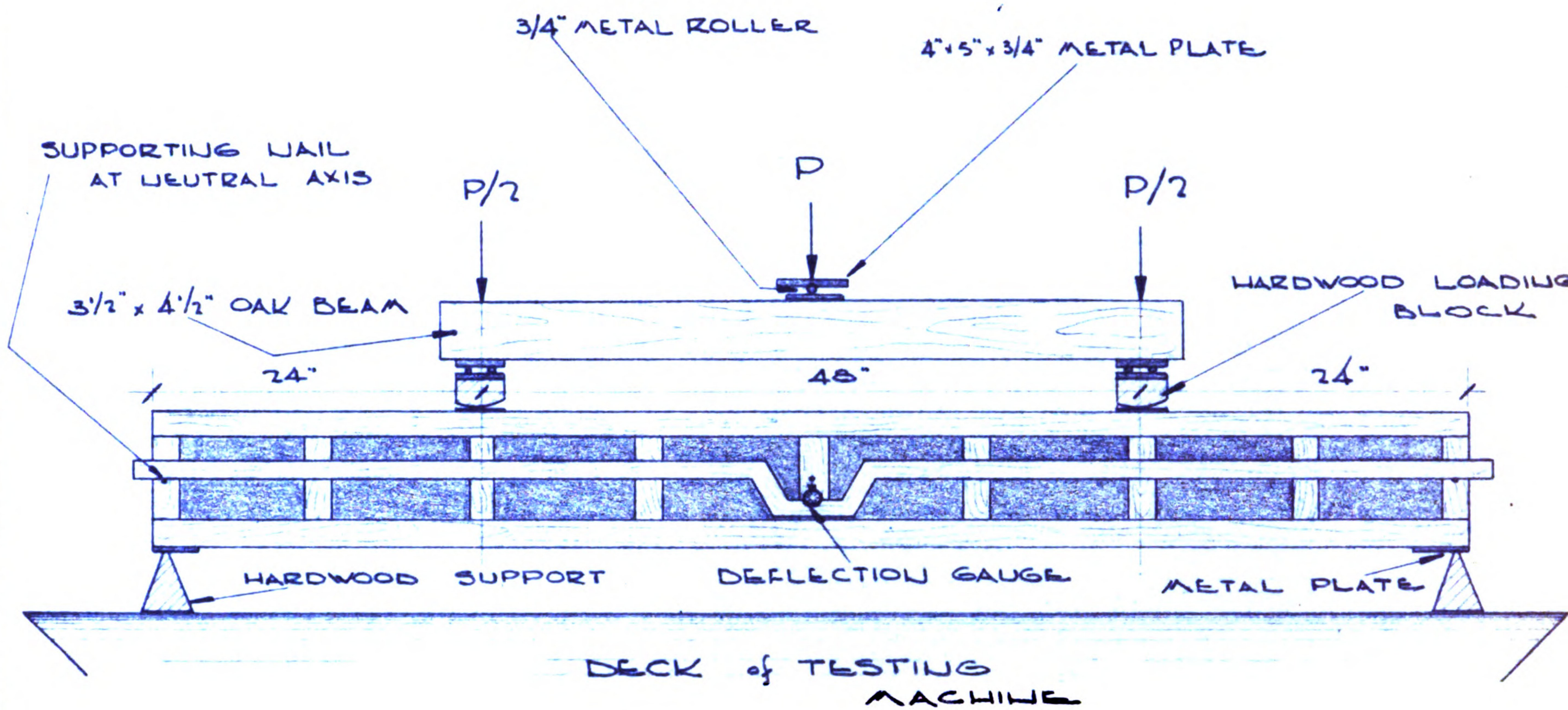

FIGURE : 3

MODEL BEAM TESTING PROCEDURE

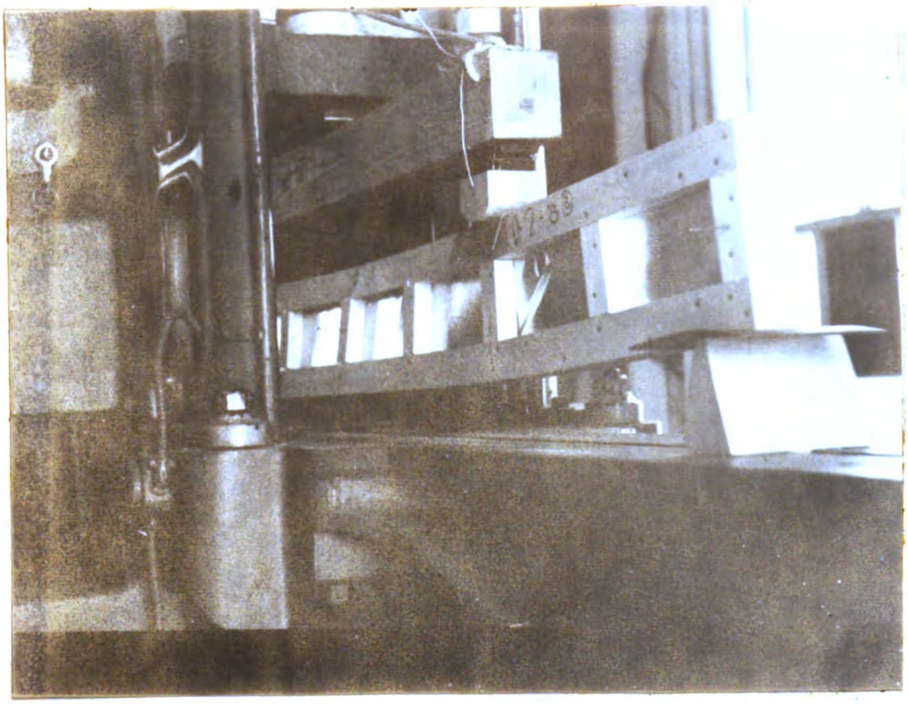

Figure 4

Half Scale Model Beam Under Test

Figure 5

Model Beam Showing Failure At Point Of

Loading

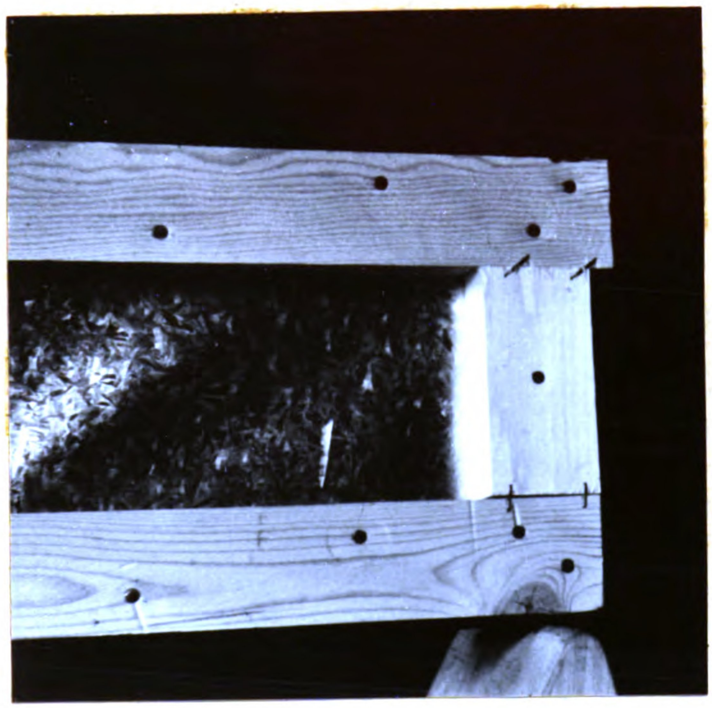

Figure 6

Flange Slippage, Model Beam, Light Nailing.

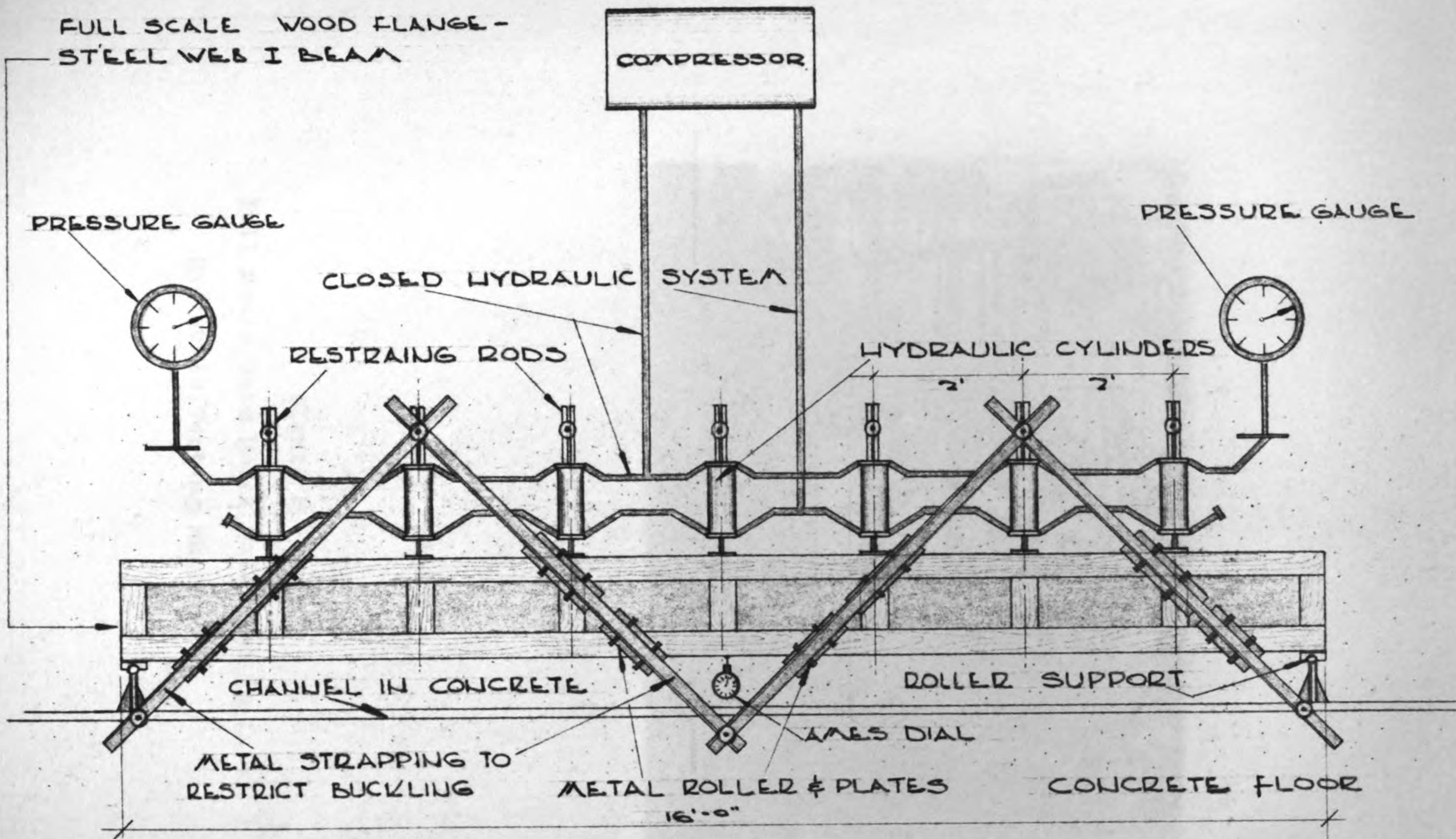

FIGURE: 7

FULL SCALE BEAM TESTING PROCEDURE

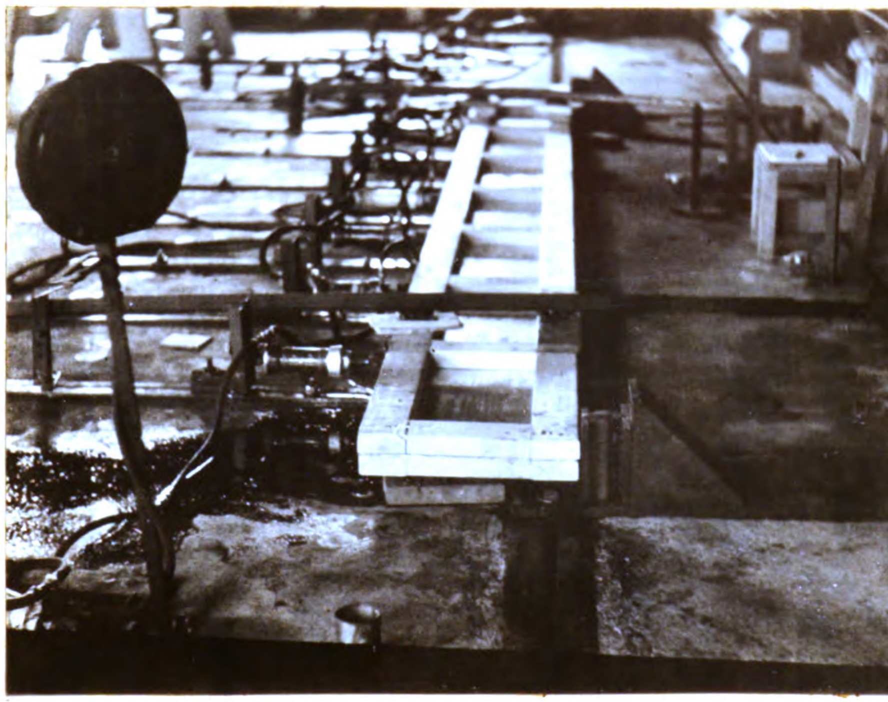

Figure 8

Full Scale Beam Under Full Load in

Hydraulic Testing Machine

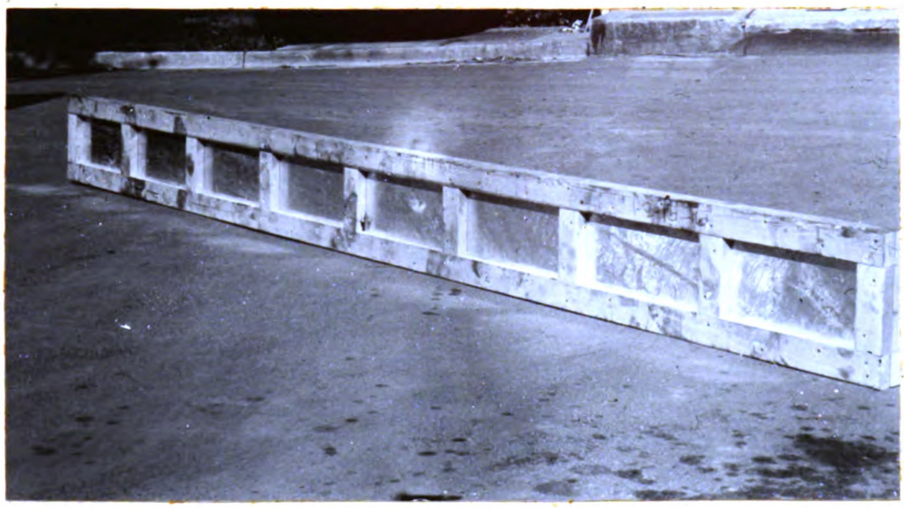

Figure 9

View of Full Scale I-Beam After Test

TEST RESULTS

Loads at Allowable Deflection

A load versus mid span deflection curve was plotted for both half scale and full scale beams. The graphs are shown in figures 10, 11, 14.

Graphs were plotted to show the relative stiffness comparison of both half scale and full scale beams due to spacing arrangement of nails. Results are shown in figures 11 and 14.

Modulus of Elasticity

An average modulus of elasticity (Ea) was established for the half scale and full scale beams. Small bending samples of the same stock were used for this purpose. The formula used was:

$$E = \frac{PL^3}{48\ I\Delta}$$

Where:

E = Modulus of Elasticity, p. s. i.

P = Total Load on Sample, pounds.

L = Length of Span, inches.

I = Moment of Inertia, inches^4.

Δ = Deflection, inches.

The average modulus of elasticity for the half scale beams was found to be 1.95×10^6. For the full scale beams it was 1.68×10^6. Moisture content readings were also taken and the average for the half scale beams was found to be 8%. For the full scale beams the average was 13.2%. Refer to tables 1 and 2.

Moment of inertia was calculated for the small

TABLE 1

MODULUS OF ELASTICITY

MODEL BEAMS

Sample No.	Modulus of Elasticity	b Least Dim.	d Great.Dim.	Moisture Content
1.	2.07×10^6	.95"	1.91"	6.0%
2.	2.28	.95	1.90	8.0
3.	1.79	.96	1.92	6.5
4.	1.73	.97	1.89	7.0
5.	2.49	.95	1.92	8.5
6.	2.14	.96	1.92	7.5
7.	2.45	.97	1.89	8.5
8.	2.51	.97	1.89	8.5
9.	2.59	.94	1.90	7.5
10.	1.68	.98	1.89	6.5
11.	1.62	.96	1.92	6.5
12.	1.85	.97	1.91	6.5
13.	1.82	.97	1.91	6.5
14.	1.58	.97	1.91	6.5
15.	1.72	.97	1.91	6.5
16.	1.57	.98	1.93	6.5
17.	2.72	.93	1.90	7.0
18.	1.53	.98	1.92	6.5
19.	1.56	.98	1.90	7.0
20.	1.74	.98	1.88	7.0
21.	2.24	.95	1.93	8.0
22.	2.16	.97	1.89	6.5
23.	2.58	.98	1.90	9.0
24.	1.48	.97	1.93	6.5
25.	2.49	.93	1.89	7.5
26.	1.97	.95	1.92	7.0
27.	1.99	.93	1.89	7.0
28.	1.51	.97	1.94	7.0
29.	1.97	.93	1.93	8.0
30.	2.25	.95	1.93	7.5
31.	1.52	.98	1.96	11.0
32.	1.85	.99	1.97	11.0
33.	1.53	.99	1.99	11.5
34.	1.70	1.00	1.98	11.0
35.	1.79	1.00	1.99	10.0
36.	1.75	.98	1.98	12.0
37.	1.93	.98	1.97	12.5
38.	2.11	.97	1.98	11.0
39.	1.83	.98	1.98	12.0
40.	2.04	.98	1.98	10.5
Total:	78.15×10^6	38.70"	77.05"	327.5%
Average:	1.95×10^6	.97"	1.93"	8.2%

TABLE 2

MODULUS OF ELASTICITY

FULL SCALE BEAMS

Sample No.	Modulus of Elasticity	b Least Dim.	d Great.Dim.	Moisture Content
1.	1.85×10^6	1.60"	1.70"	15.5%
2.	1.40	1.61	1.72	14.8
3.	1.40	1.61	1.73	15.5
4.	2.10	1.59	1.77	15.5
5.	1.92	1.52	1.72	11.0
6.	1.92	1.58	1.75	11.0
7.	1.55	1.55	1.74	11.5
8.	1.36	1.60	1.77	10.8
Total:	13.50×10^6	12.66"	13.90"	105.6%
Average:	1.68×10^6	1.58"	1.74"	13.2%

These calculations were derived from eight small bending samples, 30 inches in length, using the equation:

$$E = \frac{P\,L^3}{48\,I\,\Delta}$$

Where:

E = Modulus of Elasticity, p. s. i.

P = Load, pounds.

L = Length of Sample, inches.

I = Moment of Inertia, inches^4

Δ = Deflection, inches.

bending samples for the purpose of determining the above modulus of elasticity (Ea) using the equation:

$$I = \frac{bh^3}{12}$$

Where:

I = Moment of inertia, inches^4.

b = Least dimension, inches.

h = Greatest dimension, inches.

Equivalent Sections

In order that standard equations could be used in comparing wood flange-steel web I-beams with nail-glued wooden I-beams of similar section it was necessary to convert the steel web to an equivalent section of wood. This was done by using the equation:

$$t_w = \frac{E_s}{E_w} \times t_s$$

Where:

t_w = Thickness of equivalent web of wood, inches.

E_s = Modulus of elasticity of steel, p. s. i.

E_w = Modulus of elasticity of wood, p. s. i.

t_s = Thickness of steel web, inches.

This method was used for both half scale models and full scale beams in order to calculate the theoretical moment of inertia (I_{th}) and the theoretical stiffness factor (EI_{th}). For these values refer to tables 3 and 4.

Moment of Inertia

The theoretical moment of inertia was calculated for each beam using the equation:

$$I_{th} = \frac{b\,h^3 - 2\,b_1\,h_1^3}{12}$$

Where:

I_{th} = Theoretical moment of inertia, inches^4.

b = Total width of beam, including both flanges and web, inches.

h = Total depth of beam, inches.

b_1 = Flange width, inches.

h_1 = Total beam depth less twice flange depth, inches.

These theoretical moment of inertia (I_{th}) values were used in computing the theoretical stiffness factors (EI_{th}). These values can be found in table 3.

Stiffness Factor

For each model beam and full scale beam a stiffness factor, both theoretical (EI_{th}) and actual (EI_{act}), was computed. A theoretical EI value was also calculated for an equivalent nail-glued I-beam of similar section. This EI value was calculated without subtracting that amount due to shear so that a more accurate comparison could be made with the wood flange-steel web I-beams being tested in this research. The equation used to

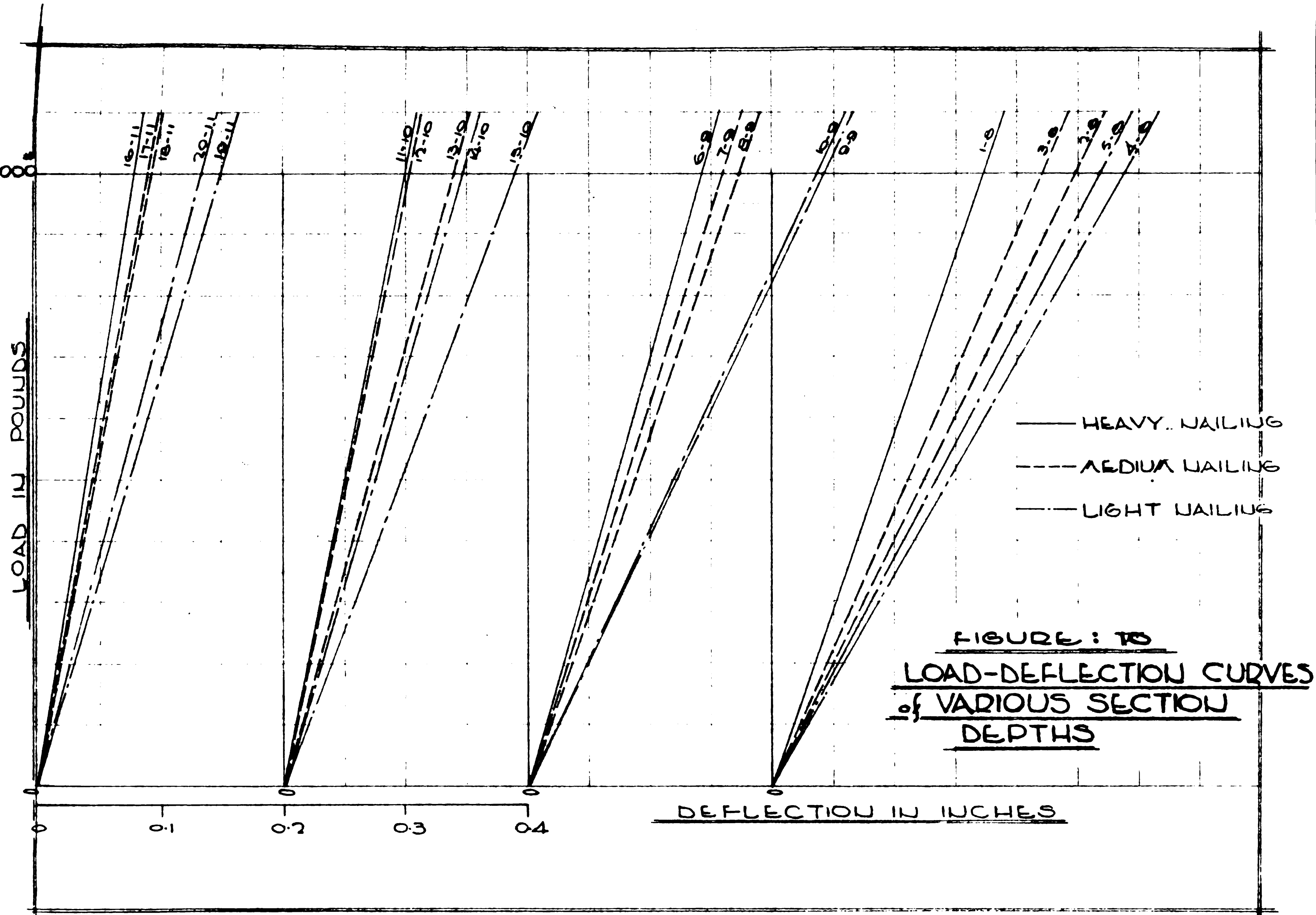

FIGURE: 10

LOAD-DEFLECTION CURVES of VARIOUS SECTION DEPTHS

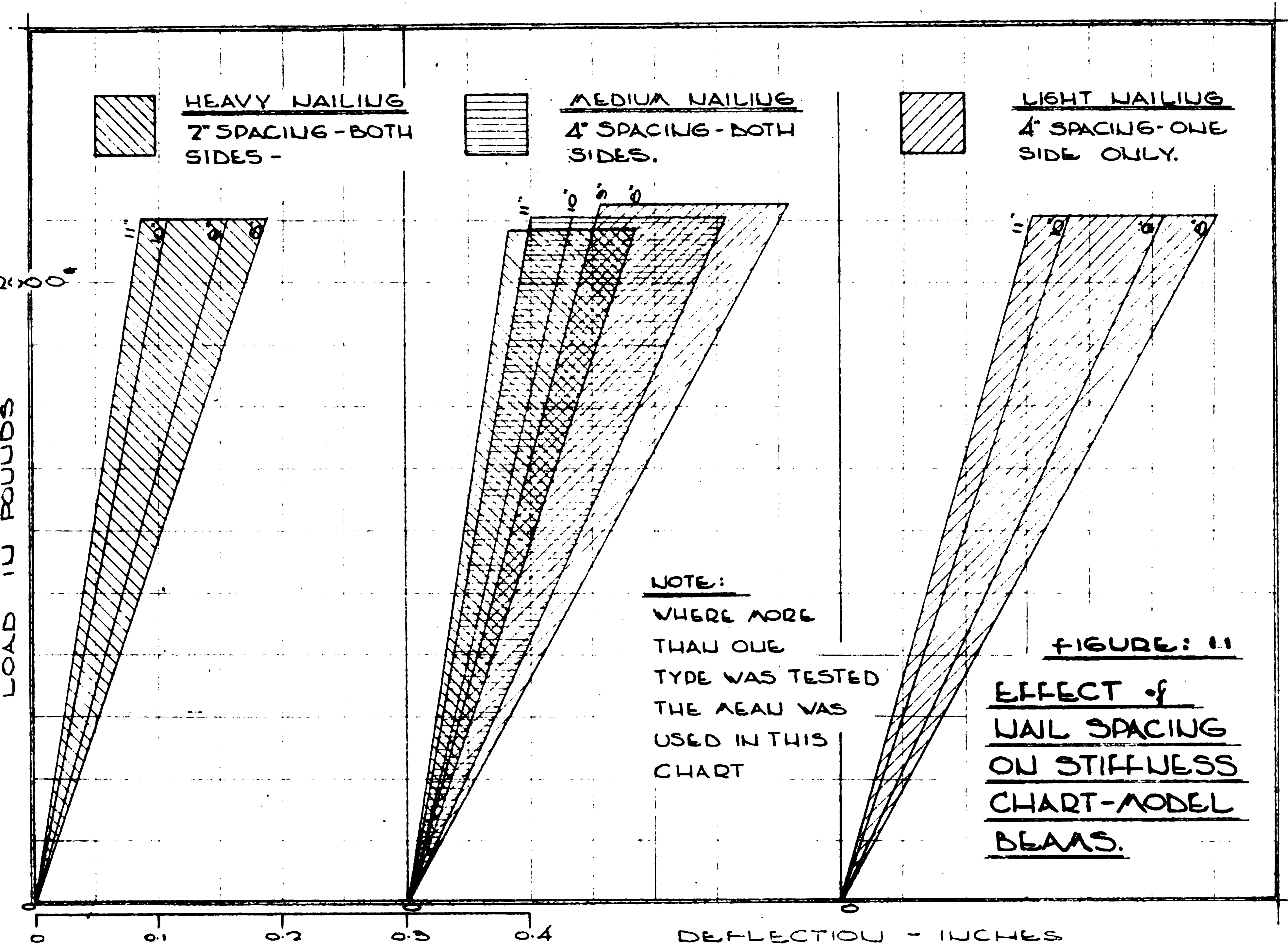

FIGURE: 11
EFFECT of NAIL SPACING ON STIFFNESS CHART-MODEL BEAMS.

calculate these EI values was:

$$EI = \frac{\Sigma A \bar{X}}{\Delta}$$

Where:

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

I = Moment of inertia, inches^4.

$A\bar{X}$ = The sum of ½ of the area of the bending moment diagram, shown in figure 15, multiplied by the distances of their respective centroids from the left edge of the diagram, inches^3. This is sometimes referred to as the second area moment theorem.

Δ = Total deflection measured at mid span, inch.

For results of this EI comparison refer to table 4. Graphic illustrations of EI plots are shown in figure 13. The effect of nail spacing on the stiffness factor can also be seen in figure 13.

Shear Deflection

Shear deflection was calculated for one model test beam to show that it was insignificant and thus could be ignored. For the method of calculation refer to Appendix.

In order that a true comparison of these beams with a nail-glued I-beam of similar section could be made, it was necessary to calculate the shear deflection of an equivalent nail-glued I-beam and add it to the deflection attributable to bending. It was felt that only in this way, could a true comparison be made. Refer to Voelker thesis for calculations[18].

Percentage of Efficiency

The effect of nail spacing on the stiffness factor of both model and full scale beams, EI_{act} values were divided by the EI_{th} values to find the percentage of efficiency. These results are shown in table 5.

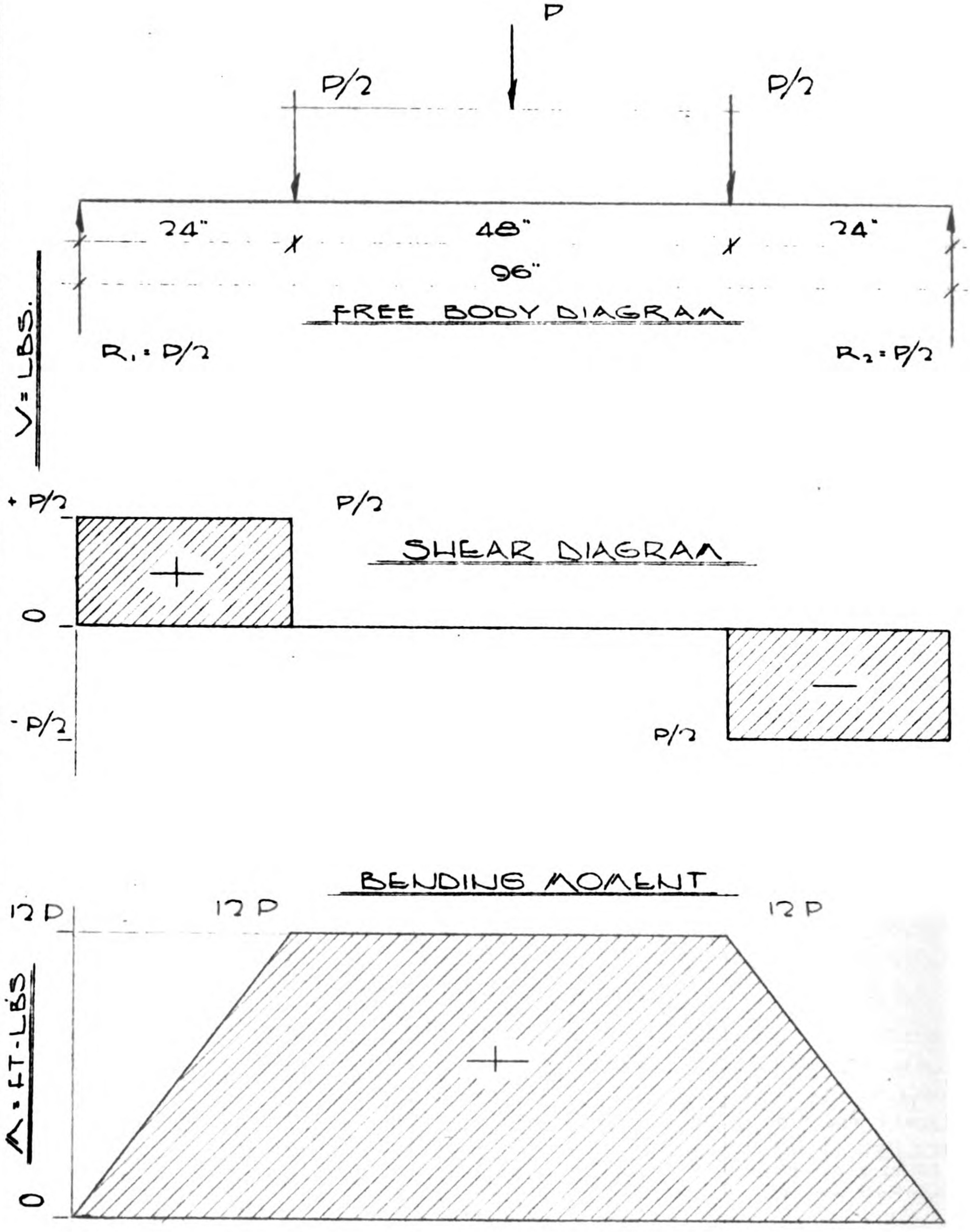

FIGURE: 12
SHEAR & BENDING MOMENT DIAGRAMS
FOR 1/4 POINT LOADING - MODEL BEAMS.

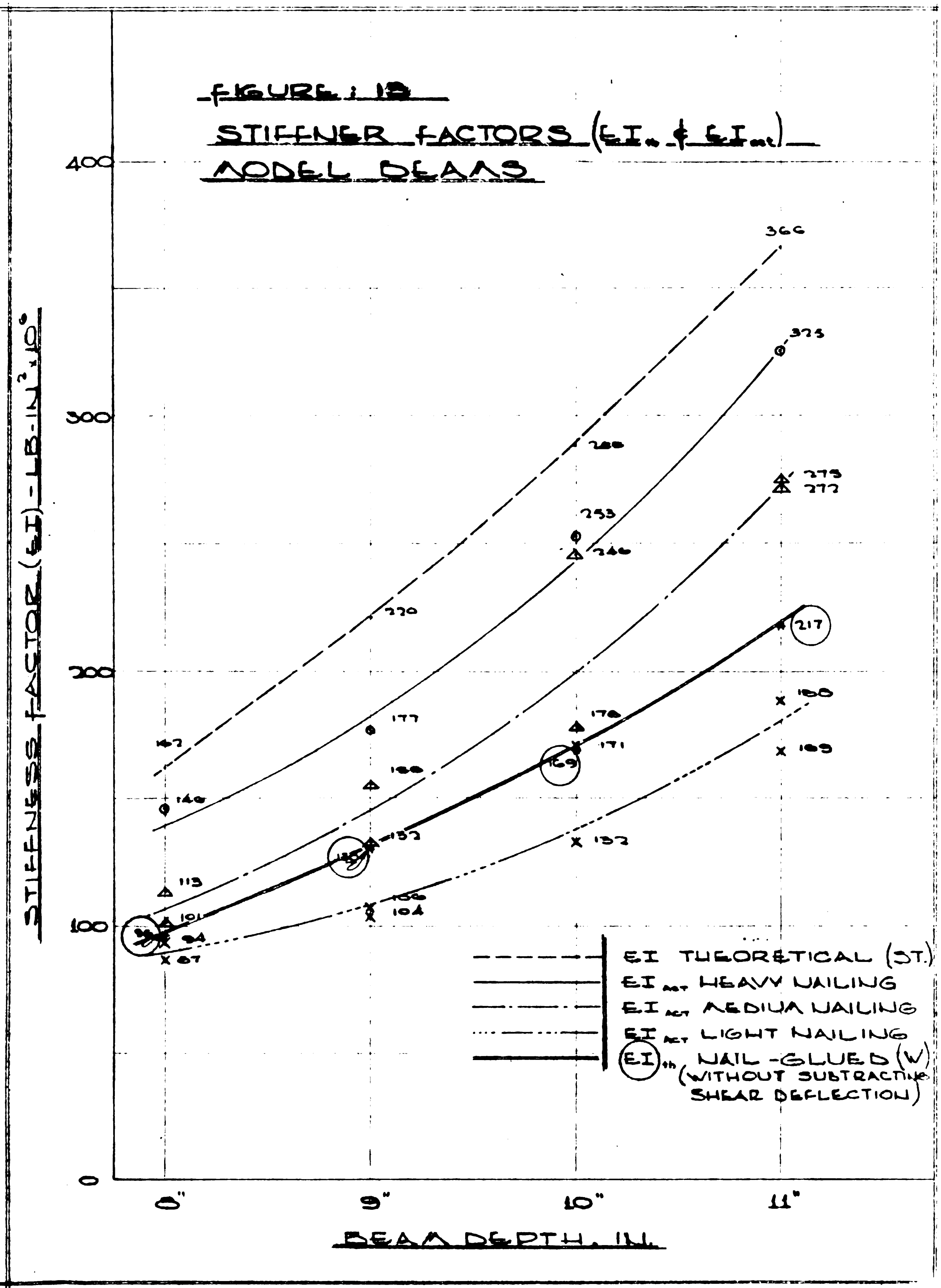
FIGURE: 13
STIFFNER FACTORS (EI & EI act)
MODEL BEAMS
STIFFNESS FACTOR (EI) - LB-IN² x 10⁶
BEAM DEPTH, IN.
0
100
200
300
400
8"
9"
10"
11"
366
325
288
275
272
253
246
220
217
188
177
178
171
169
167
158
146
132
132
128
113
106
104
101
98
94
87
EI THEORETICAL (ST.)
EI ACT HEAVY NAILING
EI ACT MEDIUM NAILING
EI ACT LIGHT NAILING
(EI)th NAIL-GLUED (W)
(WITHOUT SUBTRACTING SHEAR DEFLECTION)

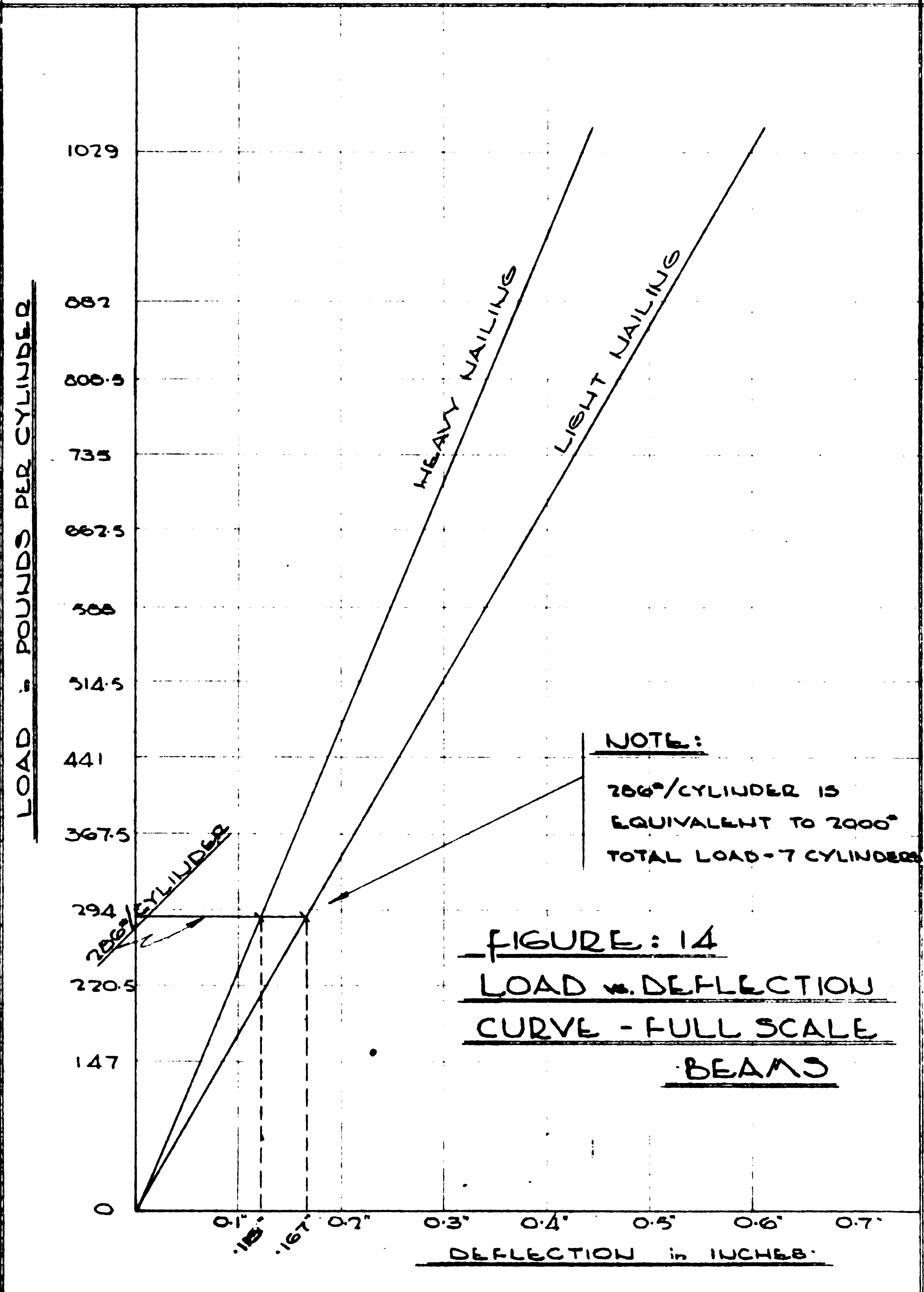

FIGURE: 14
LOAD vs. DEFLECTION CURVE - FULL SCALE BEAMS

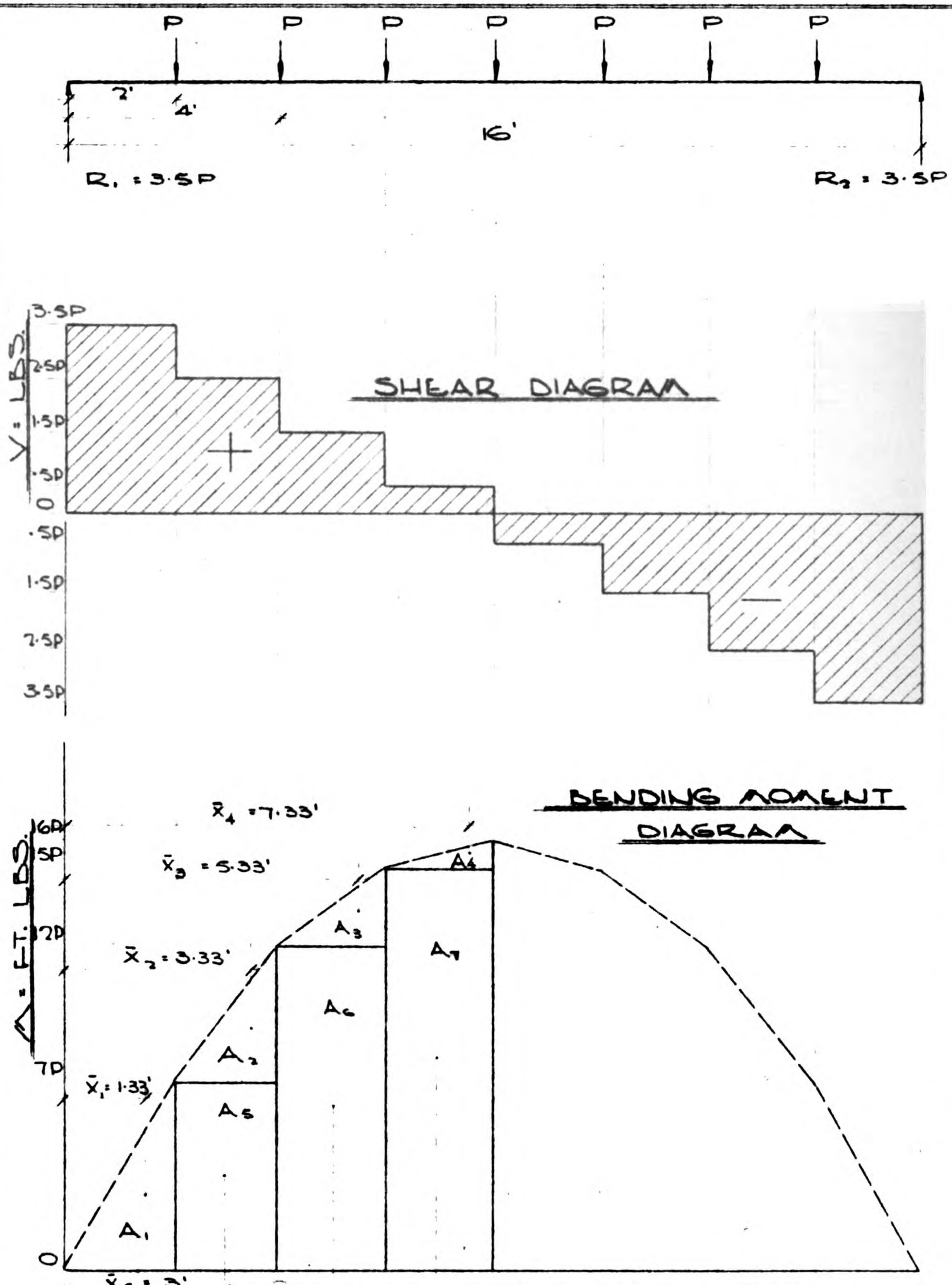

FIGURE : 15
SHEAR & BENDING
MOMENT DIAGRAMS for FULL SCALE BEAMS

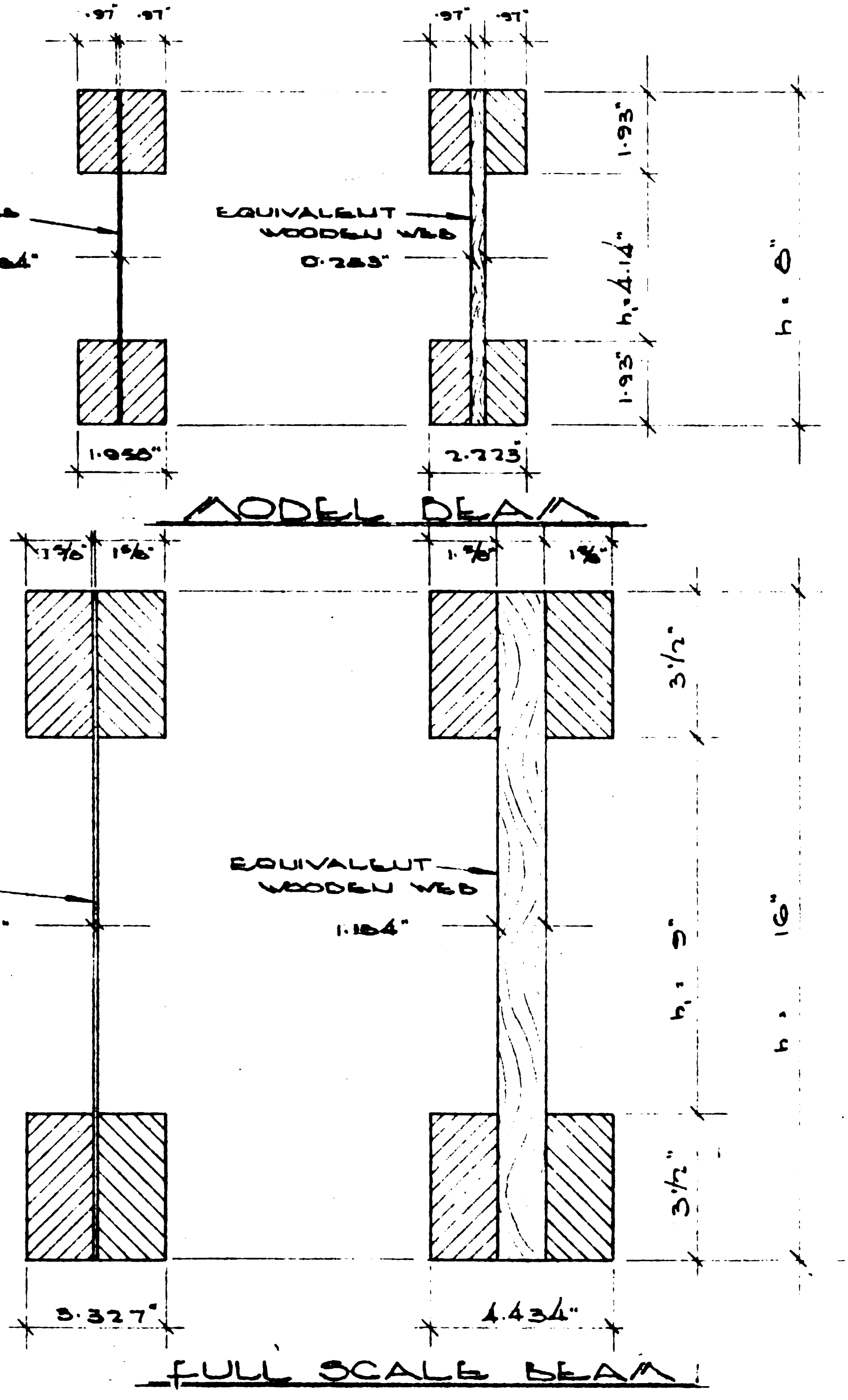

FIGURE: 16

METHOD of EQUIVALENT SECTIONS

TABLE 3

THEORETICAL AND ACTUAL MOMENTS OF INERTIA

MODEL BEAMS

Beam Depth	Beam No.	Nailing* Pattern	Δat 2000#	Av. Δ	$I_{act.}$	$I_{th.}$
8"	1-8	H	.174"	.174"	75 in^4	
8"	2-8	M	.250"	.237"	55	83 in^4
8"	3-8	M	.224"			
8"	4-8	L	.290"	.280"	46	
8"	5-8	L	.270"			
9"	6-9	H	.143"	.143"	91 in^4	
9"	7-9	M	.162"	.177"	73	113 in^4
9"	8-9	M	.192"			
9"	9-9	L	.243"	.241"	54	
9"	10-9	L	.238"			
10"	11-10	H	.100"	.100"	130 in^4	
10"	12-10	M	.103"	.123"	106	148 in^4
10"	13-10	M	.142"			
10"	14-10	L	.148"	.170"	76	
10"	15-10	L	.191"			
11"	16-11	H	.078"	.078"	167 in^4	
11"	17-11	M	.092"	.093"	140	188 in^4
11"	18-11	M	.093"			
11"	19-11	L	.150"	.143"	91	
11"	20-11	L	.135"			

* H = Heavy Nailing; M = Medium Nailing; L = Light Nailing

TABLE 4

COMPARISON OF STIFFNESS FACTORS x 10^6

Beam Depth	Beam No.	Nailing Pattern	Δ at 2000#	$EI_{act.}$	$EI_{av.}$	$EI_{th.}$
8"	1-8	H	.174"	146	146	
8"	2-8	M	.250"	101	107	
8"	3-8	M	.224"	113		162
8"	4-8	L	.290"	87	91	
8"	5-8	L	.270"	94		
9"	6-9	H	.143"	177	177	
9"	7-9	M	.162"	156	144	
9"	8-9	M	.192"	132		220
9"	9-9	L	.243"	104	105	
9"	10-9	L	.238"	106		
10"	11-10	H	.100"	253	253	
10"	12-10	M	.103"	246	212	
10"	13-10	M	.142"	178		288
10"	14-10	L	.148"	171	152	
10"	15-10	L	.191"	133		
11"	16-11	H	.078"	325	325	
11"	17-11	M	.092"	275	274	
11"	18-11	M	.093"	272		366
11"	19-11	L	.150"	169	178	
11"	20-11	L	.135"	188		

TABLE 5

PERCENTAGE EFFICIENCY COMPARISON

<u>Model Beam</u> (8')

Beam Depth	Nailing Pattern	% Efficiency
8"	Heavy	86%
9"	"	82
10"	"	83
11"	"	89
8"	Medium	66%
9"	"	65
10"	"	69
11"	"	74
8"	Light	56%
9"	"	49
10"	"	48
11"	"	49

<u>Full Scale Beam</u> (16')

16"	Heavy	78%
16"	Light	57%

<u>Equation</u>:

$$\frac{EI_{act}}{EI_{th}} \times 100$$

Where:

EI_{act} = Actual Stiffness Factor

EI_{th} = Theoretical Stiffness Factor

DISCUSSION OF RESULTS

A. Model Beams

This study was centered around three objectives: 1) to compare theoretically predicted stiffness factors (EI_{th}) with actual stiffness results (EI_{act}) derived from tests, 2) to evaluate the percentage of efficiency of these beams in relation to the number of nails used in fabrication, and 3) to compare the effectiveness of these beams with a nail-glued plywood I-beam of similar section in terms of theoretically predicted stiffness factors derived by use of standard engineering equations.

All beams tested ultimately failed through buckling. There was no evidence, whatever, of shear failure in the web. This was due to the very high shear stress resistance of steel. All buckling occurred in the outer 25% of the beam span, without exception. Buckling of the metal web was most noticeable at both points of loading and from those points to the supported ends of the beam. No buckling was evidenced at mid span. There was some evidence of flange slippage, as is shown in figure 6, but it was considerably reduced through heavy or more concentrated nailing.

All beams reacted to buckling and twisting in much the same manner. Some failed under far less load than others. This can be explained through lack of lateral support and in defects in lumber of the flanges. Because of defects in the flange material, some beams were constructed with a slight twist and thus, because of the rigidity of the metal web, tended to twist and spring out of the testing machine at a point far below the proportional limit. This absence of lateral support in the testing machine gave rise to rather eratic results as the load was increased. This variable was eliminated in the full scale tests.

Load vs. deflection curves can be noted in figure 10 and these will show that all test beams, with the exception of the eight inch beams with light nailing, were well above the standard allowable deflection of 1/360 of the span. As the depth-span ratio increased and the concentration of nails became greater, the actual deflection was much less than the standard allowable deflection.

Referring to figure 13, showing the theoretically predicted stiffness factors (EI_{th}) and the actual stiffness plots (EI_{act}) for the different range of nailing;- heavy, medium and light, it will be noted that as the nailing concentration increases, the EI_{act} approaches the EI_{th}. Table 5 will show the percentage of efficiency due to nailing. This runs from a low of 48% in the case of light nailing, to a high of 89% in the case of heavy nailing. Therefore, in situations where a greater stiffness is required, the concentration of nails should be increased.

Shear deflection for one model beam was calculated and this can be found in the Appendix. It was found that the deflection caused through shear was less than 2% and thus could be ignored. In comparing EI_{th} and EI_{act} of the wood flange-steel web I-beams with a nail-glued plywood I-beam of similar section, reference is made to results obtained by Voelker[18] in another research project which was conducted at the same time . In those results the deflection which was caused through shear of the plywood web, was subtracted from the total observed deflection to arrive at a bending deflection. Therefore, since shear deflection was of no consequence in the steel web type beam, the only true comparison of the two beams was to add the deflection due to shear to that deflection due to bending to arrive at a theoretically true stiffness factor (EI_{tt}). This was done and the results can be observed in figure 13. The method of calculating the shear and bending deflection can be found in the Appendix.

It was found that 39% of the total deflection in a nail-glued plywood I-beam is attributable to shear whereas less than 2% is due to shear in the steel web type beam. Therefore, if shear deflection is neglected, the steel web I-beam produces a 78% efficiency in the case of heavy nailing compared to 61% for nail-gluing. The efficiency of light nailing drops down to 57%, however, and is slightly less efficient than a nail-glued plywood I-beam of similar section.

The effect of nail spacing on deflection can be seen garphically illustrated in figure 11. The range of deflections for eight, nine, ten and eleven inch beams and in groups of light, medium and heavy nailing have been superimposed and the effect can be readily observed from the same graph.

Buckling was the most important consideration in the use of these beams. They tend to be considerably more rigid than a comparable nail-glued beam. Their effectiveness, however, would be seriously curtailed if used in a situation where the lateral stability could not be controlled.

B. Full Scale Beams

In testing the two full scale beams, every attempt was made to control lateral stability in order that results would be more uniform. One beam was heavy nailed; the other was light nailed.

The resulting observed deflection in these tests correlated very closely with results from model beam tests. EI_{th} and EI_{act} were calculated and the percentage of efficiency worked out and compared with model beam results. The correlation was excellent. These can be seen in table 5.

Because of the load limitations of the testing machine used, the proportional limit was not evidenced. At no point was any damage to the beams observed and only a very slight buckling occurred when maximum load was applied. When pressure was released, no damage of any description was evidenced.

In neither the model nor the full scale beam tests was an attempt made to correlate the modulus of elasticity values. Average modulus of elasticity values were determined for both types of beams and an analysis of variance calculated for small bending samples of the same stock as used in model beam fabrication. The results of this analysis can be seen in the Appendix. 99% of the samples fell within two standard deviations with the majority falling within one standard deviation.

CONCLUSIONS AND RECOMMENDATIONS

As a result of this research project, it is concluded that:

1. Wood flange-steel web I-beams are stiffer than nail-glued plywood I-beams of similar section. This is true where heavy or medium nailing is used. However, where relatively few nails are used, as in the case of light nailing, the nail-glued I-beam appears to be slightly superior. This conclusion is based on the assumption that shear deflection is not deducted from total deflection in calculating the stiffness of the nail-glued plywood I-beam. It is felt that this is the only accurate method of comparison since the ultimate " on-job " strength should be the only consideration in evaluating the two types.
2. Nailing is an effective method of fastening the wood flanges and stiffners to the sheet steel web. The efficiency of this method of bonding increases proportionately with the number of nails used. Gluing would add materially to the cost because of the type of adhesive required to give an adequate bond between wood and metal.
3. Shear deflection is of practically no consequence in design of this type of beam and can be ignored since it is less than 2%.
4. That lateral instability is a most important consideration in the design of this beam for ultimate failure. It becomes more critical as the span-depth ratio increases. However, it should be noted that it does not become a problem until well beyond the recognized allowable design

deflection which is normally taken to be 1/360 of the span.

The results obtained in this study tend to indicate that the beam has considerable merit. The negligible effect of deflection due to shear as well as the fact that an adequate bond can be achieved without the use of an adhesive, would make this type of beam more practical, than the nail-glued plywood I-beam, for many job situations.

It is felt that the cost of these beams may be slightly higher than for the nail-glued beam. However, the added strength, stiffness and the ease of fabrication may well offset the additional cost. Further steps should be taken to evaluate the comparative cost factors involved.

Further study should be undertaken with a much larger sample to arrive at more conclusive results. The results presented, indicate the need for further work, to arrive at empiracle design equations. Further experimentation could be done with different spacing of stiffners, i. e., four feet on center instead of two feet. Other types and gages of steel could be tried.

APPENDIX

ANALYSIS OF VARIANCE

N = 40 Small Bending Samples.

E = Modulus of Elasticity

No.	E	No.	E
1.	2.07×10^6	21.	1.24×10^6
2.	2.28	22.	1.16
3.	1.80	23.	2.58
4.	1.73	24.	1.48
5.	2.50	25.	2.49
6.	2.14	26.	1.97
7.	2.45	27.	1.99
8.	2.51	28.	1.51
9.	2.59	29.	1.97
10.	1.68	30.	2.25
11.	1.61	31.	1.52
12.	1.85	32.	1.85
13.	1.82	33.	1.53
14.	1.58	34.	1.70
15.	1.72	35.	1.79
16.	1.57	36.	1.75
17.	2.72	37.	1.93
18.	1.53	38.	2.11
19.	1.56	39.	1.83
20.	1.74	40.	2.04

$E_{average} = 1.95 \times 10^6$

$$\sigma = \sqrt{\frac{\Sigma x^2}{N} - \left(\frac{\Sigma x}{N}\right)^2} = \sqrt{\frac{150.4113}{40} - \left(\frac{76.14}{40}\right)^2}$$

$$= \sqrt{3.760,283 - 3.623,312} = \sqrt{.082,971}$$

$$\sigma = \pm .29$$

Therefore, 1 standard deviation $= \pm .29$

SHEAR DEFLECTION

$\Delta = \Delta_b + \Delta_s$

and for uniform loading, simply supported beam

$$\Delta_b = \frac{5\,W\,L^4}{384\,E\,I}$$

$$\Delta_s = \frac{K\,L^2\,W}{8\,A\,G}$$

Where:

Δ = Deflection at mid span, inches

W = Load, lbs. per inch of span

L = Span, inches

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

I = Centroidal moment of inertia of area, inches^4

A = Cross-sectional area, inches^2

G = Modulus of rigidity, p. s. i.

K = Shear deflection constant depending on geometry of section.

For the case of an I-beam section:

$$K = \frac{A}{b_1 I}\left[\frac{bh^2}{8} - \frac{h_1{}^2}{8}(b - b_1)\right]$$

Where:

b_1 = Web thickness, inches

b = Flange width, inches

h_1 = Web height, inches

h = Total height of section, inches.

For the case of a beam with 14 gage steel web and nominal 2" x 4" flanges, an equivalent section is used for the case of computing the relative portion of deflection due to shear.

Thus, for a beam of 16 inch total depth:

$b_1 = 0.0766$ inches

$b = 0.0766 + 2(1.63)\ \dfrac{2.0 \times 10^6}{30 \times 10^6} = 0.295''$

$I = \dfrac{0.295\ (16)^3 - 0.0766\ (7.26)^3}{12} = 96.8\ in^4$

$G = 12 \times 10^6$ p. s. i.

$E = 30 \times 10^6$ p. s. i.

$h = 16.0''$

$h_1 = 8.74''$

$K = 2.44$ (from solution of the above equation)

$A = 2.77\ in^2$

For an arbitrary load, W, below the proportional limit Δ_b and Δ_s may be computed. For the fraction due to shear deflection as a percentage:

$$\%\ \Delta_s = \frac{\Delta_s}{\Delta_b + \Delta_s} \times 100\% = 2\% \text{ for this case.}$$

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