NAIL-GLUED WOOD-PLYWOOD I-BEAMS FOR RESIDENTIAL CONSTRUCTION

Thesis for the Degree of M. S.

MICHIGAN STATE UNIVERSITY

Donald Frederick Luebs

1959

NAIL-GLUED WOOD-PLYWOOD I-BEAMS
FOR RESIDENTIAL CONSTRUCTION

By

Donald Frederick Luebs

A Thesis

Submitted to the Faculty

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

1959

ACKNOWLEDGEMENTS

The writer is deeply indebted to Professor B. M. Radcliffe for his direction, assistance and continued interest in the preparation of this paper.

Sincere thanks are also due Doctor A. W. Sliker for his helpful suggestions and valuable advice.

TABLE OF CONTENTS

LIST OF FIGURES

ABSTRACT

There are many places in the structure of a house where clear-span wood-and-plywood girders can be used to eliminate supporting walls and columns and in general allow more design flexibility and facilitate construction. Appreciable savings in material and weight are possible by using a built-up construction which places material where it may resist stresses most efficiently. It has been demonstrated that costs may be reduced in many cases by replacing other structural members with such beams.

The basic purpose of this investigation was to evaluate the practical application of certain plywood beams of simple construction for residential use. Such a beam must be easy to construct and must employ locally available materials to be of real utility. In addition, it is desirable that no special equipment be required in assembly.

Full scale tests were run on eighteen beams of various section depth; the test method simulated actual use conditions. Load-deflection data was taken, performance was observed and the nature and location of failures was noted.

It was determined that nail-gluing with casein glue was an efficient and practical method of joining members which seemed ideally suited to this particular application. Shop fabrication where conditions are more easily controlled is recommended. The beams should be used in protected ventilated locations and should not be subjected continuously to high humidity conditions.

It was concluded that beams of this description should be designed on the basis of allowable deflection at midspan. No failures occurred in the web due to horizontal shear stress even though calculations indicated values much in excess of the arbitrarily accepted allowable stresses. A well-balanced beam would not result if such values were used in the design. Compressive stress at supports should be checked on heavily loaded beams to eliminate the possibility of crushing of the wood.

Sections of the web may be cut out subsequent to fabrication if the proper load-reduction factor is employed. The removal of such panels might be necessary in construction to accommodate heat ducts, plumbing, etc.

These nail-glued plywood girders should prove more economical than steel beams or wood-steel flitch plate beams in many applications in residential construction. They will certainly be much easier to handle and position as well as fasten in place and apply finishing materials to.

NAIL-GLUED WOOD-PLYWOOD I-BEAMS FOR RESIDENTIAL CONSTRUCTION

INTRODUCTION

There are many places in residential construction where built-up wood-plywood girders may be advantageously used to replace more expensive structural members. Such installations would include headers above garage door and other wide openings, main floor support girders and wall and ridge beams for post and beam construction. The use of such beams for clear-span floor and roof systems is also a promising application.

Because the design of these girders places material where it may be stressed to the best advantage, considerable savings in wood as well as weight are possible. This combined with the high natural strength-weight ratio of wood is an important factor; handling ease is promoted while dead weight is reduced. For example, a wood-plywood girder for a 16 ft. garage door header weighs less than half its steel I-beam counterpart of similar strength and stiffness.

The fastening problem encountered with steel members is greatly simplified since the plywood girder may be nailed in place. Finishing materials may also be nailed directly to the beams. This is a great contribution to flexibility of construction.

Working stresses for any grade of lumber within a species are dependent upon a number of possible defects, particularly in the center third of the span. The built-up construction of the plywood beam promotes an increase in strength as the randomly occurring defects in the members glued together reinforce one another. Shrinkage and twisting tendencies found in solid wood members are also resisted by the built-up construction.

One of the biggest problems with any built-up construction is adequate joining of the members so that stresses may be distributed directly to the members which are designed to resist them. Often unnecessary costs are incurred by oversizing members merely to accommodate fasteners. A good glue joint of sufficient area is the most efficient device for joining these members, but the use of presses or clamps is often a hinderance, especially with larger assemblies. It has been demonstrated that the use of nails to apply pressure to glued joints is a simple yet effective method.[10] Such nail-gluing seems to be ideally suited to built-up plywood beams.

In the series of tests reported in this paper it was the object to evaluate the practical application of certain selected nail-glued, wood-plywood I-beams for residential construction. The beams were to be simple in design and constructed of stock-size materials with no special equipment. Full-scale tests which simulated actual use conditions were to be run and data was taken to evaluate the performance. The nature and location of the failures were to be observed.

PREVIOUS WORK

History

There has been much development in structural uses of plywood in the past 25 years. Previous to 1935 little such development had taken place, partially due to a limited supply of plywood and partially due to the lack of suitable adhesives. Actually, development was taking place only slowly previous to World War II.

One of the earliest intensive uses of plywood beams in this country was in a 125,000 sq. ft. warehouse built in 1942 for the RCA Manufacturing Co. at Camden, New Jersey.[19] A total of 198 identical plywood girders, 36 ft. 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.

With the advent of World War II a search for structural members other than steel took place. The necessity of conserving materials was a guiding policy in this search.

Design Data

Extensive work was done in the years 1943-1944 in the design of box beams by the Forest Products Laboratory under the supervision of the Aeronautical Board of the U. S. government. They applied conventional engineering calculations using allowable design values of wood and made recommendations concerning the webs, flanges, and stiffeners of box beams.

They were largely concerned with buckling of the web[14,15] and horizontal shear stresses.[12] 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°, however, were found to be substantially more efficient.[15] They also found that for thin beam webs significant increases in web shear resistance could be secured by reducing stiffener spacing.[13]

David Countryman in full-scale tests of plywood beams in 1944, found that nail-gluing was an effective method of fabrication.[10] 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.

The Forest Products Laboratory has published some approximate methods of calculating the strength of plywood based on the basic allowable stresses for Douglas-fir.[16] They allow a 25% increase in recommended working values for use in dry locations (M.C. 16% or less) with a reduction depending on the grade of plywood used. These allowable working stresses are widely applied today and are recommended in other countries[1] as well as in later publications in the United States.[6,9]

The Douglas-fir Plywood Association has conducted additional tests and developed simplified design methods for plywood box and I-beams.[9] The following procedure is recommended. The type of beam and method of manufacture is decided upon, then a preliminary cross-section is selected and checked for flexure, horizontal shear, shear between the web and flange and deflection. In addition it is recommended that the upper flanges be considered as columns to compute the degree of lateral stability.

David Countryman and Vernon D. Haskell have presented this same information for D.F.P.A.[11] It is in a more generally usable form with tables of design factors presented to further simplify it.

The Wood Handbook contains a unit on the design of wood-plywood beams which is a collection of data from several sources.[7] It is in a very usable form simplified to the point where beams of constant cross-section are demonstrated. It is suggested that the sizes of beams may be limited by: (1) Clearance or other headroom limitations (2) Width of plywood economically available, (3) Ratio of height to width and span for lateral stability. These factors should be evaluated previous to considering beam design.

Most of the beams originally tested were constructed to resist stresses in the most traditionally efficient engineering manner. Many of them had complicated cross-sections which varied along the length so that flange area increased toward the center to resist bending stresses while web area increased toward the ends to resist shear. This would require non-stock specially-milled lumber in addition to complicated fabrication procedure.

Many of these beams did not fail as the engineering data had predicted. This indicated that either the data did not accurately picture the stresses present or that the usual arbitrarily selected allowable stresses for the materials were at fault; very possibly both conditions were present. Far more realistic design is possible by basing criteria on actual performance.

While knowledge of built-up plywood beams is fairly widespread and recognition of their attributes is given, it appears that relatively little work has actually been done on them. Most design data and recommendations originate directly from one or two sources.

MATERIALS AND METHODS

Fabrication

Test beams were constructed in 24 ft. lengths and depths ranging from 14 to 22 inches. Referring to Fig. 1, a constant, balanced I-section was used with nominal 2 x 4 members glued to either side of the 3/8" plywood webs to form the top as well as the bottom flange. This construction gave a beam of 3-5/8 in. thickness which would fit conveniently into a conventional stud wall.

The flanges consisted of central 2 x 4 - 16's with 2 x 4 - 4's spliced to each end to form the 24 ft. length. Splices were directly opposite each other.

The flange members for the test beams were of mixed Douglas-fir and White Fir of No. 1 grade. No attempt was made to select lumber except that bright lumber was specified. Low-grade material was considered suitable for the 2 x 4 stiffeners. Moisture content of the lumber ranged from 14% to 21% with an average of 17%. Modulus of Elasticity values averaged $.94 \times 10^6$ in ten full-sized 2 x 4 samples tested which were obtained from the same source (See Appendix A).

Stiffeners were placed at two-foot intervals along the length of the beam; thus, a stiffener occurred at the flange splices and at splices in the plywood web. Stiffeners were fitted accurately between top and bottom flange members to restrain their movement toward each other as the beam was stressed. Their main purpose, however, was to resist buckling of the web.

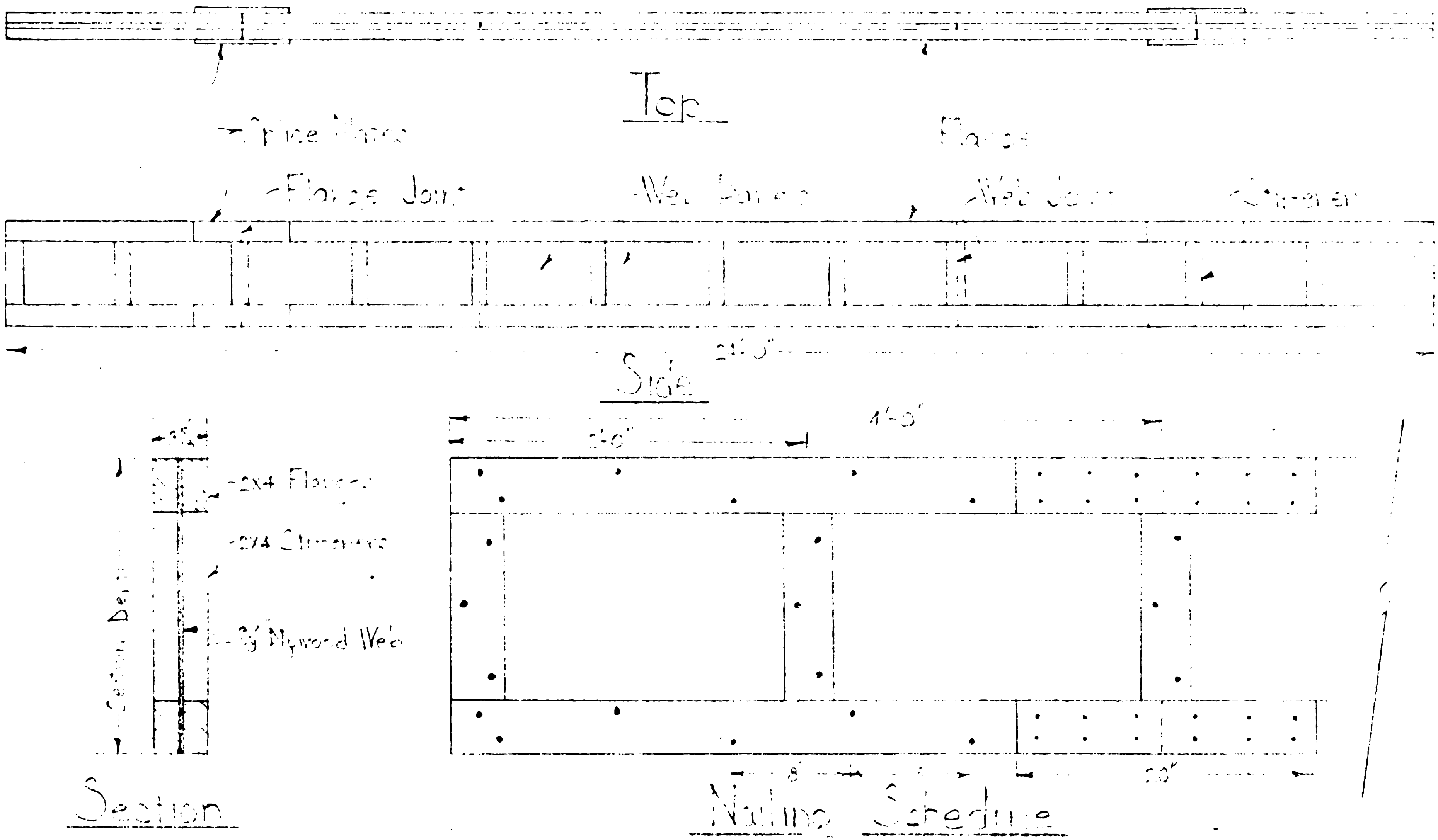


Figure -1-

CONSTRUCTION DETAILS and NOMENCLATURE of NAIL-GLUED I-BEAMS

Casein glue meeting U. S. specification MMM-A-125 was used as the adhesive. A relatively heavy mixture ($2:1\frac{1}{2}$, water to glue by weight) was used to increase gap-filling qualities. A single spread of 110 lbs./1000 sq. ft. was used. The ambient temperature was 75°F. during fabrication. Sufficient glue squeeze-out evidenced adequate contact.

In the assembly of the girders, the flanges and stiffeners for one side only were first laid out and tacked together. Glue was then spread on these members. The three plywood web sections, each 8 ft. long, were subsequently laid on this assembly and tacked in place. These consisted of 3/8 in. Douglas-fir interior grade Plyscord. The flange members and stiffeners for the remaining side of the beam were then spread with glue and laid in place.

Pressure was applied by 16d common nails driven down through the entire assembly from the one side only. Nails were spaced at 8 in. and staggered; they were driven hard to promote good contact at the glue line. Three nails were driven into each pair of stiffeners. The nails are to bring pressure to bear on the glue line; they do not contribute to the strength of the beam except to support it in moving immediately after assembly.

The last step in fabrication was to spread the splice plates with glue and secure them by nailing to both sides of the beam. The plates were 20 in. long with 10 in. over each of the butted flange 2 x 4's. Six 6d common nails were used on each side of the butt-joint. Splice plates were of 1 x 4 soft pine to resist splitting; 3/8 in. plywood was used in several beams to observe its performance.

Beams were stock-piled immediately after fabrication. Care was used in stacking to avoid deformity which would remain after the glue had set. A minimum curing time of 24 hours was allowed before testing.

Method of Test

The girders were erected in pairs 20 in. O.C., simply supported at the ends on 2 x 4 "plates" resting on concrete blocks; cross-bracing was used at the ends only. A 1/2" plywood roof was tacked across the pair to form a loading platform. A typical test set-up is shown in Figure 2.

Nails were driven into the stiffeners at either end of each beam just above the supports at the neutral axis. A nylon line was then stretched along the span of each beam by tying to the nail at one end and suspending a weight over the other. A scale with 1/10 in. increments was fastened to the stiffener at the mid-span of each beam so that deflection readings could be taken by referring to the nylon string. The string was positioned as close to the scale as possible without touching the beam. Figure 3 shows this deflection-measuring device.

A "zero" deflection reading was taken previous to loading the beams. The load was then applied in the form of 8 in. concrete blocks which weighed 38 lbs. each as determined from a representative sample. Blocks were placed two abreast along the length of the beams. A deflection reading was taken after each layer of 74 blocks (1400 lbs. per beam) had been applied. Figure 4 shows a pair of test beams being loaded.

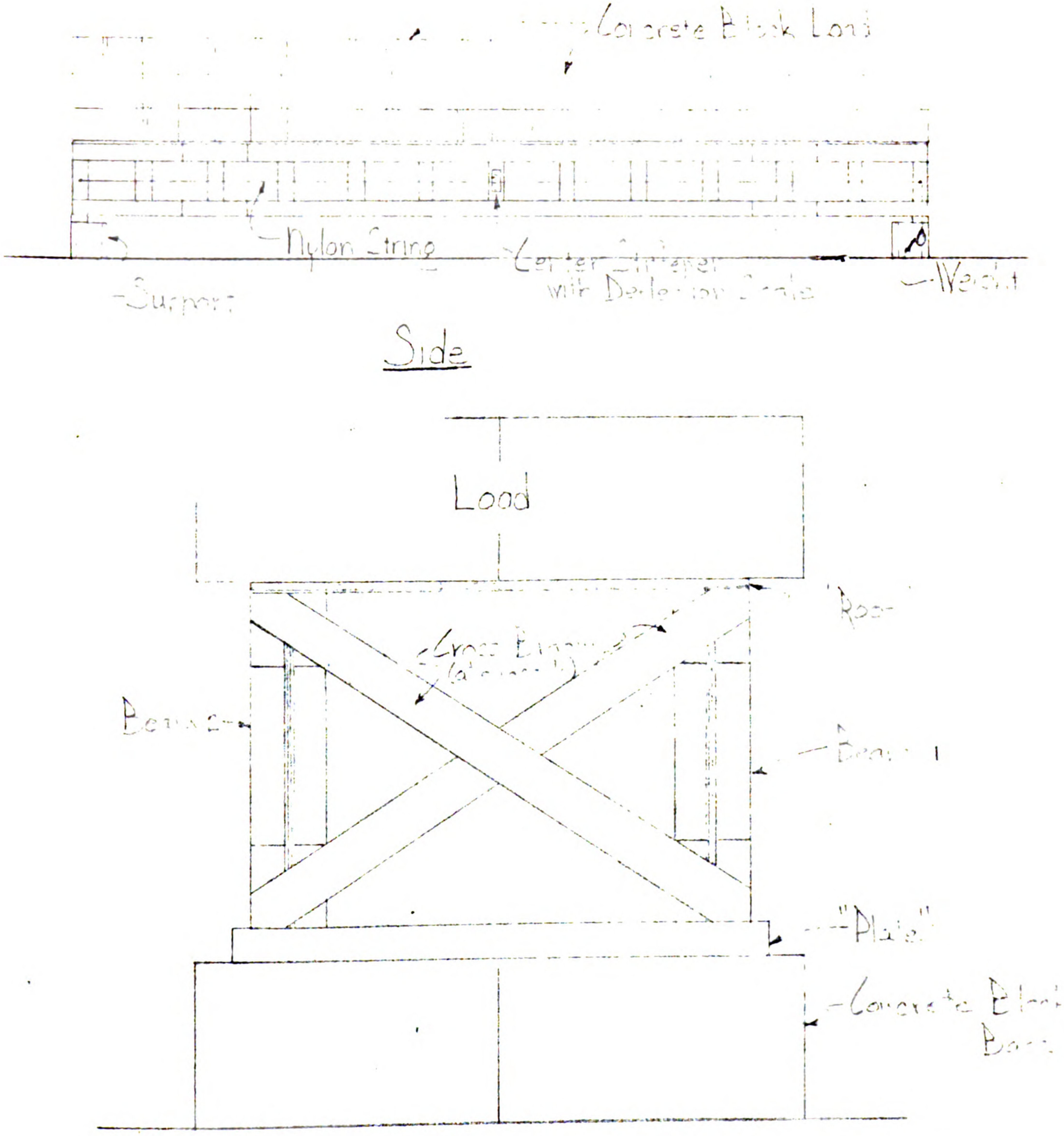


Figure -2-

METHOD OF TESTING BEAMS

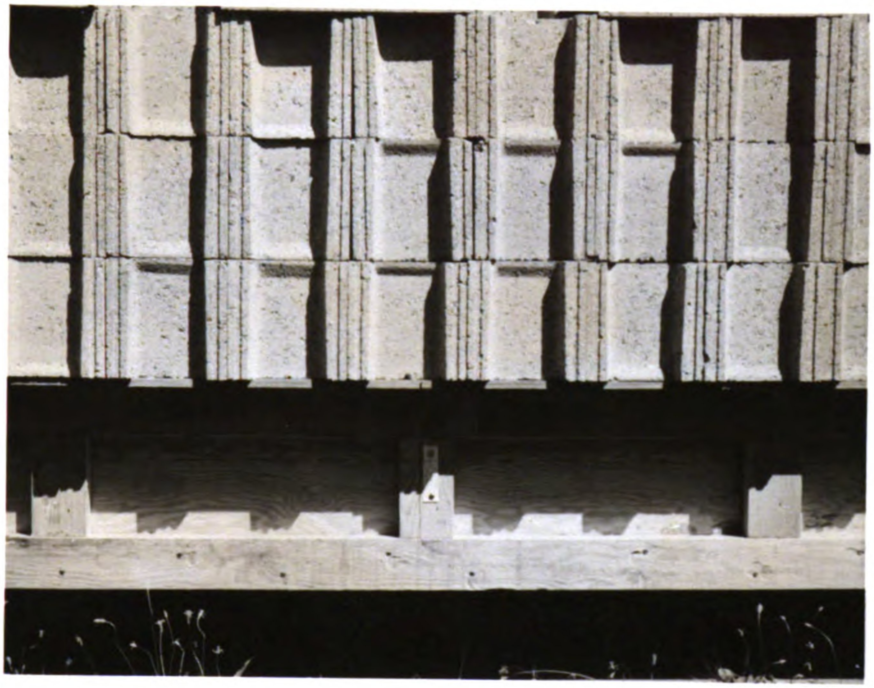


Figure 3

Deflection Measuring Device

Figure 4

Beams Being Loaded

Test to failure was conducted on beams of 14 in. and 16 in. section depths. Figure 5 shows a typical test pair after failure. For beams of deeper sections six layers of blocks were applied. This was sufficient to cause several times the allowable deflection without collapse. Figure 6 shows such a typical test pair under full load.

A subsequent series of tests was conducted to determine the effect of removing panels of the plywood web between adjacent stiffeners. In actual usage such openings might be required to provide for heat ducts, plumbing or other utilities.

Panels were removed from various positions in the four 18 in. beams for these tests. The beams had been previously loaded for deflection data as described above,but stresses had not been allowed to approach proportional limits.

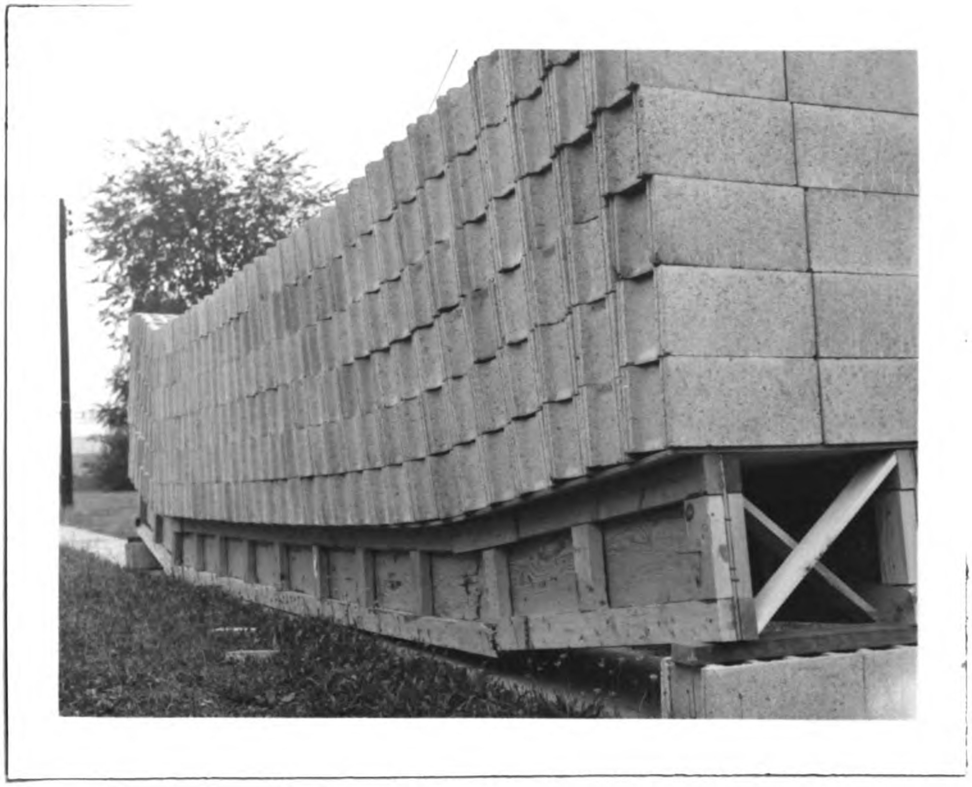


Figure 5

Smaller Beams After Failure

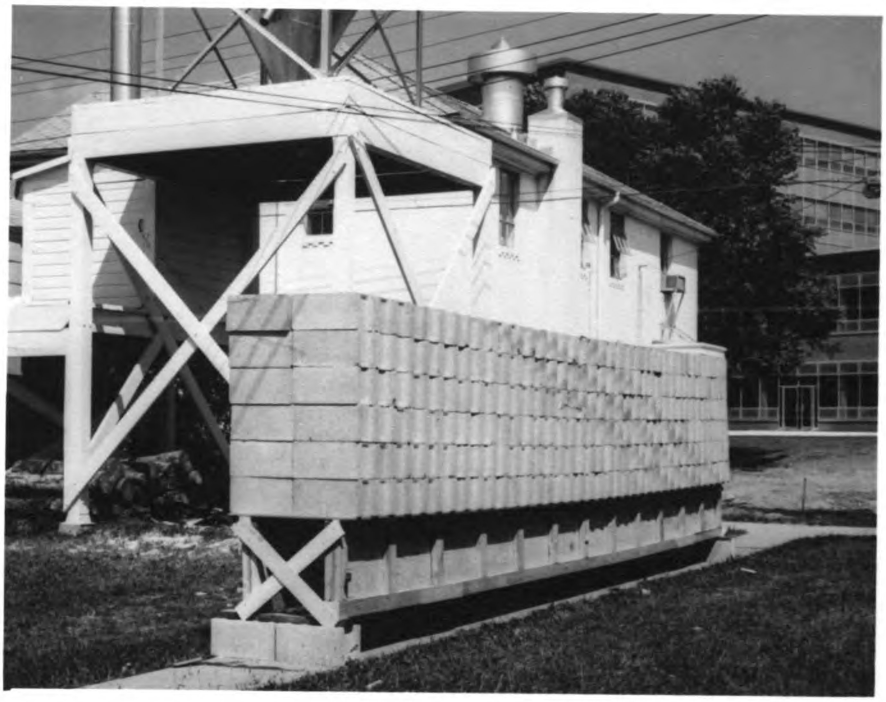


Figure 6

Deeper Beams under Full Load

TEST RESULTS

Loads at Allowable Deflection

A load versus mid-span deflection curve was drawn for each beam tested. The plots are shown in Figure 7. The loads at which a deflection of 1/360 of the span occurred (.8 in. for a 24 ft. span) are noted in Table 1 of Appendix B.

Stiffness Factor

For each beam a deflection at corresponding loads was used to compute a stiffness factor according to the equation for a simply supported beam under a uniformly distributed load.

$$EI = \frac{5wl^4}{384\Delta}$$

where:

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

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

W = load, lbs. per lin. in.

l = span, in.

Δ = deflection, in.

These experimental stiffness factors are given in Table 2, Appendix B. They are also compared with expected values in Figure 9.

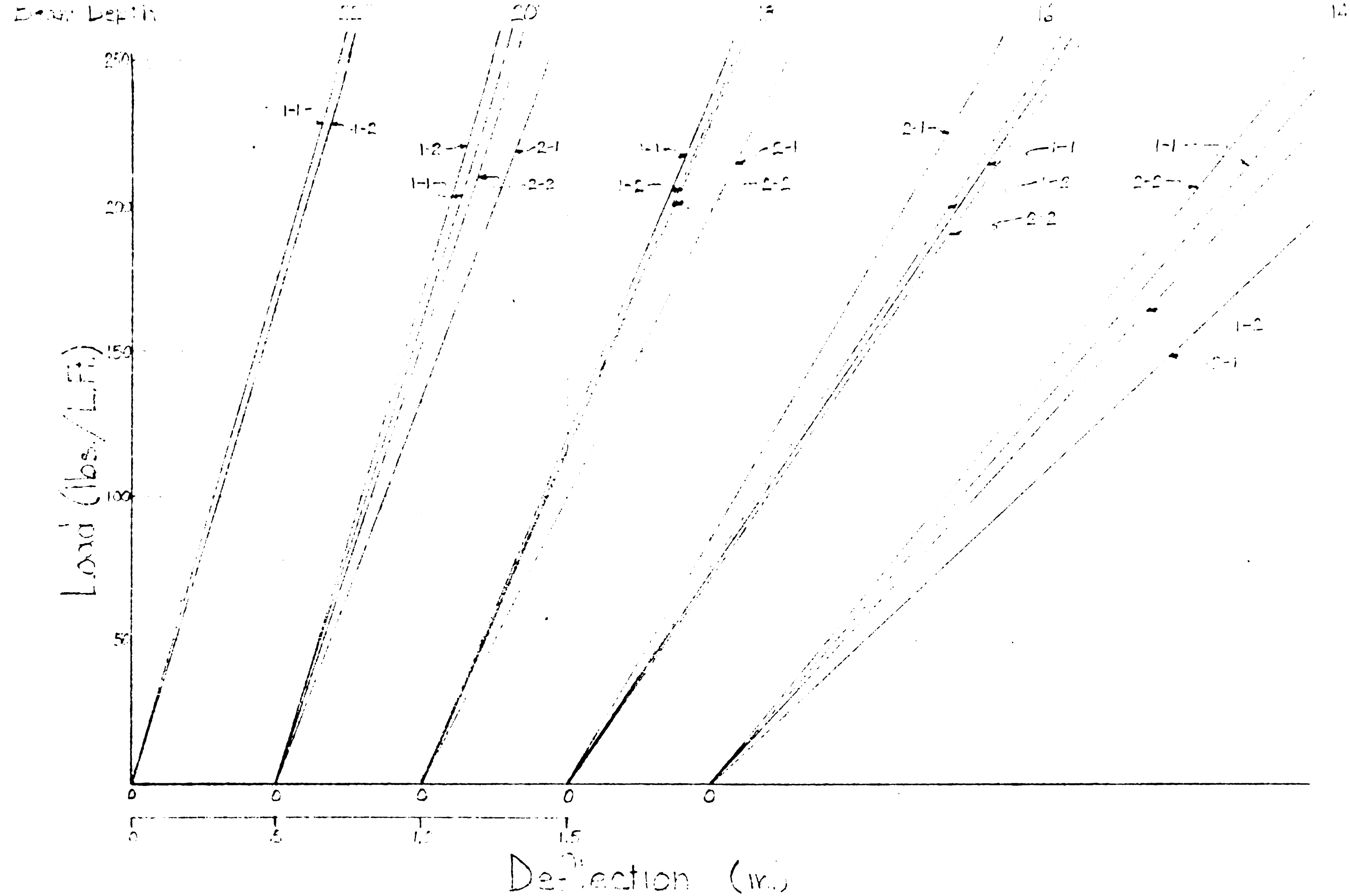
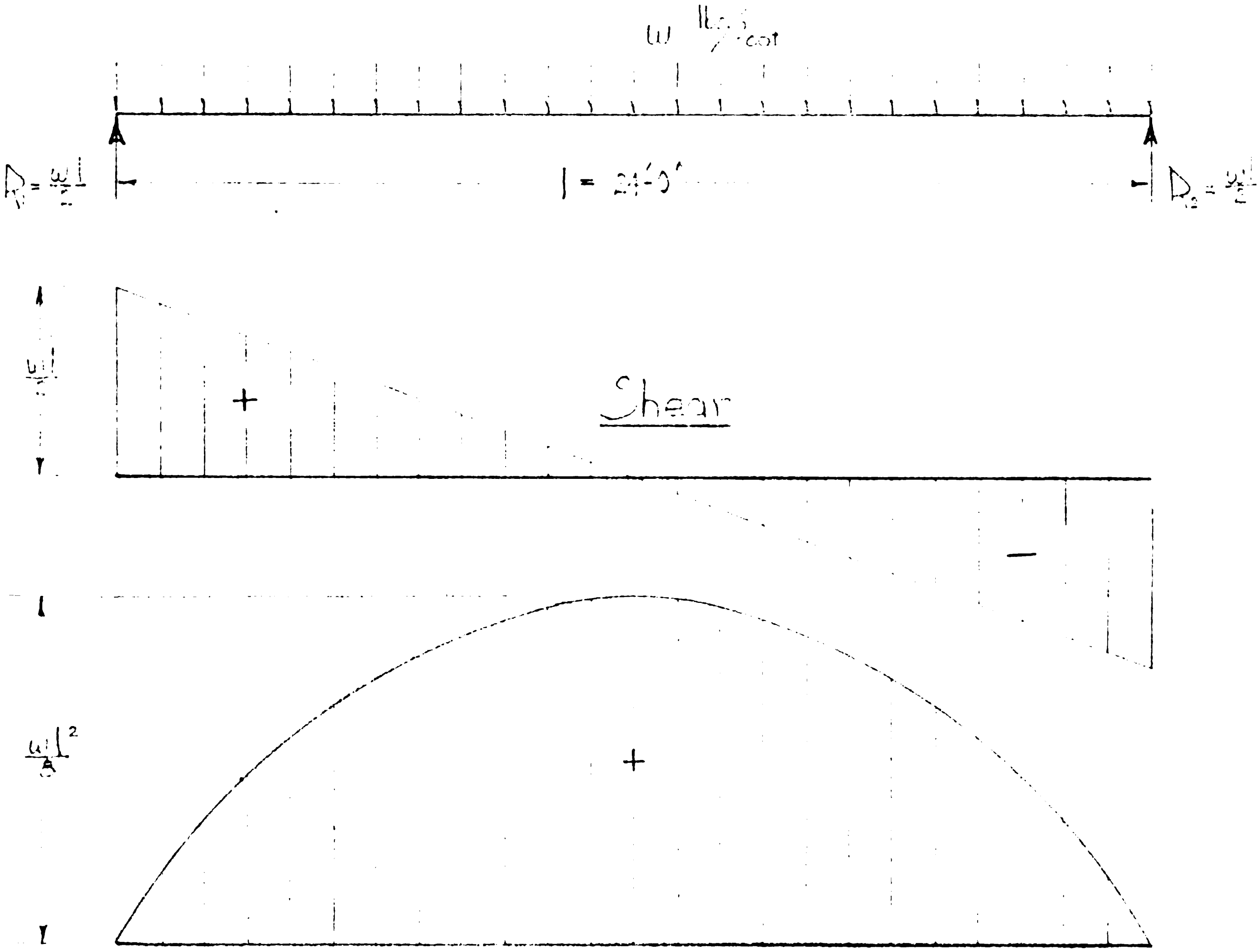


Figure -7-

LOAD-DEFLECTION CURVES for BEAMS
of VARIOUS SECTION DEPTHS



Bending Moment

Figure -8-

SHEAR and BENDING MOMENT DIAGRAMS
for UNIFORM LOADING

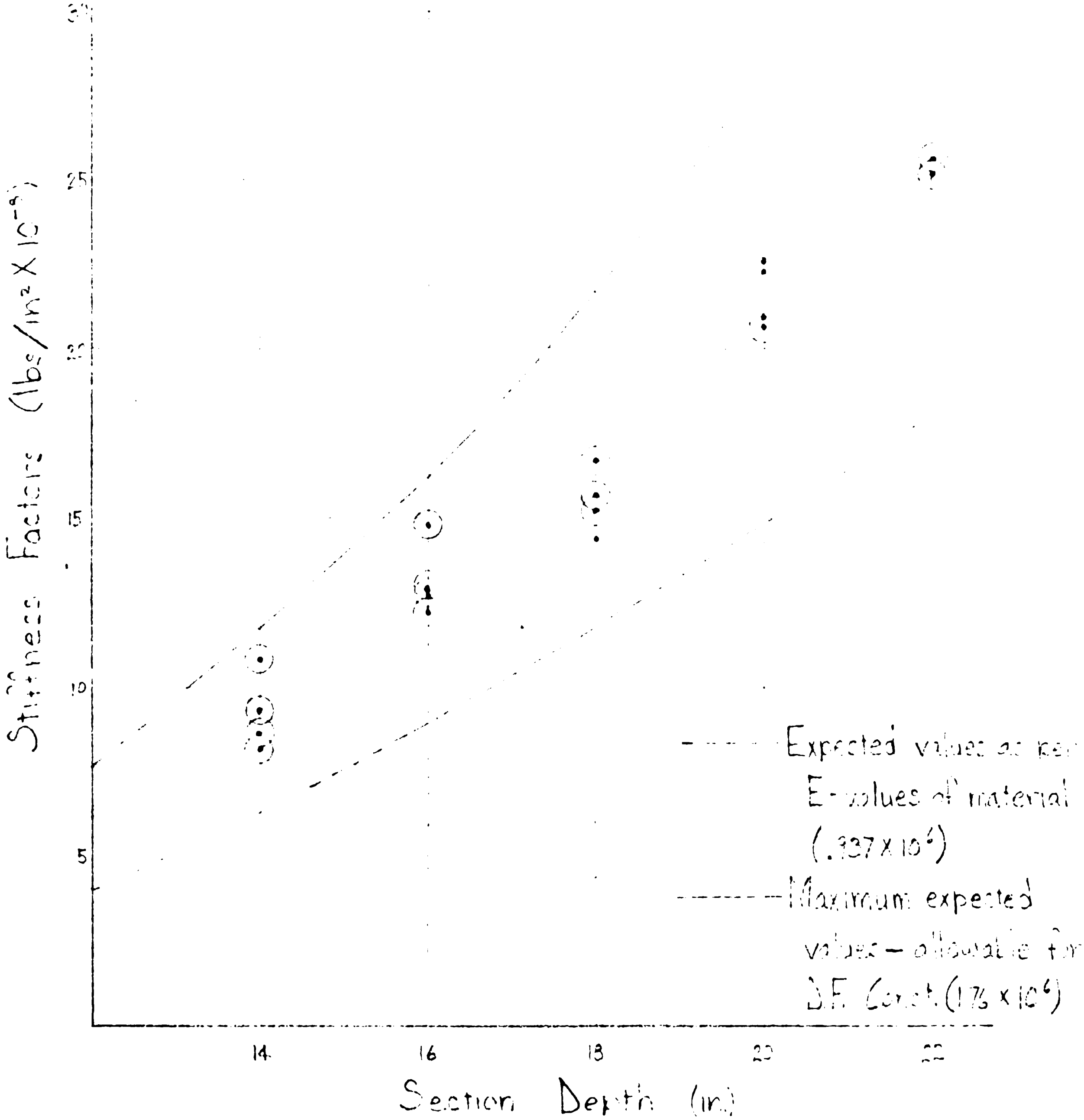


Figure -9-

EXPERIMENTAL STIFFNESS FACTORS — BEAMS

Moment of Inertia

The theoretical moment of inertia was computed for each beam.

$$I = \frac{b(h^3 - h'^3)}{12}$$

where:

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

b = thickness of flanges, in.

h = depth of beam, in.

h' = distance between flanges, in.

The web area was neglected in these computations; only the 2 x 4 flange members were considered. The I-values for the beams of various section depth are given in Table 3, Appendix B.

Modulus of Elasticity

The effective modulus of elasticity was then determined for each beam by the deflection formula as used for computing the stiffness factor (see Table 4, Appendix B). The term "effective" is used since the approximate equation for deflection does not take into account deflection due to shear. Inasmuch as the approximate deflection equation will be used for design along with performance data, based on the same assumption, the inclusion of the shear deflection component is not necessary.

Shear Deflection

As a matter of interest the component of the allowable deflection (.8 in.) attributable to shear was computed by a theoretical method.

$$\Delta_s = \frac{P\,l\,K\,h^2 C}{G\,I}$$

where:

Δ_s = deflection due to shear, in.

P = total load on beam, lbs.

l = span, in.

K = section constant

h = depth of the beam, in.

C = uniform load coefficient

G = shearing modulus of the webs, p.s.i.

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

This theoretical deflection due to shear is noted in Table 5, Appendix B. However, Figure 10 demonstrates that in this investigation there is no definite relationship between apparent stiffness of the material and beam depth as the shear deflection formula would indicate. This is due to the variable performance of non-uniform materials.

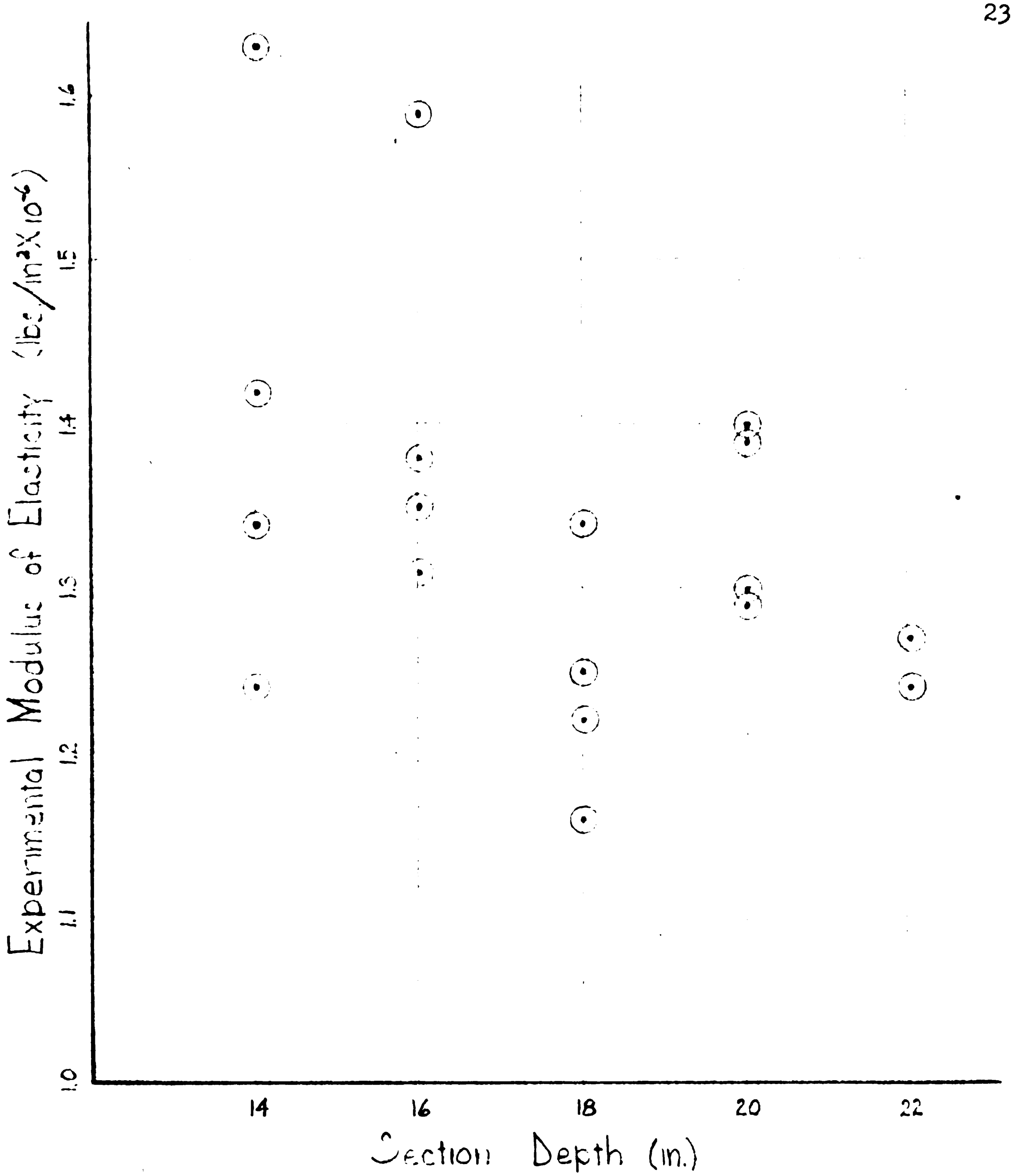


Figure -10-

CHART SHOWING LACK of CORRELATION BETWEEN BEAM DEPTH and M.O.E.

Fiber Stress in Flanges

The extreme fiber stress in the flanges at mid-span corresponding to the loading which caused a deflection equal to 1/360 of the span was calculated by the conventional flexure formula.

$$\sigma = \frac{M\,c}{I}$$

where:

σ = extreme fiber stress, p.s.i.

M - bending moments, in.-lbs.

c = distance from neutral axis to extreme edge, in.

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

Computed values may be found in Table 6 of Appendix B.

A form factor may be included in the flexure formula to increase the estimate of fiber stress.[9]

$$\sigma = \frac{M\,c}{F\,I}$$

The form factor for an I-section may be considered as the ratio of the fiber stress (at the proportional limit in its application to working loads) to the similar property of a rectangular beam 2 in. x 2 in. which allowable loading is based upon. The inclusion of a form factor was deemed unnecessary, however, because the neglected web area in the previous computation of the moment of inertia has served the same function of increasing apparent flange stress. In addition, reduction of 9/10 is already included in allowable working stresses for wood to compensate for the depth of rectangular beams commonly used in construction. Further penalizing of the strength of the wood would seem unrealistic.

Fiber Stress in Splices

The extreme fiber stress in the 1 x 4 splice plates was calculated in the same manner as for the flanges. The plates were considered as forming an I-section with the plywood web; web area (face plies only) was included in the calculation of moment of inertia for the computation. Values are given in Table 7, Appendix B.

Horizontal Shear Stress

The maximum horizontal shear stress in the plywood web was computed for the test beams for loads at allowable deflection.

$$\tau = \frac{V\,Q}{I\,b}$$

where:

τ = horizontal shear stress, p.s.i.

V = vertical shear, lbs.

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

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

b = web thickness, in.

These values may be found in Table 8, Appendix B.

Rolling Shear Stress

Where the flanges and webs are joined by glue, the joint is parallel with the plane of the plies. The rolling shear stress in plywood core developed under these circumstances was checked for loads at maximum deflection.

$$\tau_R = \frac{V\,Q}{I\,d}$$

where:

τ_R = flange-web shear, p.s.i.

V = vertical shear, lbs.

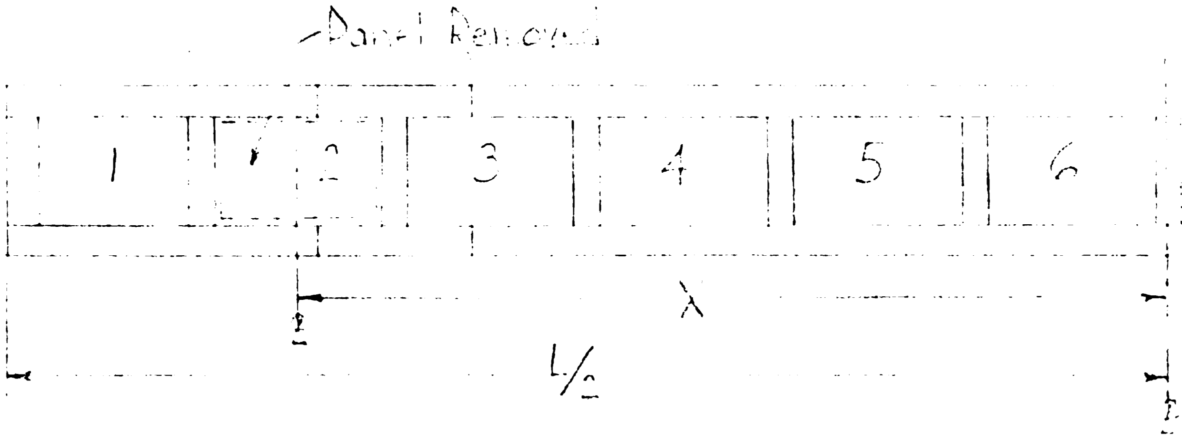
Q = statical moment of a flange member, $in.^3$

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

d = depth of the flange member, in.

Beams with Panels Removed

The effective modulus of elasticity for each beam with a panel removed was determined as described previously. The position of removed panels was designated as shown in Figure 11. In each case a ratio of the modulus of elasticity of the beam to that of the same beam intact was computed. A plot of this ratio versus the relative position of the removed panel was made. The curve, whose values are suggested as a reduction factor for allowable loading, is also given in Figure 11.



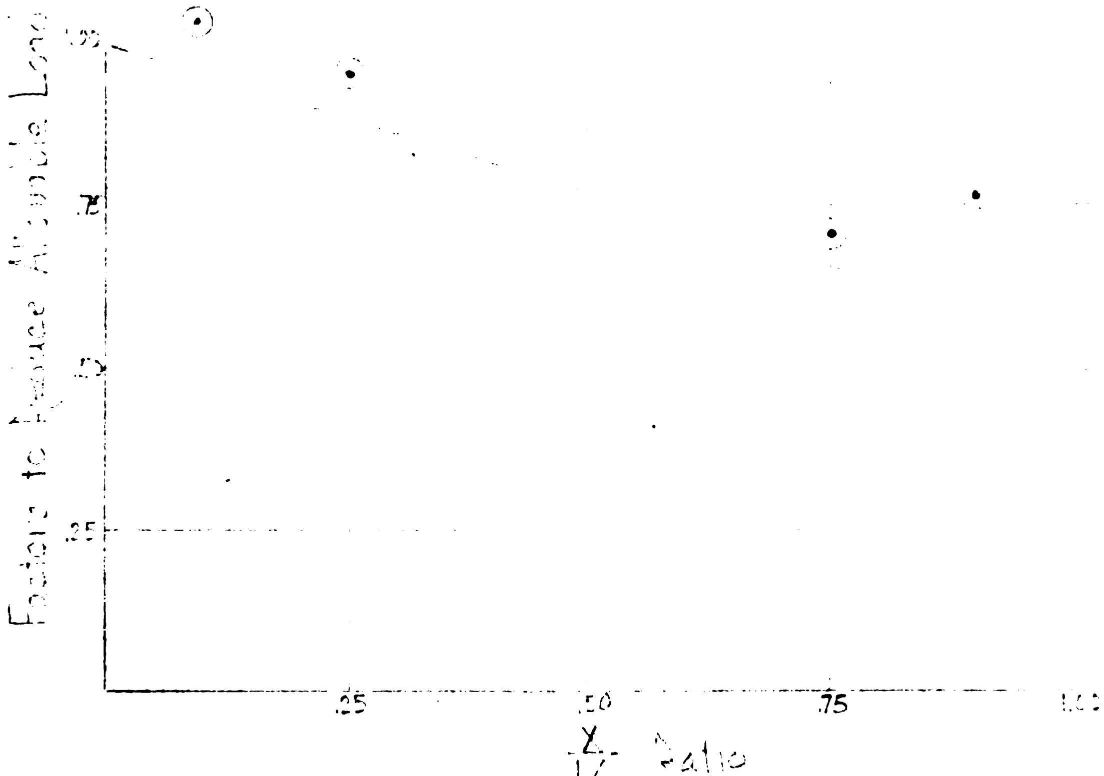


Figure -11-

LOAD-REDUCTION FACTORS — BEAMS
— PANELS REMOVED — FABRICATION

DISCUSSION OF RESULTS

The most significant observation drawn from the tests was that deflection at mid-span should be the design criterion for these girders. Also in the relatively short, deep beams capable of carrying greater loads there must be sufficient bearing to prevent compression at the supports.

All beams tested to failure failed at the tension flange splices at a load of about three times that at allowable deflection (see Figures 12 and 13). No other factor appeared to contribute to failure.

No buckling of the web was observed in the course of testing, nor did any shear failures occur although the usual computation of horizontal shear stress indicated values much in excess of the accepted allowable. At and near the web stiffeners the shear stress distribution would be quite complex. The usual equation should apply between stiffeners, however.

It is suggested in some publications that loads within a distance from both supports equal to the height of the beam should be neglected in vertical shear calculations which are used in the computation of horizontal shear.[7] This would yield somewhat lower calculated values but still would not explain the great difference between theory and performance.

Inasmuch as no failures occurred in such a significantly large number of tests, it was concluded that horizontal shear was not the controlling factor for beams within the range of sizes tested. This

Figure 12

Beam Failure in Tension
Flange Splice in 14 in. Beam

Figure 13
Beam Failure in Tension
Flange Splice in 16"
Beam

may be attributed to the fact that very large safety factors are used in determining an allowable shear strength for Douglas-fir plywood.

All other calculated stresses in the test beams were within accepted allowable values at a deflection of 1/360 of the span. The overall aspect was a rather well-balanced beam of a very practical nature.

The deflection of the beams appeared to be regular so as to form a smooth curve; no points of concentrated bending were apparent at splices or other locations. This feature is demonstrated in Figure 14.

The reduction in stiffness of the beams with panels removed was as expected. The reduction factor increased with the distance from mid-span, becoming a maximum at the last panel over the support. Thus, the increase in deflection is primarily the result of horizontal shear stress.

Figure 14 - Loaded Beams Forming a

Smooth Curve of Deflection

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the observations made and the results found in the investigation, the following conclusions have been drawn:

1. Nail-gluing is an effective as well as practical means of joining members. A slightly heavier-than-recommended casein glue mixture promoted good gap-filling characteristics while the 8 in. staggered nail spacing provided adequate pressure.
2. Beams of this description should be designed on the basis of allowable deflection at midspan. The usual methods of structural analysis would seem to apply.
3. Butt-joining of the 2 x 4 flanges using 1 x 4 lumber was of sufficient strength to support loads of 2½ to 3½ times those at allowable deflection. Splices should be located as close to the ends of the beam as possible, preferably not within the central two-thirds.
4. No failures occurred in the web due to horizontal shear stress as conventional calculations and design load would indicate. A well-balanced beam would not result if design was based upon such unrealistic values.
5. Compression at the supports may occur in the case of heavily loaded beams. Adequate bearing must be provided to limit this crushing stress.
6. Panels in the web may be cut out to accommodate heat ducts, etc. if the proper load-reduction factor is employed.

7. Beams of this description should be used in protected, ventilated locations. They should not be subjected continuously to high humidity conditions.

8. Shop fabrication where conditions may be controlled is the recommended procedure. Labor specialization and a permanent set-up are other inherent advantages which may be gained.

9. These nail-glued plywood girders should prove more economical than steel beams or wood-steel flitch plate beams in many applications in residential construction.

L I T E R A T U R E   C I T E D

LITERATURE CITED

1. Pearson, R. G., Kloot, N. H., Boyd, J. D., Timber Engineering Design Handbook, Cambridge University Press, New York, 1958.

2. Perry, T. D., Modern Plywood, Pitman Publishing Corporation, New York, 1948.

3. Timoshenko, S., Strength of Materials, Part II, D. VanNostrand Company, Inc., New York, 1941.

4. Laurson, P. G., Cox, W. J., Mechanics of Materials, John Wiley & Sons, Inc., New York, 1947.

5. Parker, H., Gay, C. M., MacGuire, J. W., Materials and Methods of Architectural Construction, John Wiley & Sons, Inc., New York, 1958.

6. Timber Engineering Company, Timber Design and Construction Handbook, F. W. Dodge Corporation, New York, 1956.

7. ____________________________, Wood Handbook, U. S. D.A. Handbook No. 72.

8. Freas, A.D., Selbo, M.L., Fabrication and Design of Glued Laminated Wood Structural Members, U. S.D. A. Technical Bulletin No. 1069, 1954.

9. ____________________________
Technical Data on Plywood, Douglas Fir Plywood Association, Tacoma,Washington, 1948.

10. Countryman, D., The Fabrication and Testing of Full Scale Plywood Beams, Report No. 30, Douglas Fir Plywood Association, Tacoma, Washington, 1944.

11. Countryman, D., Haskell, V. D., Plywood Beam Design Factors, Report No. 58, Douglas Fir Plywood Association, Tacoma, Washington, 1952.

12. ____________________________, Design of Plywood Webs in Box Beams, F. P. L. Report No. 1318, 1943.

13. Lewis, W. C., Dawley, E.R., Stiffeners in Box Beams and Details of Design, F.P.L. Report No. 1318-A, 1943.

14. ____________________________, Heebink, T. B., Cottingham, W. S., Dawley, E. R., Stiffeners in Box Beams and Details of Design, F. P. L. Report No. 1318-A, 1943

15. ______________, Heebink, T. B., Cottingham, W. S., Buckling and Ultimate Strengths of Shear Webs of Box Beams Having Plywood Face Grain Direction Parallel or Perpendicular to the Axis of the Beams, F. P. L. Report No. 1318-D, 1944.

16. Markwardt, L. J., Freas, A.D., Approximate Methods of Calculating the Strength of Plywood, F. P.L. Report No. R1630, 1950.

17. Radcliffe, B. M., Granum, H., Design of Nail-Glued Plywood Gusset Plates and Solid Wood Splice Plates for Softwoods, Purdue University Agricultural Experiment Station Bulletin No. 613, 1954.

18. Angleton, H.D., How to Nail-Glue Hardwoods and Softwoods, Purdue University Agricultural Extension Service Bulletin No. 441, 1957.

19. Anonymous, "Plywood Girders Roof a Warehouse", Engineering News-Record, 128:625-6, April 23, 1942.

A P P E N D I X

APPENDIX A

Moisture Content and Modulus of Elasticity

Value of Ten Representative 2 x 4's

Obtained from the Same Source as Members
Used in Beams

Sample	M.C.	E	Species
1	17%	1.37×10^6 p.s.i.	D.F.
2	16	.98	Wt.F.
3	15	1.01	"
4	21	1.36	D.F.
5	14	.98	Wt.F.
6	19	.54	D.F.
7	16	.93	"
8	18	.69	"
9	18	.60	"
10	16	.91	"
Ave.	17%	$.937 \times 10^6$ p.s.i.	

Testing Conditions --

Center Loading -- $E = \dfrac{p\,l^3}{48\,y\,I}$

Span -- 48 in.

Rate of Loading -- 1/8 in./min.

APPENDIX B

TEST Beam	Beam Depth (in.) 14	16	18	20	22
1-1*	94	130	166	237	272
1-2	87	134	162	240	267
2-1	100	158	154	220	---
2-2	115	136	178	223	---

Table 1
Load at 1/360 Span Deflection (lbs./lin. ft.)

Test Beam	Bean Depth (in.) 14	16	18	20	22
1-1	884×10^6	1222	1560	2228	2557
1-2	818	1260	1523	2256	2510
2-1	940	1485	1448	2068	----
2-2	1081	1278	1673	2096	----

Table 2
Experimental Stiffness Factors (lbs.-in.^{2})

*First digit refers to test pair, the second to individual beam.

Section Depth (in.)	:	I Values
14	:	661 $in.^4$
16	:	929
18	:	1245
20	:	1608
22	:	2018

Table 3

Moments of Inertia

Test Beam	Beam Depth (in.)				
	14	16	18	20	22
1-1	1.34×10^6	1.31	1.25	1.39	1.27
1-2	1.24	1.35	1.22	1.40	1.24
2-1	1.42	1.59	1.16	1.29	----
2-2	1.63	1.38	1.34	1.30	----

Table 4
Effective Modulus of Elasticity (lbs./in.^{2})

Test Beam	Beam Depth (in)				
	14	16	18	20	22
1-1	.119	.150	.190	.256	.277
1-2	.111	.162	.185	.259	.272
2-1	.127	.191	.176	.238	---
2-2	.146	.165	.203	.241	---

Table 5
Shear Deflection Component of Total Allowable Deflection (in.)

Test Beam	Beam Depth (in.)				
	14	16	18	20	22
1-1	860	967	1937	1273	1281
1-2	796	997	1012	1287	1257
2-1	915	1175	962	1181	----
2-2	1052	1012	1112	1197	----

Table 6
Extreme Fiber Stress in Flanges at Allowable Deflection (p.s.i.)

Test Beam	Beam Depth (in.) 14	16	18	20	22
1-1	839	936	989	1194	1202
1-2	777	965	966	1210	1180
2-1	893	1138	918	1109	----
2-2	1027	979	1061	1124	----

Table 7

Fiber Stress in 25/32" Wood
Splice Plates at Allowable Deflection
(p.s.i.)

Test Beam	Beam Depth (in.) 14	16	18	20	22
1-1	279	328	363	457	469
1-2	259	338	354	463	460
2-1	297	398	337	424	---
2-2	342	343	389	430	---

Table 8

Horizontal Shear Stress
at Allowable Deflection (p.s.i.)

Test Beam	Beam Depth (in.)				
	14	16	18	20	22
1-1	14.44	17.00	18.72	23.61	24.15
1-2	13.36	17.53	18.27	23.90	23.71
2-1	15.36	20.67	17.37	21.91	-----
2-2	17.66	17.79	20.08	22.21	-----

Table 9

Rolling Shear at Joint Between

Flange and Web at Allowable Deflection (p.s.i.)