ENGINEERING REPORT

ON

PURDUE RESEARCH FOUNDATION EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PREPARED BY JULES F. REITHER
Technical Associate

APPROVED BY CARL F. BOESTER
Executive in Charge of Developments

OFFICE OF PRODUCTION RESEARCH AND DEVELOPMENT CONTRACT NO. WPB-312

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana
March 1, 1944

PREFACE

Progress and change mean deviation from accepted practices, not only in the thing produced, but in its method of production as well.

In the development of the various components employed in this low-cost housing research a purpose and theory were permitted to develop which, while formative, were sufficient to unite the engineers, designers and craftsmen on the basis of common interest in the best use of the mediums employed. In practice, the results of this method have affected not only initial design and engineering, but over-all planning as well.

Obviously the delegation of responsibility for housing research inevitably means delegation of responsibility for architectural as well as for structural quality. It is further recognized that if the structural components result in adequate, economical and easily erected living units, the medium of architecture can develop the artistic qualities which might be desired in the appearance of the structure. If, however, economical structural values are subordinated to architectural pattern and picture, then we cannot consider the approach to the solution of the problem as being practical.

This report deals entirely with the results determined from research experimentation with the structural components. It is recognized that architectural refinement would, of necessity, need to be incorporated into the final usage of the various practices discussed herein.

It is felt an intelligent approach to the final product of the housing unit will result in satisfaction to the owners and occupants and profit to those involved within the industry. The limits of this enormously creative medium of research are not even in sight.

TABLE OF CONTENTS

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana
March 1, 1944

SECTION I

FOUNDATIONS

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana

March 1, 1944

FOUNDATIONS
PROCEDURE

Average residential construction offers very light loads. In the past, standards were developed and introduced into building codes which provided a substantial factor of safety in order to make certain that resulting foundations would be sufficiently heavy to carry any structural load. Foundation unit costs have always been among the highest in a structure. These frequently affected the total price to the point where a sacrifice was made in the living part of the structure in order that the owner might be able to pay the high price of the foundation required. Where foundations were carried below frost line with the necessary over-cut in excavation to allow for footing and forms or to permit working space for the laying of masonry units, very little additional money was required to provide an entire basement excavation. The elimination of these costly factors needed to be considered in the initial analysis contributing to speedy low-cost temporary housing.

Foundations were considered as falling into these general classifications: beams supported by piers; floating-type beam, having the characteristics of a footing; a floating concrete slab; a soil stabilized slab.

The Portland Cement Association, in their concrete information bulletins, have thoroughly covered the problem of light wall and slab construction. It was not believed necessary to go into a great deal of study regarding mixes, water-cement ratios and other established practices which, through the research of the Portland Cement Association, have become recognized standards.

In all of these studies, reinforcing was considered as unavailable and mass concrete would of necessity need to off-set reinforcing values. Of course, mass cannot entirely off-set the absence of reinforcing.

However, on the theory research must be conducted to arrive at results employing a minimum amount of labor and materials, the engineer for the foundations was called upon to prepare calculations based upon the actual bearing properties of the soil and strength of the concrete, disregarding some of these standard practices. Incorporated herewith are these basic calculations which are interesting in that they disclose an excessive amount of concrete being used so far as engineering data are concerned. It becomes very apparent that some of these components constructed on the basis of such calculations would become too small in section to offer adequate supporting area, and that other elements such as handling and pouring would affect the ultimate strength of the component should these minimum dimensions be employed.

TEST #146

For example, in the study of foundations for Unit B-13 it becomes very apparent the actual cross section required in a pier would result in such a small component it would be difficult to form and pour. Equally so, the footing required, 144 sq. in., would be most impractical in view of the fact these piers are of the precast type and the footings are poured around a pier after its placement within the hole. The post hole digger normally employed for the placement of either the precast or site poured pier would not permit the digging of a hole small enough to provide a pier of adequate strength for the loads imposed upon it.

TEST #147

In the employment of the monolithic piers and beam the space between the piers was excavated to a depth 6" greater than that required for the actual concrete of the beam. After the forms were constructed, this overcut of 6" received cinders, shaped up to produce the bottom of the beam and the pier brackets. Cinders were covered with a watertight paper to prevent water from the concrete dissipating into the cinders at the time of pour.

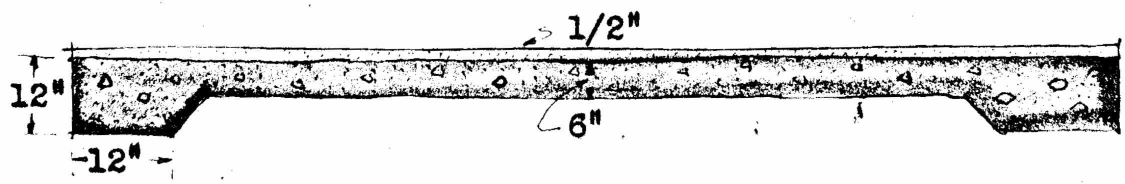
TEST #148

Probably the most interesting of these foundations is that in which the floating type beam is employed. Observation of highway slabs, garage footings and floors, and light-weight concrete construction employed in small agricultural structures lead to the belief that if the body of the slab could be cushioned against frost action, then the walls carrying the peripheral loads could also be cushioned in a similar manner. This amounted to the construction of a monolithic footing at a surface rather than a sub-grade. Such a footing would need to be poured on a cushioning material. The Portland Cement Association had employed sand. It appeared cinders would do the job as well.

As the enclosed calculations indicate, this concrete footing was formed over a cinder bed which extended 6" beyond the limits of the footing in all directions. That is, the excavation was carried 6" deeper and 12" wider than required for the actual concrete. This trench was then filled to the height of 6" with cinders, forms were constructed and the pour made. Upon removal of the forms, cinders were employed for back fill. The success of this type of beam lies in pouring it so that no work lines occur and in designing the utilities so that no sleeves or pipes pass through the concrete.

While it is not the practice to discuss methods in this report, it becomes apparent that the concrete must be segregated from the cinders at the time of pour. Either roll roofing, asphalt-impregnated papers or 30-pound slater's felt may be employed at the bottom of the beam, turned up slightly onto the forms to prevent the dissipation of the water from the concrete into the cinders.

- - -



TEST #149

The floating slab constituted a light concrete floor extended in width and depth around the perimeter to produce a footing offering cross section area sufficient to resist the loads to be imposed upon it. This combined into a light weight monolithic slab the footings and slab normally segregated as two separate components. The particular material employed for this test was vermiculite concrete in which vermiculite aggregate and admix were combined with Portland Cement and a carefully calculated ratio of water to develop what the Vermiculite Institute represents as being a concrete of high insulating value. The soil beneath this slab was shaped exactly to the contour of the concrete, there being no cushion material introduced. However, two layers of waterproofed fabric were laid over the entire surface receiving concrete and carried up in to the forms along the perimeter of the excavation. As the Vermiculite Research Institute has gone into very careful detail as to the characteristics of their material, these will not be reviewed.

TEST #150

Since slabs were to be considered for quick wartime housing and the Federal Public Housing Authority was recognizing the type of concrete identified as "porous" (Division T-3, Masonry and Concrete, Section 4.1(a)0, it appeared some consideration should be given to a slab based upon soil stabilization practices. In following out this undertaking the work was based upon the recommendations of the Portland Cement Association in their construction handbook for soil-cement roads. The cross section of the slab was the same as that employed for the vermiculite slab. The work was done by hand, the soil at the site being dug up to the required depths, carefully pulverized, raked through so that the resultant aggregate was of uniform proportion. Cement, using a ration of 10% by volume, was introduced, worked into the soil, then the entire mass thoroughly wetted down with the stipulated amount of water and finally levelled off to the limits of the forms.

This briefly is the procedure followed in a limited soil stabilization area. In both the vermiculite and the soil-cement slabs the surfaces were roughened and, immediately following initial set, a one-half inch topping applied.

FOUNDATION
UNIT B-13 **TEST #146**

SUBJECT:

Foundations employing concrete piers $6\frac{1}{2}$" x $7\frac{1}{2}$" spaced 8' on center, and composite wooden girder beams. Three 2" x 10" glued and nailed to form one beam 4-7/8" x $9\frac{1}{2}$".

PROCEDURE:

Basic Assumptions:

Weight of structure	17,875#
Weight of foundations	7,767
Total	25,642#

10 piers in front and back walls (5 in each)

4 piers in end walls (2 in each)

Footings below piers - 8" x 12" x 12"

Calculations

Area of pier = 7.5 x 6.5 = 48.75 sq. in.

Area of footing = 12" x 12" = 144 sq. in.

Assuming end piers to support a negligible part of total, an assumption substantiated by final deflection readings:

$$\text{Load per pier} = \frac{25{,}642}{10} = \frac{2564\#}{\text{Pier}}$$

Footings 144 sq. in.

We get $\frac{2564\#}{\text{sq.ft.}}$ which is allowable.

Ground is good for 4000 #/sq.ft.

Piers were precast and footings were used to prevent settling, inasmuch as exact depths of excavation for precast piers are not practicable.

Load in pier:

$$\frac{2564}{48.75} = 52.5\ \#/\text{sq.in.}$$

This is far below allowable; however, smaller piers are subject to breakage in handling.

Unit stress in wooden beams

M = Bending moment (# in.)

$$M = \frac{PL}{8} = \frac{4(2000)\ 96}{8} = 96{,}000 \text{ \# in.}$$

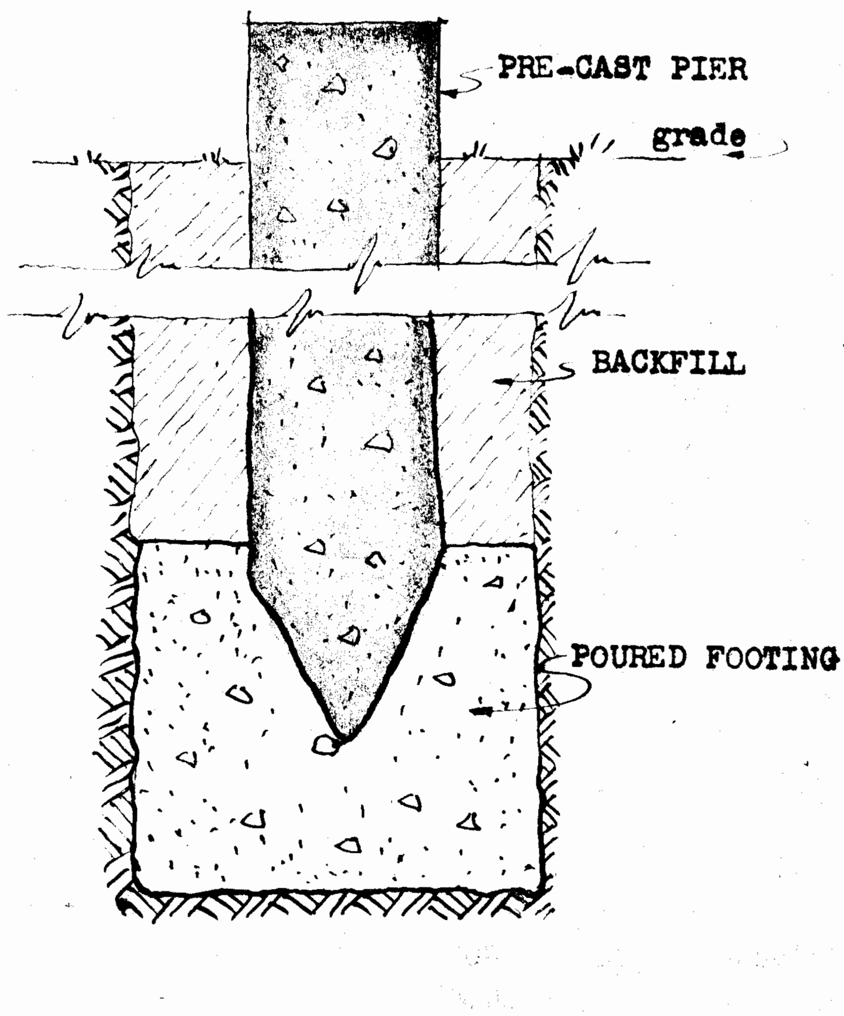
S = Unit compressive stress (#/sq.in.)
c = Distance from neutral axis to outer fibers (inches).
I = Moment of inertia $(\text{in.})^4$

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

b = Width of beam
h = Depth of beam.

$$I = \frac{3(1.625) \times (9.5)^3}{12} = 349$$

$$S = \frac{Mc}{I} = \frac{96{,}000 \times 4.75}{349} \qquad 1300 \text{ \#/sq.in.}$$



FOUNDATION
UNIT B-15 TEST #147

SUBJECT:

Foundations employing beam and pier construction. Monolithic. 8" diameter piers placed 48" on center. No reinforcing steel permitted.

PROCEDURE:

Basic Assumptions:

Weight of structure	15,500
Weight of foundations	12,570
Total	28,070

18 piers total (9 in each wall)

No footings employed.

Assume cross section of beam 12" x 12"

Calculations:

Area of piers - .706 sq. ft.

101 sq. in.

$$\frac{28{,}070}{18} = 1560 \text{ \#/pier}$$

$$\frac{1560}{.206} - 2210 \text{ \#/sq. ft. on soil}$$

Soil encountered: 4000 #/sq.ft.

$$\frac{1560}{101} = 15.5 \text{ \#/sq.in. compressive stress in piers.}$$

I = Moment of inertia of beams $(\text{in})^4$
M = Moment (# in.)
S = Unit Stress #/sq.in.
c = Distance from outer material to neutral axis

$$I = \frac{bh^3}{12} = \frac{(12)^4}{12} = 1728 \text{ in.}^4$$

$$M = \frac{PL}{8}$$

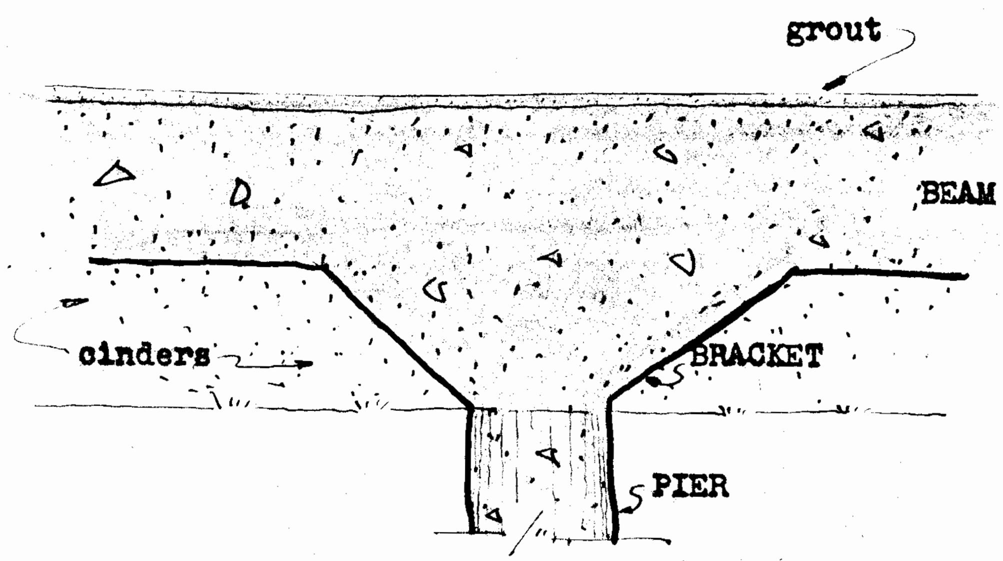
Where P = Load of one truss wall ~~roof~~, etc.
P = 2000 #

$$M = \frac{2000 \times 4 \times 12}{8}$$

$$= 12{,}000 \text{ \# in.}$$

$$S = \frac{Mc}{I} = \frac{12{,}000 \times 6}{1728} = 41.7 \text{ \#/sq.in.}$$

Concrete has an allowable stress in tension equal to 40 - 50 #/sq.in. Therefore, the assumed beam is sufficient cross section to carry the load imposed.



FOUNDATION
UNIT B-24 TEST #148

SUBJECT:

Monolithic concrete - no reinforcing steel allowed. No piers. Floating type, to be carried on a cushion of sand or cinders.

PROCEDURE:

Basic Assumptions

Weight of structure	-	31,910 - 000
Weight of foundation	-	14,136 - 420
	Total	46,046 - 420 = 45,626

Assume cross section of beam to be 8" deep x 12" wide

Calculations

Cross section area	8 x 12 = 96 sq. in.
$\frac{1}{2}$ Shear area	9.25 x 12 = 111 sq. in.
Shear area	222 sq. in.

Concrete is assumed to shear along surfaces 60° to horizontal.

Each truss carries the following:

Dead load	1830#	
Live and snow load	2700	
	4530#	Or 2265# per truss end.

$$\frac{2265}{222} = 10.2 \text{ \#/sq. in.}$$

Ultimate strength of concrete - 2000 #/sq. in.

Allowable shear = 2% of ultimate

$$S_s = .02 \times 2000 = 40\text{\#/sq. in.}$$

Allows a factor of safety (F.S.) of 4 in this respect.

$$11.75 \times 12 = 141 \text{ sq. in.}$$

$$\frac{2265}{\frac{141}{144}} = 2320 \text{ \#/sq. ft. bearing on earth.}$$

Allowable for clay 4000 #/ft. Allows a factor of safety of 1.7.

At point where truss bears on concrete wall

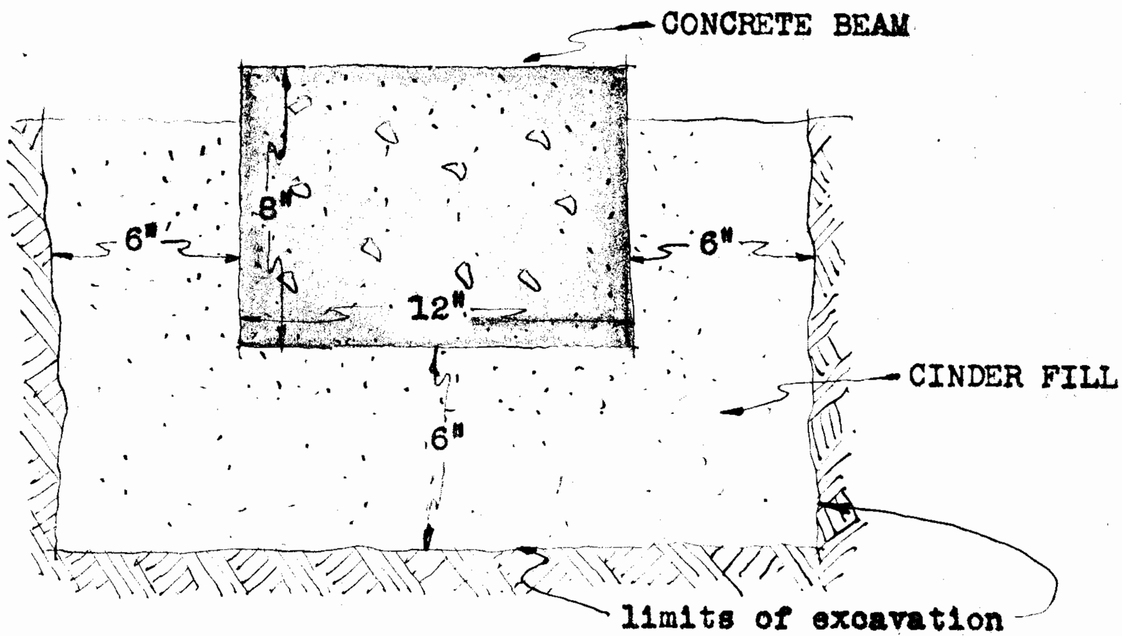
Area in contact = 2-7/16 x 12 = 29.2 sq. in.

$$\frac{2265}{29.2} = 77.6 \text{ \#/sq. in.}$$

Allowable compressive = 40% of ultimate

$$S_c = .4 \times 2000 = 800 \text{ \#/sq.in.}$$

Allowed factor of safety (f.S.) here is about 10.



SUMMARY

The use of the precast concrete pier seemed impractical for small work. The weight of the pier made its handling difficult, holding it to the desired grade as well as the necessity of retaining it in a plumb positionduring pouring seemed to offset any savings which might have resulted from eliminating forms for small piers. Such forms do carry a high unit cost, but where mass construction is being undertaken light demountable pier forms usually are employed, frequently of metal when such is available. Reuse of these forms lowers the basic form cost.

The pouring of piers and beam offers some merit. It is a quick method by which the post hole provides the form for the pier and a limited amount of form work is necessary to create the beam. This construction would be greatly benefitted by a modest amount of reinforcing, an item which was not permitted in our study of these foundations. In all of these low-cost components it was necessary to hold the tops of the beams reasonably true and to introduce any bolts or straps immediately following the pour. No shims were permitted under trusses.

The continuous beam cushioned on cinders was carefully studied during all phases of construction. As the deflection readings for B-24 shown in Section 5 will indicate, there was some settlement on this beam when the load of the walls was applied. Indications are that this settlement was uniform throughout one 32'0" length of wall. From a careful analysis of all the factors involved it is believed this displacement resulted from permitting a work line to occur at the northwest corner of the foundation at the time pour was made. This work line did not become immediately apparent and it was not until displacement was observed and the cinders were dug away from the beam at this corner that it was noticed the bond was not all that it might be. This displacement was not radical but was sufficient to justify the importance of the requirement that the beam be monolithic and that no work lines or disrupting of the mass be permitted in this type of foundation.

Both the concrete and the soil stabilized slabs seemed to offer a quick solution for floors for temporary housing. The vermiculite slab offered a very quick floor. In the employment of these slabs care must be taken to carefully tamp the back-fill into the trenches bringing in the utilities and sewer. The observation is made again that even a limited amount of reinforcing in these slab footings would be of definite benefit.

This report deals only with the structural factors involved. Floor temperatures, air circulation, etc., will not be discussed.

In the soil stabilization slab the limited amount of excess dirt resulting from the back filling of the trenches caused some unnecessary handling of soil as an effort was made to work this into the soil to be employed in stabilization. Where manual labor is to be employed in preparing such a slab it is believed advisable to remove surplus soil along with sod, roots or any deleterious material, bringing the area to a rough grade before undertaking the actual preparation of the soil. The working of the soil to a depth of six inches offered no serious problem, but it was found necessary to dig out to a depth of 6" and about 18" from the perimeter in order to process the lower half of that part of the slab which constituted the footing. This operation might have been simplified if the perephial area and then the slab area had been treated.

As a final solution employing a slab for temporary types of housing it would be well to consider carrying the slab 6" or 8" beyond the limits of the walls, tapering this projecting part of the slab away from the structure. This would offer a certain amount of protection for the wall.

It is recognized work must of necessity proceed as quickly as possible over foundations. As long as ordinary precautions are observed there is no need to delay the employment of these units more than one day after their initial pour.

All foundations of the type described within this report have been in place since the Summer and Fall of 1943. All have been subjected to temperatures as low as $0^{o}F$. Readings taken at outside temperatures of $+40^{o}F$. followed by readings taken at a temperature of approximately $+15^{o}F$. and finally those taken at $+50^{o}F$. indicate no serious displacement in any of these foundations.

One observation that has been made in connection with the continuous beam foundation is the inability to excavate within the limits of the foundation to a depth greater than two or three inches below the top of the concrete due to the necessity of retaining the inside cinder fill in position. The removal of too much soil from beneath structures employing the concrete beam and pier construction would permit the backfill around the exterior to wash under the beams, causing water to stand and ultimately carrying in enough soil that the effectiveness of the excavation would be lost.

SECTION II

FLOOR TRUSSES

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana
March 1, 1944

DEFLECTION TESTS
METHOD OF TEST LOADING

Two trusses are constructed in accordance with the engineer's sketches. The #1 yellow pine is culled out to provide the best pieces for the long flange members. The remaining material is cut up for the diagonals and other small components. Those pieces which too closely approach #2 grade are not employed. In addition to the members called for in the initial design, frequently a 1" x 4" is secured flatwise to the lower flanges or chords for the purpose of carrying the ½" gypsum plenum members or ceiling. These units are assembled in a jig on a shop table.

Floor truss assembly provided one horizontal upper and lower member be put in place on jig. These receive glue where the web diagonals are to make contact. The web diagonals are put into position, and glue is applied at their ends where the second 1" x 4" flange members are to engage. All components are nailed with 10d box nails, there being no fewer than four at each point of contact.

As this report deals with the engineering factors of the truss rather than the manufacturing problems involved, the details and time studies in connection with the actual assembly of the truss do not receive any comment.

The truss is permitted to set for a period of not less than eight hours. Two identical units are placed on solid bearings simulating foundation contact points, are spaced 24" on center from each other, and their ends plumbed and secured with plywood or gypsum simulating the wall construction which will be adjacent to the trusses in their actual use within a structure. A yellow pine board decking 48" wide is laid along the trusses so that there is a projection of 12" beyond the center line of each of the trusses, and this decking is carried full length. The structure resulting from this assembly is then checked by instrument in order to determine its being level. Reading points are established at the center of and at distances equal to one fourth of the span to the right and left of the center and on the lower flange of the floor truss or lower chord of the roof truss. Check readings are made on these lower chords to determine any action which might have resulted from the application of the decking. Such initial displacement, if apparent, is recorded.

Loads are applied by using one foot square concrete blocks of known weights, the loading being by the checkerboard method. An initial set of blocks is laid down the center of the deck with the first and last blocks at the limit of the deck and with intermediate blocks spaced one foot apart. Initial deflection and weight readings are made and recorded. Additional blocks are then laid so that they occur opposite and alternate to the open spaces of the first row of blocks, and, upon their placement, readings of weights and deflections are made and recorded. The remaining opposite, alternate spaces adjacent to the open spaces of the first row receive blocks. The third set of readings and recordings is made. This pattern is followed until either excessive deflection, actual failure, or satisfactory loading has been accomplished.

DEFLECTION TESTS
FLOOR TRUSSES

I. SOLID-WEB FLOOR TRUSSES

In the initial studies on floor trusses first consideration was given to solid web trusses employing 1" x 4" yellow pine flanges, the Douglas Fir Plywood Association Engineer's Handbook, Section VII, offering a graphical solution for the form factor of "I" and box beams with plywood webs.

On the basis of this method, a truss was computed (Calculation A). The application of this form factor formula indicated the ¼" plywood web truss 15" deep and 24'0" long, employing two 1" x 4" yellow pine flanges, was not sufficiently strong to receive the imposition of a 100# to the square foot design load over a span of 24'.

Calculation B employing a more orthodox formula resulted in an entirely satisfactory value. It became apparent the only satisfactory way to determine what results might be obtained with light, deep, clear span floor trusses would be to construct pairs of these trusses and actually load them, carefully determining the loads and deflections as they occurred.

There was no data available on the characteristics of gypsum or asbestos board when employed as a web member in a truss.

In order to have results which would contribute to comparative values, three sets of trusses were constructed:

TEST #120 - Plywood web floor truss, ¼" plywood.

TEST #121 - Gypsum web floor truss, ½" gypsum board, i.e., Gyplap.

TEST #122 - Asbestos board floor truss, asbestos board 3/16" thick.

The construction of these trusses is indicated in their respective deflection test drawings.

As deflection tests indicate, the solid web in combination with 1" x 4" #1 yellow pine and 15" deep offered a solution of no particular value. The units themselves became heavy and cumbersome.

Since the desire was to obtain a comparative value in the three materials, no effort was made to glue any of the components. It is quite apparent that gluing would be entirely possible in the case of plywood trusses, and additional structural value would no doubt be obtained as a result of such gluing.

II. OPEN-WEB FLOOR TRUSSES

After a study of these three solid-web trusses and a review of other factors influenced by their use, further development of this type truss was temporarily discontinued. The engineers were instructed to develop a truss in which the web would be more in the nature of a lattice through which pipes might pass and air could be caused to circulate. The further restriction placed upon design was that material would be limited to 1" stock, the grade not to exceed that of #1 yellow pine. If this requirement was to be met and the minimum amount of lumber employed, the first analysis indicated diagonal members having some of the characteristics of the web would need to be on the outside of the flange members.

Since the three previous trusses had a depth of 15", this was carried on into the calculations for the open-web truss.

<u>TEST #123</u>

As the graph indicates, the first result with a truss employing diagonal members resulted in early failure, the truss losing its crown of 1" at a load only slightly greater than 1 kip. Continued loading was carried on until a complete breakdown of the truss occurred. Examination of the components indicated the horizontal members had failed to adequately support the loads imposed. The idea of diagonal components glued and nailed into place seemed to have merit. Only one glued joint was torn from its horizontal member. This resulted more from the blocks twisting it off when they became displaced upon the failure of the truss, rather than from any inherent failure within the joint. With this exception, the glued joints held up remarkably well.

Nailing had been driven from one side of the truss only. Those diagonal members which were on the far side of the nailing side of the truss seemed to have received little benefit from the nails driven into them. The excellent condition of the diagonal members substantiated the engineer's analysis to the effect that the 1" x 4" 's were greater in section than needed but should be employed in order to gain gluing and nailing surface.

From the information obtained in Test #123, calculations were made and a design identified as Test #125 was developed.

<u>TEST #125</u>

The engineer reviewed all of the data resulting from the tests made on the various 15" deep trusses and from this information went into a more carefully engineered and calculated component.

<u>TEST #124</u>

At the time Test #125 was developed, test truss #124 was being designed which would have a depth of only 12" but with the same general characteristics of test truss #125. This truss was to be supported on parallel foundations which would be 48" in from each end of the truss. Each truss was to be capable of supporting 800 pounds at each of its overhanging ends.

This truss was constructed in the manner shown, and its loading developed very satisfactory characteristics. The deflection tests taken in Unit

B-15 after occupancy indicate the success of this truss. Reference sketch accompanying the deflection tests indicates the position of the supporting beams which constituted the foundation of these trusses and provides identifying lines indicating locations along which deflection readings were taken after this unit was occupied.

TEST #126

After the solid web (Test #120) had been employed, no other use of plywood was studied until Test #126 was undertaken. In the original plywood truss test it was recognized the absence of gluing in this initial test had a material effect on the inability of the truss to sustain the loads imposed upon it. Without making any effort to calculate the values in the components, a truss was developed which possessed the characteristics of that of Test #123; that is, a 15" high truss with a single horizontal top and bottom member and with diagonal members on the outside, both glued and nailed. Failure of this truss in the early stages of loading was anticipated, but the result being sought was a determination of the variables which entered into the calculations in which plywood would be employed.

The diagonal members in particular were difficult of calculation due to the direction of fibers within these members. The glued joint became of empirical value. As the test indicates, failure occurred early with a distortion rather than a breaking down of the truss, twisting in the absence of bridging becoming so great it was impractical to attempt to continue breaking down the member.

Examination of the structure indicated the small diagonal members were sufficient in cross section to perform the function of the web, but, regardless of the size of these diagonals, a greater cross section was necessary in the horizontal components. As previously observed, there was no failure in the glued joint. In this truss, nailing had been employed from both sides; consequently the nails were functioning in the manner expected. As a result of all of these floor truss tests, the engineers developed a truss Test #127.

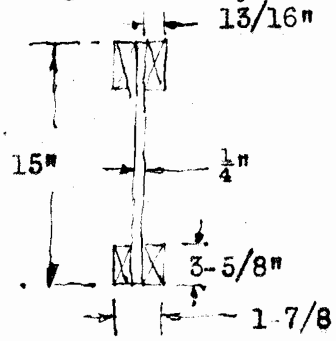
TEST #127

This floor truss represented a design in which the depth continued to be 15", the diagonal members 1" x 4", calculated 45° from the vertical. Flanges were 2" x 4". The web members were placed on the outside of the 2" x 4" 's and secured in place by both gluing and nailing. The truss was constructed with a 1" crown.

Deflection test on this component indicates the most satisfactory solution obtained from any of the floor truss tests.

DEFLECTION TESTS
FLOOR TRUSS, TEST #120
CALCULATION A:

Douglas Fir Plywood Association Method



t_1 = Total width of web - $\frac{1}{4}$"
t_2 = Total width of beam - 1-7/8"
d = Depth of compression flange - 3-5/8"
h = Depth of beam

$$\frac{t_1}{t_2} = \frac{.250}{1.8750} = .1333 \qquad \frac{d}{h} = \frac{3.625}{15} = .2415$$

Referring to: Graphical solution of U. S. Forest Products Laboratory formula for "form factor at proportional limit wood I and box beams." Douglas Fir Plywood Association Engineer's Handbook, Section VII.

F_{pl} = (Form factor at proportional limit)

$$F_{pl} = .750$$

$$\frac{2(.8125)(15 - 7.75)}{12} + \frac{2(.75)(15)^3}{3(12)} =$$

$$393.5 + 140.6 = 534.1 \text{ in.}^4$$

$$M = \frac{10 \times .75 \times 1600 \times 534.1}{9 \qquad\qquad 12} = 59{,}400 \text{ in. lb.}$$

Then, taking a 24' span (uniformly loaded)

$$M = \frac{WlX}{2} - \frac{WX^2}{2}$$

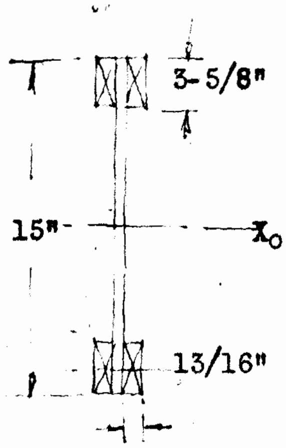
where X = 12 ft.
l = 24 ft.
W = #/ft.

$$59{,}400 = \frac{WlX}{4} = \frac{W \times 24 \times 12 \times 12}{4}$$

$$W = \frac{59{,}400 \times 4}{24 \times 12 \times 12} \qquad \frac{68.75\#}{Ft}$$

This is not sufficient for the 100 #/ft. design load being employed.

DEFLECTION TESTS
FLOOR TRUSS, TEST #120
CALCULATION B:



$$I_x = \frac{bh^3}{12} = \frac{13}{8} \times \frac{(3.625)^3}{12} = 6.45$$

$$I_{Xo} = 2(I_X + Ad^2)$$

$$I_{Xo} = 2(6.45 + 190) = 394$$

$d = 7.5 - 3.625$

$= 5.6875$

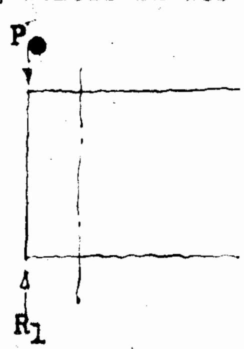
$S = \frac{Mc}{I}$ or assuming a value of $S = 1600$

$$M = \frac{SI}{c} = \frac{1600 \times 394}{7.5} = 83{,}500 \text{ \# in.}$$

$$M = 83{,}500 = \frac{W \times 24 \times 12 \times 12}{4}$$

$$W = \frac{83{,}500 \times 4}{24 \times 12 \times 12} \qquad \frac{96.5 \ \#}{\text{Lineal Ft.}}$$

Shearing stress in web



P_o approaches 0 at support

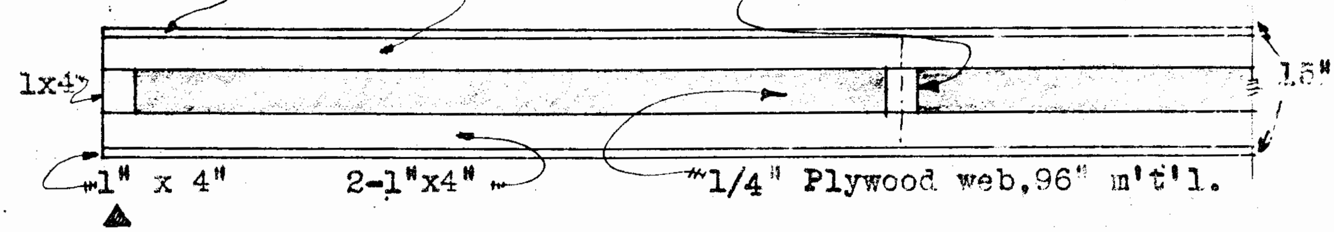
$$V = R_1 - P_o = 1200 - P_o$$

$$V = 1200 \ \# = P$$

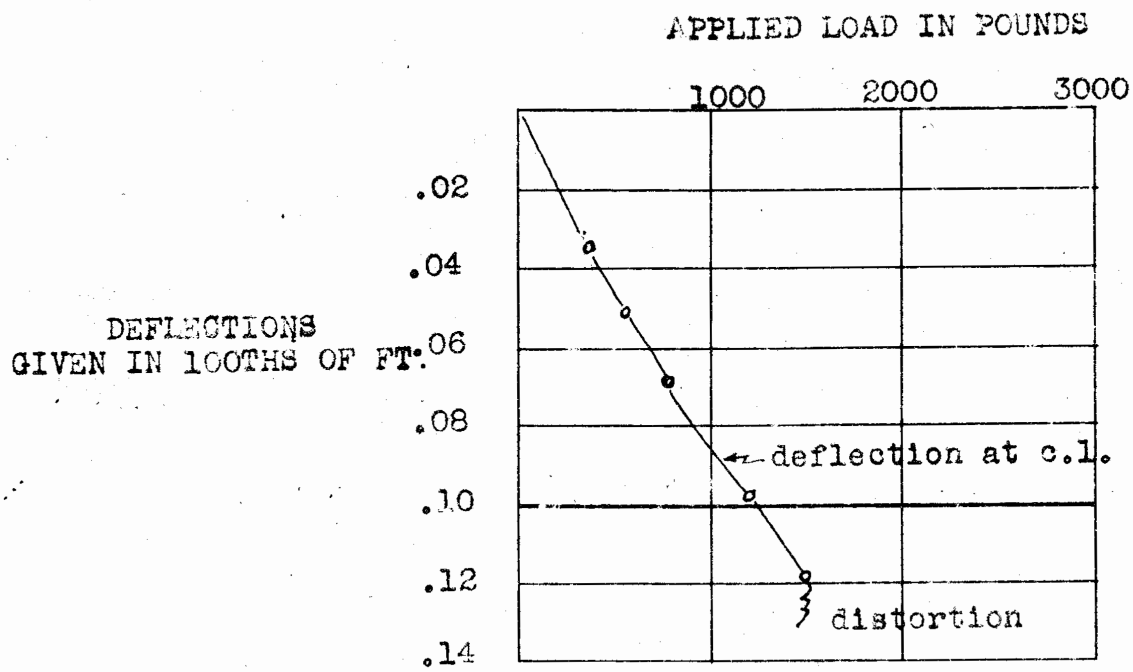
$$S = \frac{P}{A} = \frac{1200}{15 \times .25} = 480 \text{ PSI, a safe factor.}$$

DEFLECTION TEST
FLOOR TRUSS, TEST #120

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0"



Based upon preceding calculations covering this test.



MAXIMUM DEFLECTION AT c. l. FROM UNIFORM LOAD OF 1490# - .118'

WEIGHT OF TRUSS UNIT - 138#

MATERIALS -

MEMBERS: 13/16" x 3-5/8" #1 YELLOW PINE

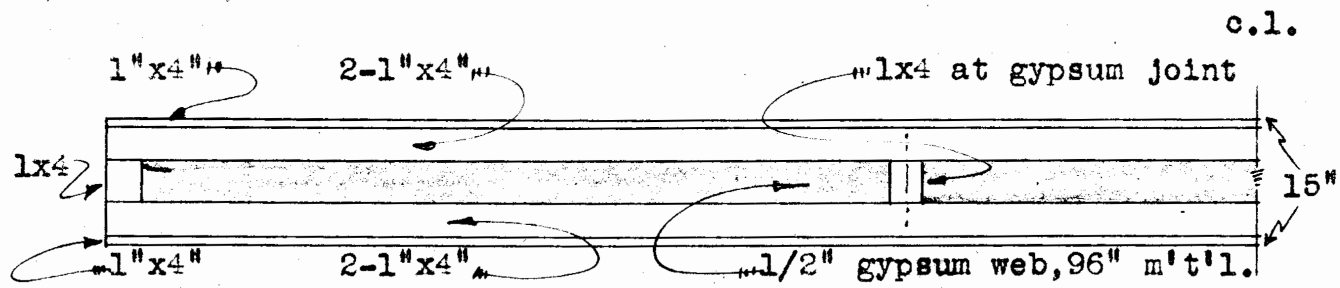
WEB: ¼" x 15" x 96" FIR PLYWOOD

NAILS: 10d BOX

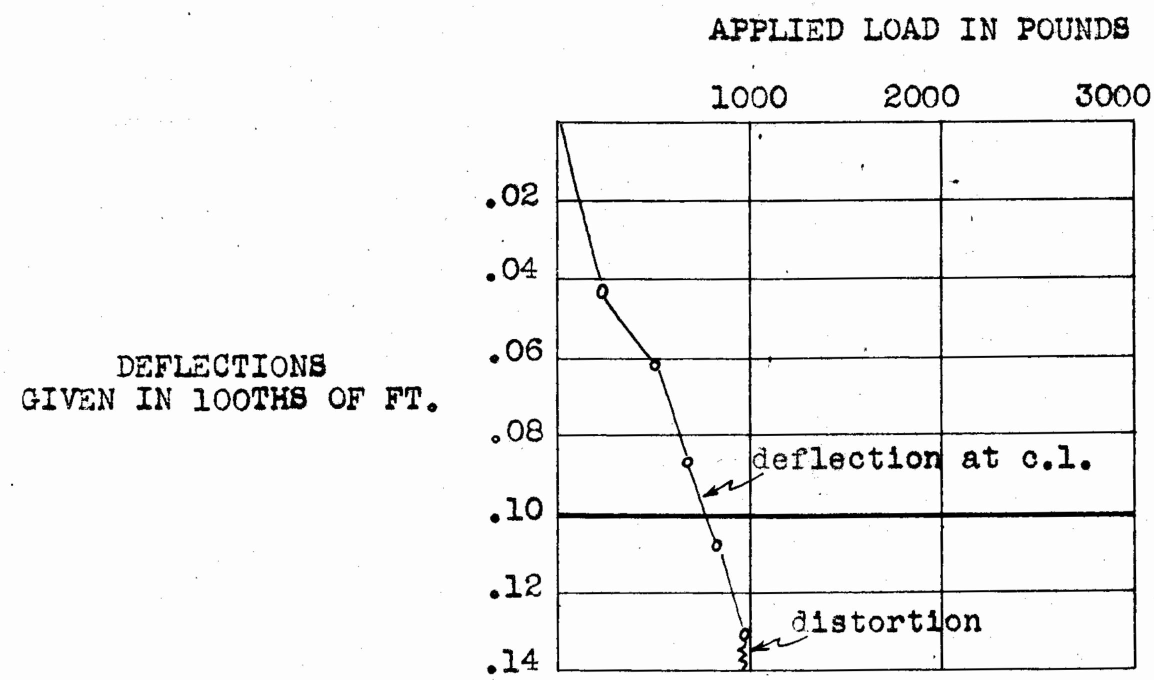
GLUE: PHENOLIC RESIN

DEFLECTION TEST
FLOOR TRUSS, TEST #121

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0"



Based upon calculations made for Floor Truss Test #120. No basis for allowing values for web material.



MAXIMUM DEFLECTION AT c. l. FROM UNIFORM LOAD OF 980# - .13'

WEIGHT OF TRUSS UNIT - 173#

MATERIALS -

MEMBERS: 13/16" x 3-5/8" #1 YELLOW PINE

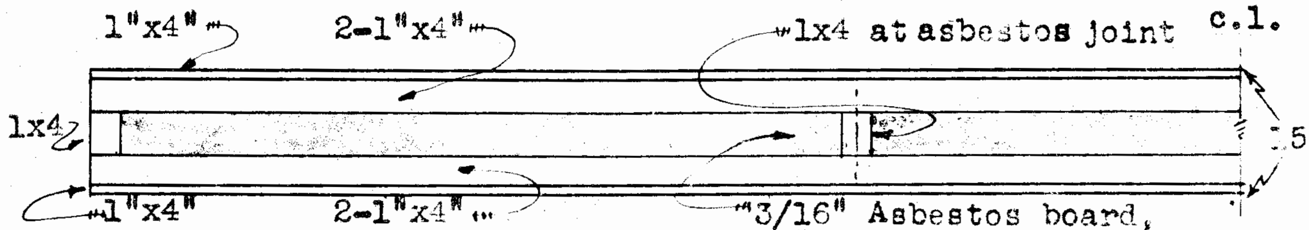
WEB: ½" x 15" x 96" GYPSUM

NAILS: 10d BOX

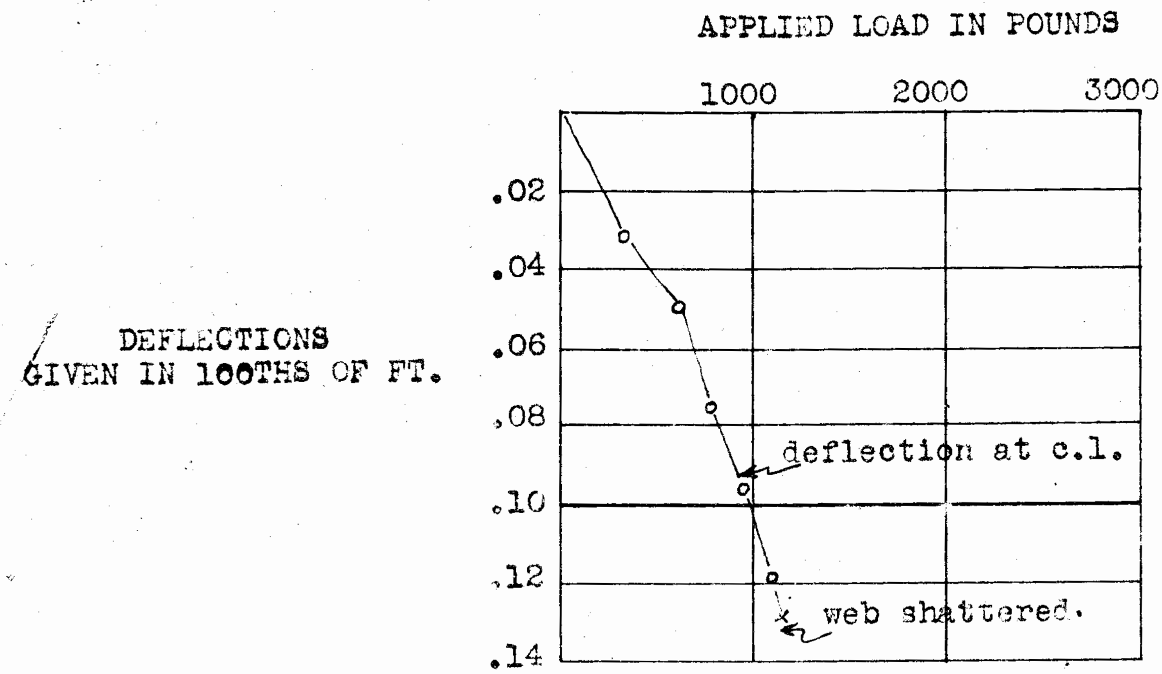
GLUE: PHENOLIC RESIN

DEFLECTION TEST
FLOOR TRUSS, TEST #122

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0"



Based upon calculations made for Floor Truss Test #120. No basis for allowing values for web material.



MAXIMUM DEFLECTION AT c. l. FROM UNIFORM LOAD OF 1190# - .118'

WEIGHT OF TRUSS UNIT - 168#

MATERIALS -

MEMBERS: 13/16" x 3-5/8" #1 YELLOW PINE

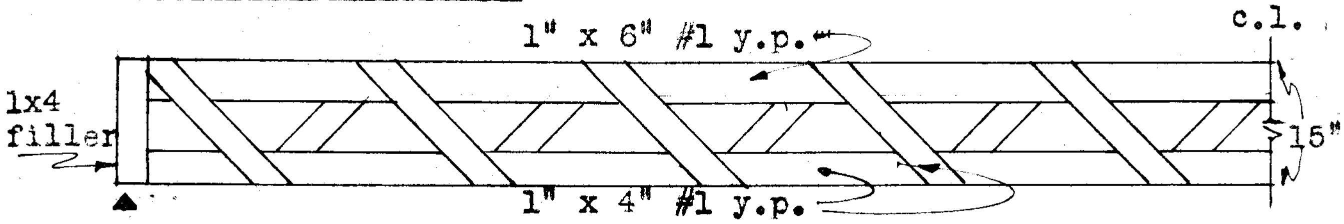
WEB: 3/16" x 15" x 96" ASBESTOS BOARD

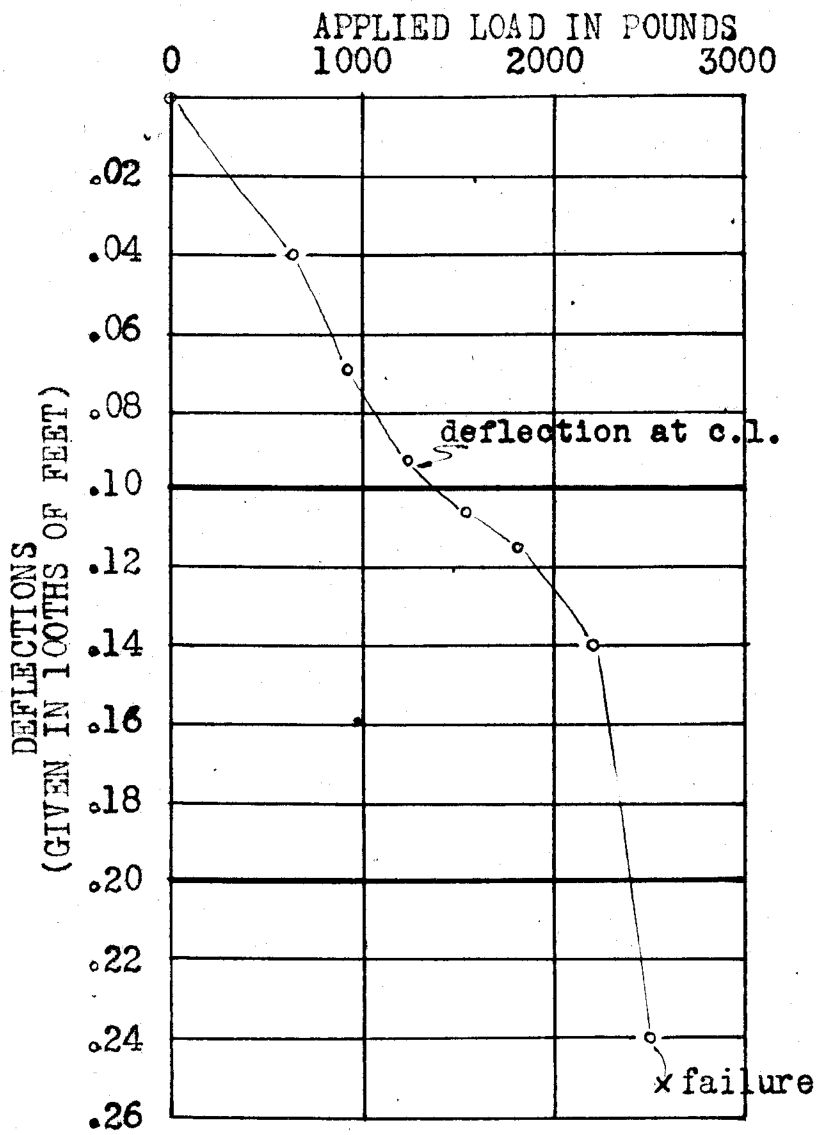
NAILS: 10d BOX

GLUE: PHENOLIC RESIN

DEFLECTION TEST
FLOOR TRUSS, TEST # 123

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0".





MAXIMUM DEFLECTION AT c.l. FROM UNIFORM LOAD OF 2486# - .24'

WEIGHT OF TRUSS UNIT - 130#

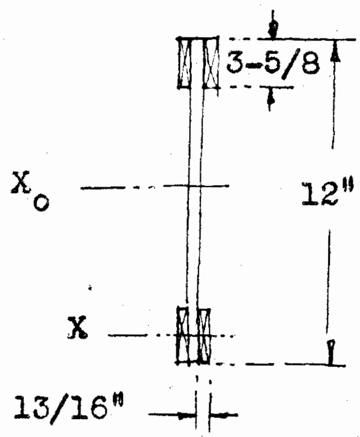
MATERIALS:

TOP FLANGE: 1" x 6" #1 Yellow Pine
BOTTOM FLANGE: 1" x 4" #1 Yellow Pine
Diagonals and Fillers: 1" x 4" #1 Yellow Pine

NAILS: 10d Box

GLUE: Phenolic Resin

DEFLECTION TESTS
FLOOR TRUSS, TEST #124
CALCULATIONS:

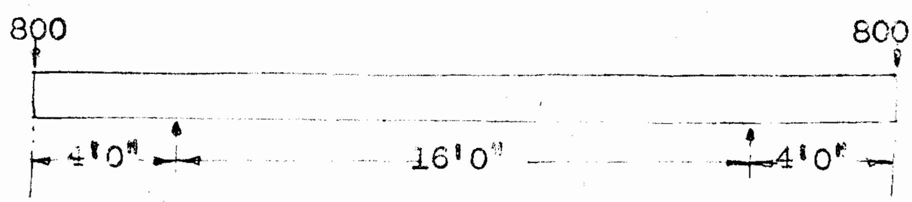


I_x = Moment of inertia of flange
I_{xo} = Moment of inertia of truss
b = Width of two 1" x 4"
h = Height of flange

$$I_x = \frac{bh^3}{12} = \frac{13}{8} \times \frac{(3.625)^3}{12} = 6.45$$

$$I_x = 2(6.45 + \frac{13}{8} \times 3.625 \times 4.1875^2) = 219$$

Maximum moment occurs where shear > 0



Inasmuch as this truss carried a balanced load, the reactions are equal and based upon the assumptions of 96 #/lineal foot, total load is found to be:

$$2300\# + 2(800) = 3900\#$$

and $R_1 = R_R = 1950\#$ each

Maximum moment could occur over supports or at center.

Trying the supports first

$$M = 12\,(800 \times 4 + 4 \times 96 \times 2)$$
$$= 12(3200 + 768) = (3968)12 = 47{,}616 \text{ \# in.}$$

Assuming a value of 1600

$$S = \frac{Mc}{I_{xo}}$$

when M = Moment in #
C = Distance from neutral axis to outer fibers
I_{xo} = Moment of inertia

$$I_{xo} = \frac{47{,}616 \times 6}{1600}$$
$$= 179$$

But, using 1" x 4", then

$$I_{xo} = 219$$

$$S = \frac{47616 \times 6}{219} = \frac{1300\#}{\text{Sq.in.}}$$

Basing calculations on a value of S = 1600, this is a safe figure. The War Production Board has recently (August 1943) issued a directive allowing a value of S = 1250.

Vertical shear must be taken up in the 45° diagonals.

When X = Distance from end of truss

$$V = W_1 + 96X$$

V = Maximum where X = 4

For beyond this point

$$V = W + (4 \times 96) - 96X_1$$

Where X = Distance from support toward the center

$$V = 800\# + 384$$

$$= 1184\#$$

If 1" x 4" diagonals are employed

Stress in diagonal

$$\frac{1184}{\text{Sine } 45^\circ} \times \frac{1}{\text{Area of diagonal}}$$

$$= \frac{1184 \times 1.414}{.8125 \times 3.625} \times \frac{570\#}{\text{Sq.in.}}$$

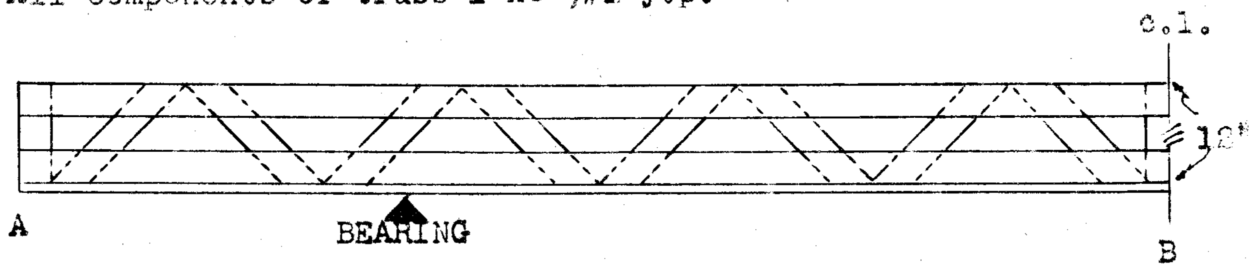
DEFLECTION TEST
FLOOR TRUSS, TEST # 124

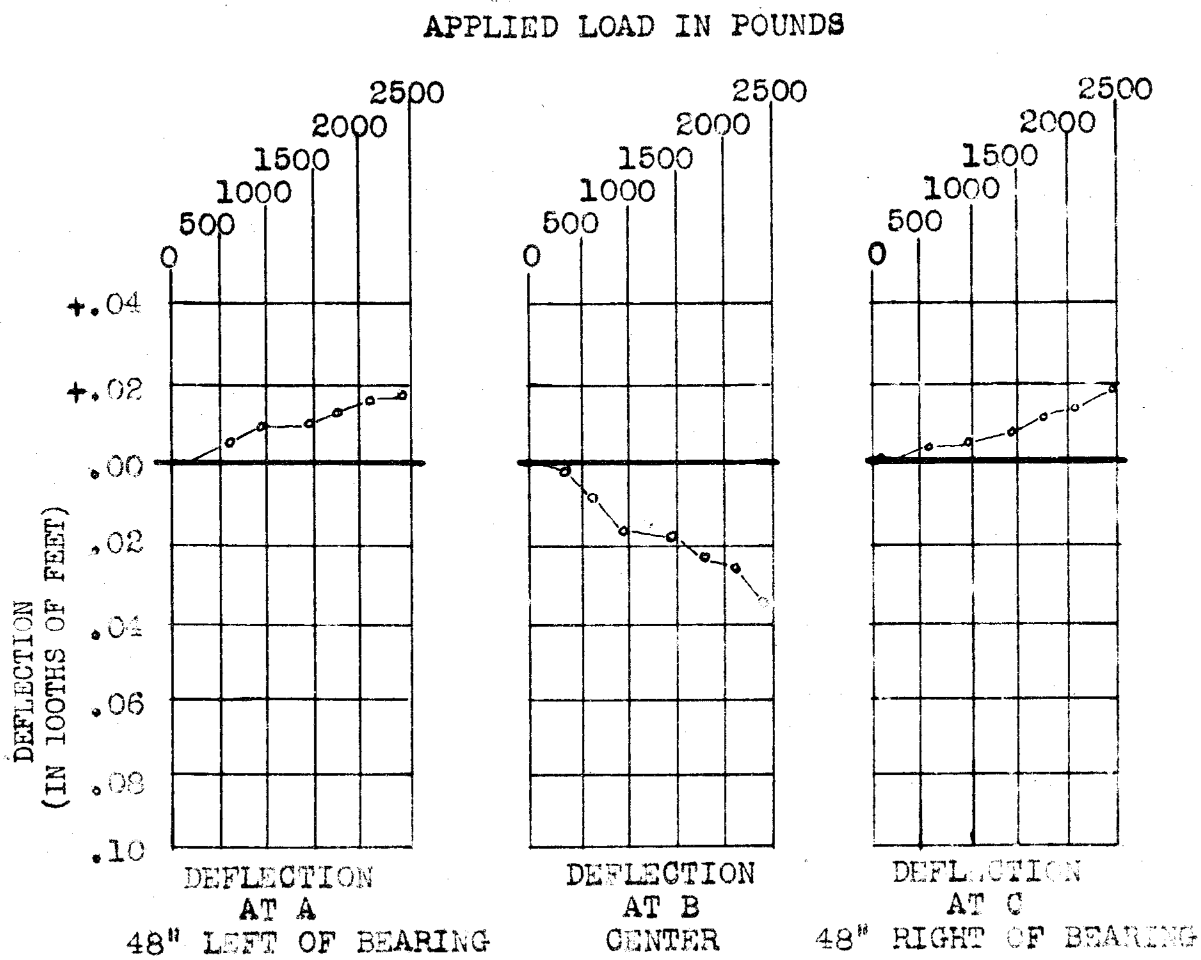
TRUSS SUPPORTED 48" FROM EACH END. SPAN: 24'0".

Based upon preceding calculations covering this test.

UNIFORMLY APPLIED LOAD

All components of truss 1"x4", #1 y.p.





WEIGHT OF TRUSS UNIT - 121.56#

MATERIALS:

MEMBERS: 13/16" x 3-5/8" #1 Yellow Pine
NAILS : 10d Box
GLUE: : Phenolic Resin

DEFLECTION TESTS
FLOOR TRUSS, TEST #125
CALCULATIONS:

Open-web truss to span 24'0". To be constructed of standard #1 Yellow Pine.

PROCEDURE:

Basic assumptions:

Truss to be 15" deep overall
Loading
Live load - 40#/sq.ft.
Floor load - 5#/sq.ft.
Wt. of truss - approximately 140#
Spacing
Trusses to be placed on 24" centers.

Calculations:

24 x 2 = 48 sq. ft. floor supported per truss

$\frac{45\#}{\text{sq.ft.}}$ x 48 sq.ft. = 2160#

Wt. of truss = 140#
2300# Total load per truss

or 96# to the lineal foot.

Assuming a uniformly distributed load of 96#/ft.

M = Moment
W = Load in pounds/ft.
X = Distance from end of truss to point under consideration
l = Length of truss (24'0")

In a uniformly-loaded, simple beam supported at each end the reactions at the ends are equal, and the maximum moment occurs at the center.

$$M = \frac{WlX}{2} - \frac{WX^2}{2} \qquad \text{but} \quad X = \frac{1}{2}(l)$$

$$M = \frac{WlX}{2} - \frac{WlX}{4} = \frac{WlX}{4}$$

$$M = \frac{96 \times 24 \times 12 \times 12}{4} = 82{,}800 \text{ \# in.}$$

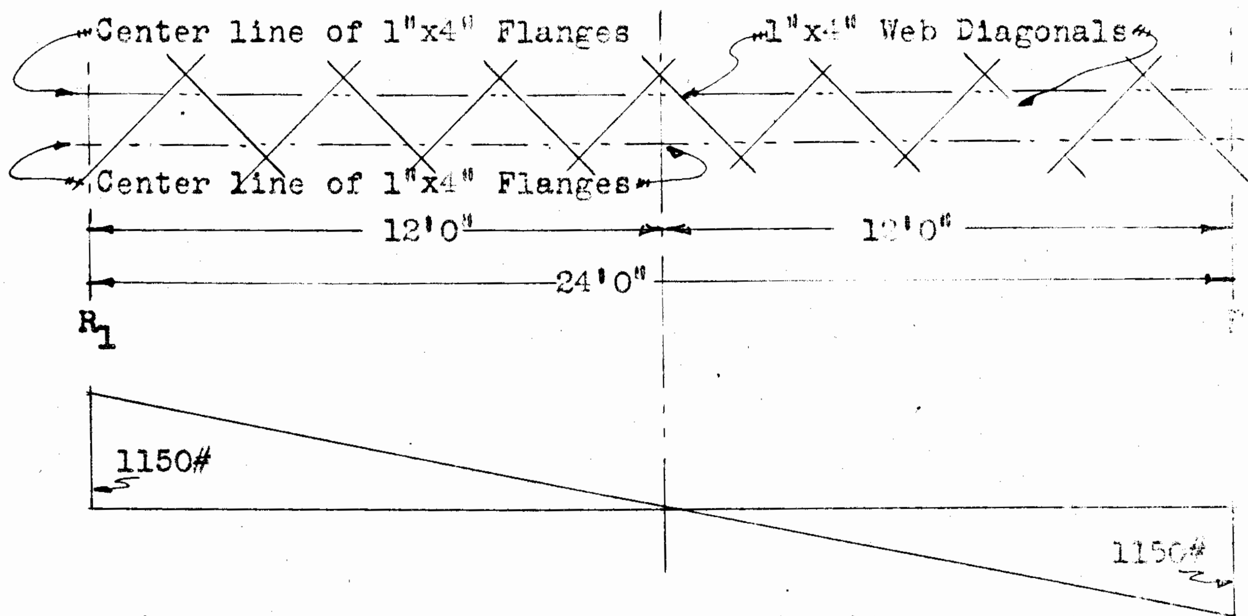
Referring to Southern Pine Manual of Standard Wood Construction, we use a value of

S = 1600#/sq.in.
S = Unit stress in pounds/sq.in.
M = Moment, pound inches

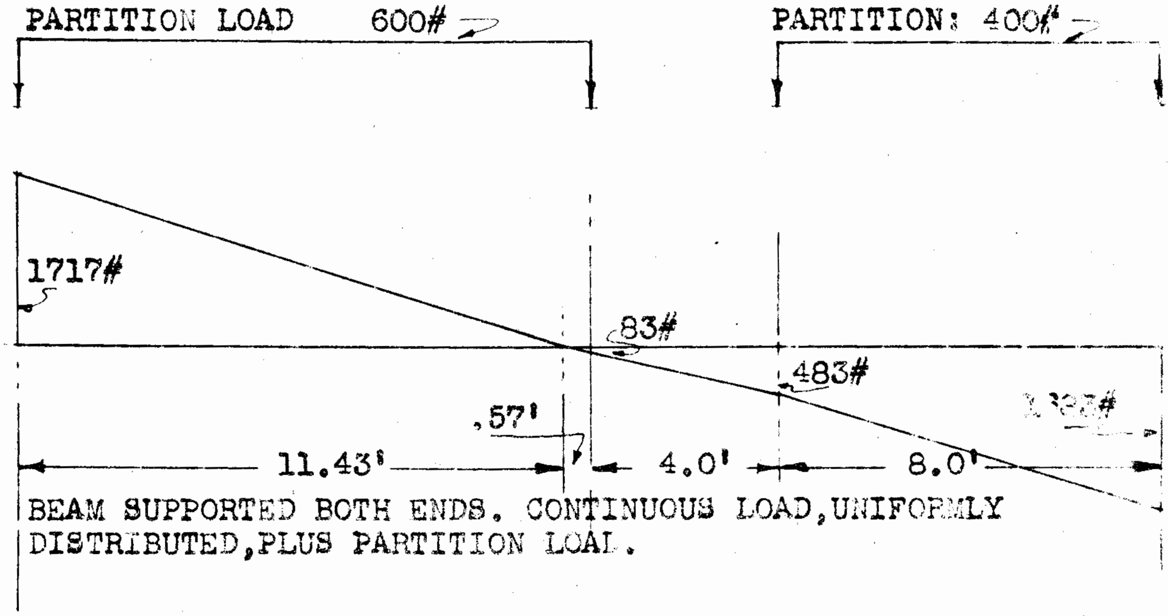
Diagonals could be of 1" x 2" material. To provide sufficient nailing and guling area a 1" x 4" was selected (3.95 sq.in.)

$$S = \frac{P}{A} = \frac{1460}{3.95} = \frac{370\#}{\text{sq.in.}}$$

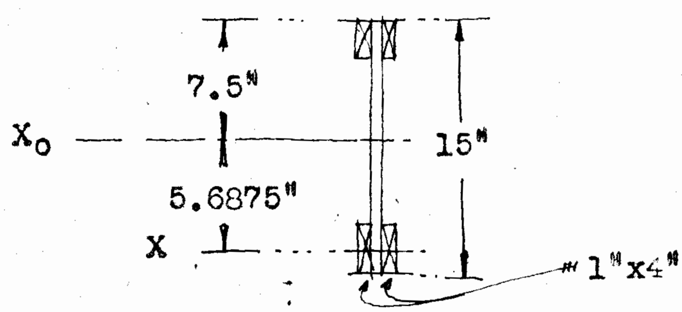
Results:



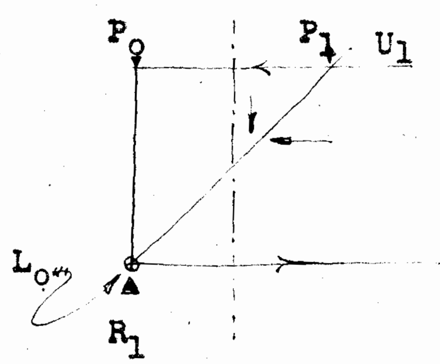
BEAM SUPPORTED BOTH ENDS, CONTINUOUS LOAD, UNIFORMLY DISTRIBUTED.



BEAM SUPPORTED BOTH ENDS, CONTINUOUS LOAD, UNIFORMLY DISTRIBUTED, PLUS PARTITION LOAD.



But there is another consideration. The web, composed of diagonals at 45° with the horizontal, must be able to withstand the vertical shear, which is a maximum at the support.



V - Vertical shear (pounds)
R_1 - Left reaction (pounds)
P_o - Weight of half panel which bears over support (pounds

$$V = R_1 - P_o$$

$$(L_o - U_1)_v = V$$

But diagonal is at 45°

$$(L_o - U_1) = \frac{V}{\text{Sine } 45^\circ} = 1.414V$$

$$R_1 = 1150$$

$$P_o = 120$$

$$V = 1150 - 120 = 1030$$

$$L_o - U_1 \doteq 1030 \times 1.414$$
$$= 1460 \#$$

c = Distance from neutral axis to outer fibers
I_{xo} = Moment of inertia of entire truss (inches)^4

$$S = \frac{Mc}{I_{xo}} \quad \text{or} \quad \frac{I_{xo}}{c} = \frac{M}{S}$$

$$\frac{I_{xo}}{c} = \frac{82,800}{1600} = 51.8$$

C = 7.5 assuming a 15" deep truss

$I_{xo} = 51.8 \times 7.5 = 390$ and

$I_{xo} = 2I_x + 2Ad^2$

I_x = Moment of inertia of one flange
A = Area of one flange
d = Distance between neutral axis and truss and neutral axis of flange

$$I_{xo} = 2\left[I_x + Ad^2\right]$$

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

b = Width of flange
h = Height of flange

We must assume a value for either b or h. Using a double flange with web in between members, we shall assume a value of 3-5/8" for h. Then

$$I_{xo} = 390 = 2\left[\frac{b \times (3\text{-}5/8)^3}{12} + b \times 3\text{-}5/8 \times (5.6875)^2\right]$$

$$\frac{390}{2} = \left[\frac{47.4}{12} + 117.0\right] b$$

$$b = \frac{390}{2(120.95)} = 1.62 = 1\text{-}5/8 \text{ approx.}$$

$$\frac{b}{2} = \frac{13}{16}$$

Therefore we can employ 2 1" x 4" pieces to form one flange.

TEST #125
ADDITIONAL PARTITION LOAD

REACTIONS:

$$24\ R_1 = 4 \times 400 + 12 \times 2300 + 18 \times 600$$

$$R_1 = \frac{1600 + 27600 + 10800}{24} = \frac{40{,}000}{24}$$

$$R_1 = 1666$$

$$24\ R_2 = 6 \times 600 + 12 \times 2300 + 20 \times 400$$

$$R_2 = \frac{3600 + 27600 + 800}{24} = \frac{39200}{24}$$

$$R_2 = 1634$$

$$R_1 + R_2 = 2300 + 600 + 400 = 3300$$

$$1666 + 1634 = 3300$$

Maximum moment occurs where $V = 0$

V = Vertical shear
R_1 = 1666 Lb.
X = Distance from left end of truss
W = 146 #/Ft.

$$V = R_1 - 146X = 0$$

$$X = \frac{R_1}{146} = \frac{1666}{146} = 11.4 \text{ Ft.}$$

Maximum moment becomes

$$M = \left[R_1\ (11.4) - \frac{146\ (11.4)^2}{2}\right] \times 12$$

$$M = (18{,}950 - 9470) \times 12$$

$$M = (9480) \times 12 = 114{,}000$$

Stress at this point (S)

S = Unit stress (#/Sq.in.)
M = Moment (# inches)
I = Moment of inertia (in.^{4})
c = Distance from neutral axis to outer fibers

$$S = \frac{Mc}{I} = \frac{114{,}000 \times 7.5}{393} = \frac{21800\ \#}{\text{Sq.in.}}$$

Allowable S = 1600 (Southern Pine Manual)

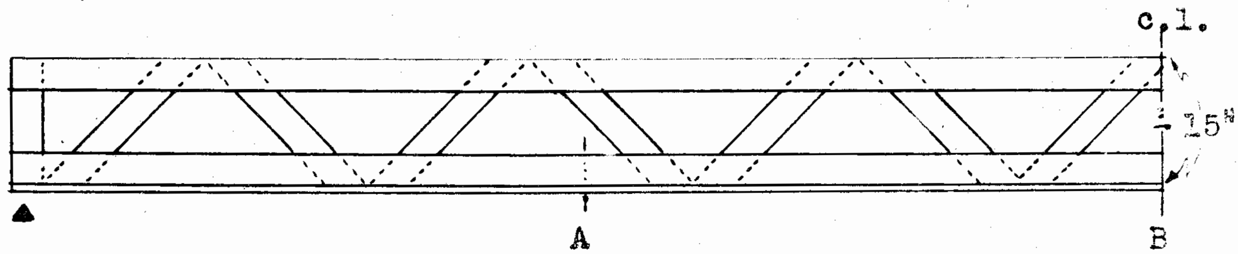
Use two trusses under partition wall.

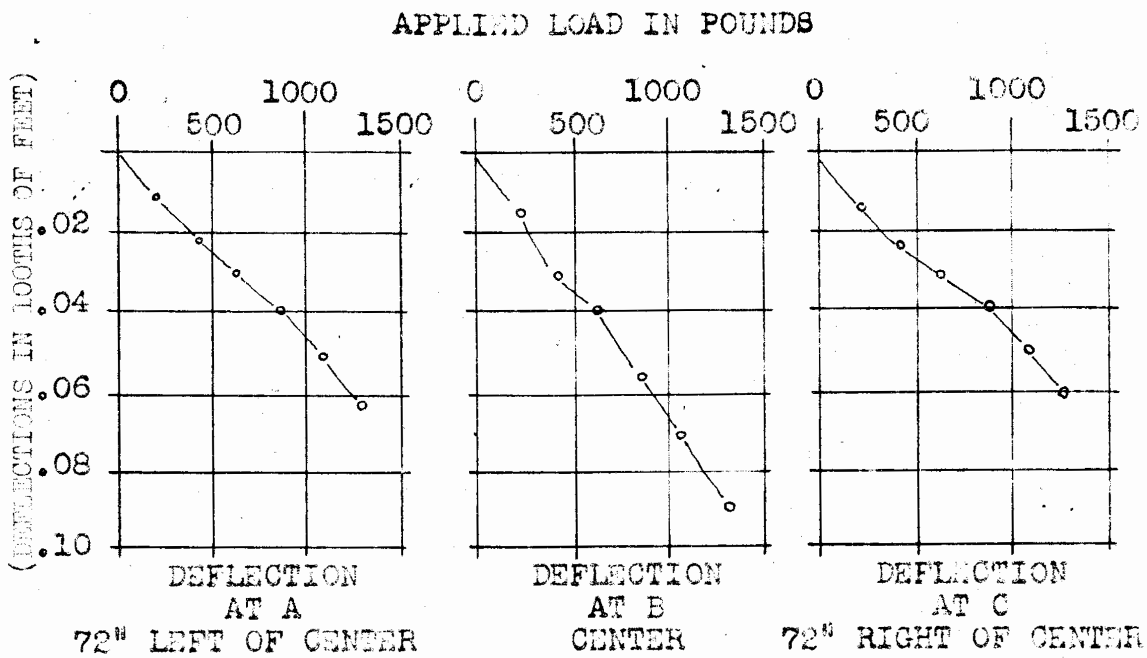
DEFLECTION TEST
FLOOR TRUSS, TEST # 125

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0"

UNIFORMLY APPLIED LOAD

All components of truss 1"x4",#1 y.p.





MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 1290#

AT A - .0625'   AT B - .09'   AT C - .06'

WEIGHT OF TRUSS UNIT - 119.88#

MATERIALS:

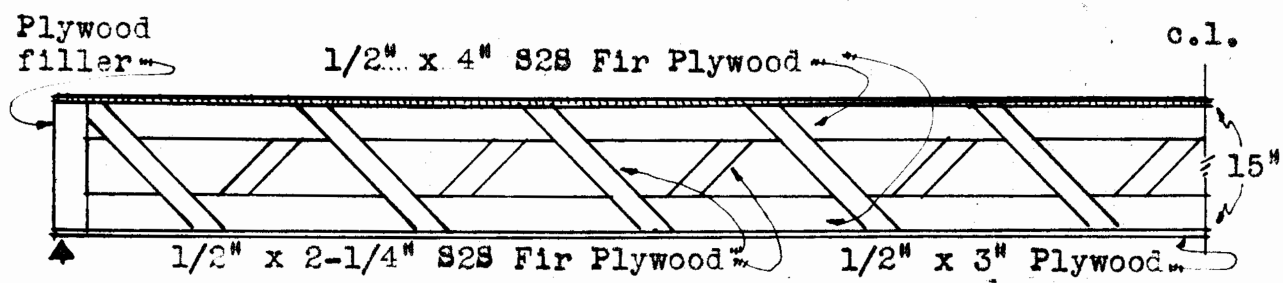
MEMBERS: 13/16" x 3-5/8" #1 Yellow Pine

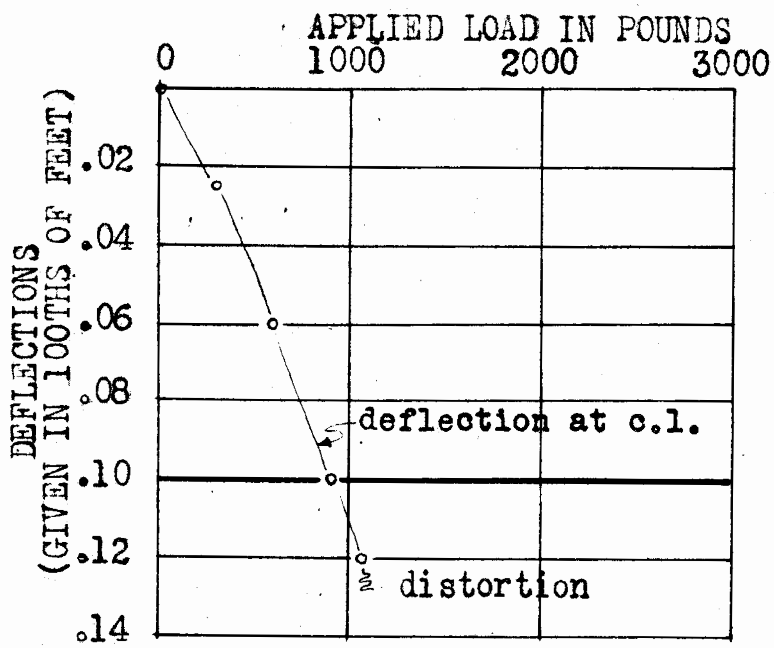
NAILS: 10d Box

GLUE: Phenolic Resin

DEFLECTION TEST
FLOOR TRUSS, TEST #126

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0".





MAXIMUM DEFLECTION AT c.l. FROM UNIFORM LOAD OF 980# - .12'

WEIGHT OF TRUSS UNIT - 56#

MATERIALS:

MEMBERS: ½" PLYWOOD

NAILS: 8d BOX

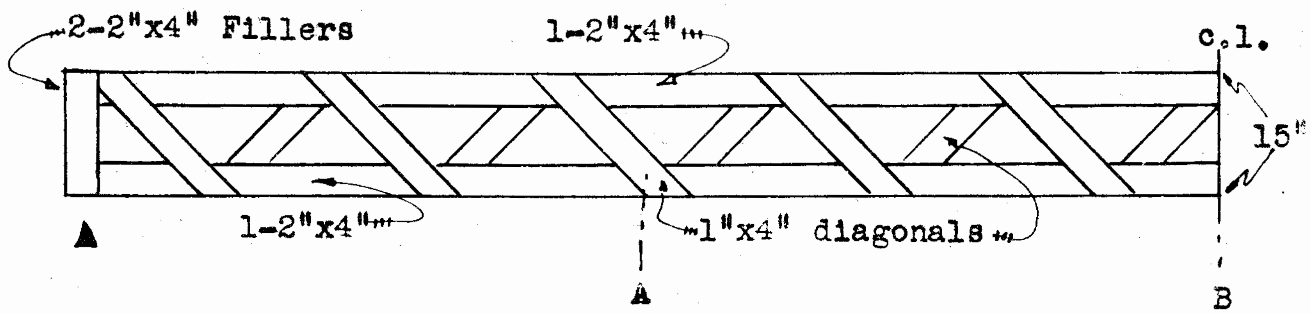
GLUE: PHENOLIC RESIN

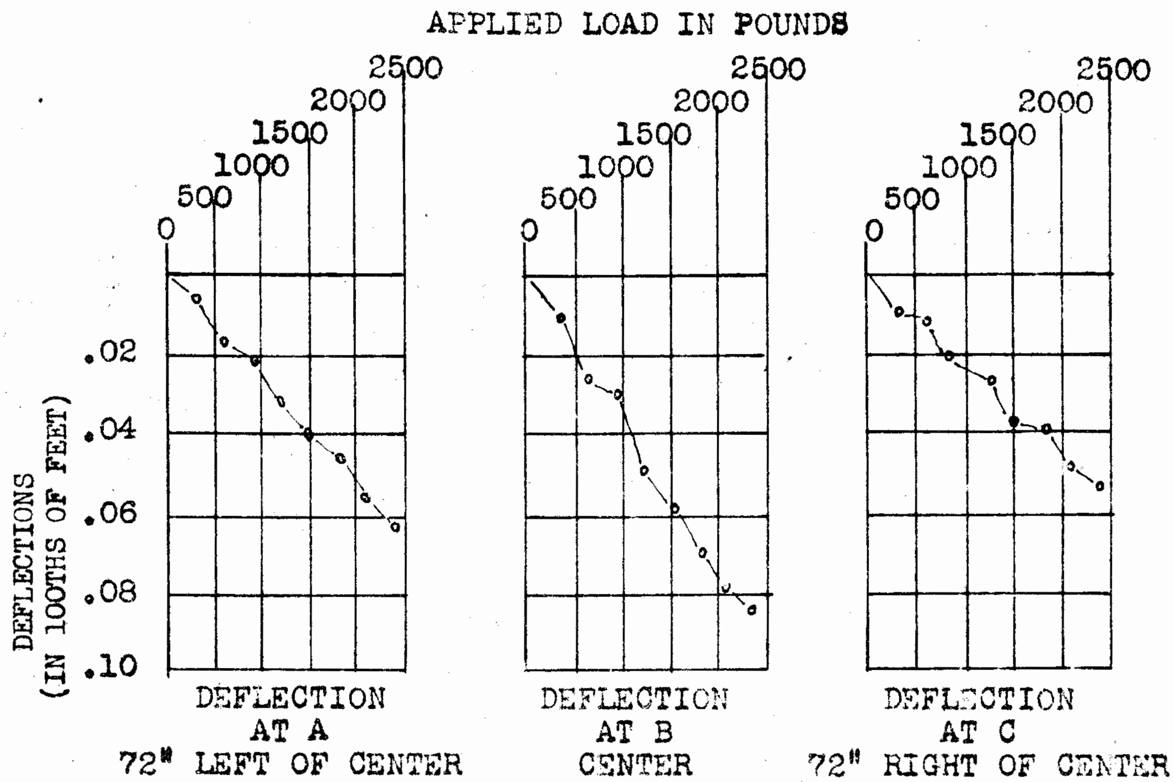
DEFLECTION TEST
FLOOR TRUSS, TEST # 127

TRUSS SUPPORTED AT EACH END.
SPAN, 24'0"

Based upon calculations made for Floor Truss Test #125. Employing single 2" x 4" #1 yellow pine flanges with diagonals on the outside.

UNIFORMLY APPLIED LOAD





MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 2386#

AT A - .062'   AT B - .084'   At C - .054'

WEIGHT OF TRUSS UNIT - 120#

MATERIALS:

FLANGES: 2" x 4" #1 Yellow Pine
DIAGONALS: 1" x 4" #1 Yellow Pine
FILLERS: 2" x 4" #1 Yellow Pine

NAILS: 8d BOX
GLUE: PHENOLIC RESIN

FLOOR TRUSSES
SUMMARY

In coming to any conclusions regarding supports for low-cost housing floors, there must be a definite division involving the types of supports being considered.

In the first group are combined components which propose to free span an area, these components to rest on some type of foundation or bearing and be off the ground, permitting air circulation under them.

In the second group are components which in themselves would produce a floor or a sub-floor. These are, of course, some types of slab. The second group will be discussed in connection with the foundations. This discussion will therefore involve only those components which might be classified in the first group.

Comparative figures between orthodox foundation and framing, and trusses employing limited foundation indicated there was a definite advantage in time and materials with subsequent savings, in the employment of clear-span trusses.

Where a volume of material would be employed, it becomes apparent the diagonal members in the open-web truss could be cut from salvage ~~at a~~ substantial saving. As the engineer's comments indicate, these components need not be uniform as to width as long as there is not too great a discrepancy which might tend to throw the truss off its structural balance.

Truss units can be standardized and assembled cheaply and quickly with a minimum amount of skilled labor. When conveyed to a site ready to receive them, they can be put into place without any delay and with a minimum amount of bridging, the only bridging employed being that which holds up the first two trusses. These provide a base for the running strips which are carried across the top of the trusses and act as spacers, remaining until floor components are introduced.

All tests indicate truss components can be designed very light in weight and standardized to a high degree if in their use in the designing of a living unit the factors which contribute to concentrated loads within the structure are carefully considered and compensated for either in special trusses built to carry these additional loads or in doubling up of the standard trusses.

The improved results in load-bearing values with a truss having a glued joint are worth the investment.

Where nails pass through more than two members, any members beyond these first two derive greatly diminished benefits from the nails which enter them. It seems more advisable to use a smaller nail and to nail from both sides despite the fact this requires either the turning up or turning over of the component during its assembly.

The open-web truss permits knob and tube wiring to be carried through on knobs secured on the under edge of the upper flange and alongside the truss. The nailing of knobs on the lower flange of the solid truss would offer difficulties, particularly where limited excavation had been carried out. The stouter flange with web members on the outside produces a more satisfactory component than that resulting from web members placed between two light flange members.

Where light exterior walls are employed in connection with these trusses it would appear to be a more practical construction to carry wall materials to the bottom of the trusses, permitting nailing to occur where the supporting members of the walls fit against the ends of the floor trusses and allowing the walls to rest on the same foundation which supports the trusses.

As Test #124 indicates, it is entirely possible to carefully design a truss of minimum height and employ smaller components, so balancing this truss in cantilever that reactions are set up which result in a very satisfactory component which not only is capable of carrying floor and live loads but ceiling and roof loads as well.

The solid-web truss would appear to be more practical for spans beyond those in which research has been undertaken, and it should not be dismissed too lightly. These solid-web trusses do have a tendency to block off piping and wiring, resulting in increased labor cost when the introduction of utilities is necessary under the floors. If a satisfactory, light-weight, solid-web truss was developed, it would be necessary that utilities either be carried under these truss components or that holes be established by the engineer through which wiring and piping could be conducted. Unless such a precaution was taken, the inherent strength of the truss could be affected by holes drilled by mechanics for their convenience rather than to an engineering advantage.

SECTION III

ROOF TRUSSES

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana
March 1, 1944

DEFLECTION TESTS
ROOF TRUSSES
PROCEDURE:

In the very beginning of the analysis of the small house, it was determined the partitions would not be load bearing and the ceiling joist and roof rafters were to be so coordinated as to create a free span over the floor area. This required that a truss be designed to perform this function.

The interesting observation here is the initial tendency to create a truss exceeding the required structural values. As the following studies would indicate, the original trusses not only employed considerably more material than was necessary, but the use of solid panels and intermediate members was found to be unnecessary and produced an unduly heavy truss.

These studies indicate that careful analysis of light truss design results in their being a very economical method of ceiling and roof construction.

TEST #128

The initial roof truss was designed for a 24' span. It was desired to introduce two solid webs and to support these solid webs with material not exceeding 1" x 4". Asbestos board not exceeding 3/16" in thickness was to be the web member. The truss was to have a pitch permitting a 4' x 12' sheet of asbestos to be cut to produce a 24' component.

Initial calculations indicated an excessive dead weight in the truss, but since the information sought had more to do with the values of the solid web than the values which might be necessary to a successful and usable truss, construction of two of these trusses was carried out and load tests made.

It will be seen the truss was much too heavy for practical use and greatly exceeded the needed structural values for the 24' span.

The next three tests were based upon the use of ½" gypsum in the web.

TEST #129

This truss was designed for a span of 16', employing a maximum rise of approximately 36". Design developed indicates a rise of 37½", a web of 3/8" gypsum, the center vertical member being a 1" x 4", one on each side of the web. Joints were perfected by employing ¼" plywood gussets glued and nailed into place. Test loading was carried out on one side of the truss, the practice for testing roof trusses. Checkerboard pattern of loading as described was used.

The truss was loaded to a total slightly exceeding 1 kip. The interesting observation is the minimum of displacement on the unloaded flange. These flanges had been nailed with 6d box nails spaced 6" on center and staggered along the 1" x 2". Nailing was done from only one side. Upon the removal of the test load, the truss returned to its original position. An attempt was made to take this truss apart to determine the condition of the gypsum where nails had passed through it in the nailing process. This proved to be somewhat difficult, and no definite observation could be made as to the structural value of the web in relation to the total strength of the truss.

TEST #130

While the single-web truss spanning 16' gave encouraging results, it was believed some comparison should be made between the original two-webbed asbestos truss and a similar truss in which ½" gypsum would be employed in the two webs. Such a truss was constructed in a manner identical to the asbestos truss and subjected to the same type of loading. Excessive weight was again apparent. The test substantiated what had already been found out in the asbestos truss: the double web was acting as little more than a stiffener at an expense in dead weight.

TEST #131

Employing the factor obtained in the three previous tests, a roof truss was constructed which used a single, solid web of ½" gypsum cut from a 4' x 12' sheet. The characteristics of the 16' single-web truss were adapted with the exception the vertical member became a 1" x 6" instead of a 1" x 4" piece. This truss developed good characteristics but remained heavy. Because of the minimum amount of cross section in the flanges it made a very difficult component to handle. Raising these trusses to a height of 8' off a potential floor required considerably more labor than was believed to be economical. Twisting in the members after erection resulted in the introduction of spacers in the manner of bridging at points 8' in from the outer ends of the trusses. This was a slow operation requiring the fitting of pieces between the lower flanges of the truss, this causing the ultimate cost to exceed that which was believed to be justifiable for the complete operation of erecting members acting as ceiling joists and roof rafters.

In the examination of all of these solid-web trusses, it became increasingly apparent the solid web was not contributing very much structural value to the truss. The solid-web components required a vertical joint in the center. When this joint was compensated for in the calculations, a member was introduced which possessed such values that the web itself would not have been required at this point. Removal of these flange members after loading tests disclosed the nails holding to satisfaction. The nail holes through the web had become elongated, indicating the webs were offering little resistance at their point of contact with the nails. This was a bit more apparent in the gypsum than in the asbestos board, although the same factors were present.

The engineer reasoned if these webs were securely retained in position by the nails and this elongation did not occur, then the loads would be transferred to these webs, and buckling could be expected to occur. It seemed logical the roof truss next to be tested would be one with simple yellow pine members from which solid web had been eliminated.

TEST #132

In order to obtain basic comparisons, the first truss was designed for a span of 12'. As shown on the deflection test drawing, this constituted a light scissors truss. The results of this loading were entirely satisfactory although the component seemed very light for even a 12' span.

TEST #133

The scissors truss was carried into the second design for a span of 16'. This, too, proved to be entirely substantial. In both of these trusses no attempt was made to introduce a lower chord equivalent to the ceiling joist. Such studies as were being made at the time anticipated a living unit in which there would either be no ceiling, the sub-roof being the finished ceiling, or the ceiling would follow the pitch of the lower chords of the scissors truss. From a study of these trusses the next test was designed.

TEST #134

This truss was designed as a Belgian truss with four top panels and three bottom panels in its 24'0" span. This design is economical as it offers the possibility of employing short-length material with a minimum amount of waste. Structurally, it has been proved practical. Its design worked well within the limits established, i.e., a height not to exceed 4'. Tentative weight calculations indicated it would be light and easy to handle.

The first design required an offset in the upper chord to receive a certain type of roof sheathing. The practice of gussets of both 1" material and 3/8" plywood was adopted as a result of the excellent joints obtained in all of the tested trusses employing such joining. Tests indicated success of this truss, and its design was incorporated into Unit B-13, deflection tests on which are included in this report.

The assembly of the truss was simple and rapid. Sufficient rigidity was imparted so that in conveying the truss to its point of erection and the handling required in the placement caused no distortion. An excessive amount of labor was not required and the time factor was very favorable. As this report deals with the engineering features of the various components employed, the details of construction and handling of this truss into position will not be discussed.

TEST #135

In its essentials the truss tested under this number had the basic components and characteristics of that of Test #134. It will be observed that the gussets are raised up on the lower chord a distance of 1" and that the lower chord has been increased in size from 2" x 3" to 2" x 4". The lower batten of 1" x 4" material has been added as the use of this component was to be in Unit B-15 in which a ceiling was slid rather than nailed into place. When it was necessary to create this 1" space in which to glide the ceiling, the engineer increased the lower chord 1" in order not to lose bearing for the gussets which were both glued and nailed into position as usual.

TEST #136

A complete analysis of all trusses indicated the open-web truss with a rise of 4' in a span of 24' and with four upper and three lower panels was offering the most satisfactory results, both as to quantity of material employed and the rapidity with which its light weight permitted erection.

Some of the components were not doing all that could be expected of them, but the principles involved were inherently sound. The engineer recalculated this truss, and as a result of these new calculations Test #136 was constructed and loaded.

TEST #137

This was a light weight member to span 12'. The components were to be either exposed or were to carry a ceiling which would terminate at the underside of the cross member of the "A" frame. This was very cheaply and simply constructed and involved no problem in design or handling, its erection being very quick and its load-bearing properties as anticipated.

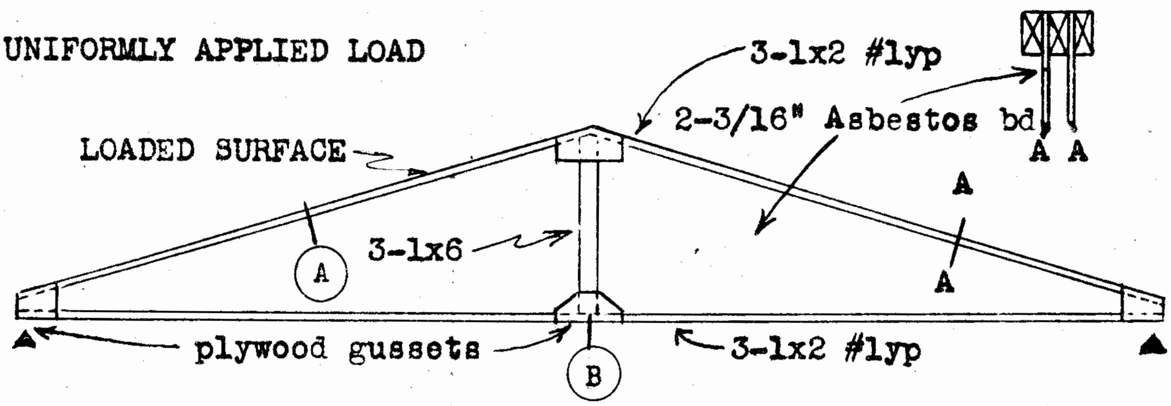
WIND PRESSURE ON ROOFS

In the low-ptiched roof normally employed in small residential structures there is no justification for calculating a downward-acting wind load. It is more essential compensation be made for lifting forces. It becomes apparent that in these small structures it is essential that a sequence of ties from the foundation through the walls and finally to the roof members be employed to offset this lifting action which can become as great as 0.7 to 1 x velocity pressure.

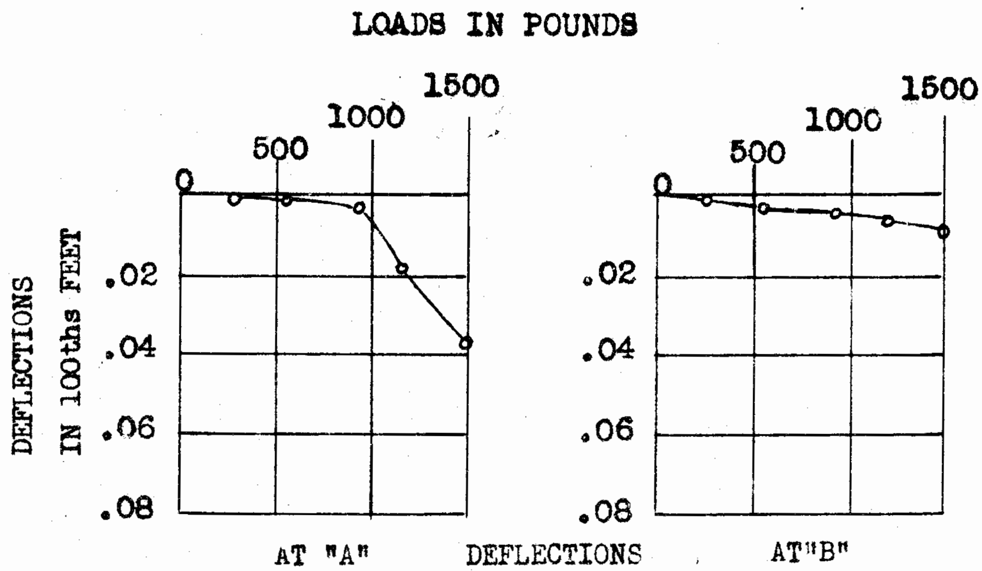
DEFLECTION TEST
ROOF TRUSS TEST #128

TWO WEB ASBESTOS BOARD

UNIFORMLY APPLIED LOAD



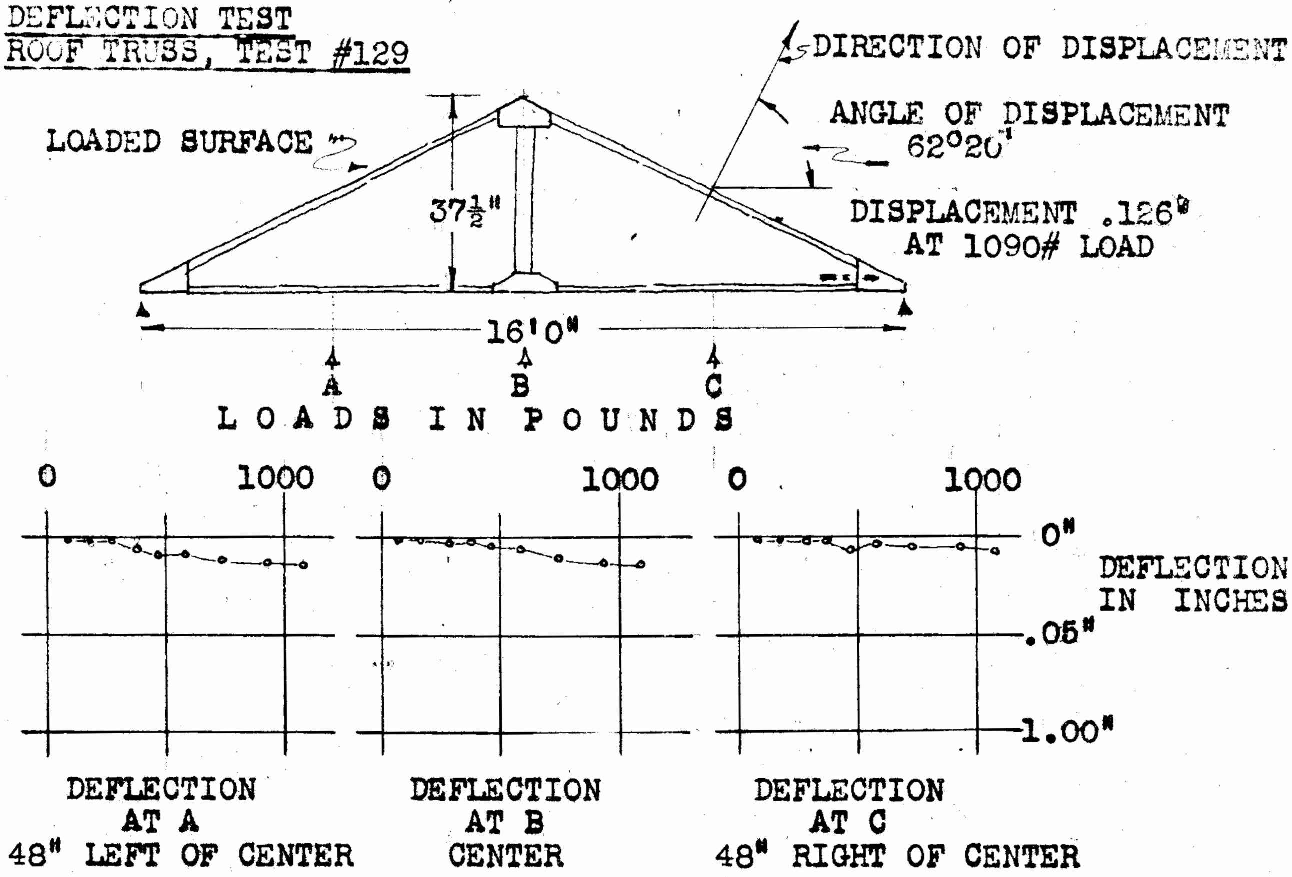
LENGTH OF TRUSS - 24'0"
HEIGHT OF TRUSS - 4'0"
WEIGHT 262#



MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 1495#
AT "A" - .038'
AT "B" - .009'

MATERIALS:
WEB: 2-3/16 x 4' x 12' ASBESTOS BOARD
FLANGES: 3 members - 1"x 2" #1y.p.
VERTICAL MEMBERS 3-1x6 #1y.p.
GUSSETS: 1/4", 3 ply, S2S FIR PLYWOOD

DEFLECTION TEST
ROOF TRUSS, TEST #129



MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 1095#

AT A - .1563"   AT B - .130"   AT C - .0628"

WEIGHT OF TRUSS UNIT - 87#

MATERIALS:

WEB: 3/8" GYPSUM BOARD

FLANGES: 1" x 2" #1 Yellow Pine

VERTICAL MEMBER: 1" x 4" #1 Yellow Pine

GUSSETS: 1/4",3 ply,S2S Fir Plywood

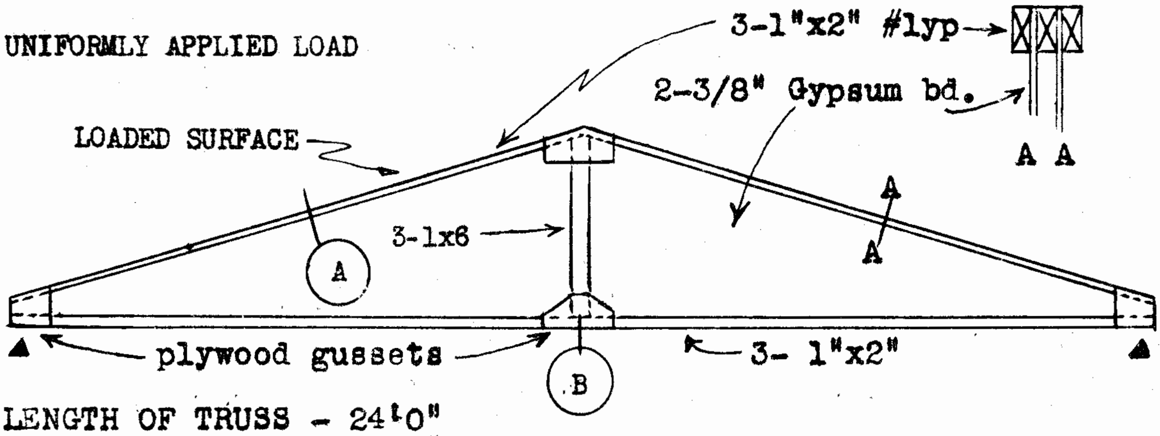
NAILS: 8d box

GLUE: Phenolic Resin

DEFLECTION TEST
ROOF TRUSS TEST #130

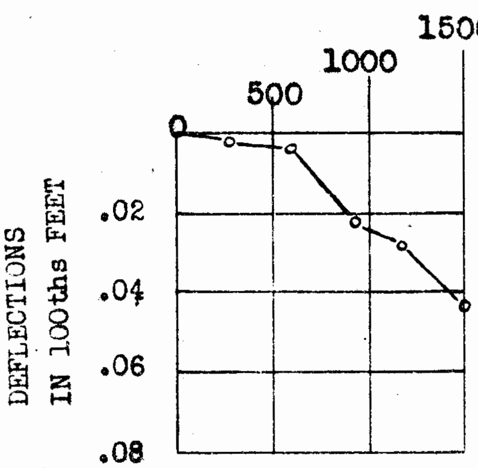
TWO WEB GYPSUM BOARD

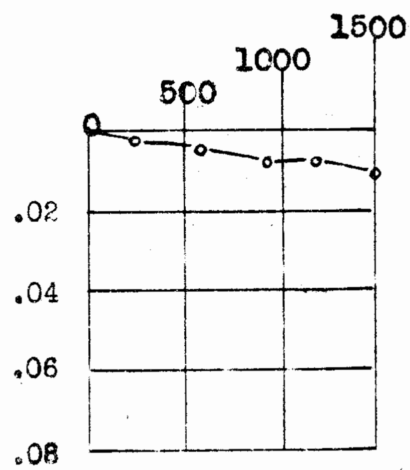
UNIFORMLY APPLIED LOAD



LENGTH OF TRUSS - 24'0"
HEIGHT OF TRUSS - 4'0"
WEIGHT 262#

LOADS IN POUNDS





AT "A" DEFLECTIONS AT "B"

MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 1490#

AT A - .044' AT B - .012'

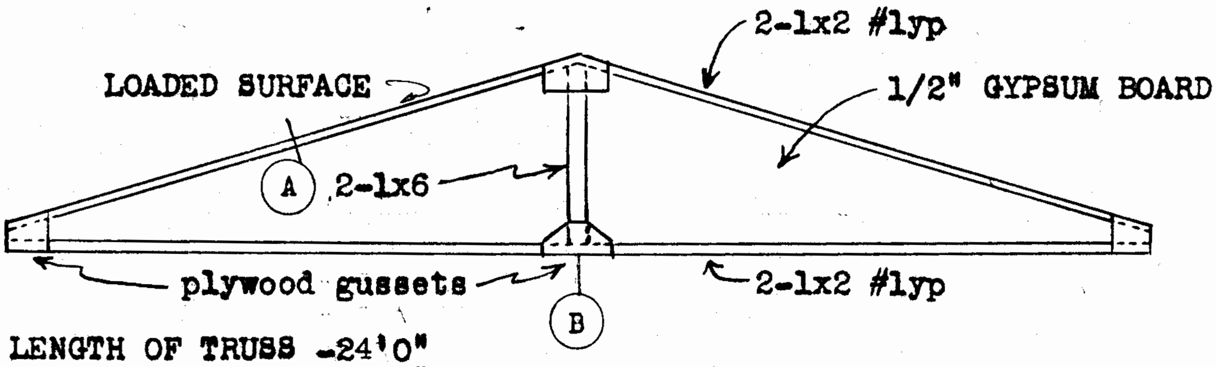
WEIGHT OF TRUSS UNIT - 262#

MATERIALS:

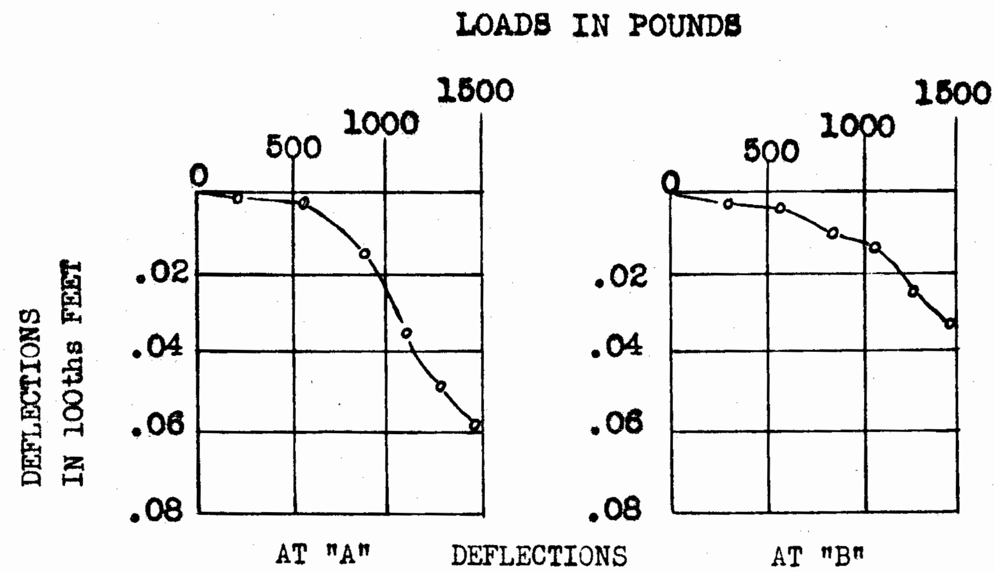
WEB: 2-3/8" x 4'0" x 12'0" GYPSUM BOARD
FLANGES: 3 members - 1"x 2" #1y.p.
VERTICAL MEMBERS: three 1" x6" #1y.p.
GUSSETS: 1/4", 3ply, S2S FIR PLYWOOD

DEFLECTION TEST
ROOF TRUSS TEST # 131

SINGLE WEB GYPSUM BOARD



LENGTH OF TRUSS -24'0"
HEIGHT OF TRUSS - 4'0"
WEIGHT -160#



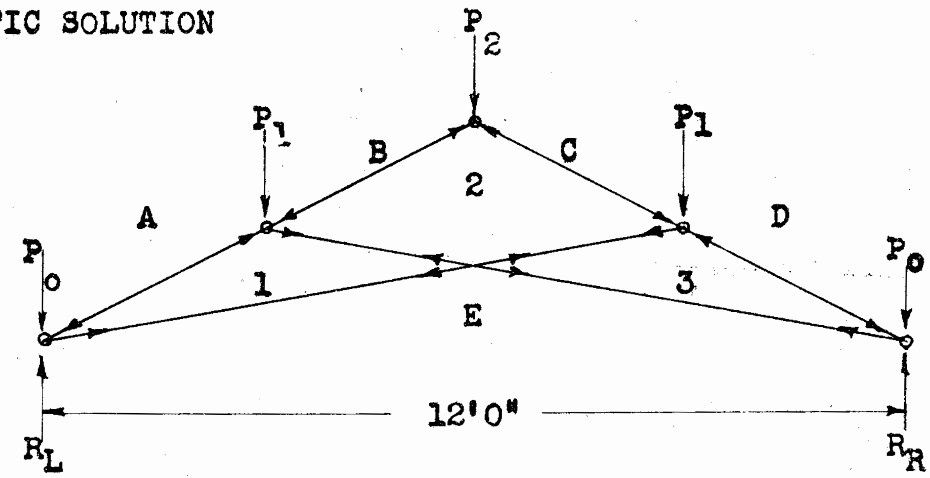
MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 1490#
AT "A" - .059'
AT "B" - .034'

MATERIALS:
- WEB: 1-1/2"x 4'x 12' GYPSUM BOARD
- FLANGES: 2 members - 1"x 2" #lyp
- VERTICAL MEMBERS: 2-1"x 6" #lyp
- GUSSETS: 1/4", 3 ply, S2S FIR PLYWOOD

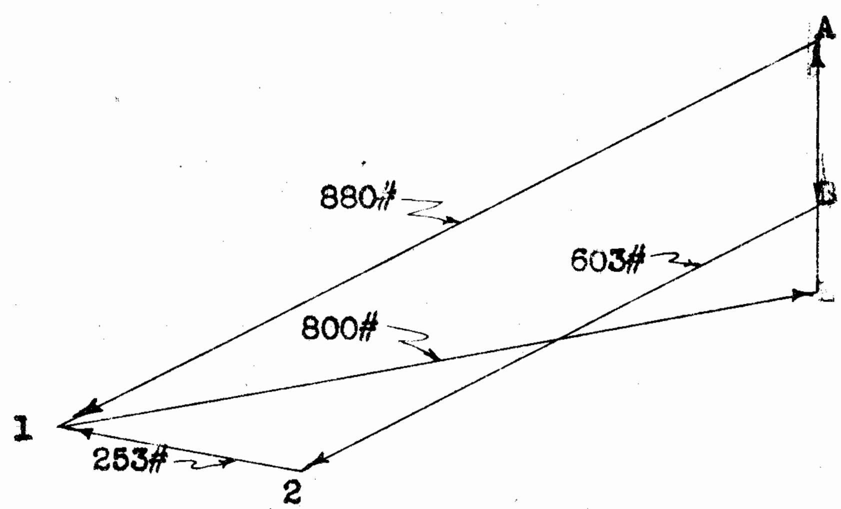
DEFLECTION TEST
ROOF TRUSS TEST #132

SCISSORS
SPAN: 12'0"

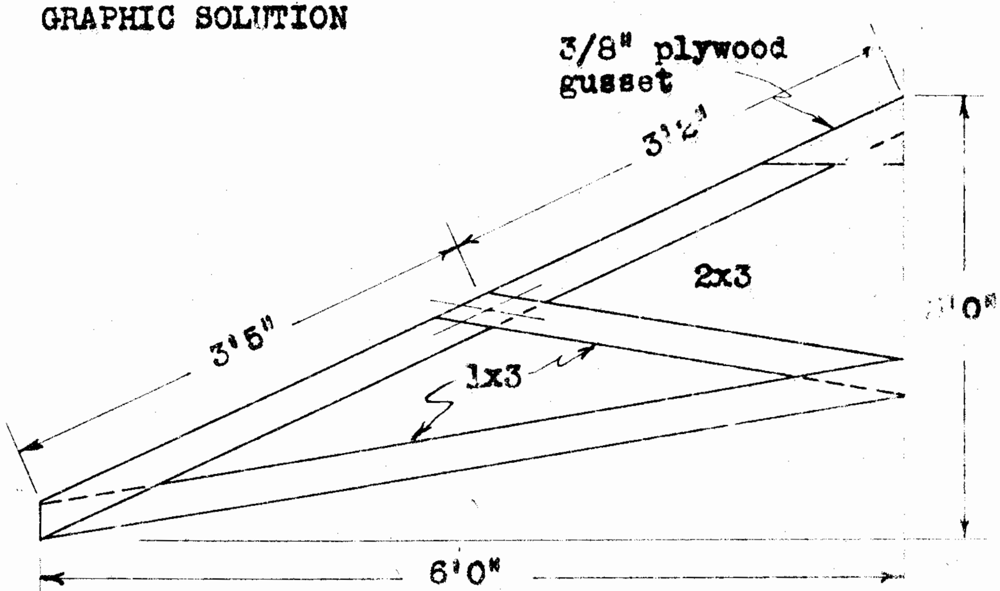
DIAGRAMMATIC SOLUTION



FORCE DIAGRAM



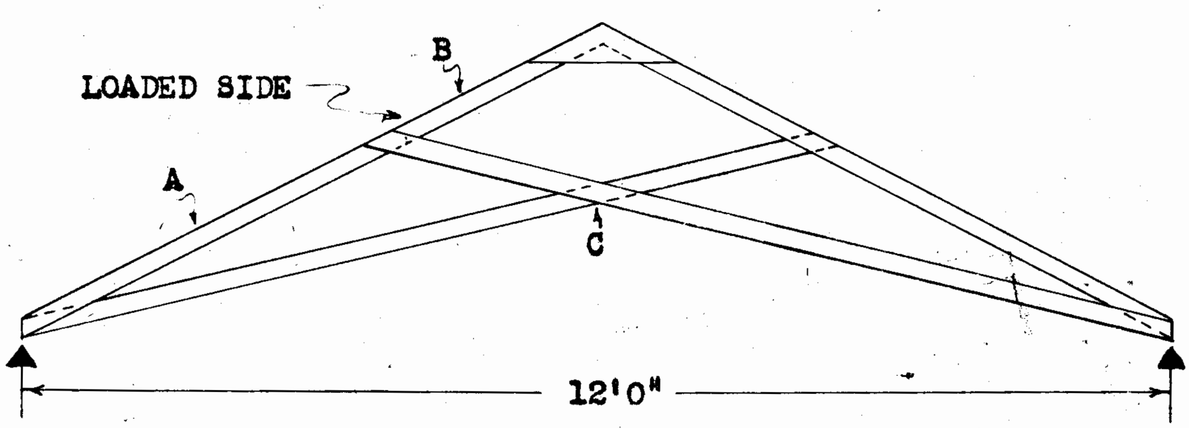
GRAPHIC SOLUTION



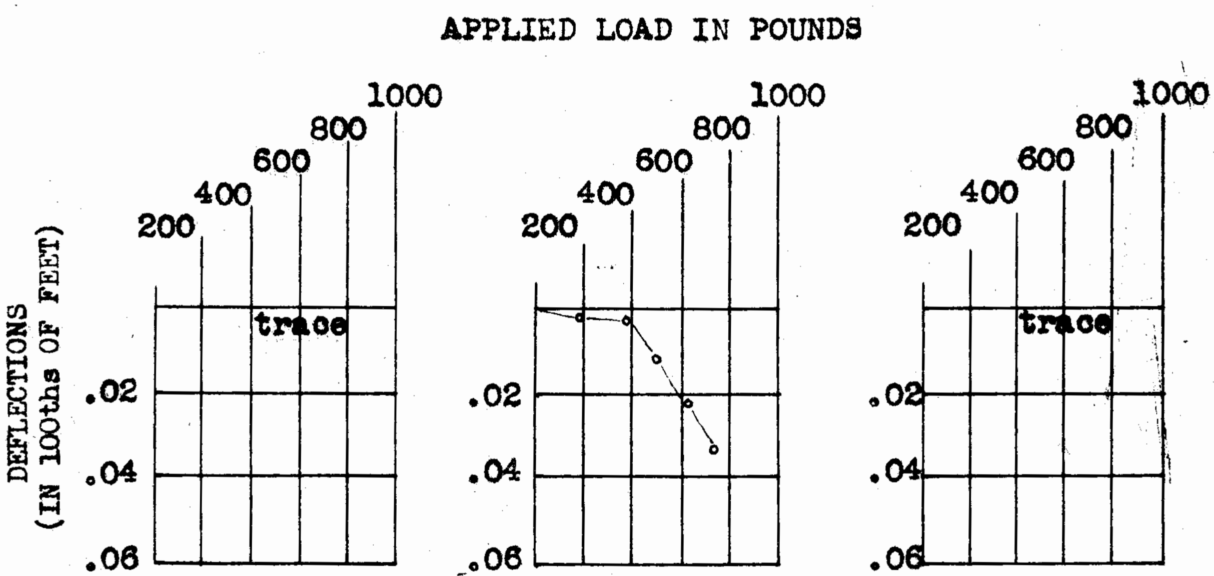
STRUCTURAL SOLUTION

DEFLECTION TEST
ROOF TRUSS, TEST # 132

SCISSORS
SPAN: 12'0"



WEIGHT OF TRUSS - 31#
HEIGHT OF TRUSS - 3'0"



AT "A"

DEFLECTIONS
AT "B"

AT "C"

MAXIMUM DEFLECTIONS UNDER UNIFORM LOAD OF 740#

AT "A" - trace
AT "B" - .033'
AT "C" - trace

DEFLECTION TESTS
ROOF TRUSS, TEST #133

SUBJECT:

Scissors truss to span 16', to be built of #1 yellow pine with plywood gussets.

PROCEDURE:

Basic Assumptions:

Weight of truss	42# approx.
Roofing load	5#/sq.ft.
Snow load	20#/sq.ft.
Wind load	Negative
Depth of truss (at center)	48"

CALCULATIONS:

See graphic solution, Fig.

In addition to these tension and compression loads, consider the bending moment and resulting stress in the upper chords of the truss.

Consider a 2" x 3", 5.4' long, for the upper chord; as a fixed beam

M = Moment # in.
W = Weight per foot of length
l = Length (ft.)

$$M = \frac{57.8 \times 5.4 \times 12}{12} = 1690 \text{ \# inches}$$

$$S_b = \frac{Mc}{I} = \frac{1690 \times 1.34}{2.64} = 860 \text{ \#/sq. in.}$$

$$S_T = 860 \quad \frac{1480}{4.37} = 1198 \text{ \#/sq.in.}$$

Use a 24" x 6" gusset at the ridge as determined by previous calculations.

Diagonals

Tentatively use a 1" x 2"

Area = 1.32 sq. in.

$$S = \frac{1370\#}{1.32} = 1040 \text{ \#/sq.in.}$$

However, to provide sufficient gluing and nailing area, employ 1" x 3" members for these diagonals.

DEFLECTION TEST
ROOF TRUSS, TEST #133

CALCULATIONS: Employed in connection with graphic solution

Basic Assumptions:

Weight of truss	42#
Roofing load	5 #/sq.ft.
Snow load	20 #/sq.ft.
Wind load	Negative
Depth of truss (at center)	48"

With trusses 2' on centers, top area carried per truss is:

$$\left[(8'8")2\right] \times 2 = 34.66 \text{ sq. ft.}$$

Total load per truss

$$\frac{5\#}{\text{Sq.ft.}} \times 34.66 \text{ sq. ft.} = 173.5$$

$$\frac{20\#}{\text{Sq.ft.}} \times 34.66 \text{ sq. ft.} = 693.0$$

$$\underline{42.0}$$

$$1008.5\#$$

$$\frac{1,008.5}{17.3} = 57.8 \ \#/\text{ft.}$$

$$\text{Panel (A)} = 5.417 \times 57.8 = 312\# = \text{Panel (D)}$$

$$\text{Panel (B)} = 3.25 \times 57.8 = 188 = \text{Panel (C)}$$

$$P_o = \frac{\text{Panel (A)}}{2} = 156\#$$

$$P_1 = \frac{\text{Panel (A)} + \text{Panel (B)}}{2} = 250\#$$

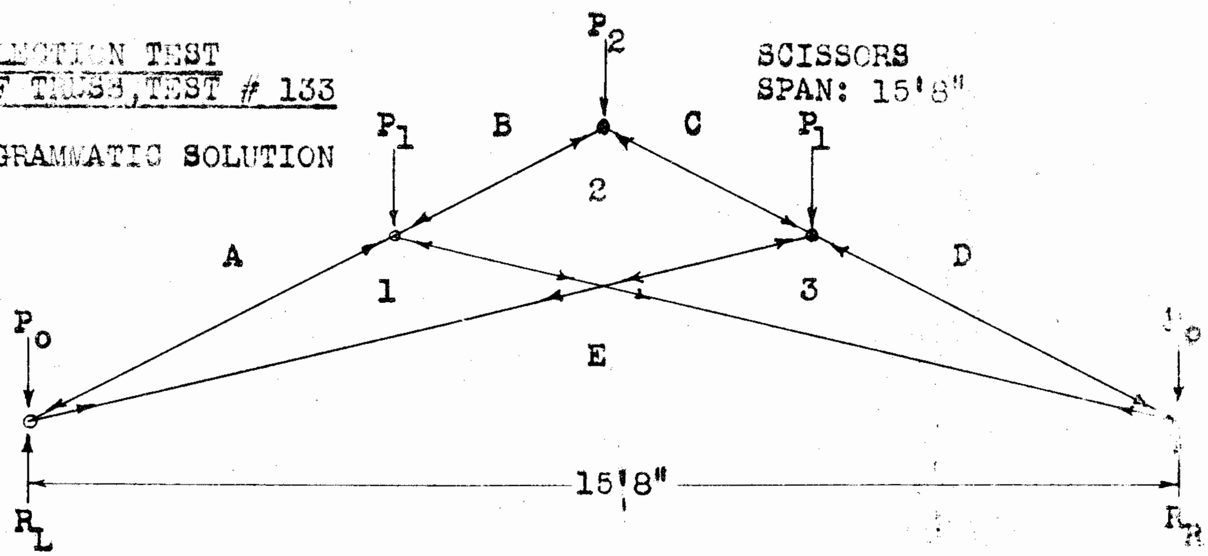
$$P_2 = \frac{\text{Panel (B)}}{1} = 188\#$$

$$R_1 - P_o = R_R - P_o = 500 - 156 = 344\#$$

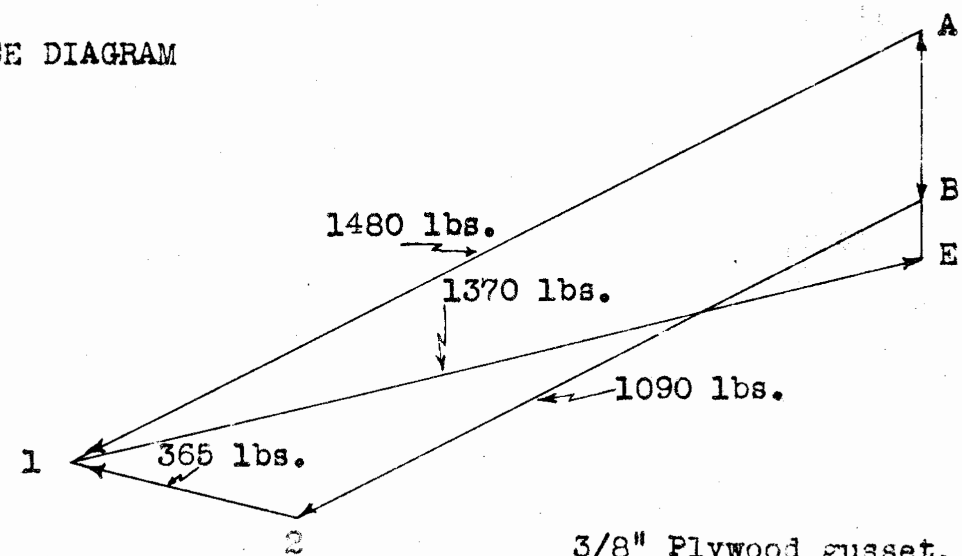
DEFLECTION TEST
ROOF TRUSS, TEST # 133

SCISSORS
SPAN: 15'8"

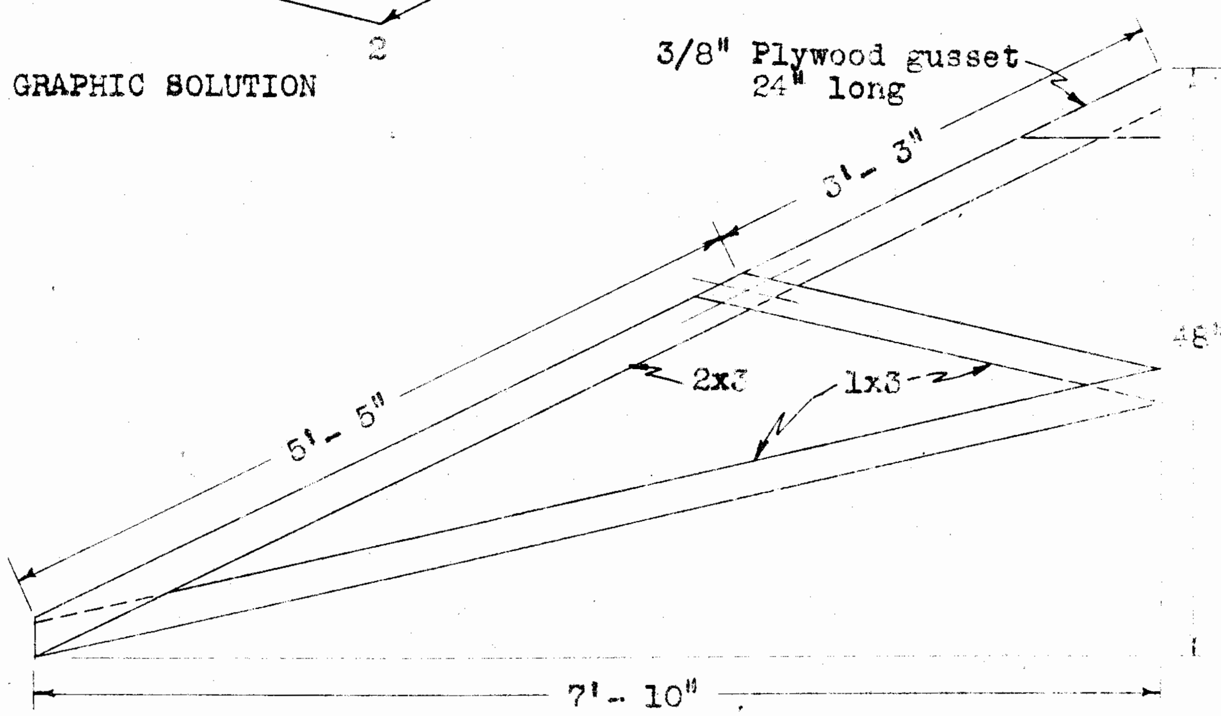
DIAGRAMMATIC SOLUTION



FORCE DIAGRAM



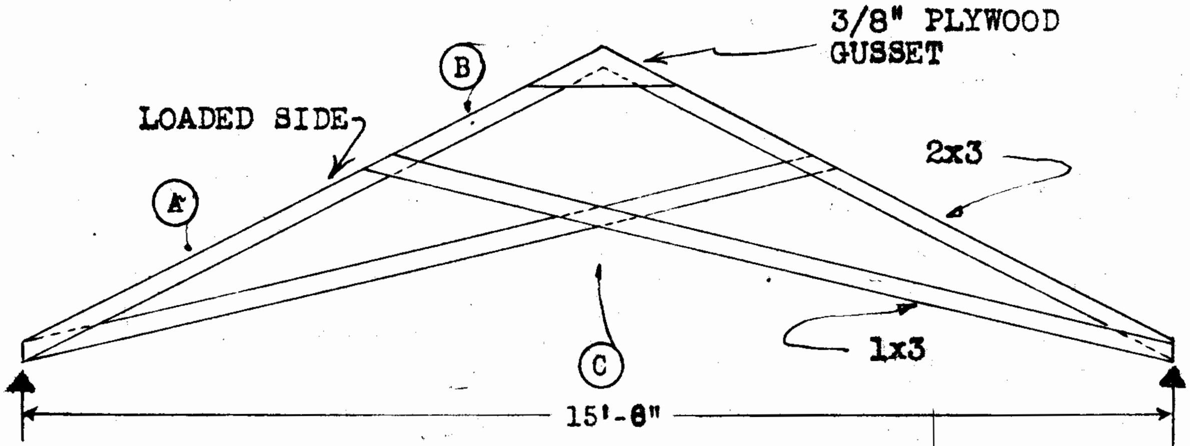
GRAPHIC SOLUTION



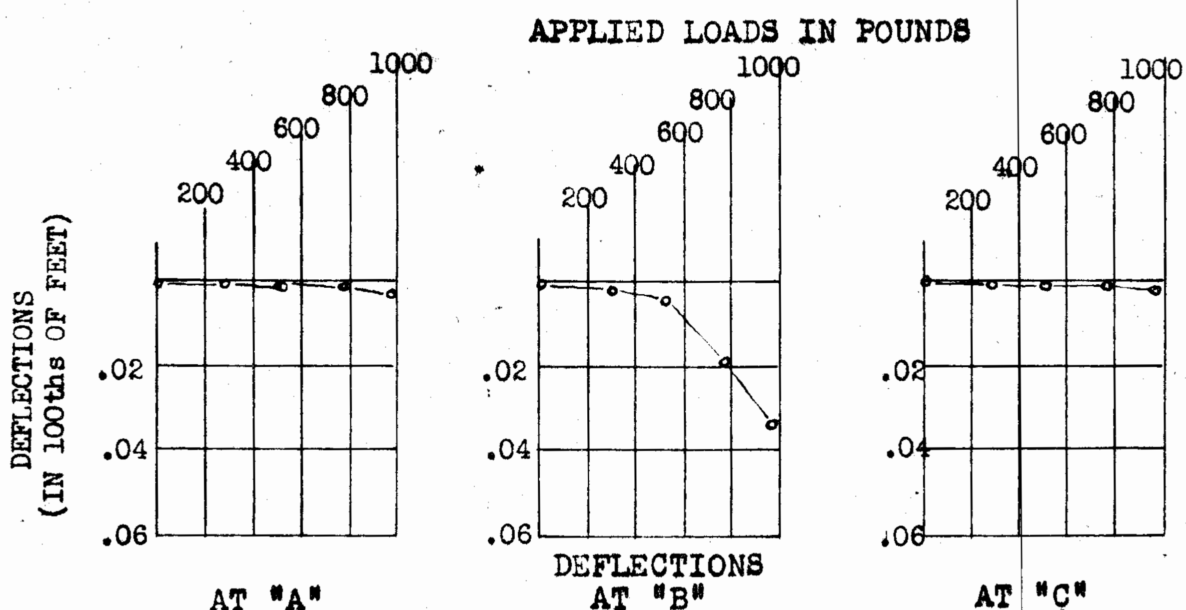
STRUCTURAL SOLUTION

DEFLECTION TEST
ROOF TRUSS TEST #133

SCISSORS
SPAN: 15'8"



WEIGHT OF TRUSS - 42#
HEIGHT OF TRUSS - 4'0"



MAXIMUM DEFLECTIONS UNDER UNIFORM LOAD OF 990#
AT "A"- .004'
AT "B"- .036'
AT "C"- .002'

DEFLECTION TESTS
ROOF TRUSS, TEST #134

CALCULATIONS

For graphic solution and data, see Fig., P. 58

Compute bending stress in member A-1. It is the critical member, being the most heavily loaded and one of the longest members in the truss. Consider this member as a fixed beam with uniform "A" load of 50 #/lineal foot.

M = Moment (bending) # inches
S = Unit stress #/sq. in.
I_x = Moment of inertia of member
c = Distance from neutral axis to outer fibers
W = Pounds per lineal foot
l = Length of member (feet)
A = Area of member (sq.in.)

$$M = \frac{Wl^2}{12} = \frac{50 \times 8 \times 12}{12} = \frac{3200}{12} \times 12$$

$$M = 3200 \text{ \# inches}$$

$$S = \frac{Mc}{I} \qquad \text{or} \qquad \frac{I}{c} = \frac{M}{S}$$

Assuming the use of a 2" x 3"

$$I_x = \frac{1.625 \times 2.6875}{12} = 2.63 \text{ in.}^4$$

$$A = 1.625 \times 2.6875 = 4.37 \text{ sq.in.}$$

$$S = \frac{3200 \times 1.343}{2.63} \quad \frac{1820}{4.37}$$

$$= 1635 + 416 = 2051 \text{ \#/sq.in.}$$

Assuming the use of a 2" x 4"

$$I_x = \frac{1.625 \times 3.625}{12} = 6.45 \text{ in.}^4$$

$$A = 1.625 \times 3.625 = 5.9 \text{ in.}^2$$

$$S = \frac{3200 \times 1.8125}{6.45} \quad \frac{1820}{5.9}$$

$$= 900 + 309 = 1209 \text{ \#/sq.in.}$$

Ceiling Members

Fixed beams - 96" long

A 1745# load in tension, plus the weight of the ceiling

Using a 2" x 3" in this position

$$I_x \text{ (for 2" x 3")} = 2.63 \text{ in.}^4$$

$$A \text{ (for 2" x 3")} = 4.37$$

$$M = \frac{Wl^2}{12} = \frac{2 \times 8 \times 12}{12} = 128 \text{ \# inches}$$

$$S = \frac{128 \times 1.343}{2.63} + \frac{1745}{4.37}$$

$$= 65 + 400 = 465 \text{ \#/sq.in.}$$

Next to be considered are members 1 - 2 and 2 - 3

Total compressive load in 1 - 2 is 300#

Using a 2" x 2"

$$I_x = \frac{1.625 \times 1.625}{12} = .584$$

$$A = 2.64 \text{ sq. in.}$$

$$S = \frac{P}{A} = \frac{300}{2.64} = 135 \text{ \#/sq.in.}$$

Total load in tension in 2 - 3 is 375#

Use 2 members in this position, one on each side of truss.

Try 1" x 2" members

$$A = 2.64 \text{ sq. in.}$$

$$S = \frac{P}{A} = \frac{375}{2.64} = 142 \text{ \#/sq.in.}$$

Considering the small load carried, eliminate one member and disregard the eccentricity of the load

Employing one 1" x 2"

$$\text{Area} = \frac{13}{16} \times 1\text{-}5/8 = 1.32 \text{ sq. in.}$$

$$S = \frac{P}{A} = \frac{375}{1.32} = 284\ \#/\text{sq.in.}$$

In determining gusset sizes three considerations were made: necessary glue area, nailing area, and shear area.

Using United States Plywood Corporation data on glued joints, safe value for the stress was determined

$$\text{Ultimate Stress} = \frac{1000\#}{\text{Sq.in.}}$$

That is, 1000# per square inch of contact.

Use a value of 250 #/sq.in. as a working value.

The force of 1870# requires

$$1870\# \times \frac{\text{sq.in.}}{250\#} = 7.5\ \text{sq.in.}$$

This does not provide sufficient nailing area.

Actually the shearing stress governs in this case as Plywood data indicates 180 #/sq.in. allowable shearing stress.

Using two 3/8" gussets must produce

$$1870\# \times \frac{\text{Sq.in.}}{180\#} = 10.4\ \text{Sq.in.}$$

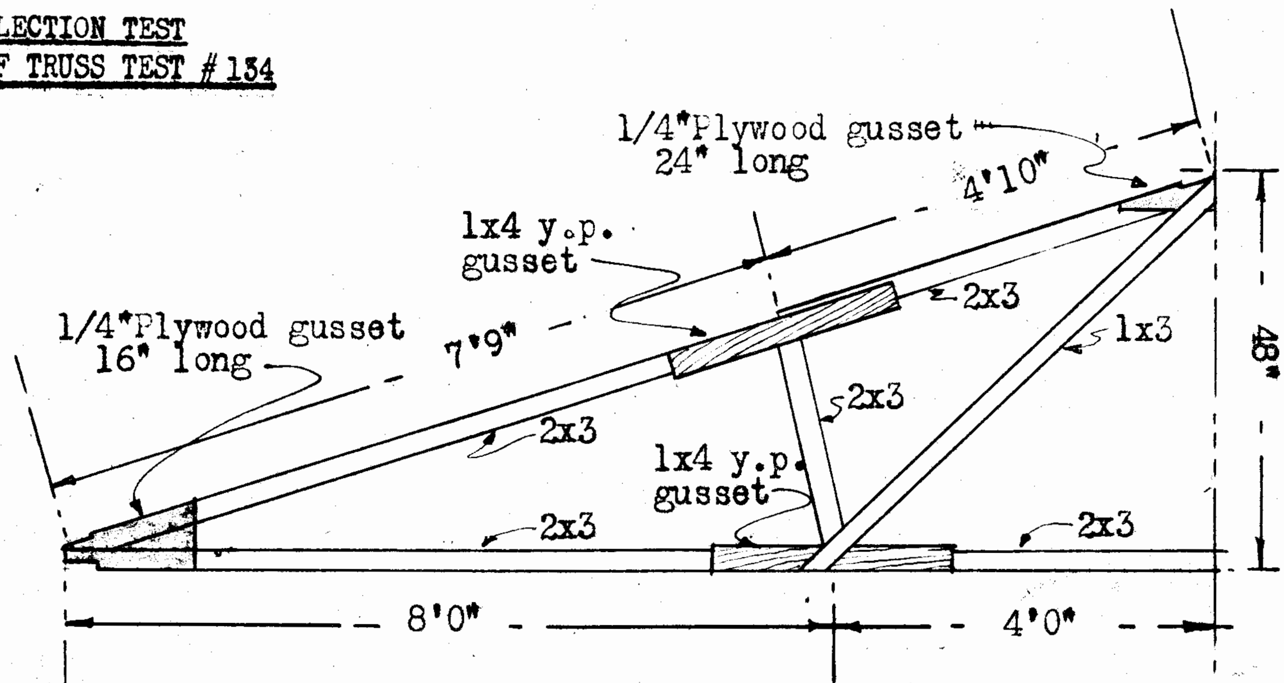
$$10.4\ \text{Sq.in.} = 3/4\ l$$

$$l = \frac{4}{3} \times 10.4 = 13.9"$$

Use two 15" gussets at eaves and a pair of 24" gussets at ridge.

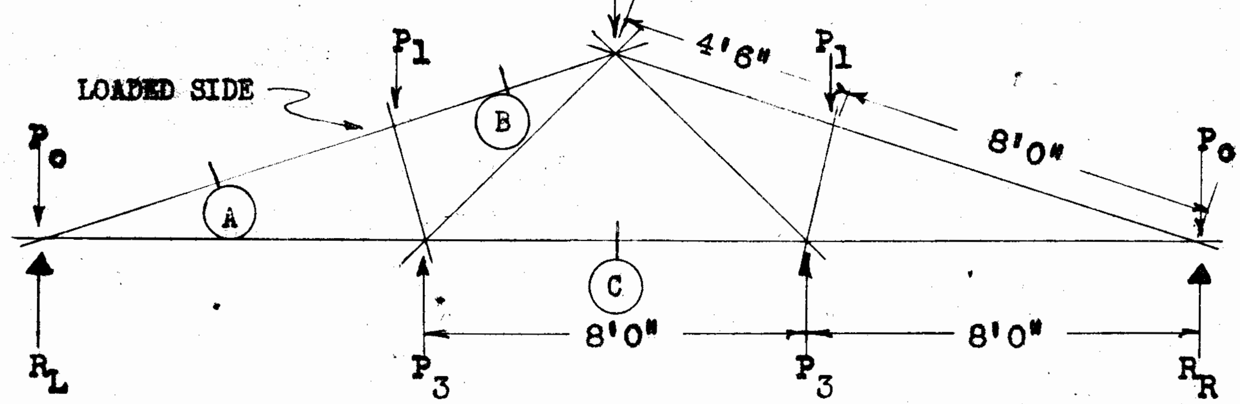
As drawing indicates, 1" x 4" members, 24" long, were used as gussets at two joints along ceiling joist.

DEFLECTION TEST
ROOF TRUSS TEST # 134

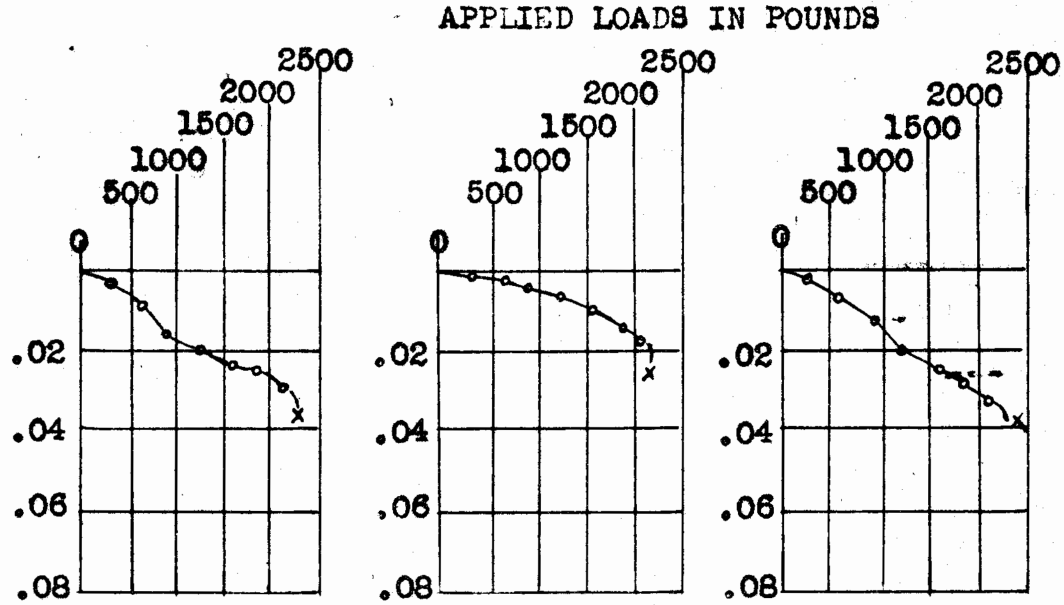


MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 2090#
AT "A" - .029'
AT "B" - .018'
AT "C" - .032'

LENGTH OF TRUSS 24'0"
HEIGHT OF TRUSS 4'0"
WEIGHT 112#



APPLIED LOADS IN POUNDS



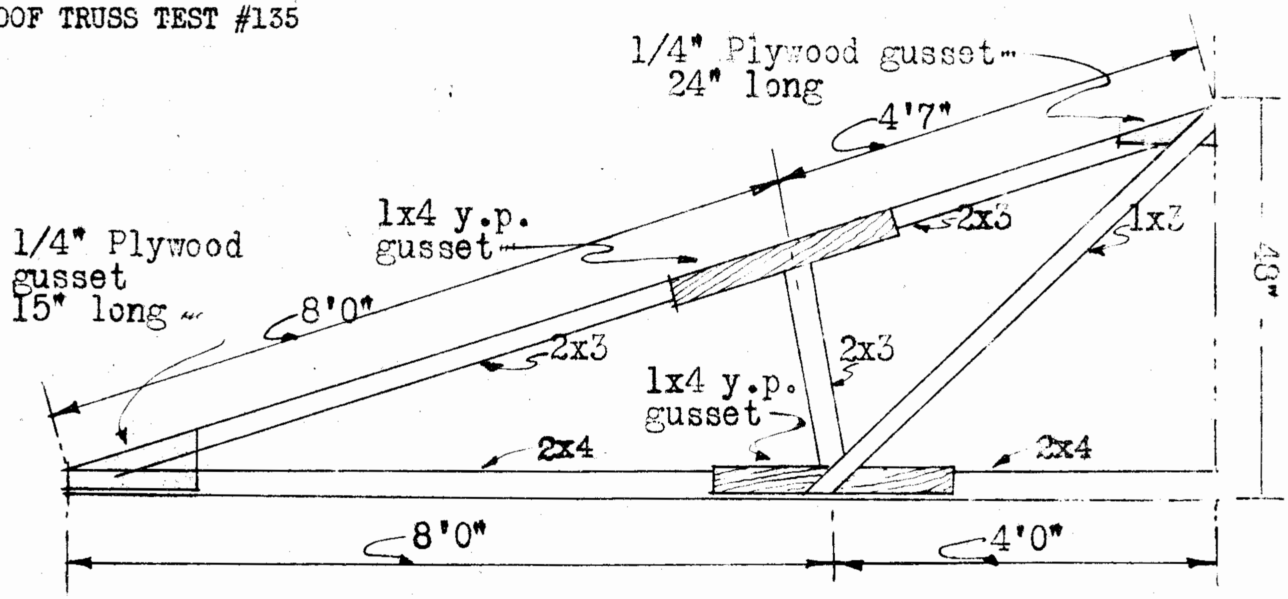
AT "A"

DEFLECTIONS AT "B"

AT "C"

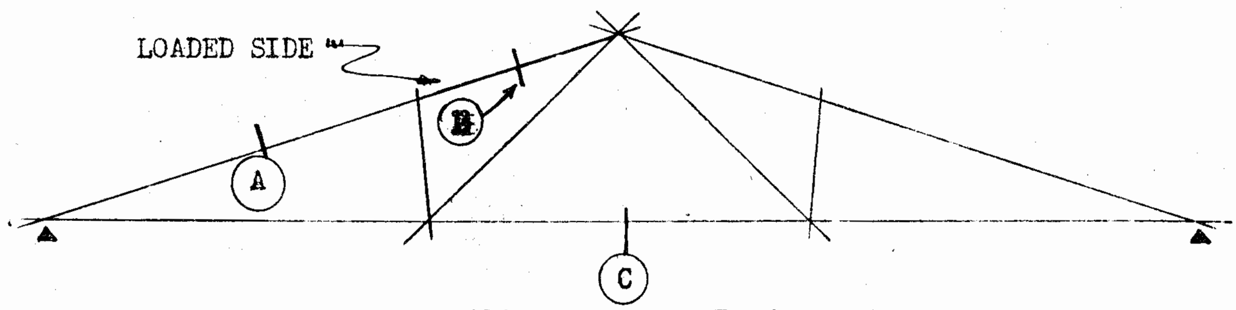
DEFLECTION TEST

ROOF TRUSS TEST #135

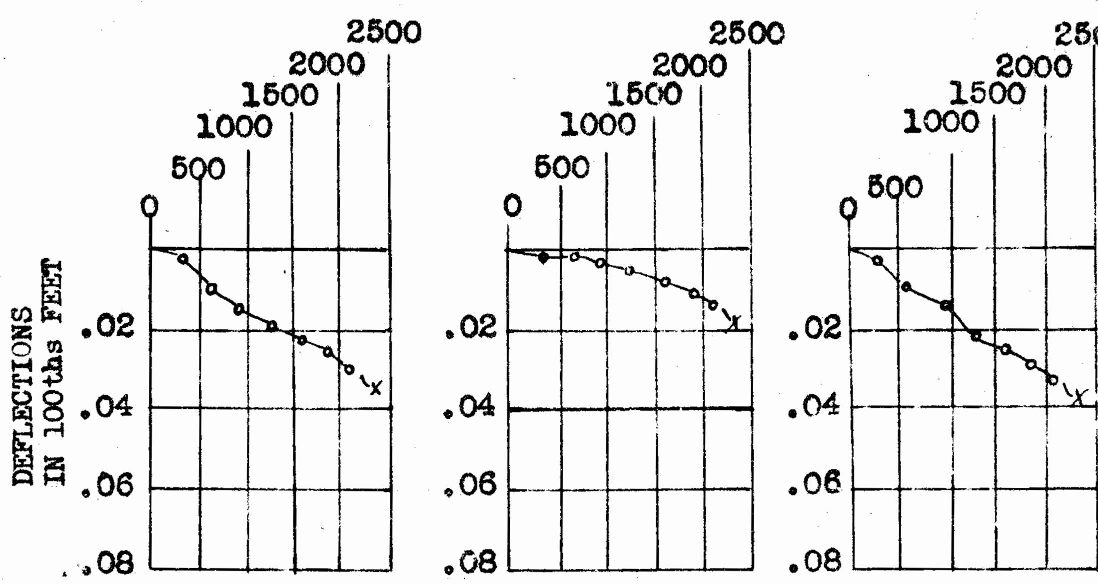


LENGTH OF TRUSS - 24'0"
HEIGHT OF TRUSS - 4'0"
WEIGHT 121#

MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 2100#
AT "A" - .029'
AT "B" - .013'
AT "C" - .033'



APPLIED LOAD IN POUNDS



DEFLECTIONS

AT "A" AT "B" AT "C"

DEFLECTION TEST
ROOF TRUSS, TEST #136

CALCULATIONS: Employed in connection with graphic solution

Basic Assumptions:

Weight of truss	124# (approx.)
Roofing load	5 #/sq.ft.
Ceiling load	2 #/sq.ft.
Snow load	20 #/sq.ft.
Wind load	Negative

Trusses to be placed on 2' centers.

$$\text{Roof load} = (25 \times 2)(5 + 20) = 1250\#$$

$$\text{Ceiling load} = (24 \times 2)(2) = 96\ \#$$

$$\text{Panel (A)} = \frac{8}{25} \times 1250 = 400\# = \text{Panel (D)}$$

$$\text{Panel (B)} = \frac{4.5}{25} \times 1250 = 225\# = \text{Panel (C)}$$

$$\text{Panel (E)} = \frac{8}{24} \times 96 = 32\# = \text{Panel (F) \& (G)}$$

$$P_o = \frac{\text{Panel (A)}}{2} = 200\ \#$$

$$P_1 = \frac{\text{Panel (A)} + \text{Panel (B)}}{2} = \frac{400 + 225}{2} = 313\#$$

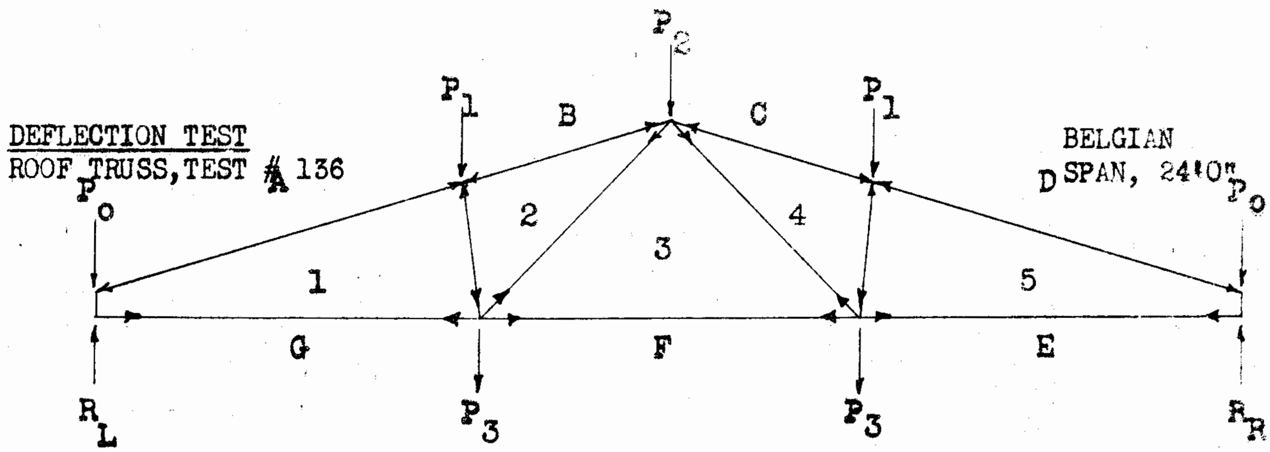
$$P_2 = \text{Panel (B)} = 225\#$$

$$P_3 = \text{Panel (E)} = 32\#$$

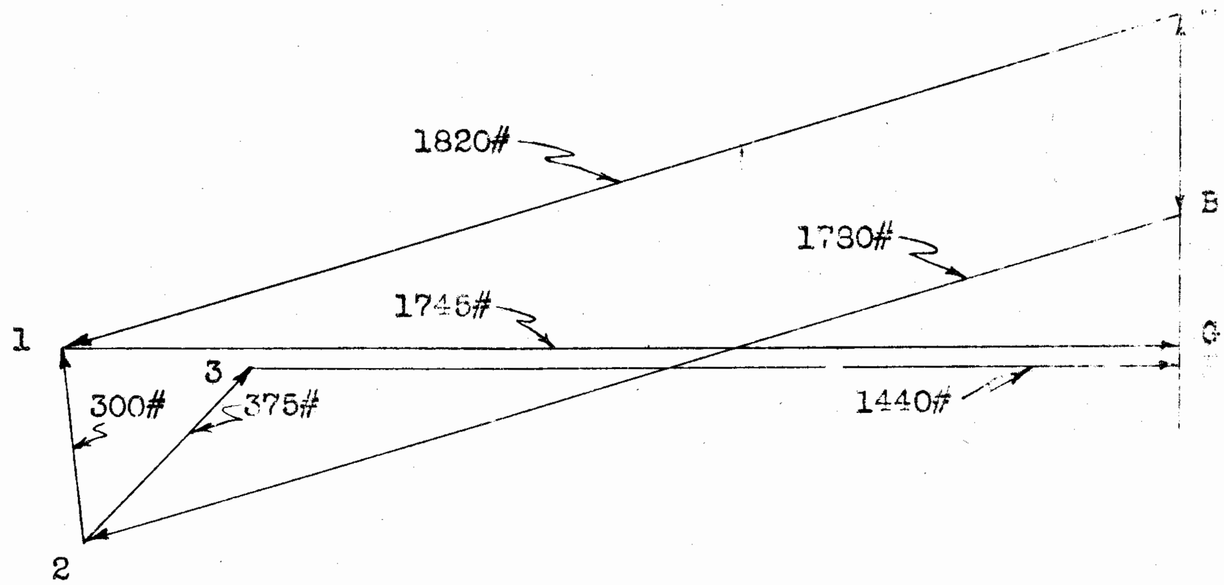
$$R_1 = R_R = \frac{1250 + 96 + 124}{2} = \frac{1470}{2} = 735\#$$

Vertical Components

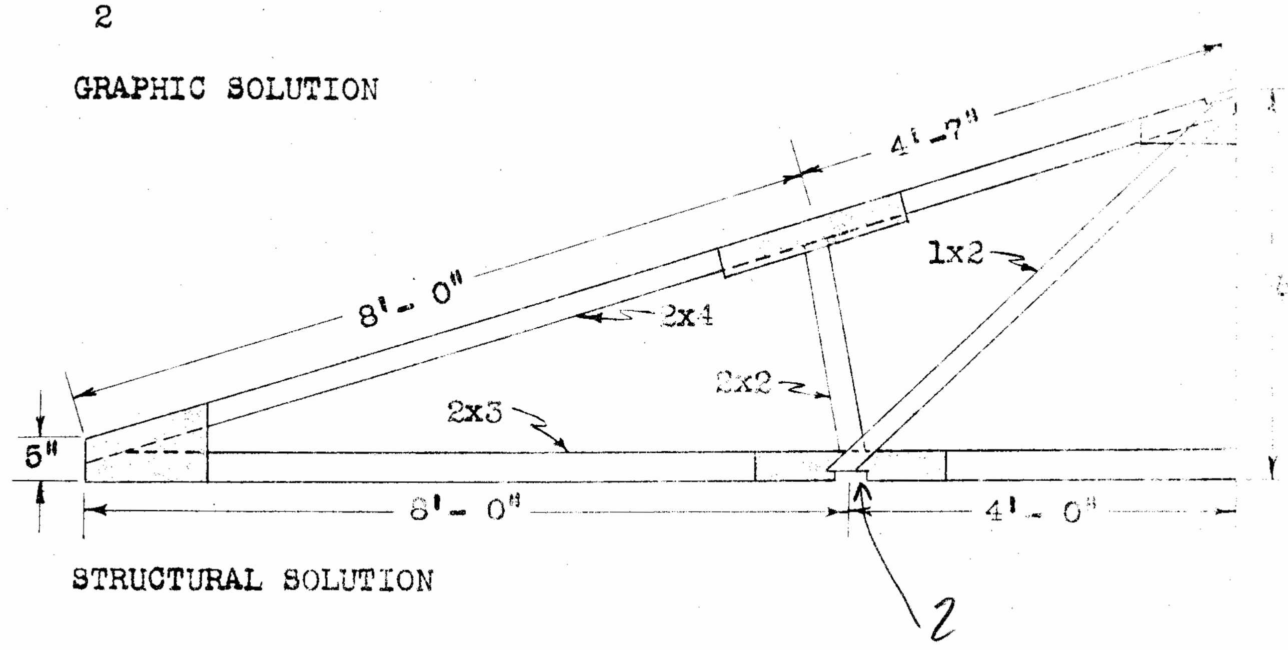
$$R_1 - P_o - \frac{P_3}{2} = 735 - 200 - 16 = 519\#$$



FORCE DIAGRAM



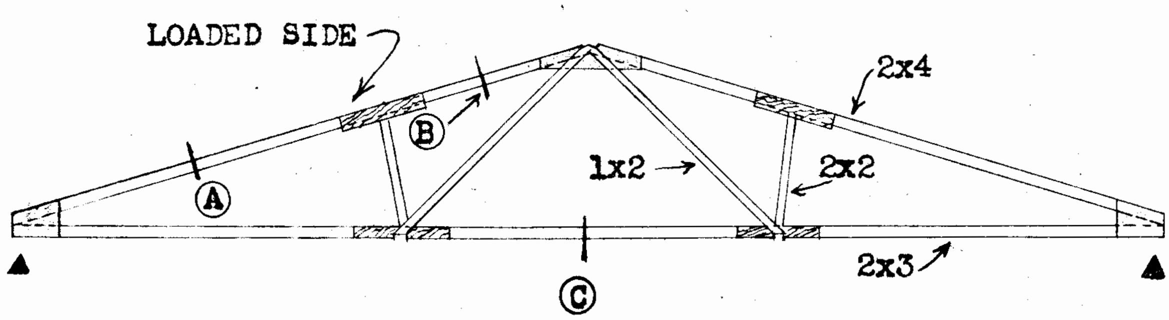
GRAPHIC SOLUTION



STRUCTURAL SOLUTION

DEFLECTION TEST
ROOF TRUSS TEST #136

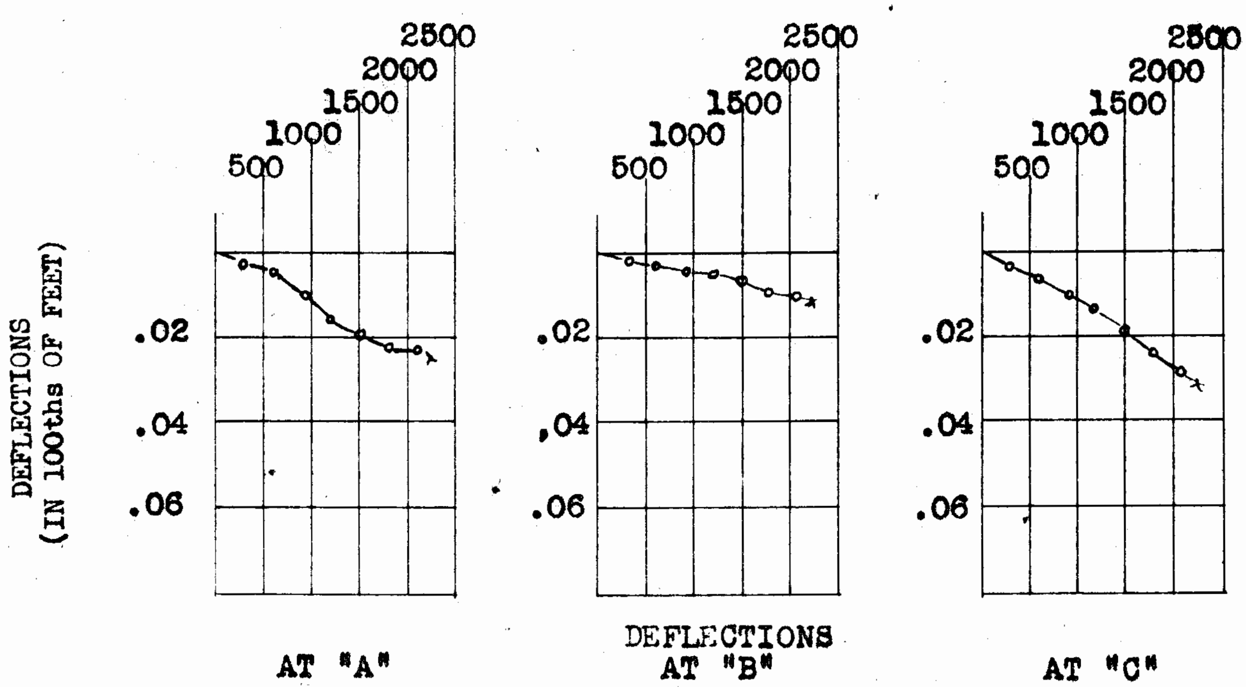
BELGIAN



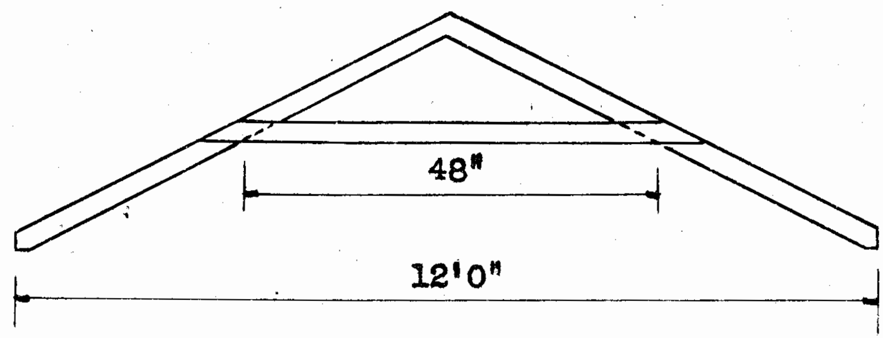
LENGTH OF TRUSS - 24'0"
HEIGHT OF TRUSS - 4'0"
WEIGHT 98#

MAXIMUM DEFLECTIONS FROM UNIFORM LOAD OF 2094#
AT "A" - .022'
AT "B" - .010'
AT "C" - .028'

LOADS IN POUNDS



ROOF TRUSS # 137 "A" FRAME



WEIGHT OF TRUSS - 30#
HEIGHT OF TRUSS - 39"

This truss was not tested in the manner employed in previous trusses as the span was nominal and the solution was obvious. Components were 2" x 4", the horizontal member being secured in place by means of a 1" x 4" nailed on the faces of this member and the two rafter components.

Where the truss was exposed, the sub-roof providing a finished ceiling, it was possible to omit the horizontal 2" x 4" and employ two 1" x 4", one on each side.

ROOF TRUSSES

SUMMARY

In the design of light-weight, short-span roof trusses there is a tendency to construct components exceeding the requirements of span and load. This over-designing invariably results in trusses which are difficult to transport and handle. This is particularly so of the solid-web trusses. These offer but little structural advantage and contribute considerable to the dead weight. This weight in turn must be transferred to the walls, with resultant undue weight carried down into the footings.

In transporting the solid-web truss its handling is awkward, and securing it on a truck is difficult as there is small opportunity for tying. Since the wood members employed are light, cleats nailed across a series of trusses to retain them during transportation tend to break these light flanges. At the site, the moving into position of such a truss is difficult as there is a tendency for its apex to want to tip. Consequently it is necessary to provide some means of balancing until the component is secured in position on the wall. This offers a problem in the solid-web truss as there is little surface to which a brace or batten can be secured or against which a board or stud can be placed for the steadying of the component while raising it into position.

The solid-web truss retards ventilation through the attic space, the webs segregating each area and preventing any movement of air except by means of some type of eaves ventilator. Tests have indicated this means of ventilation to be inadequate unless there is some relief in the ridge or gable ends.

Carefully engineered and constructed light-weight open web trusses falling into the general category of "A" frames, scissors, and Belgian can provide sufficient roof and ceiling support without too great a weight introduced into the truss.

The Belgian trusses reviewed in the series of tests shown here were particularly successful. Plywood gussets glued into position produced a rigid unit which was easily loaded and could be tied and secured for transporting without any great effort or the use of cleats or supplementary nailing.

Twelve-foot and sixteen-foot trusses were conveniently conveyed on a pick-up truck. The longer trusses required brackets to be built on the bumper, in the space between cab and truck body, and at the rear of the truck body in order to avoid distortion and deflections resulting from torque.

There has been a tendency to either shop fabricate the entire roof truss and apply the ceiling and roof material after erection or to pre-assemble ceilings and roofs, assuming normal values of 2" dimension lumber to provide ceiling joist and roof rafters. (This) disregards the fact that a modest amount of presupporting of these ceiling and roof rafters prior to

the former ?

their being secured as trusses would eliminate a great deal of unnecessary weight and handling ordinarily found where the more orthodox components are employed. It is entirely possible to design these trusses in such a manner that the components supporting the ceiling and those supporting the roof may be assembled with the ceiling material and the roof decking in place, the actual assembly of the trusses taking place after these members have been put into permanent position.

Where trusses were shop-assembled gluing, nailing, and the employing of plywood gussets resulted in a sturdy, stable unit capable of standing much abuse in transporting and handling.

Where light-weight trusses are employed, it is advisable to introduce a piece nailed flatwise on the underside of the lower chord and to allow slots in this piece to receive similar strips running across the trusses which act as spacers and nailers for ceiling materials. This not only eliminates the use of fitted bridging, but provides nailing for whatever type of ceiling might be employed without the danger of these ceiling nails splitting the structural members in the lower chord of the truss.

The observation is again made that if the design of these trusses provides joints at intervals corresponding to the sizes of the ceiling and roof sheathing materials to be used, it is possible to bring about this joining after erection has occurred.

The use of bolts and mechanical fasteners should be considered in connection with light trusses. These prove very effective in heavier work, and there is reason to believe that field joining of light truss components could benefit from such joints.

Whether a truss is assembled in the shop or after actual erection, it is imperative that the joints as designed by the engineer are reproduced in each of the final trusses. If certain gluing area is required, then supervision should be such that it is certain each truss has received the benefit of that glued area. If the design specifies certain size nails placed in certain positions, then it must be ascertained that the final nailing produces the results required. If these few simple operations are respected during the design and construction of intregal roof and ceiling load-bearing components, a light-weight, low-cost and simply-erected unit possessing the load-bearing properties required will be the result.

SECTION IV

WALLS

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana
March 1, 1944

DEFLECTION TESTS
WALLS
PROCEDURE:

The type of wall requiring considerable study and research was that in which large panels of material were to be employed as structural walls and retained in place with battens. For the purpose of identifying this type of wall, we have employed the term "sandwich". In an analysis of these walls three types were taken into consideration. The first, a three-column composite, consisted of three 1" x 4" battens and two sheets, or web members, 48" x 96" of the material being tested. These were so assembled that one set of battens was exposed to each side and the third set was between the two sheets of material. This unit was designed to take insulation between the two sheets of wall material. The data describing the method of calculating these walls will be found under "Deflection Tests, Walls, Three-Column Composite."

The next group was the two-column composite in which two 48" x 96" wall materials were placed together and these retained in position with 1" x 4" battens on the outside and the inside, there being no space between components. This calculation has been fully described under "Deflection Tests, Walls, Two-Column Composite."

Finally, a wall was analyzed in which there was but one solid wall panel retained in place by 1" x 4" on both sides and described as a two-column single panel. The factors employed in computing this panel were the same as those employed for the two-column composite.

TEST #138 - 141, INC.

These sandwich panels were built with insulation between panels (Tests #138 and 140) and with the insulation omitted (Tests #139 and 141). The insulation value within these panels will not be discussed at this time. It can be said the omission of this small amount of insulation did not appreciably affect the "k" factor.

Structurally, there were such slight differences in values in the two panels it was not believed necessary to prepare graphs for the two different types. In running these load tests, deflections were recorded on a basis of total load applied across the top of the panel, but the actual load was broken up into three relatively equal loads centered at points A, B, and C.

In the three-column composite the initial weight of the wall section totalled 162 pounds where gypsum was employed, 169 pounds using asbestos board. This produced much too heavy a wall and even in these initial test panels it became apparent that an excessive amount of material was being employed and assembly would be expensive. However, to attempt to determine values in these walls, for which there was no engineering precedent or calculations available, tests were conducted with the results shown on the respective graphs.

The total load of one and one-half kips on these panels indicated an excessive construction for the loads which a wall employing this construction would be expected to carry. The cracking of the panel in Test #141 indicated panel members were contributing something to the strength of the panel, but the contribution was not enough to justify the added weight and amount of material necessary to retain these panels in position. The adjustment of the graph indicating the deflection at B', the concentrated load at the center of the panel, can be understood. As the loads caused deflection at A' and C', the beam moved down, its own deflection not necessarily affected by this loading. As the load became greater it was necessary to observe not only this displacement of the beam due to the loadings over the columns, but to also determine the deflection that was actually occurring in the beam itself.

TEST #142 - 143

The calculations for the two-column composite are a refinement of the three-column and offered a wall in which two different materials were combined in the panel with two sets of battens retaining this material in position. These walls showed good characteristics and were employed; Test #142 in Unit B-13, Test #143 in Unit B-15.

Weight continued to be the factor in all of these components.

TEST #144

The final of these sandwich type walls was one in which the panel consisted of a 25/32" asphalt-coated fiber board having an outer surface treated with roofing granules. This resulted in a panel having a weight of approximately 20 pounds. It employed, besides this single thickness of material, the two sets of battens in the same manner as these were employed in the two-column composite. This type of panel was used in Unit B-22.

In all of these wall studies it was necessary to consider the size panel under test load as being less effective than a similar panel combined in a wall, the test panel not having the added stiffness resulting from adjacent companion panels. Primarily these tests needed to be incorporated into actual structures to determine exactly what could be expected when actual roof loads were applied, window and door openings cut, and irregular deflections encountered which can occur from unbalanced loading or foundation irregularities.

The graphs showing the action of these various types of panels in their respective structures clearly indicate the results of their employment.

TEST #145

Stud walls required no particular analysis. Single top and bottom plates were employed with stud spacings 24" on center. Double studding was omitted at window and door openings with the exception that openings exceeding 4' in width received doubled 2" x 6" lintels and double studding at the limits of the lintels. Single 2" x 4" headers installed flatwise were employed at the lintel and sill lines of other window and door openings. Corners were framed in any one of the recognized three-stud assembly methods of corner framing. No wind bracing was installed. Walls of this nature received wallboard sheathing on the outside covered with an additional sheet of wallboard as a finish wall. Space between studs received insulation. The inside walls were covered with wallboard.

As this report does not deal with the phases of fabrication, further discussion as to the manner of preparing the drawings and assembling these walls will not be made at this time.

DEFLECTION TESTS
WALLS

THREE COLUMN COMPOSITE

SUBJECT:

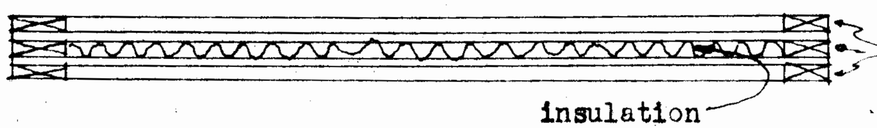
Walls 8' high built with 4' x 8' wall panels utilizing 1" x 4" battens as columns.

PROCEDURE:

Basic Assumptions:

Inasmuch as no criteria were available, first efforts were a series of cuts and tries.

The initial wall was to be so constructed as to leave a space for insulation.



Calculations:

In order to calculate the allowable load on each composite column, some assumptions were made:

To consider the 1" x 4" 's individually;

To consider them as one column three 1" x 4" 's deep;

To consider the wall panels as equivalent to a 1" x 4" wide and use their nominal thickness deep.

The stresses were computed in the columns, using the three basic assumptions.

1" x 4" 's separately

L = 8 x 12 - 7 = 89"
d = 13/16
E = Modulus of elasticity = 1,600,000
A = Area
D = Load

Basically, using Euler's formula (for long columns), the equation is

$$\frac{P}{A} = \frac{E}{2.5(\frac{l}{K})^2} \qquad \text{But } K = \frac{I}{A}$$

$$\frac{I}{A} = \frac{\frac{bh^3}{12}}{bh} = \frac{h^2}{12} \qquad K = \frac{h}{12} \qquad 2.5 = \text{Factor of safety}$$

$$\frac{P}{A} = \frac{(3.1416)^2 \times 1,600,000}{\left(\frac{12\ l}{d}\right)^2} = \frac{.329E}{\left(\frac{l}{d}\right)^2}$$

To try for ultimate, however,

$$\frac{P}{A} = \frac{.823E}{\left(\frac{l}{d}\right)^2}$$

Then, taking 1" x 4" 's as separate members

$$\frac{P}{A} = \frac{.823 \times 1,600,000}{\left(\frac{89}{\frac{13}{16}}\right)^2} = 109.5\ \#/\text{sq.in.}$$

$$A = 3(3\text{-}5/8 \times \tfrac{13}{16}) = 8.85 \text{ sq. in.}$$

$$P = 8.85 \times 109.5 = 970\ \#\text{ per column.}$$

Taking three 1" x 4" 's as a single unit

$$\frac{P}{A} = \frac{.823 \times 1,600,000}{\left(\frac{89}{3 \times \frac{13}{16}}\right)^2} = \frac{.823 \times 1,600,000}{(36.5)^2}$$

$$\frac{P}{A} = 395\ \#/\text{Sq. in.}$$

$$A = 3(3\text{-}5/8 \times \tfrac{13}{16}) = 8.85 \text{ sq. in.}$$

$$P = 395 \times 8.85 = 3500\ \#$$

Assuming the wall panels to represent pieces equal to their nominal thickness and equal to a 1" x 4" in width

$$\frac{P}{A} = \frac{.823 \times 1,600,000}{\left(\frac{89}{3\text{-}3/16}\right)^2} = 1690\ \#/\text{sq. in.}$$

$$A = 3(3\text{-}5/8 \times \tfrac{13}{16}) + 2(3\text{-}5/8 \times 3/8)$$

$$= 8.89 + 2.72 = 11.61$$

$$P = 11.61 \times 1690 = 19,650\ \#$$

The first set of calculations is obviously too low; the last set is too high. Use the second set.

Inasmuch as roof load is equal to 735 # per truss end, and in view of the fact there is one column every 4', each column will carry the equivalent of two truss ends, or 1470 #.

This allows a factor of safety of 2.38, which is adequate.

In using these assumptions, no strength has been attributed to the wall panel material. It is known from tests that the panels do contribute to the strength of the wall.

The bending moment in the upper plate of the wall is another consideration. The maximum moment occurs at the support and is

$$M = \frac{Pl}{8}$$

$$M = \frac{735\# \times 4 \times 12}{8} = 4410 \text{ \# inches}$$

Where
- S = Unit stress (#/sq.in.)
- M = Moment (# inches)
- c = Distance from neutral axis to outer fibers
- I = Moment of inertia $(\text{in.})^4$

$$I = \frac{bh^3}{12} = \frac{3\frac{13}{16} \times (3\text{-}5/8)^3}{12} = \frac{3 \times 13 \times (3.625)^3}{16 \times 12}$$

$$I = 9.65 \text{ in.}^4$$

$$S = \frac{4410 \times 1.8125}{9.65} = 830 \text{ \#/sq.in.}$$

DEFLECTION TESTS
WALLS

TWO COLUMN COMPOSITE

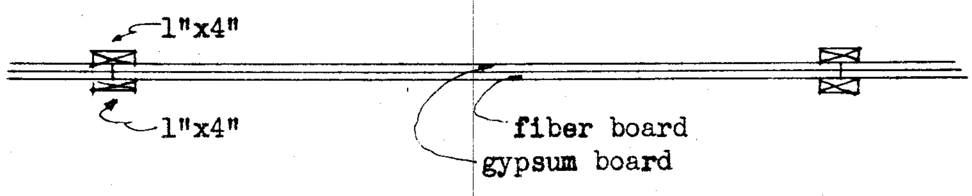
SUBJECT:

Walls 8' high, built with 4' x 8' wall panels utilizing 1" x 4" battens as columns.

PROCEDURE:

Basic Assumptions:

Inasmuch as the initial walls were too heavy, contained too many pieces, and were stronger than necessary, the next attempt at this type of wall used rigid wall board omitting the space for loose insulation. This eliminated one of the three sets of 1" x 4" 's.



Calculations:

Calculations follow the same pattern as in the first wall. Use Euler's formula, leaving out the factor of safety; where

P = Load (pounds)
A = Area (sq. in.)
E = Modulus of elasticity 1,600,000
l = Length of column
d = Least dimension (cross section)

$$\frac{P}{A} = \frac{.823E}{(\frac{l}{d})^2}$$

1" x 4" 's separately

$$\frac{P}{A} = \frac{.823 \times 1,600,000}{\left(\frac{89}{\frac{13}{10}}\right)^2} = 109.5$$

$$A = 2 \times (\frac{13}{16} \times 3\text{-}5/8) = 5.89 \text{ sq. in.}$$

$$P = 5.89 \times 109.5 = 645 \#$$

1" x 4" 's as a single unit

$$\frac{P}{A} = \frac{.823 \times 1{,}600{,}000}{\left(\frac{89}{1.625}\right)^2} = 440$$

$$A = 2\left(\frac{13}{16} \times 3\text{-}5/8\right) = 5.89 \text{ sq. in.}$$

$$P = 5.89 \times 440 = 2590$$

1" x 4" 's plus wall board thickness — assuming wall board to be equivalent to a 1" x 4" wide and its nominal thickness deep,

$$\frac{P}{A} = \frac{.823 \times 1{,}600{,}000}{\left(\frac{89}{2.6}\right)^2} = 1125$$

$$A = 2.6 \times 3.625 = 9.4$$

$$P = 9.4 \times 1125 = 10{,}600 \ \#$$

As in the first wall computations, the first calculations are too low. Discard the third set and use the second set, resulting in a factor of safety of

$$F.S. = \frac{2590}{1470} = 1.76.$$

This may be slightly low, but again no strength was attributed to the wall board although tests indicated it does contribute strength to the wall section.

The bending stress in the upper plate is:

- M = Bending moment
- P = Load (pounds)
- L = Length of span (ft.)

$$M = \frac{PL}{8} = \frac{735 \times 4 \times 12}{8}$$

$$M = 4410 \ \# \text{ inches}$$

Where

- S = Unit stress (#/sq.in.)
- M = Moment (# inches)
- c = Distance from neutral axis to outer fibers (inches)
- I = Moment of inertia (inches)^4

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

$$I = \frac{bh^3}{12} = \frac{2\frac{13}{16} \times (3\text{-}5/8)^3}{12}$$

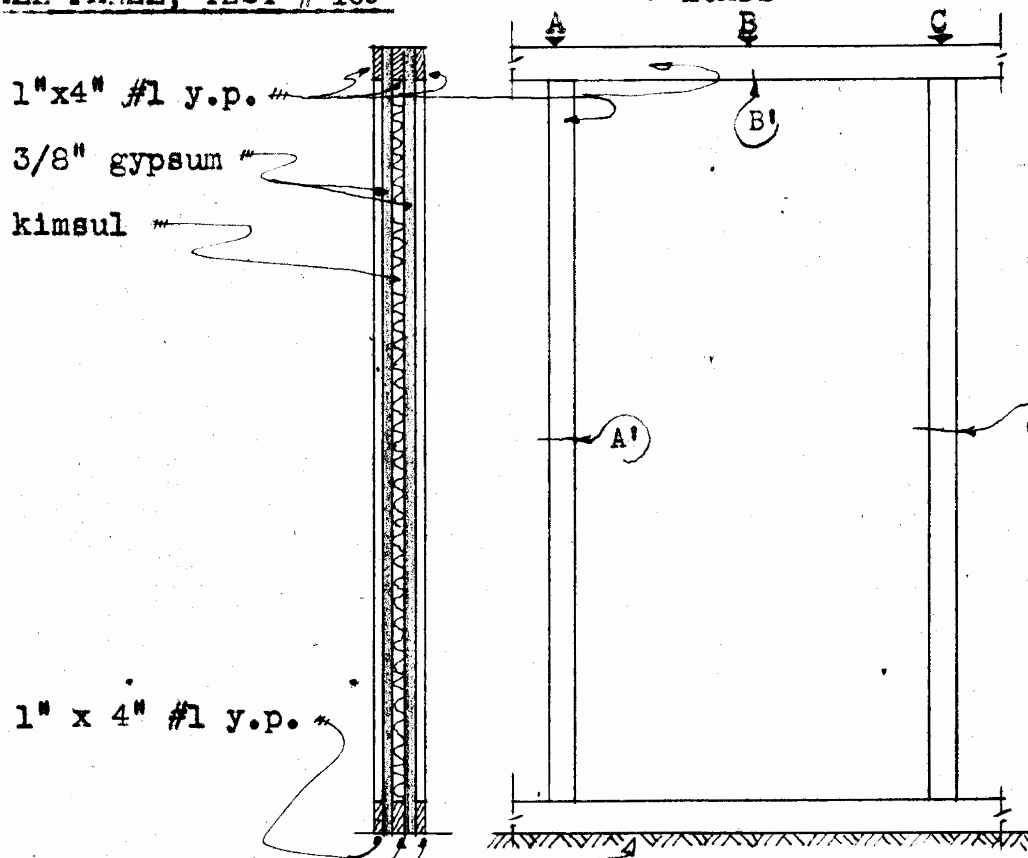
$$I = 6.45$$

$$S = \frac{4410 \times 1.8125}{6.45} = 1240 \text{ \#/sq.in.}$$

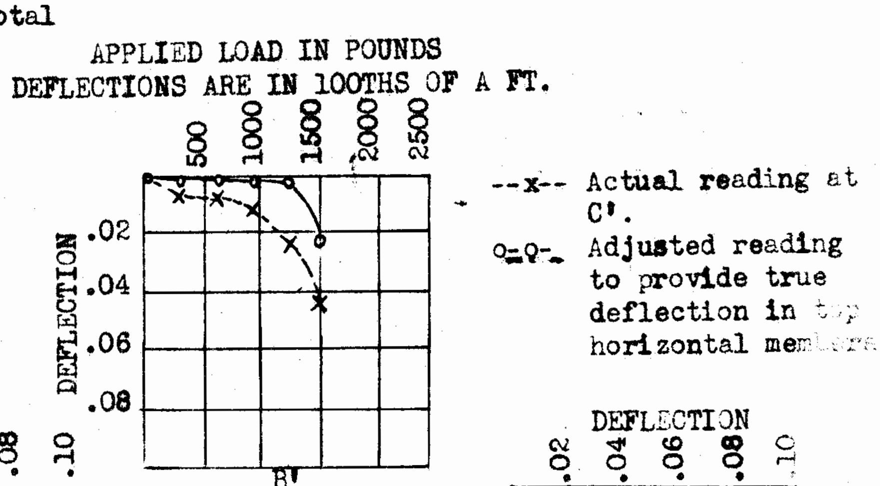
This again discounts any strength contributed to the wall from the wall panel material.

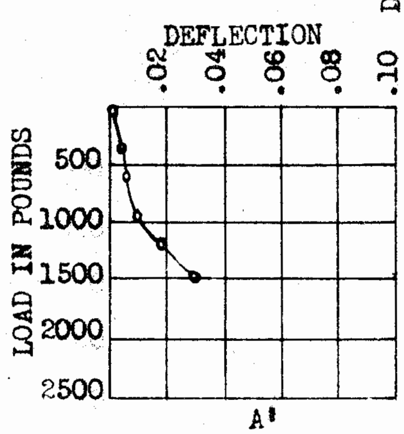
DEFLECTION TEST 138
WALL PANEL, TEST # 139

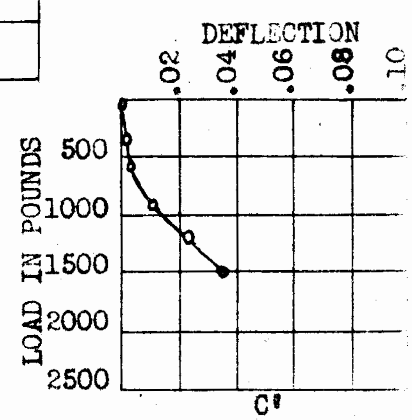
THREE COLUMN COMPOSITE.



Applied load in pounds equally distributed over points A-B-C.
Weight of wall section - 162#
Maximum deflections with total applied load of 1500#
At A' - .03
At B' - .022
At C' - .036

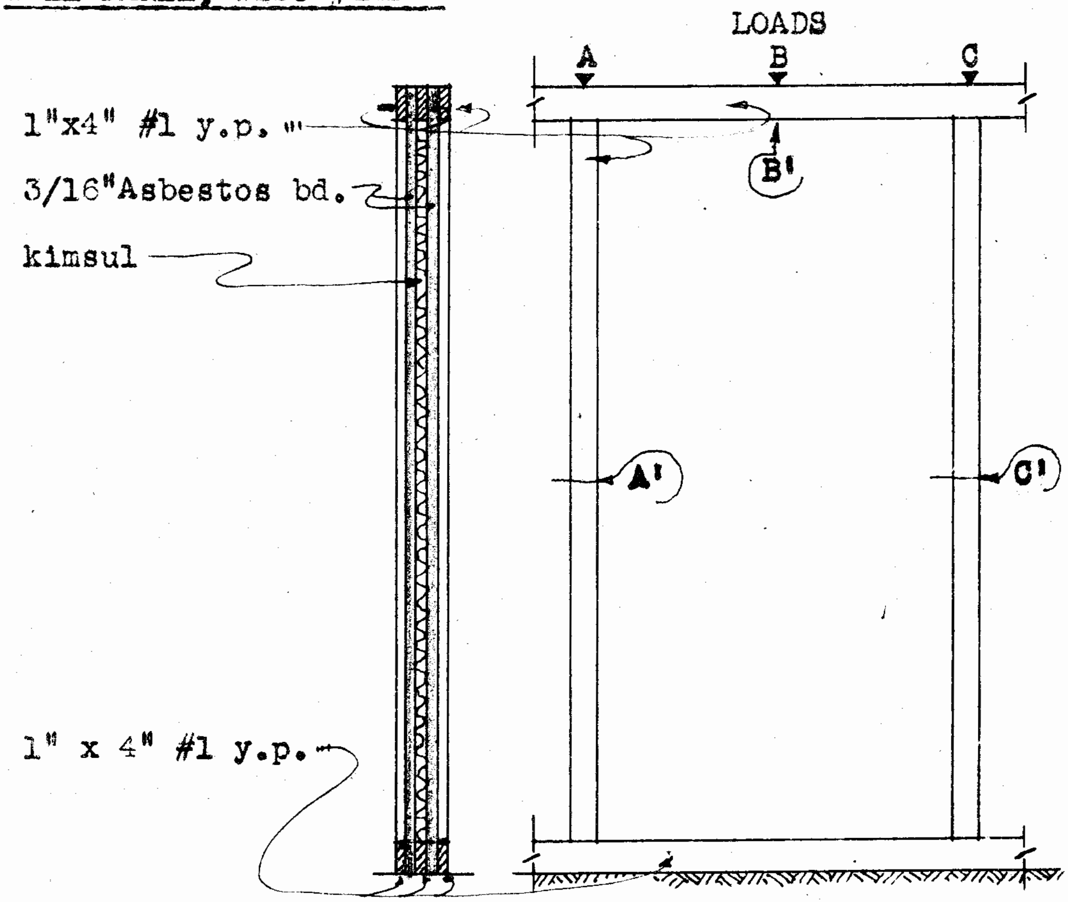






DEFLECTION TEST 140
WALL PANEL, TEST #141

THREE COLUMN COMPOSITE

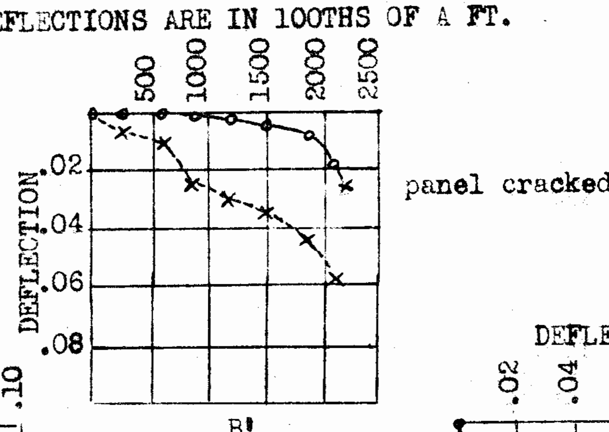


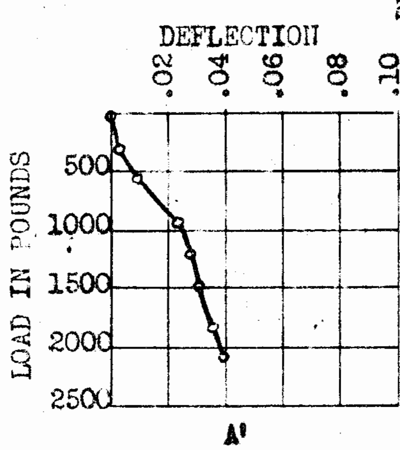
Applied load in pounds equally distributed over points A-B-C.
Weight of wall section -169#
Maximum deflections with total applied load of 2090#

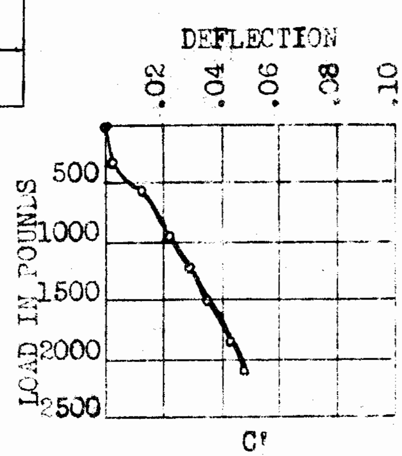
At A' - .039
At B' - .019
At C' - .048

--x-- Actual reading at C'.
c o- Adjusted reading to provide true deflection in top horizontal members.

APPLIED LOAD IN POUNDS
DEFLECTIONS ARE IN 100THS OF A FT.

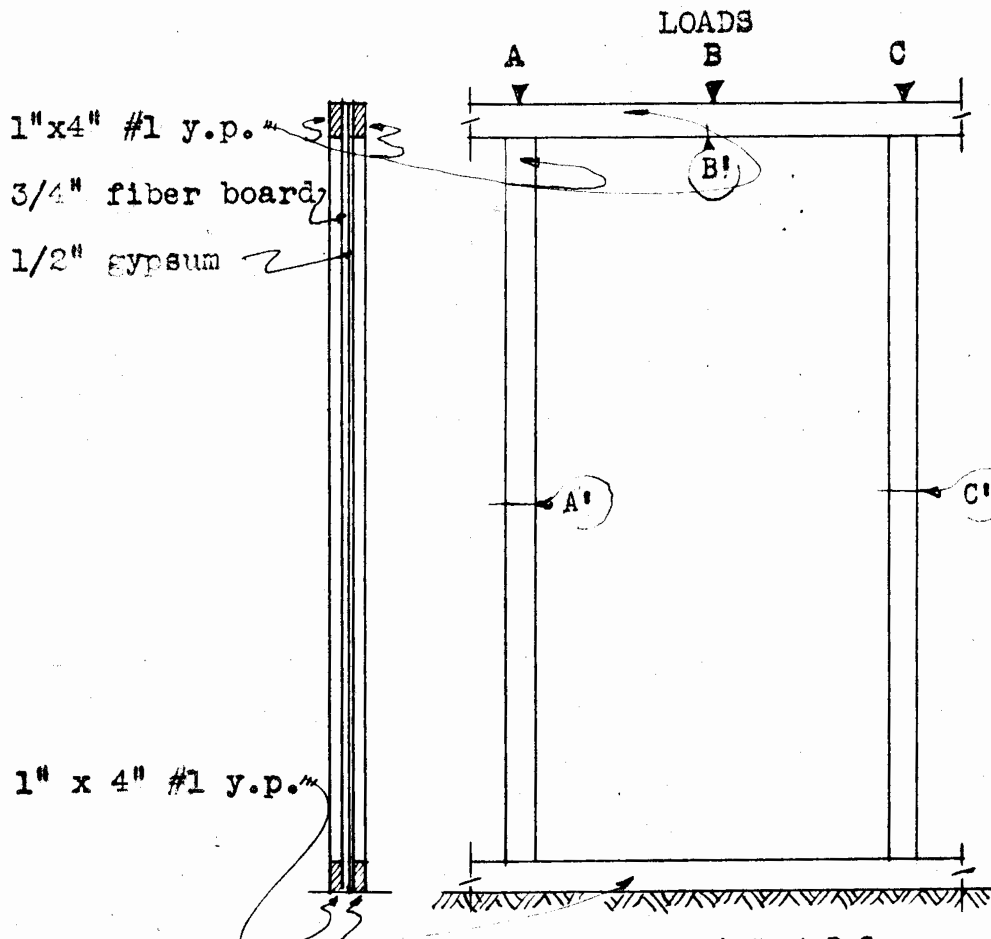






DEFLECTION TEST
WALL PANEL, TEST #142

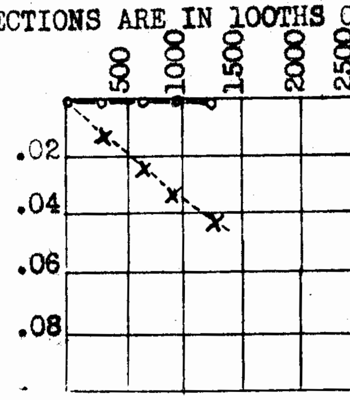
TWO COLUMN COMPOSITE



Applied load in pounds equally distributed over points A-B-C.
Weight of wall section - 144#.

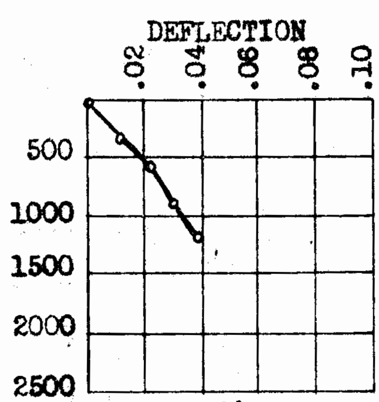
APPLIED LOAD IN POUNDS
DEFLECTIONS ARE IN 100THS OF A FT.

Maximum deflections with total applied load of 1290#
At A' - .039
At B' - .001
At C' - .042

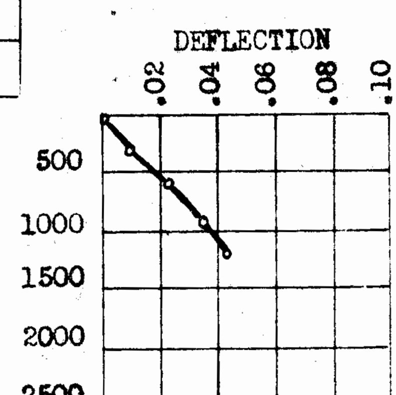


B'

--x-- Actual reading at C'.
o-o- Adjusted reading to provide true deflection in top horizontal members.



A'

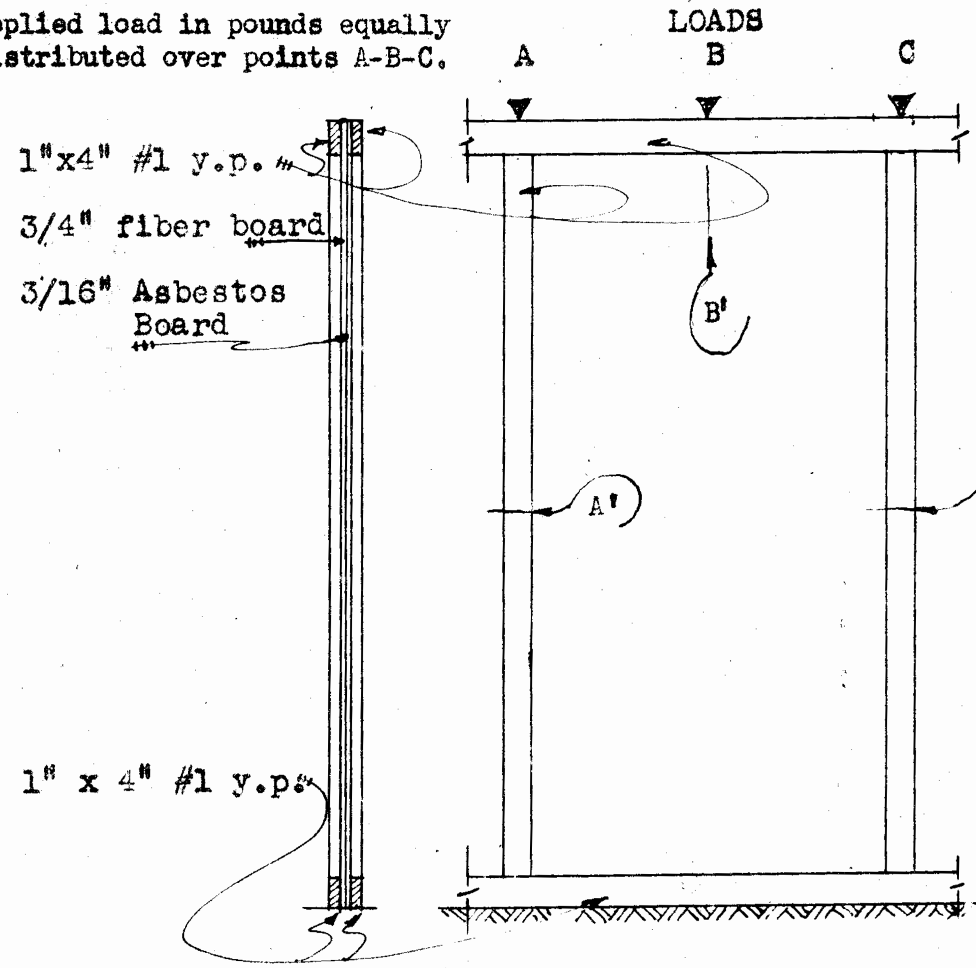


C'

DEFLECTION TEST
WALL PANEL, TEST #143

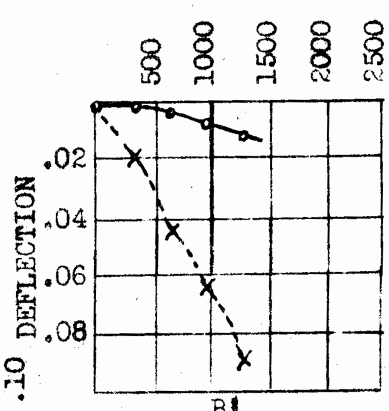
TWO COLUMN COMPOSITE

Applied load in pounds equally distributed over points A-B-C.

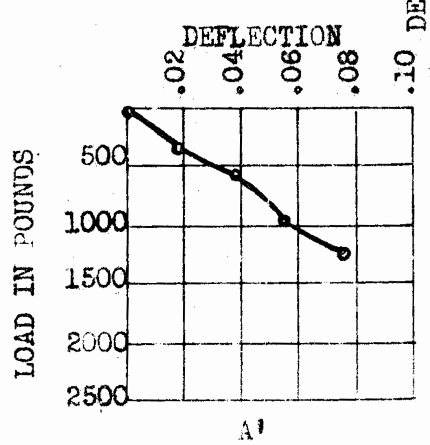


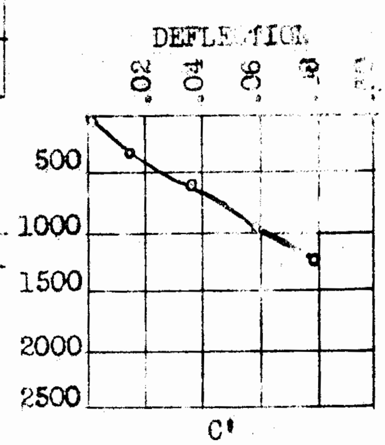
APPLIED LOAD IN POUNDS
DEFLECTIONS ARE IN 100THS OF A FT.

Weight of wall section - 137#
Maximum deflections with total applied load of 1280#
At A' - .078
At B' - .013
At C' - .079



--x-- Actual reading at C'. o-o-Adjusted reading to provide true deflection in top horizontal members.





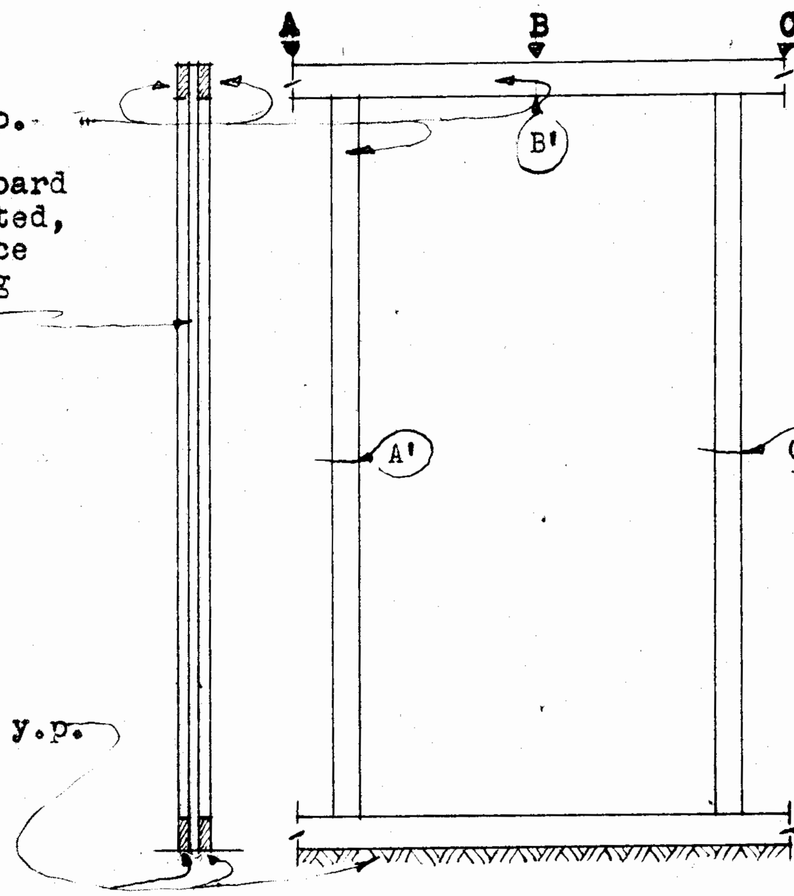
DEFLECTION TEST
WALL PANEL, TEST # 144

TWO COLUMN SINGLE WEB

1"x4" #1 y.p.

3/4"fiber board
Asphalt coated,
outer suface
with roofing
granules

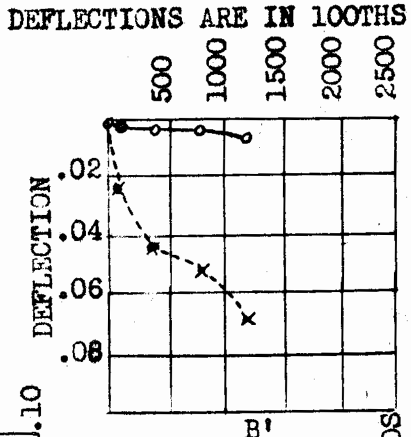
1" x 4" #1 y.p.



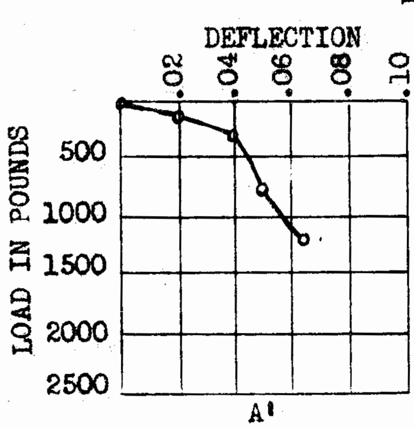
Applied load in pounds equally distributed over points A-B-C.
Weight of wall section - 82#

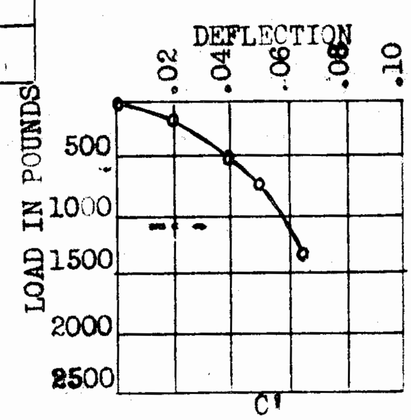
Maximum deflections with
total applied load of 1190#
At A' - .064
At B' - .006
At C' - .064

APPLIED LOAD IN POUNDS
DEFLECTIONS ARE IN 100THS OF A FT.



--x-- Actual reading at C'.
o-o- Adjusted reading to provide true deflection in top horizontal members.





WALLS
SUMMARY

The sandwich type of wall, when the weight is gotten out of it, offers a quick and not too expensive manner of creating a complete wall unit either in the shop or at the project site. Most successful assemblies of these components were those in which complete walls were put together on the previously prepared floors of the actual structure where the walls were to be erected.

The lighter wall shown in Test #144 was easily handled in lengths up to 20', but as soon as the lengths exceed this, the unit became bulky and there was a tendency to twist in handling. It would appear the practical length for a shop-fabricated unit of this type should not exceed twelve feet.

The interesting observation on this wall is that it possesses unusual structural value for the lightness of its members and offers good possibilities for further finish. Should an owner see fit to apply siding, shingles, or brick veneer to this exterior or wallboard to the interior, such finishes could easily be applied, employing the battens as studs. The further development of this type of wall could make demountability very simple. The solution lies in providing wood screws at certain points where in these initial studies nailing was employed.

Simple standardization is possible with these sandwich-type walls. The four foot component lends itself well to receiving a door frame in which the casings are sufficiently wide to engage the battens. Simple windows may be secured either between these battens or, by use of a plain plank frame, to the exterior of these battens. It should be observed that in using this wall consideration should be given to a type of jamb at the doors which would permit the application of a screen door without the necessity of extending the width of the jamb.

The window frames should be so constructed that should an additional exterior material be put over the outside battens adequate stile width would remain to receive this additional material. These stiles should be sufficiently thick (not less than 3/4") to permit the application of an outside casing if desired.

In the use of 1" x 4" in the manner described for these sandwich panels it is important that the material employed for battens be either impregnated or painted four sides so the tannic acid in the wood will not stain the wall panel material.

SECTION V

READINGS FROM COMPLETED STRUCTURES

EXPERIMENTS IN CONNECTION WITH WAR WORKERS' HOUSING

PURDUE RESEARCH FOUNDATION, G. S. MEIKLE, RESEARCH DIRECTOR

HOUSING RESEARCH DIVISION, CARL F. BOESTER, EXECUTIVE

Purdue University
West Lafayette, Indiana

March 1, 1944

WALL DEFLECTION READINGS
UNIT B-13

The following five graphs indicate the deflections presented in the walls employed in test Unit B-13, 24'0" x 32'0".

The wall construction of this unit employs the materials and construction developed from Wall Test #142.

Readings were taken vertically along the battens both inside and outside the unit, and the results recorded in 100ths of a foot.

The small plan indicates the points or stations at which these readings were taken.

Elevations of readings indicate the distances above the base of the wall at which readings were taken, given in inches.

Deflections provide the scale for the deflection readings, given in 100ths of a foot.

Each graph is marked to tell the station to which it applies.

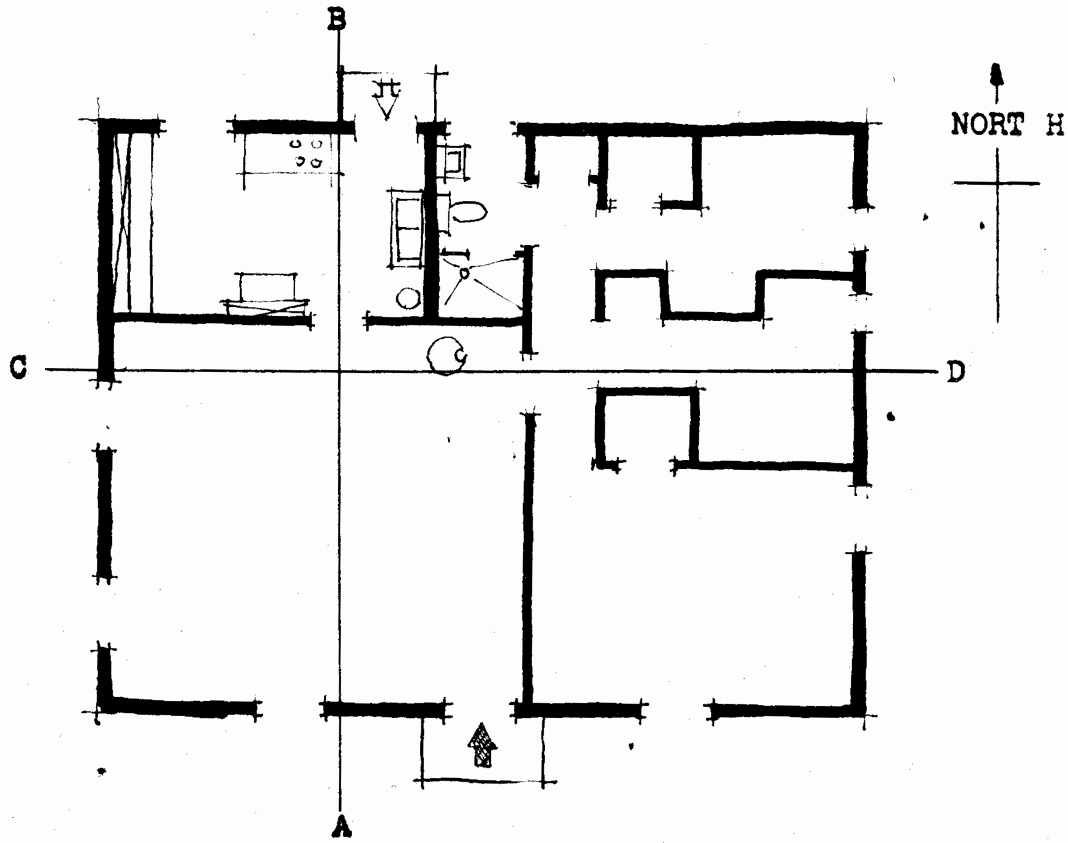
Section graphs indicate the position of the materials in the wall in relation to the larger graph. These smaller graphs also indicate the materials employed in the wall.

On the graph taken at Station A, the wall materials are described; on that taken at Station E, their thickness in 100ths of a foot are given.

The lower case notes within the body of the graphs indicate the planes along which the readings were taken. Observe the manner in which these readings reverse themselves between Stations C and D. This was done in order that a comparison might be made between the deflections shown for Stations A and B and opposite Stations D and E from a theoretical point of observation to the south of the structure.

REFERENCE SKETCH
FLOOR TRUSS TEST #125
ROOF TRUSS TEST # 134

IDENTIFYING LINES ALONG WHICH DEFLECTION READINGS WERE TAKEN. SCALE: 1/8" EQUALS ONE FOOT.



UNIT-B-13

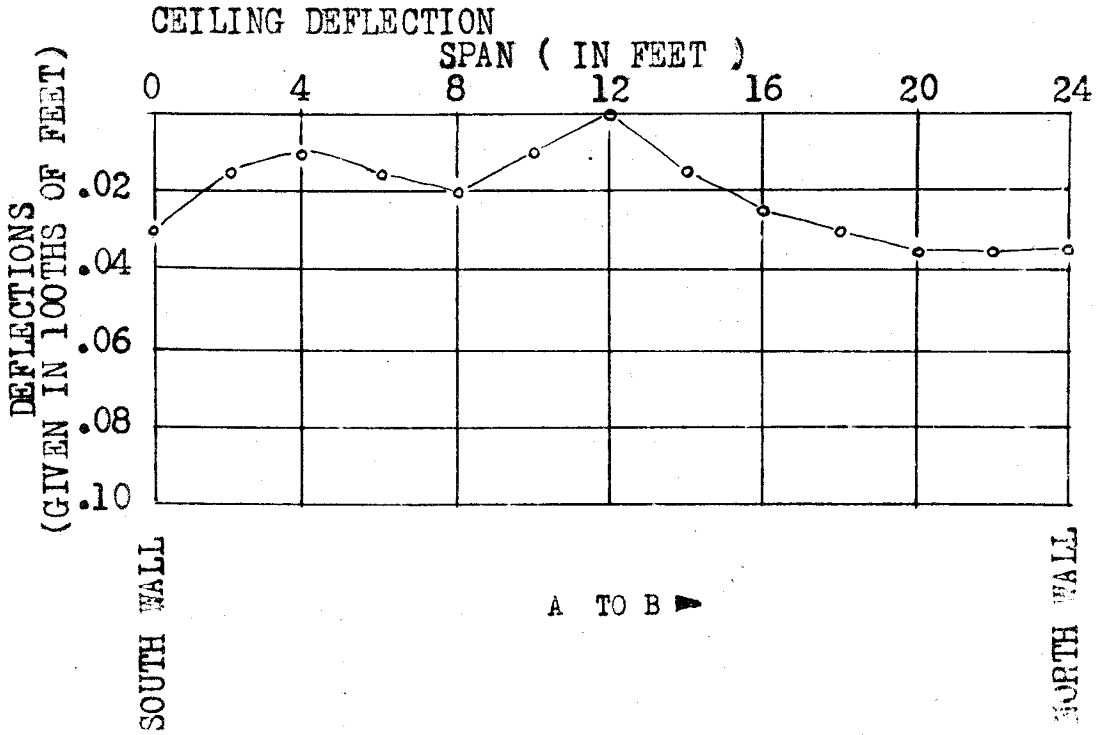
24'0" x 32'0"

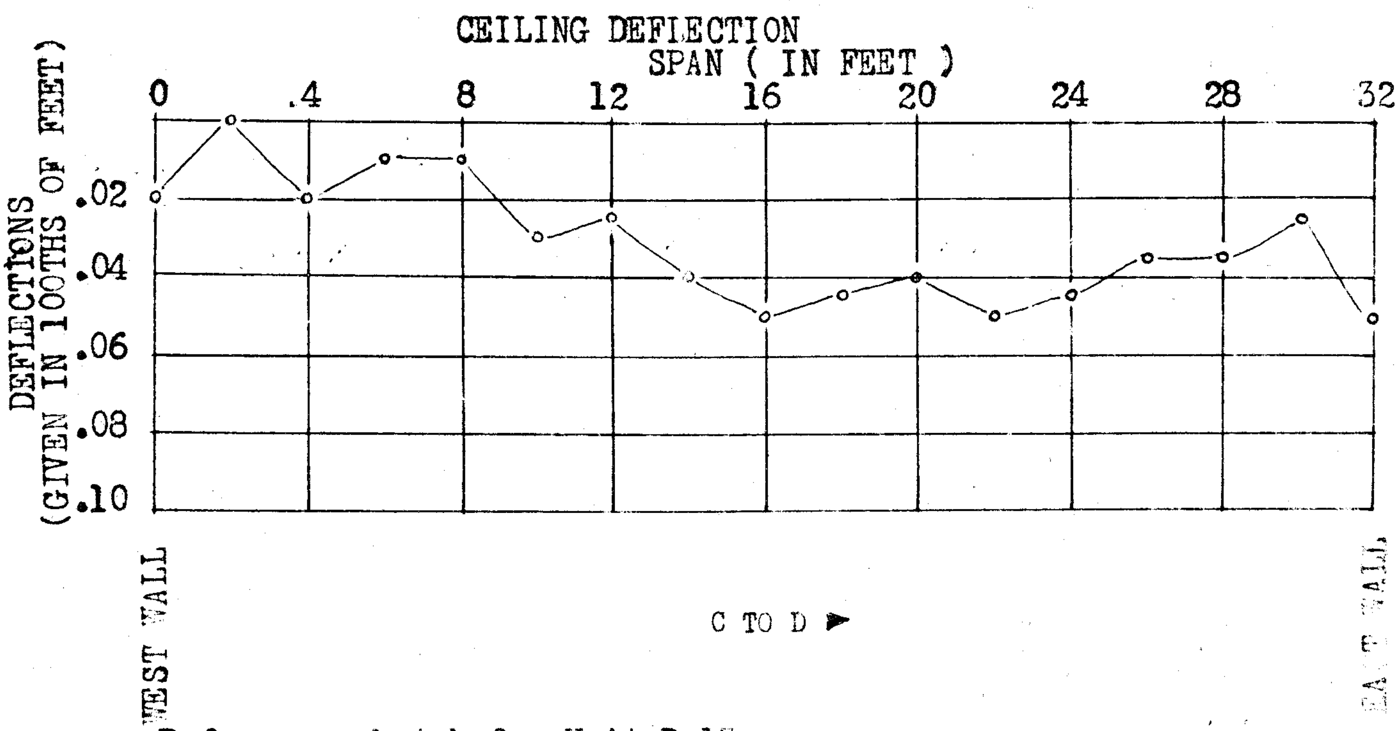
DEFLECTION TESTS
ROOF TRUSS, TEST # 134

FOR DESCRIPTION OF TRUSS SEE FIG., P.58

TRUSSES SUPPORTED AT ENDS.

CARRYING FULL SPAN CEILING, INSULATION AND ROOF LOADS.





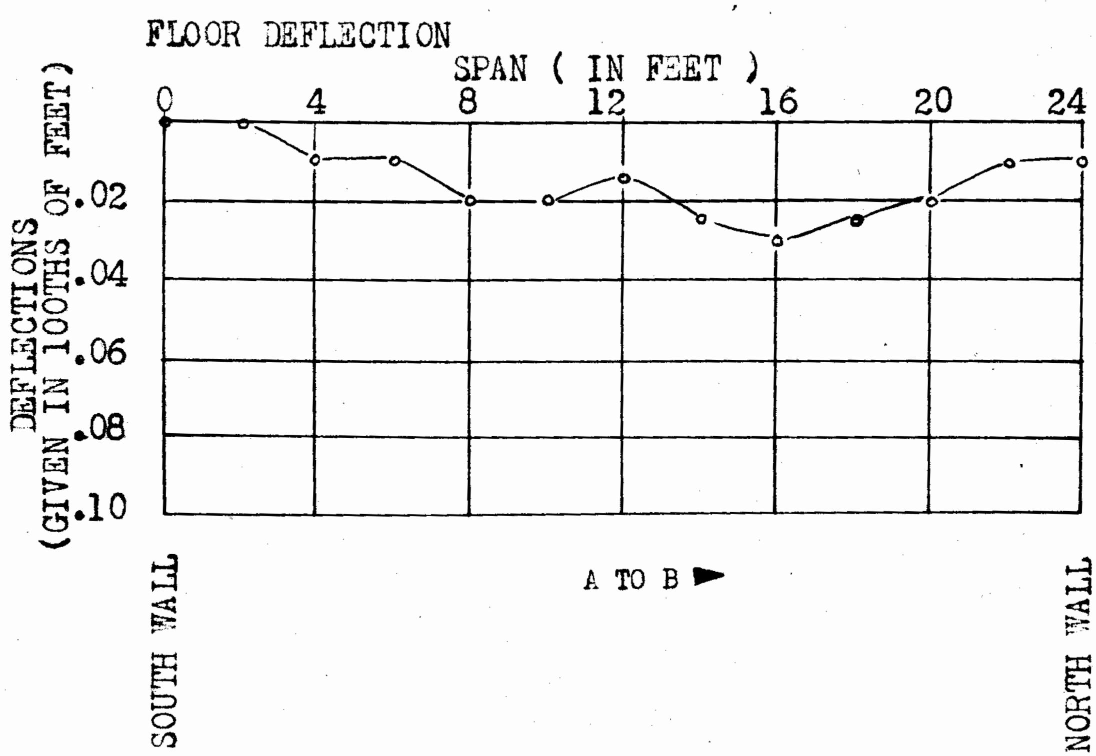
Reference sketch for Unit-B-13 identifies lines along which readings were taken

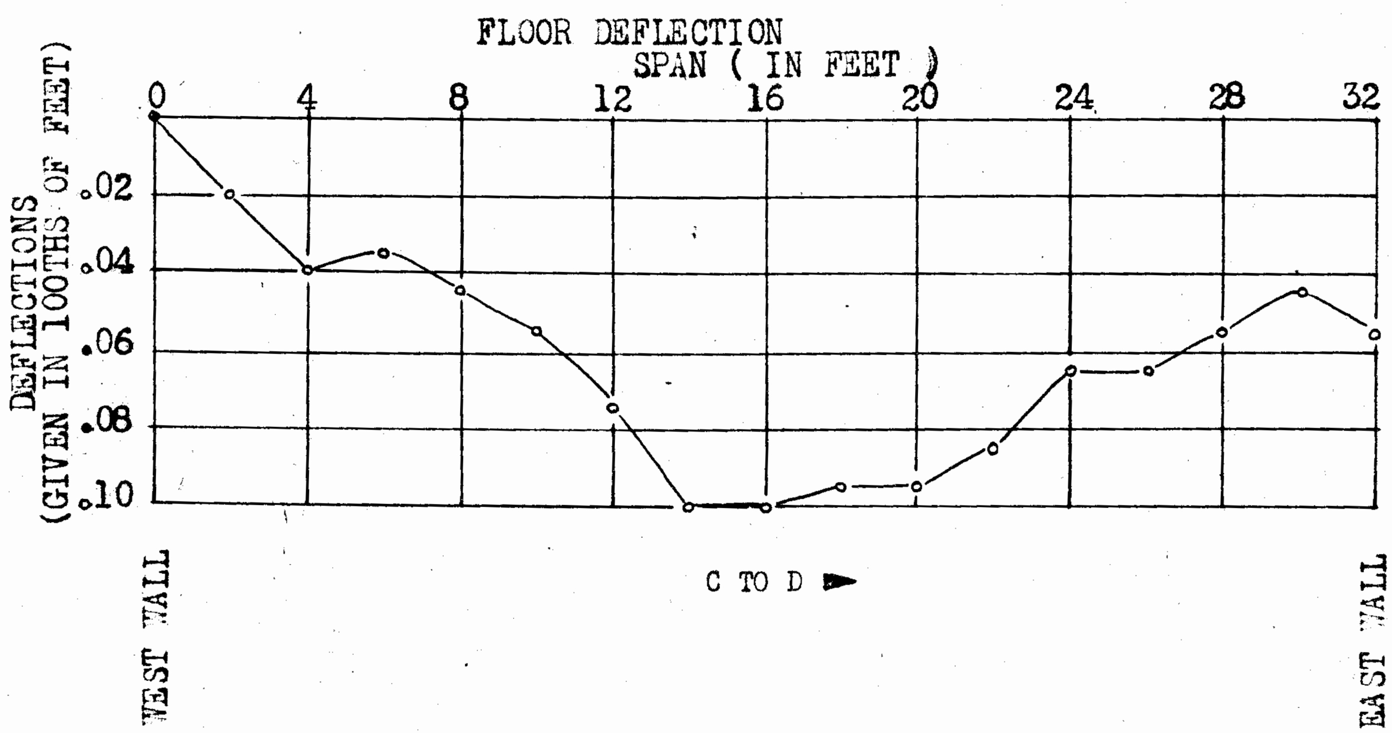
DEFLECTION TESTS
FLOOR TRUSS TEST / 125

FOR DESCRIPTION OF TRUSS SEE FIG., P.35

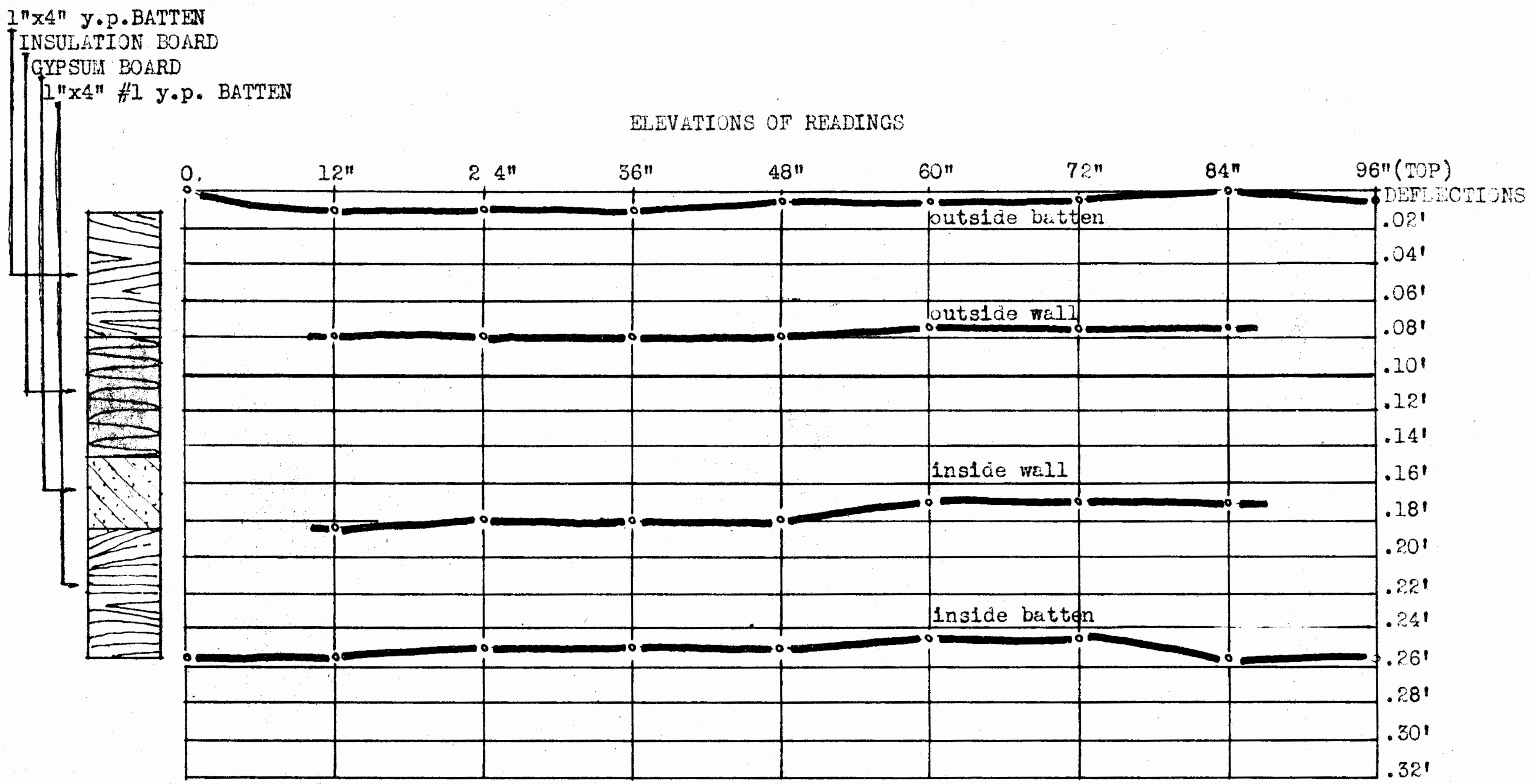
TRUSSES SUPPORTED AT ENDS.

NO ROOF OR CEILING LOADS.
CARRYING 13/16" OAK FLOOR, 1/2" GYPSUM BOARD.



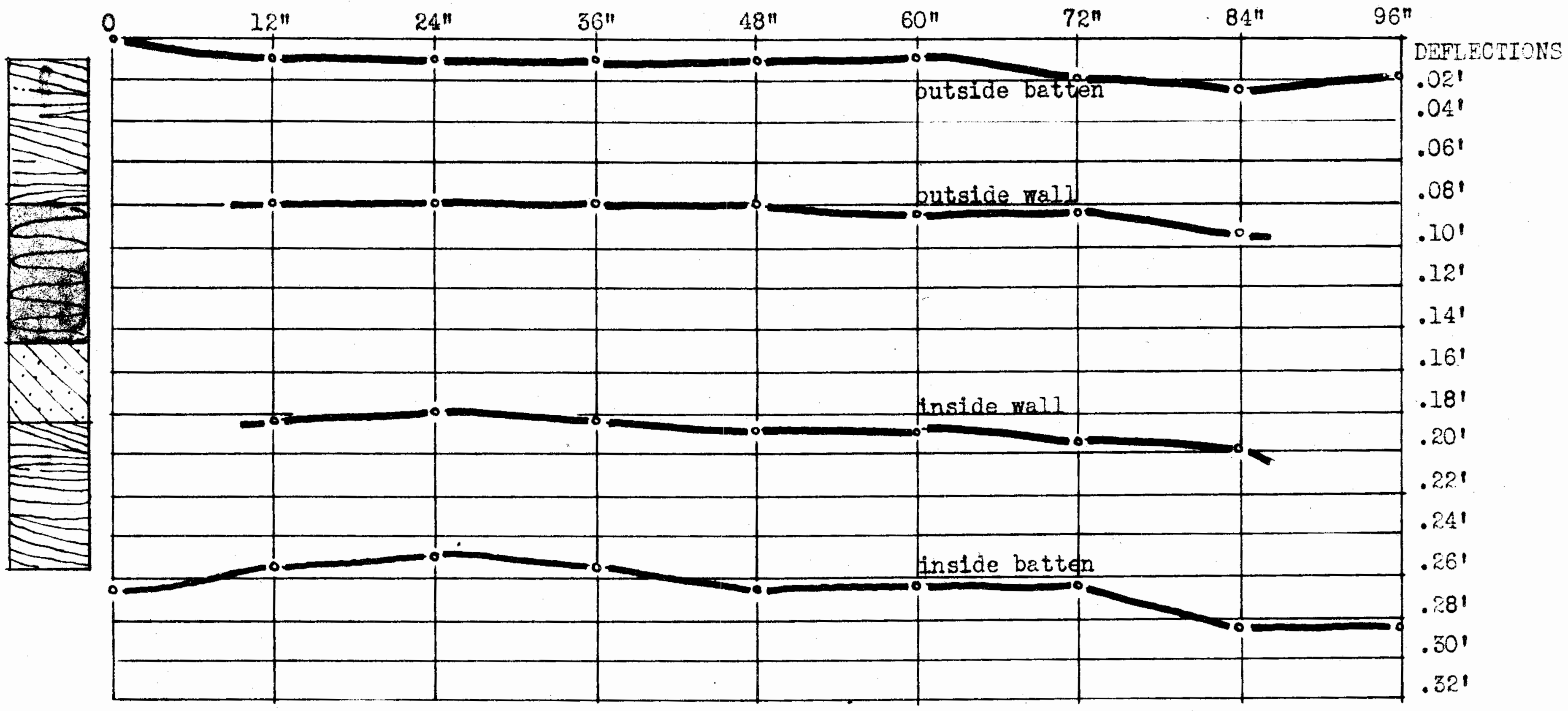


Reference sketch for Unit-B-13 identifies lines along which readings were taken



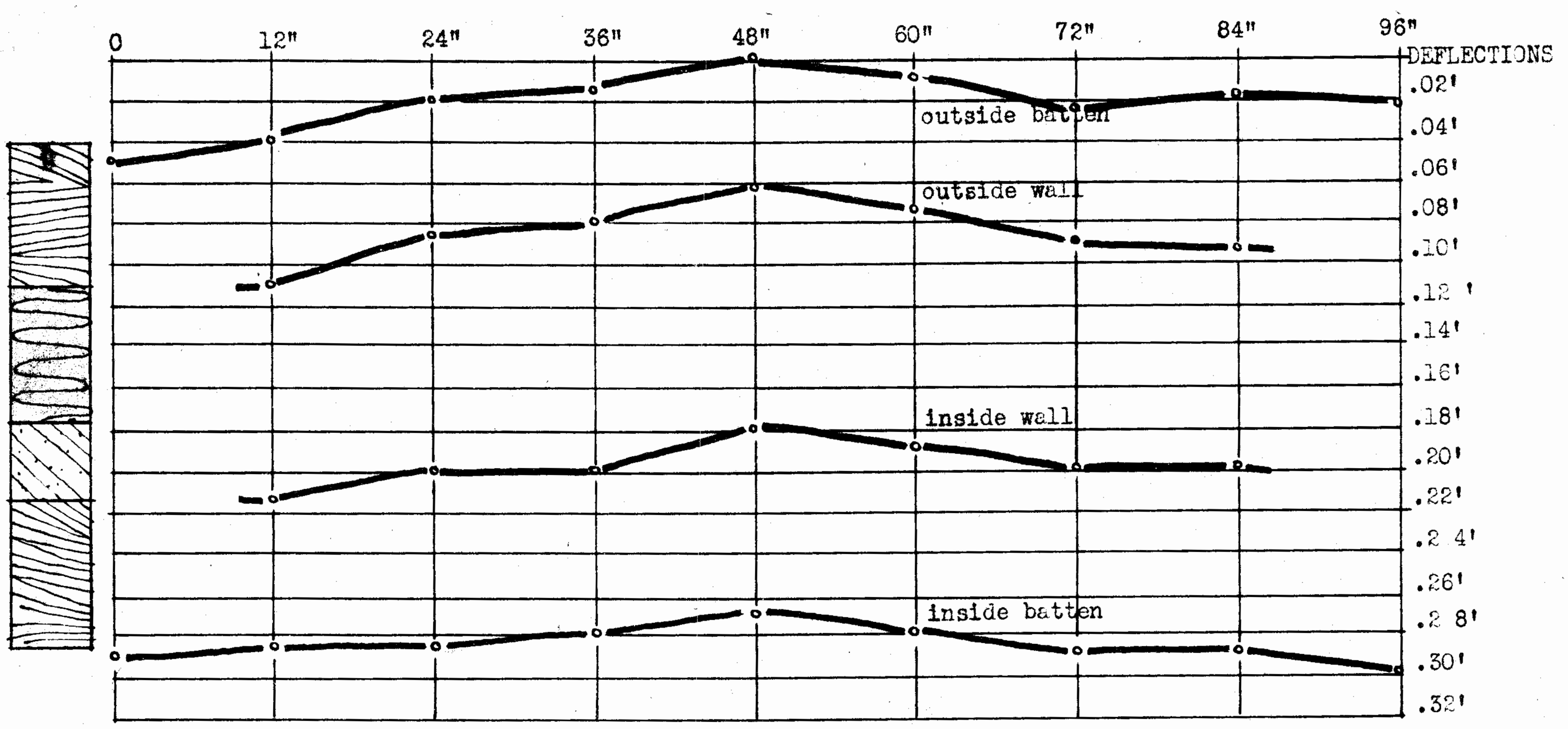
READINGS TAKEN AT STATION A

ELEVATIONS OF READINGS

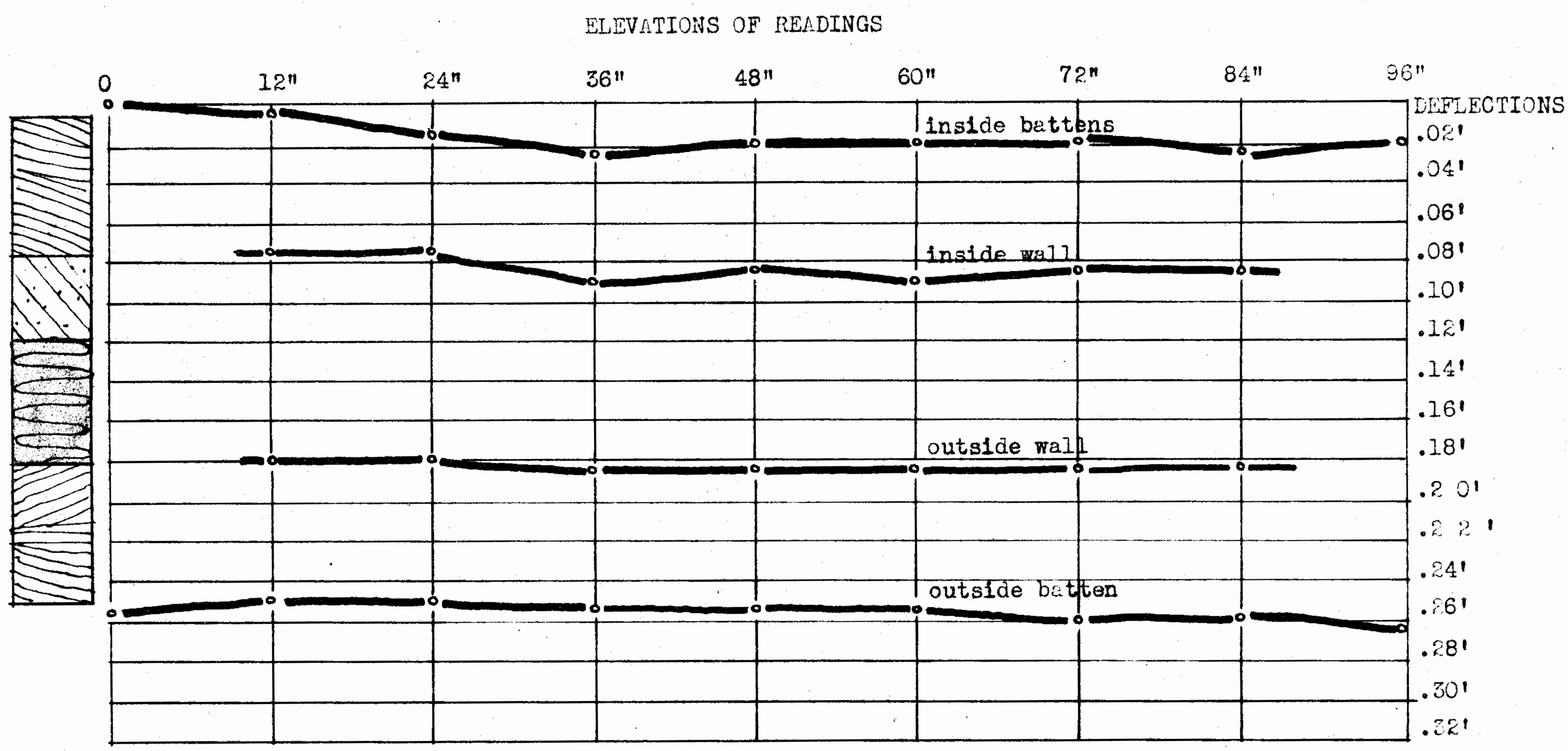


READINGS TAKEN AT STATION B

ELEVATIONS OF READINGS

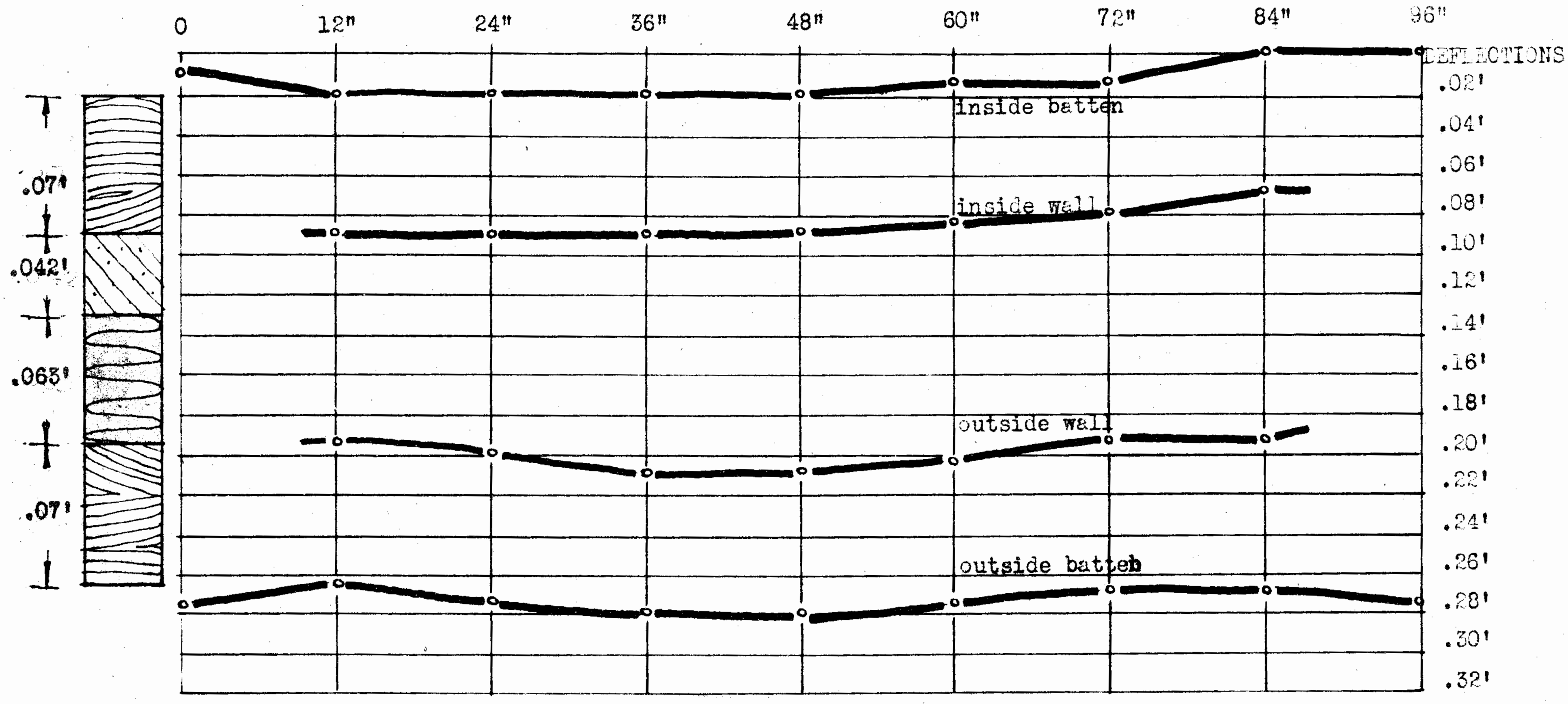


READINGS TAKEN AT STATION C

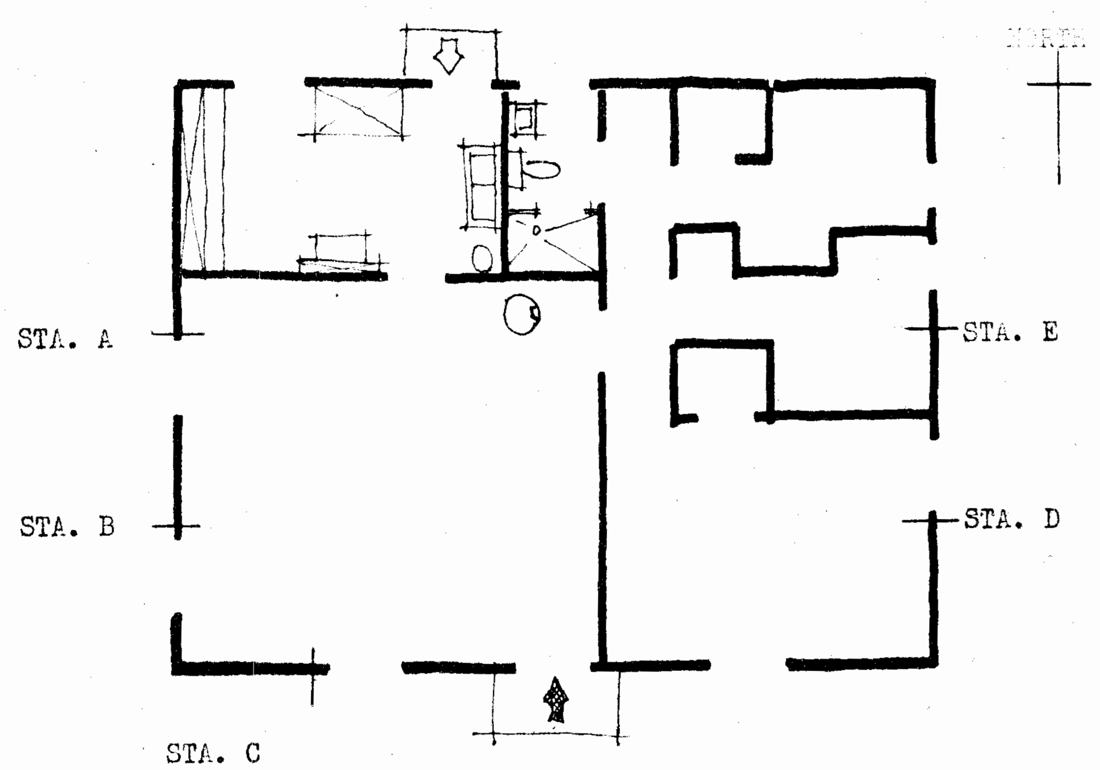


READINGS TAKEN AT STATION D

ELEVATIONS OF READINGS



READINGS TAKEN AT STATION E



LOCATIONS OF STATIONS AT WHICH READINGS WERE TAKEN.

UNIT-B-13.

WALL DEFLECTION READINGS
UNIT B-15

The following five graphs indicate the deflections present in the walls employed in test Unit B-15, 24'0" x 32'0".

The wall construction of this unit employs the materials and construction developed from Wall Test # 143

Readings were taken vertically along the battens both inside and outside the unit, and the results recorded in 100ths of a foot.

The small plan indicates the points or stations at which these readings were taken.

Elevations of readings indicate the distances above the base of the wall at which readings were taken, given in inches.

Deflections provide the scale for the deflection readings, given in 100ths of a foot.

Each graph is marked to tell the station to which it applies.

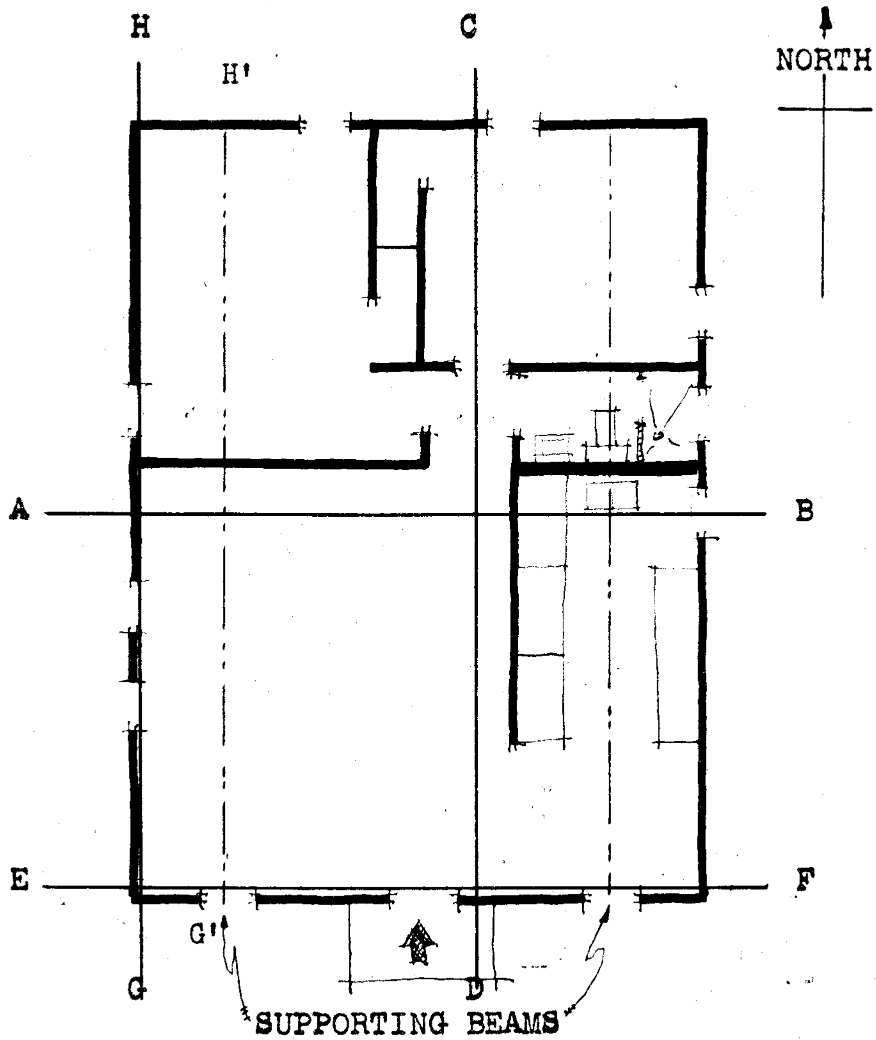
Section graphs indicate the position of the materials in the wall in relation to the larger graph. These smaller graphs also indicate the materials employed in the wall.

On the graph taken at Station A, the wall materials are described; on that taken at Station E, their thickness in 100ths of a foot are given.

The lower case notes within the body of the graphs indicate the planes along which the readings were taken. Observe the manner in which these readings reverse themselves between Stations C and D. This was done in order that a comparison might be made between the deflections shown for Stations A and B and opposite Stations D and E from a theoretical point of observation to the south of the structure.

REFERENCE SKETCH
FLOOR TRUSS TEST #124
ROOF TRUSS TEST #135

IDENTIFYING LINES ALONG WHICH DEFLECTION READINGS WERE TAKEN.
SCALE: 1/8" EQUALS ONE FOOT.



UNIT B-15

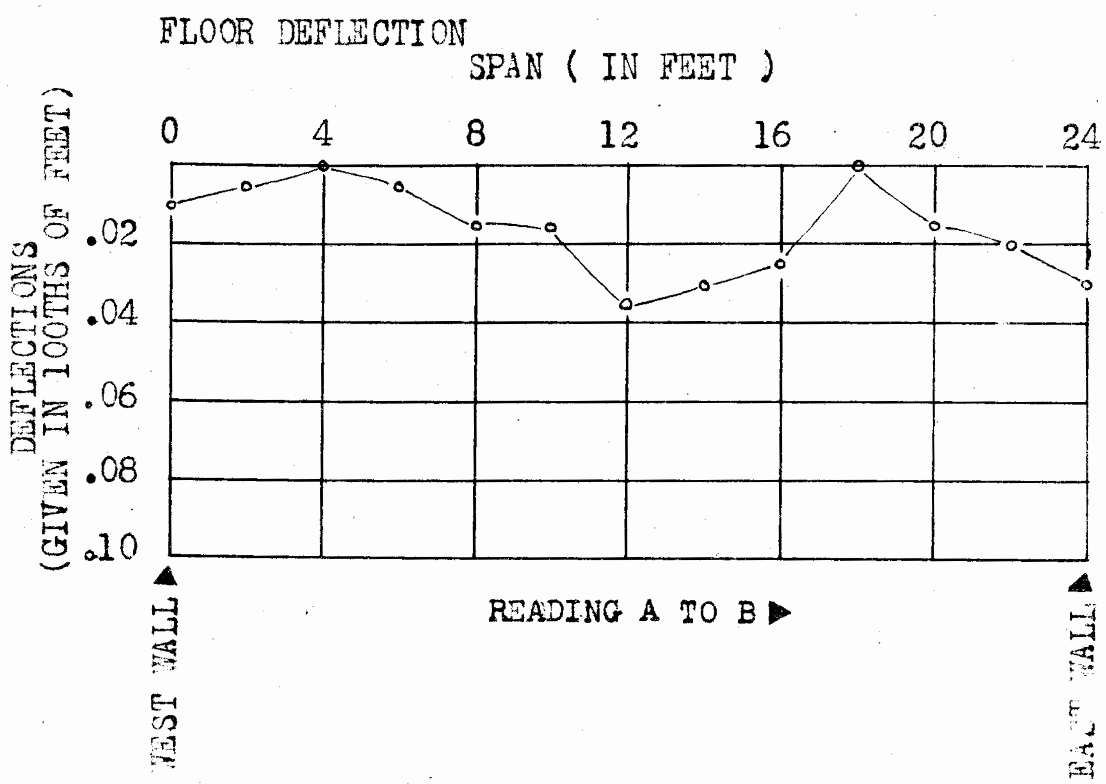
24'0" x 32'0"

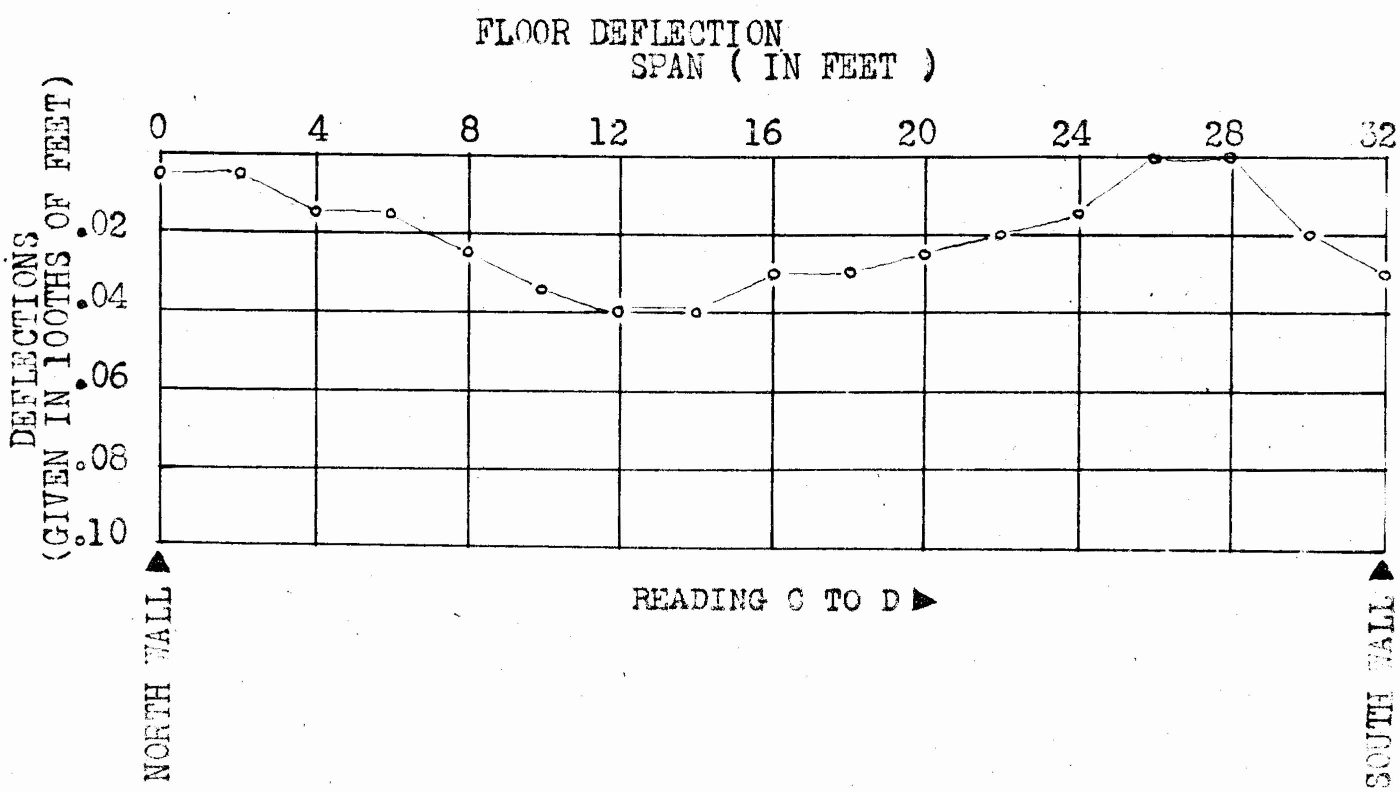
DEFLECTION TESTS
FLOOR TRUSS TEST #124

FOR DESCRIPTION OF TRUSS SEE FIG., P.29

TRUSSES SUPPORTED 48" IN FROM EACH END.

IN ADDITION TO SUSPENDED FLOOR LOAD, TRUSSES CARRY WALLS, CEILINGS, INSULATION, ROOF LOADS.





Reference sketch for Unit-B-15 identifies lines along which readings were taken.

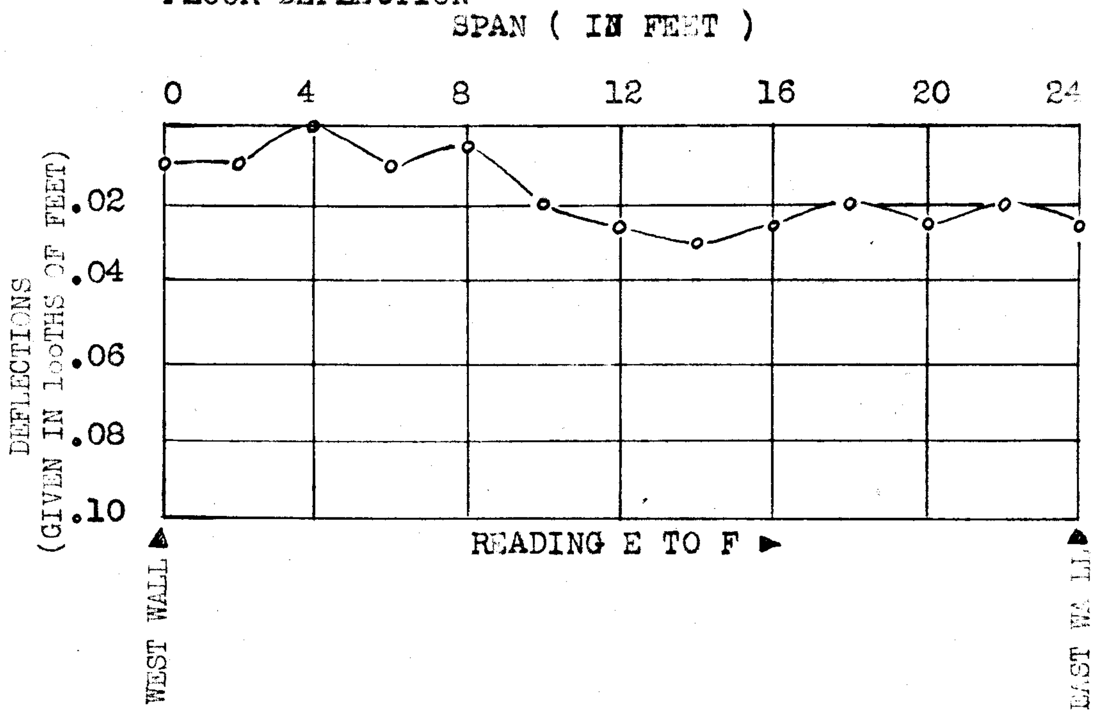
DEFLECTION TESTS
FLOOR TRUSS TEST, # 124

FOR DESCRIPTION OF TRUSS SEE FIG.

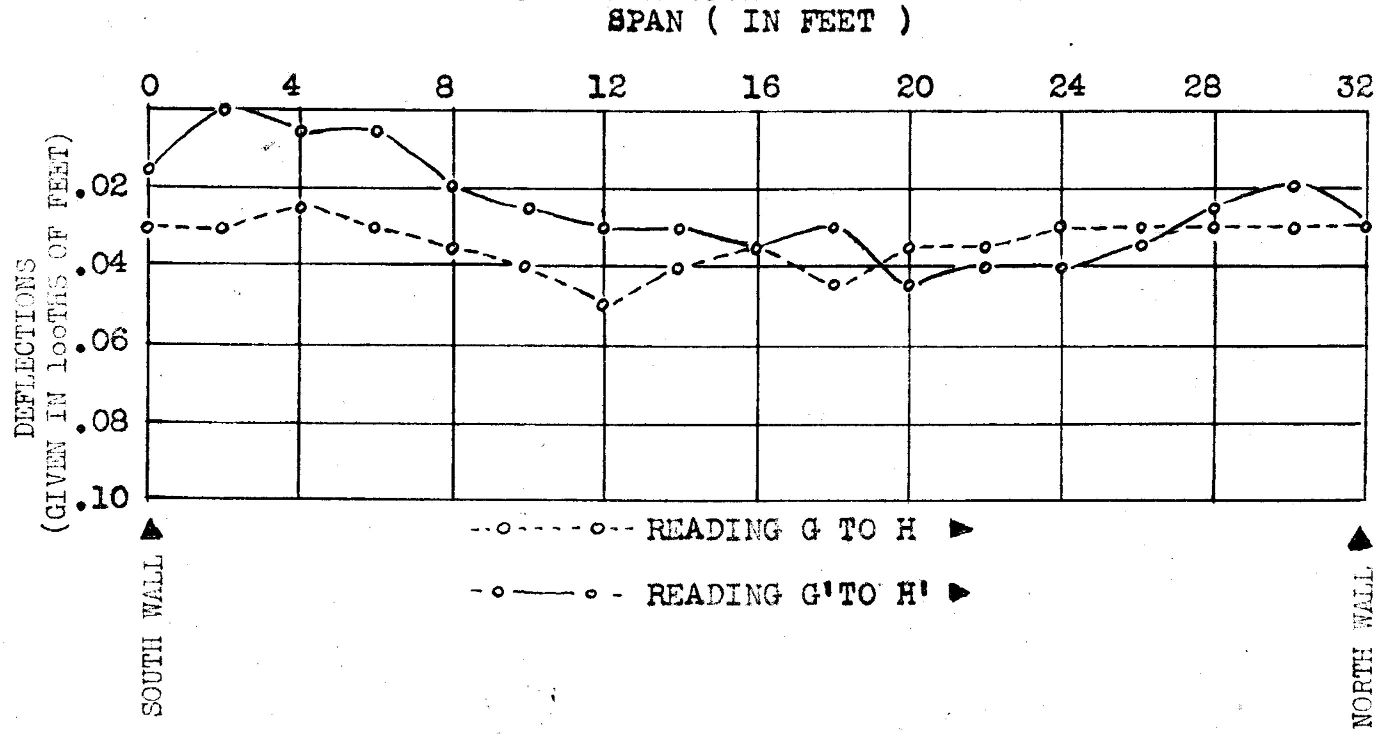
TRUSSES SUPPORTED 48" FROM EACH END.

IN ADDITION TO SUSPENDED FLOOR LOAD, TRUSSES CARRY WALLS, CEILING, INSULATION, ROOF LOADS.

FLOOR DEFLECTION



FLOOR DEFLECTIONS



Reference sketch for Unit-B-15 identifies lines along which readings were taken.

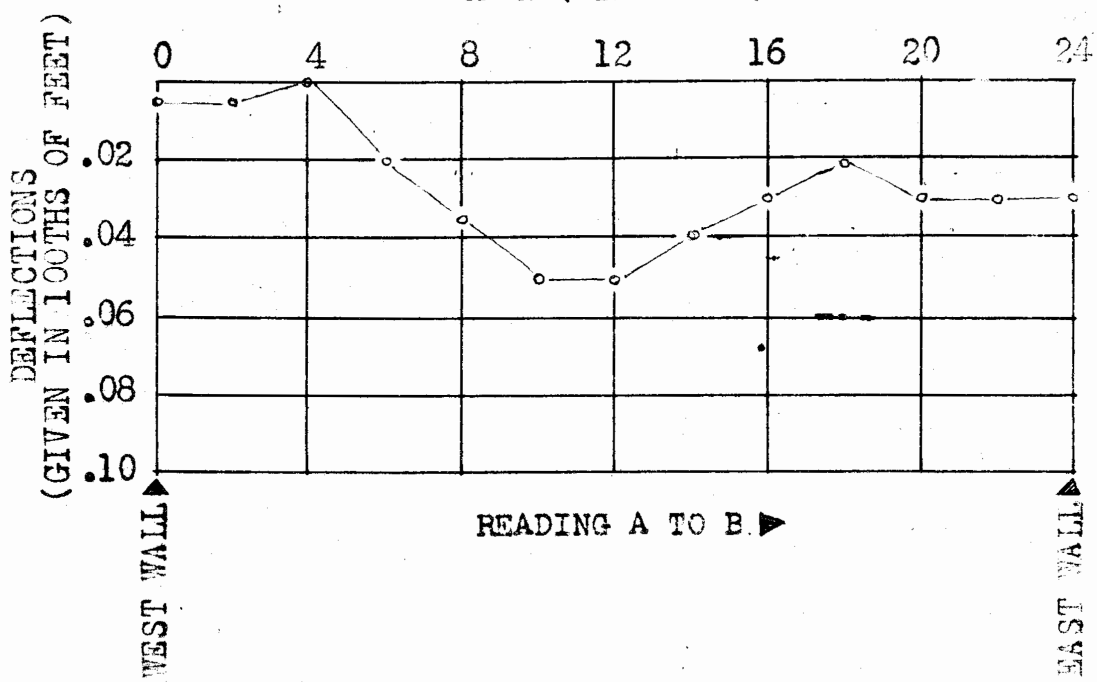
DEFLECTION TESTS
ROOF TRUSS. TEST # 135

FOR DESCRIPTION OF TRUSS SEE FIG., P.59

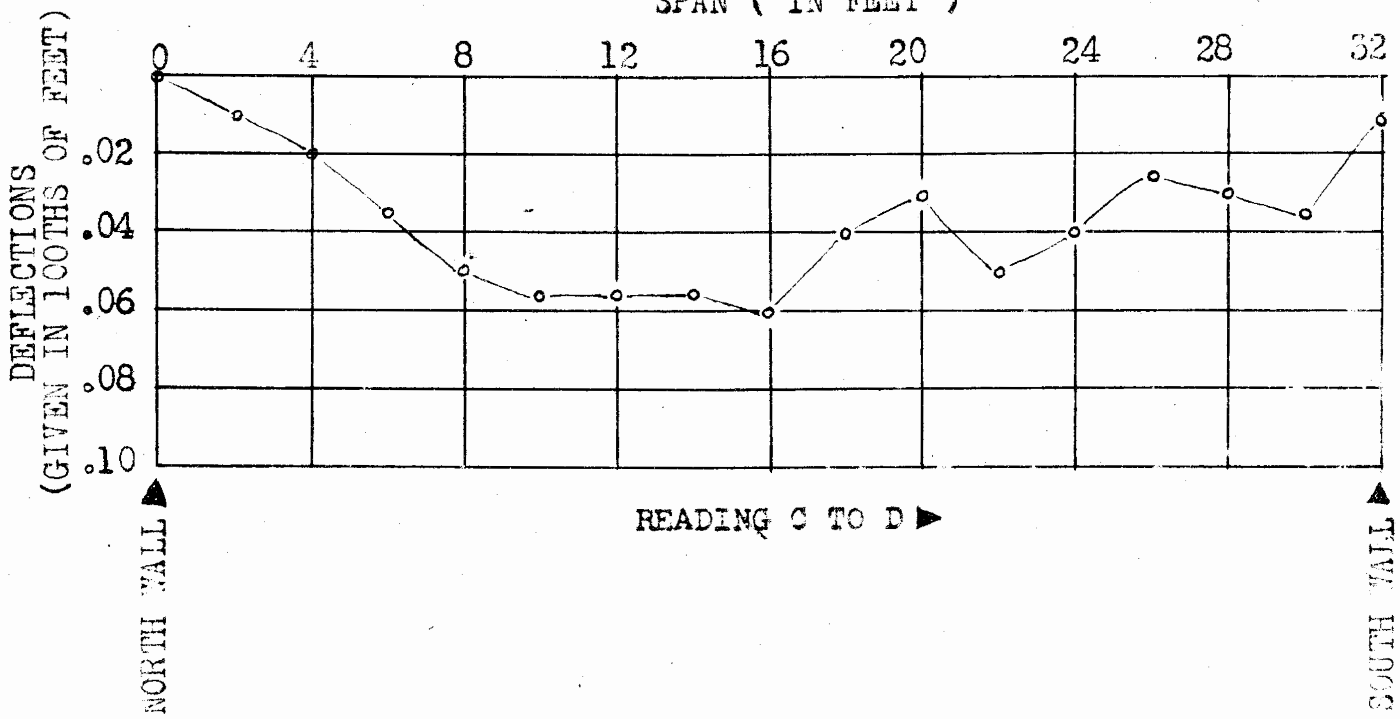
TRUSSES SUPPORTED AT ENDS

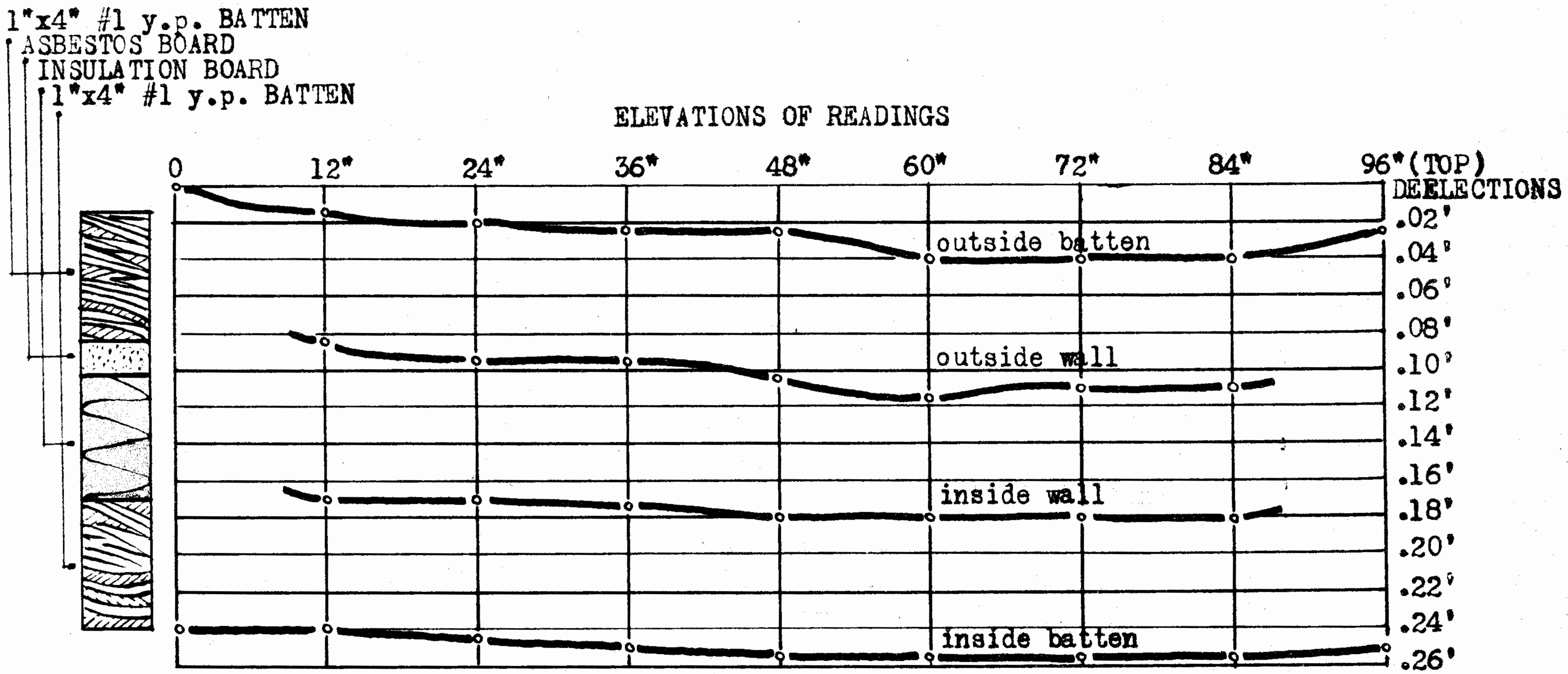
CARRYING FULL SPAN CEILING, INSULATION AND ROOF LOADS

CEILING DEFLECTION

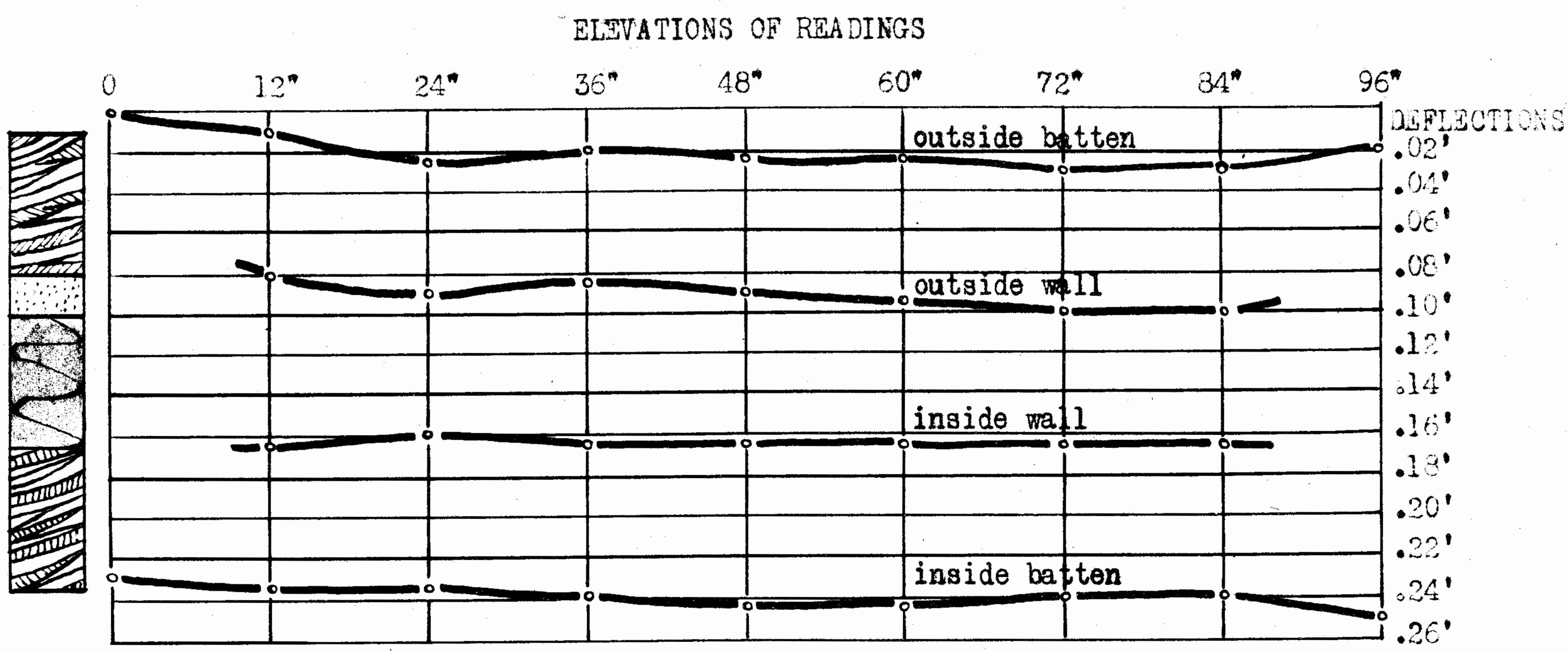


CEILING DEFLECTION

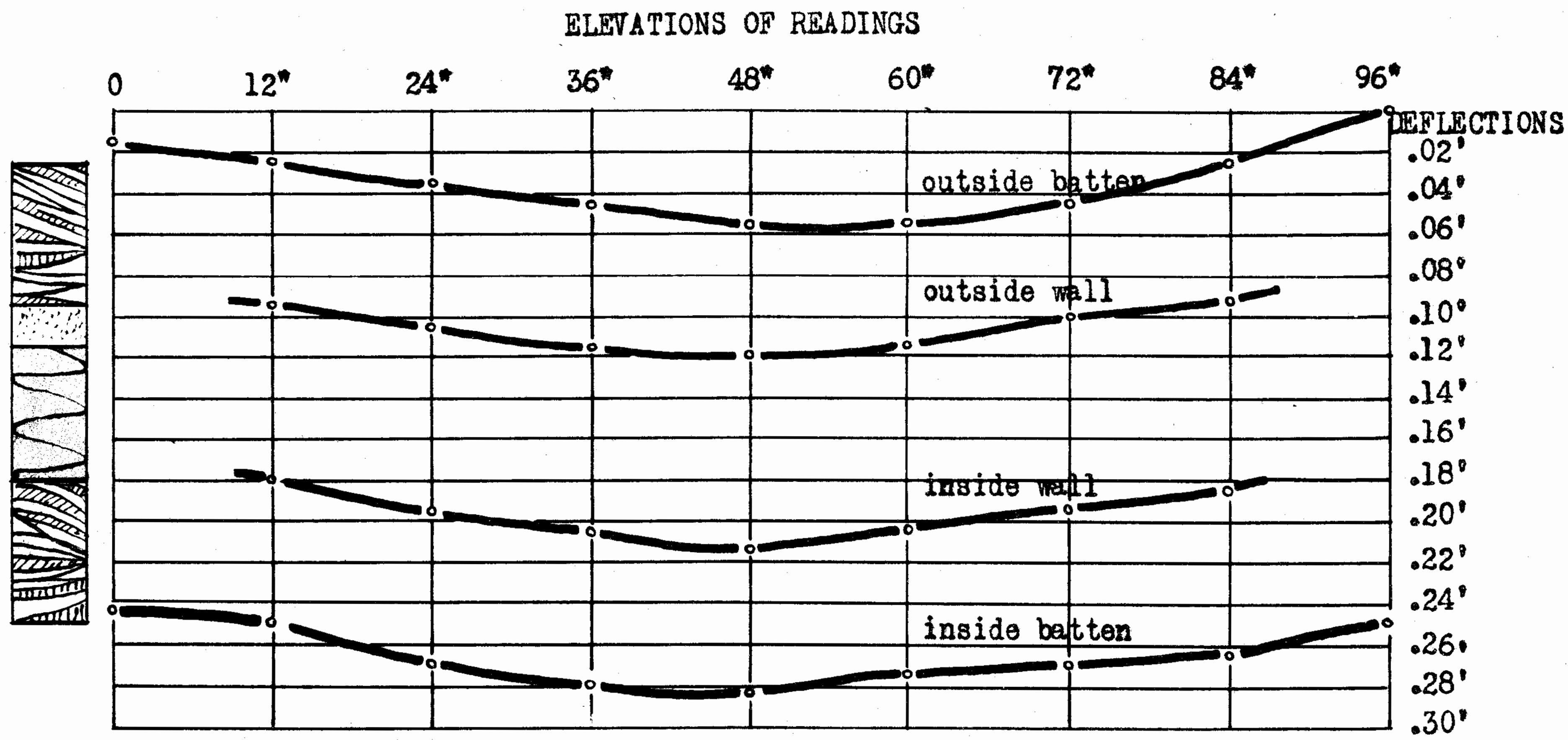




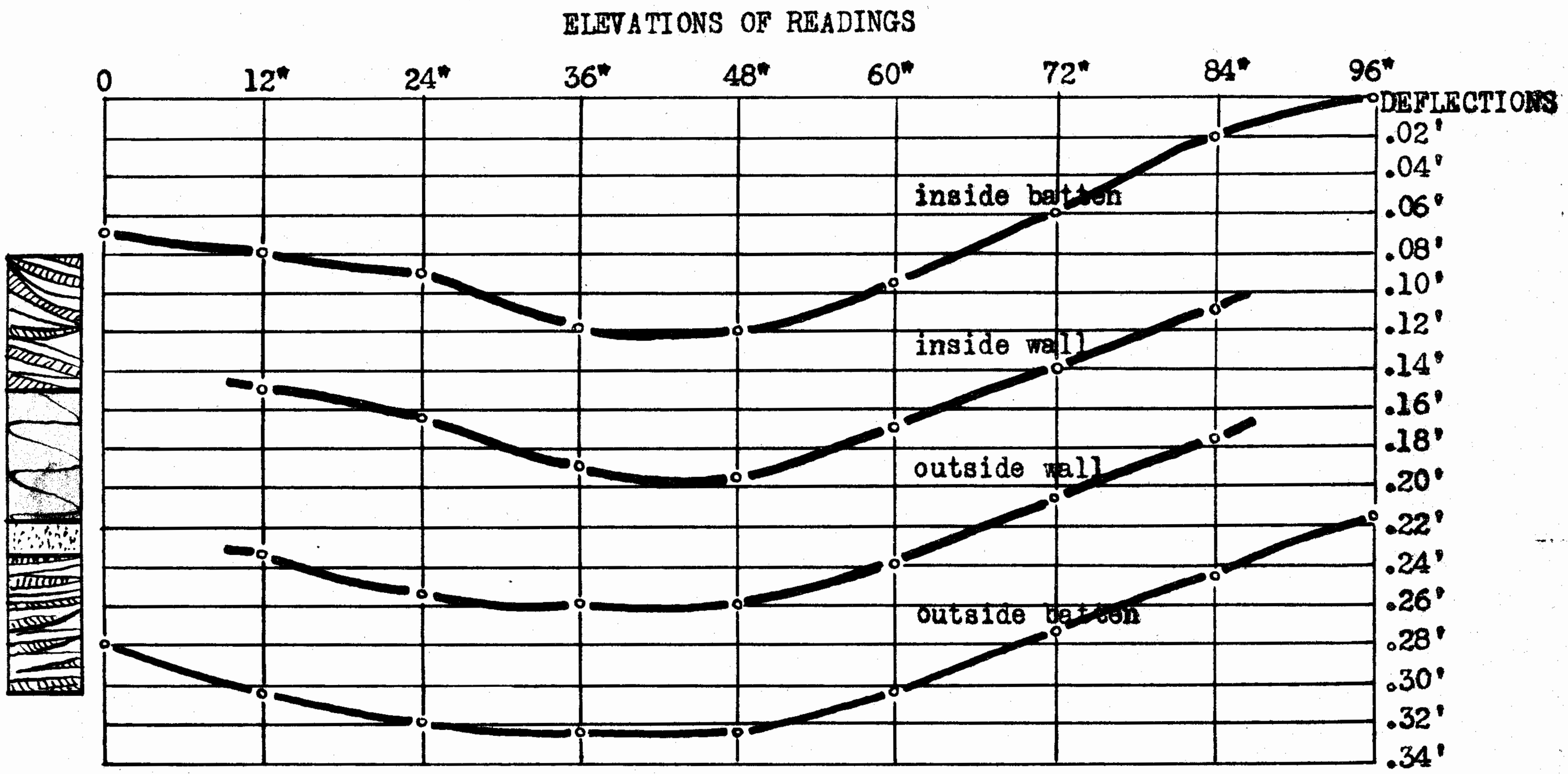
READINGS TAKEN AT STATION A.



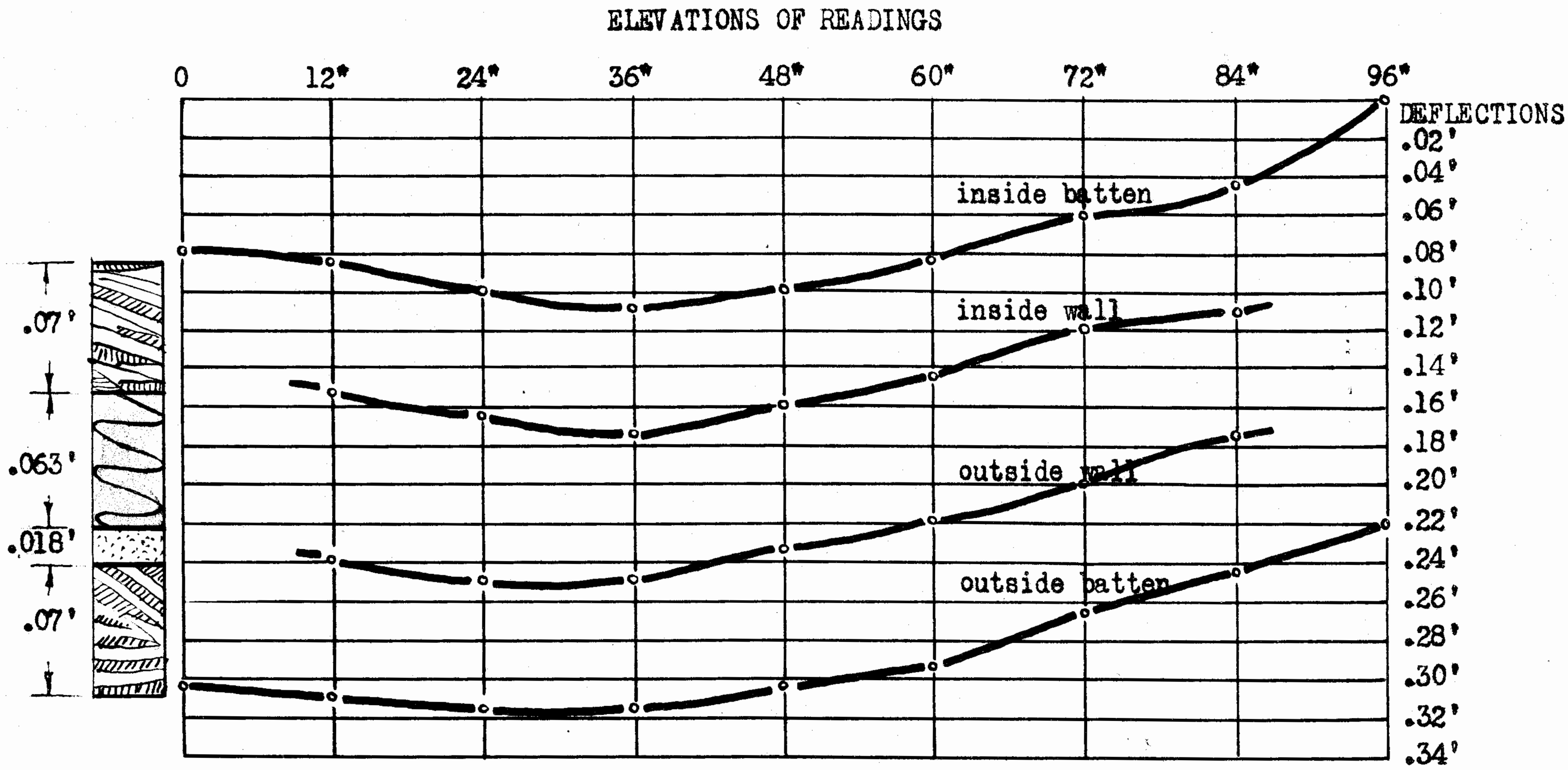
READINGS TAKEN AT STATION B



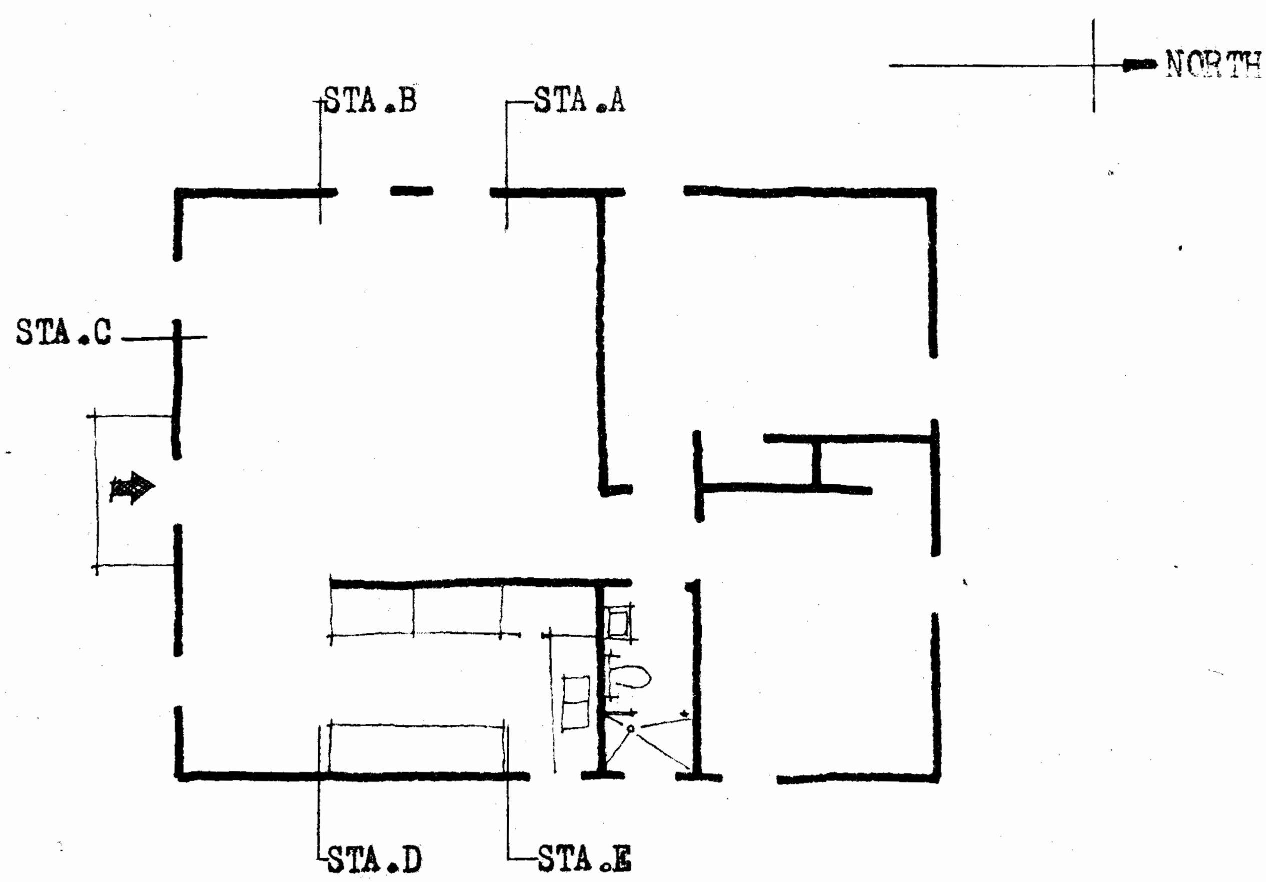
READINGS TAKEN AT STATION C



READINGS TAKEN AT STATION D



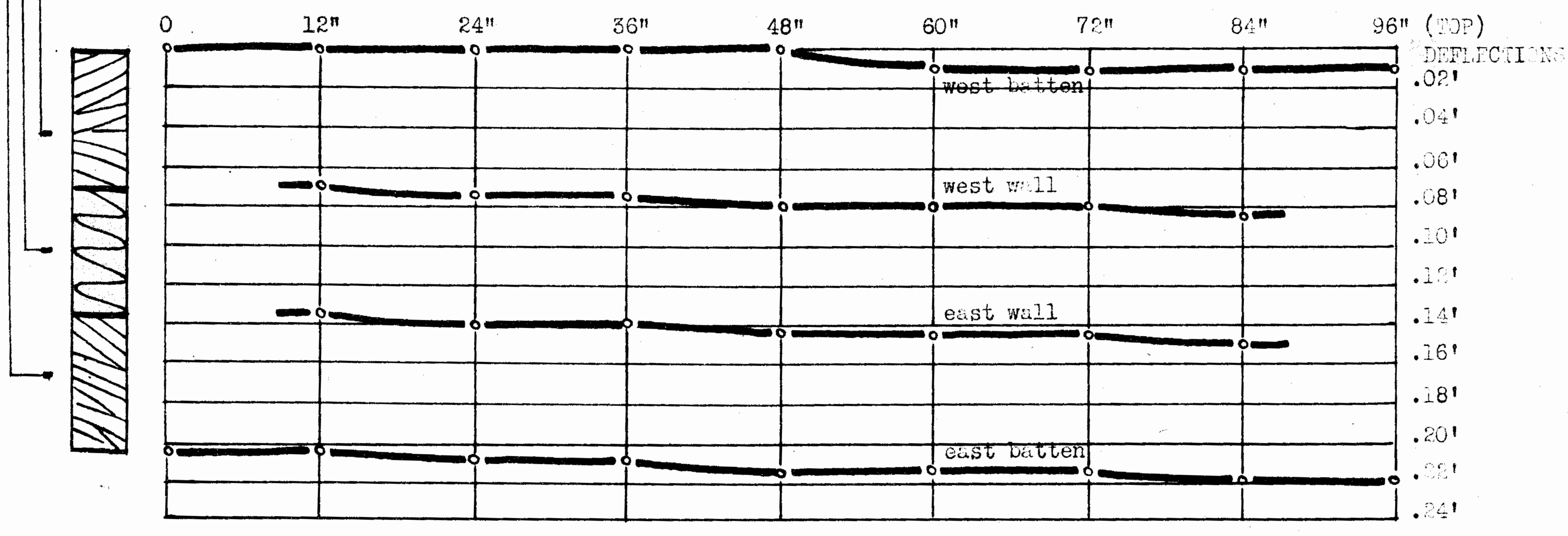
READINGS TAKEN AT STATION E



LOCATIONS OF STATIONS AT WHICH READINGS WERE TAKEN

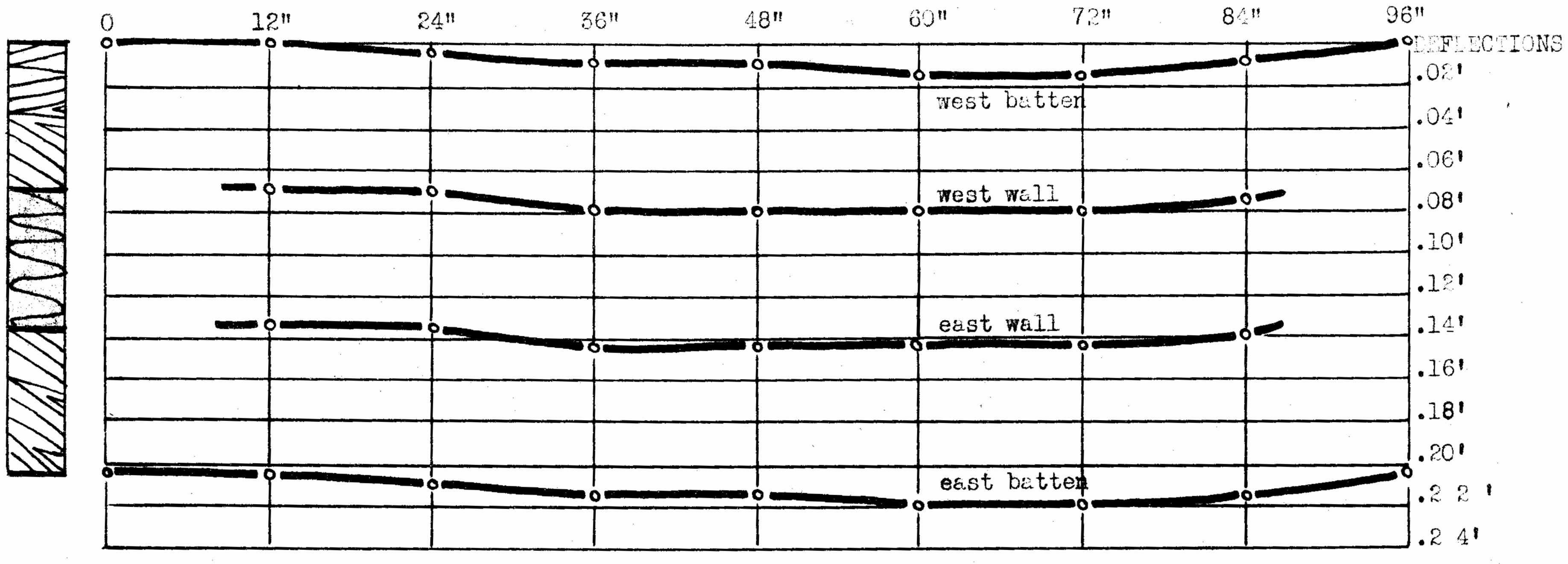
UNIT-B-15

ELEVATIONS OF READINGS



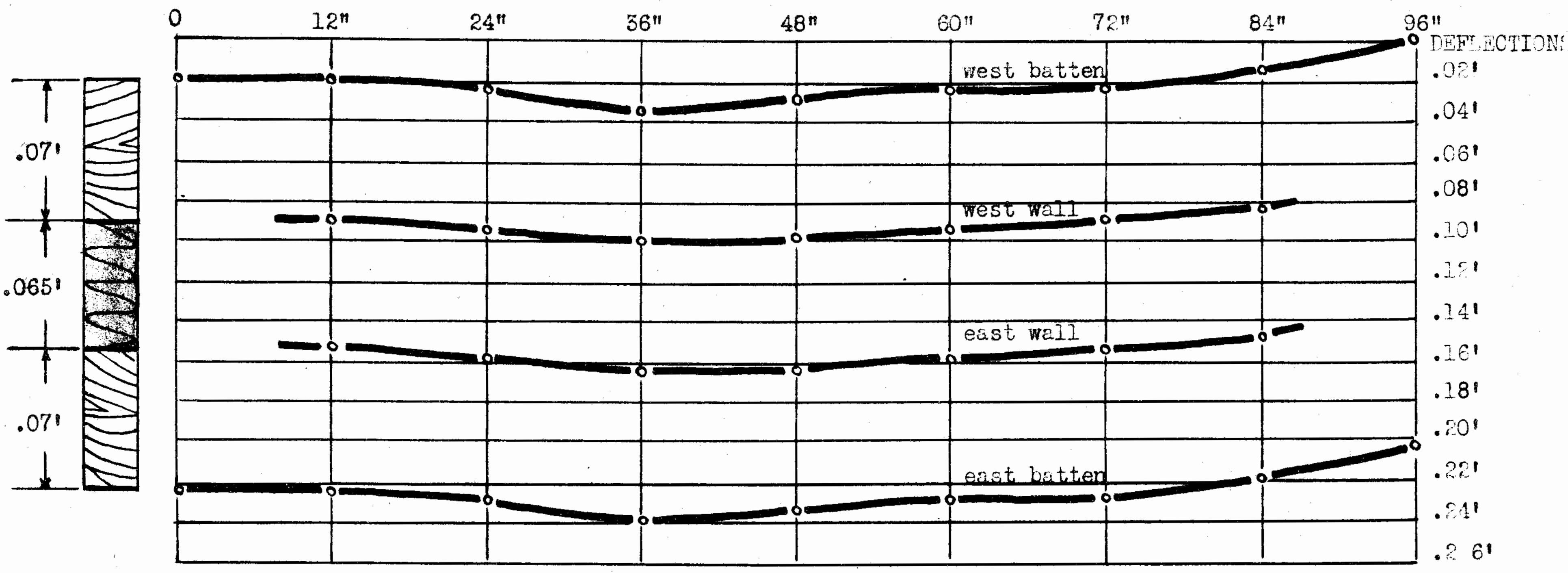
READINGS TAKEN AT STATION A

ELEVATIONS OF READINGS

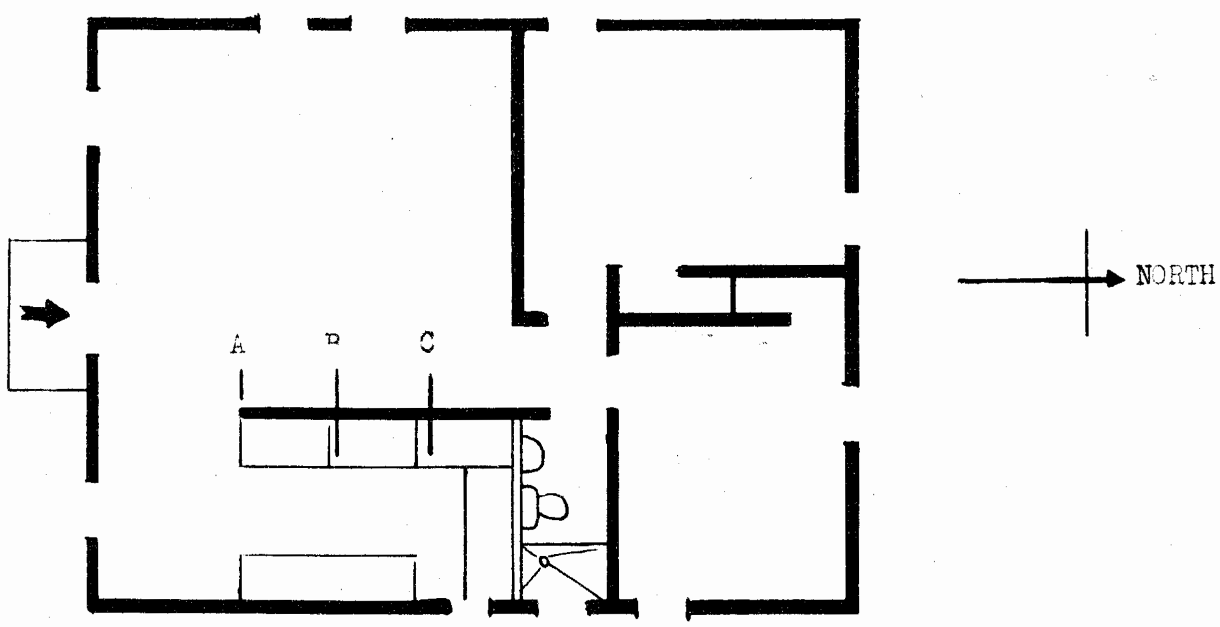


READINGS TAKEN AT STATION B

ELEVATIONS OF READINGS



READINGS TAKEN AT STATION C



LOCATIONS OF STATIONS AT WHICH PARTITION READINGS WERE TAKEN.

UNIT-B-15.

CEILING AND RAFTER DEFLECTION READINGS
UNIT B-22

The following six graphs indicate the deflections present in the ceiling and rafters employed in test Unit B-22, 20'0" x 24'0".

Readings were taken as indicated on plan showing locations of stations.

The first four graphs indicate readings taken at 24" intervals along the lower face of the 2" x 4" members acting as roof rafters over the rear portion (east half) of the Unit.

The line under each graph indicates the position and direction of the particular readings.

The inches indicate points along rafters at which respective deflections were recorded.

Deflections given are in 100ths of a foot.

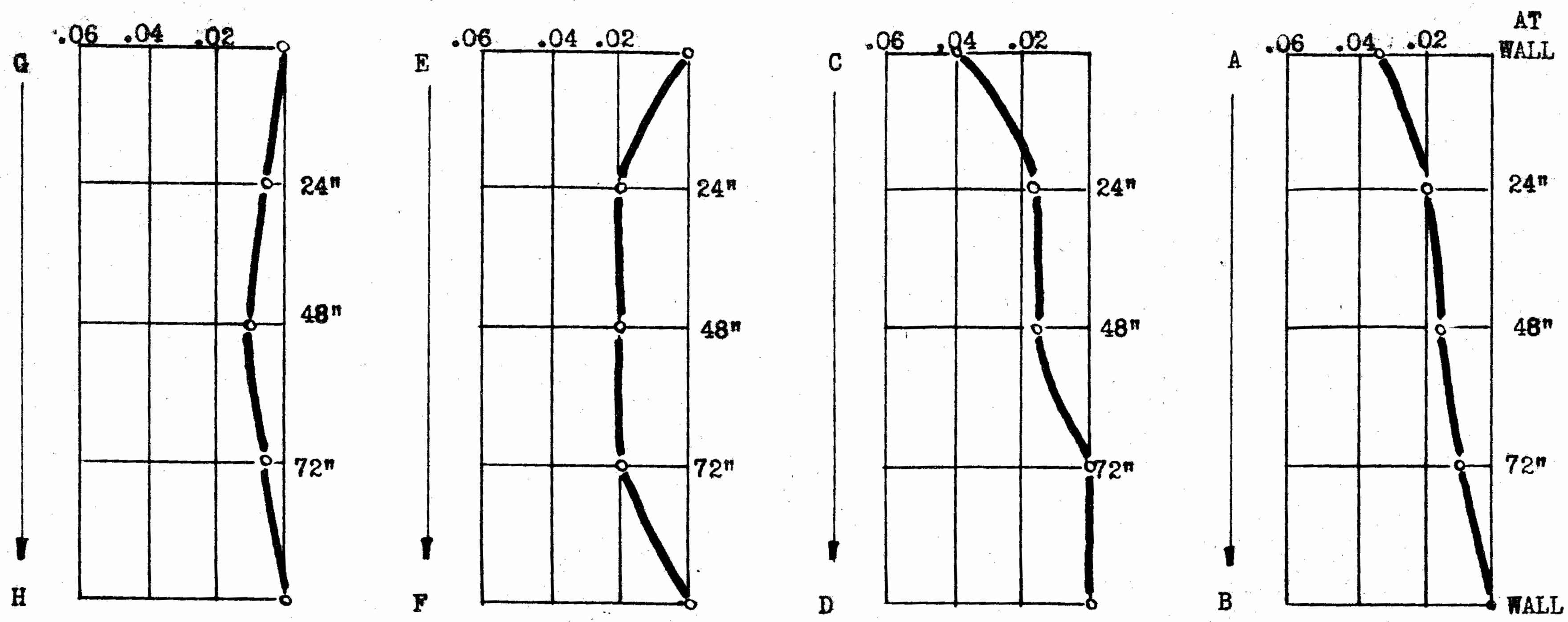
The second two graphs provide readings taken across the rafters and along the lower face of the sub-roof which acts as a ceiling in this east half of the Unit.

These readings were taken along positions indicated as K - L and M - N, at intervals as shown in sketch "Section of Ceiling."

It will be observed the odd numbered points have a definite relation to the deflection of the rafters, the readings at these odd numbered points corresponding closely to those taken along the rafters.

The line under each graph indicates the position and direction of the particular readings.

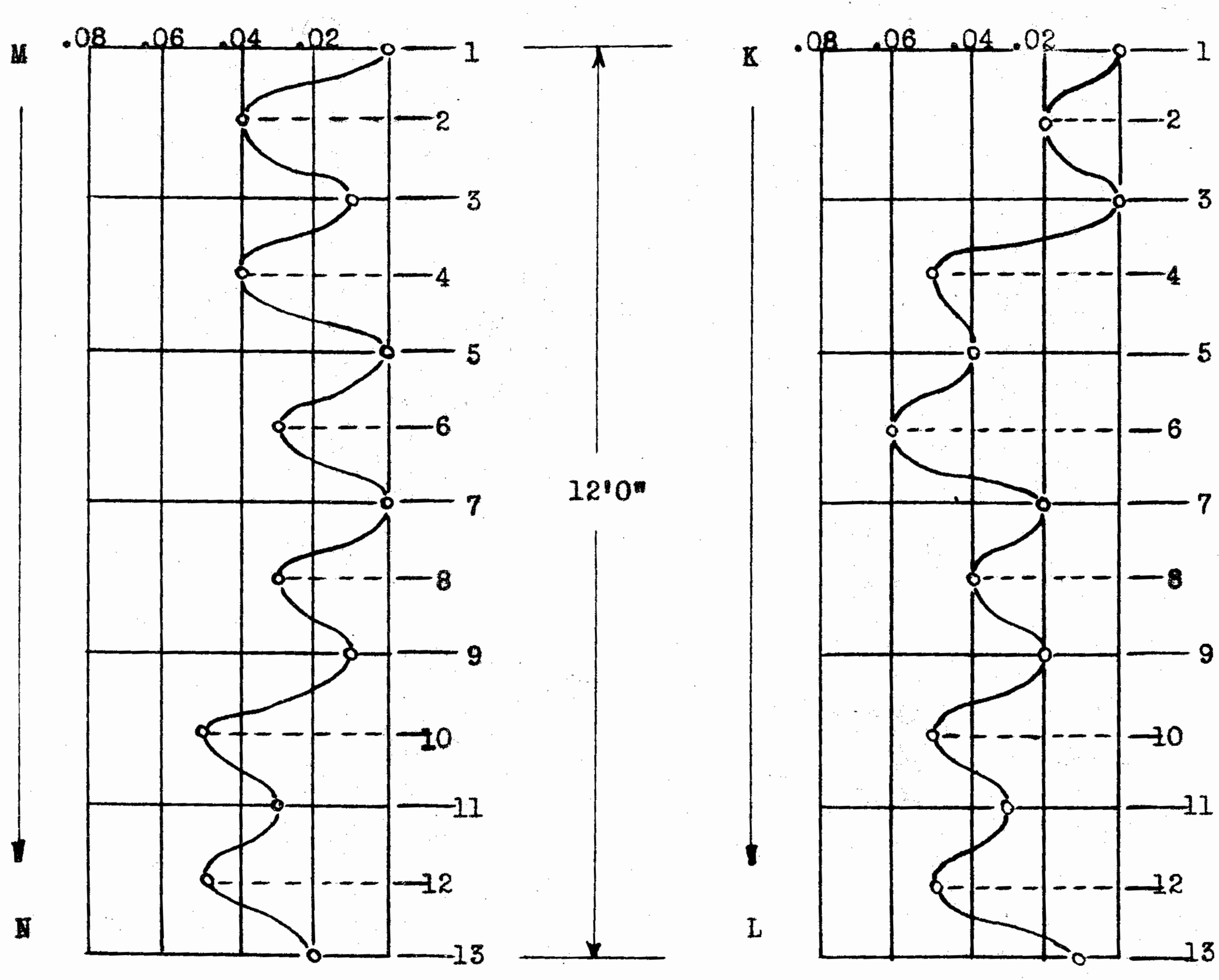
DEFLECTIONS
(IN 100ths OF FEET)



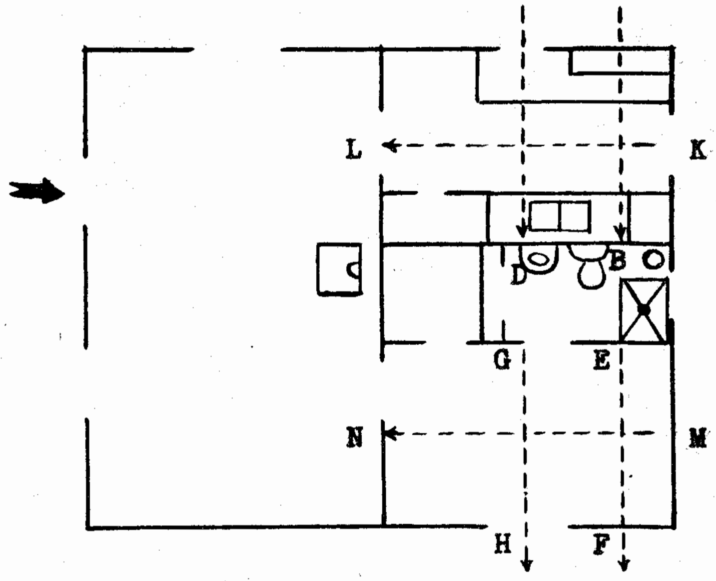
TAKEN ALONG 2" x4" ROOF RAFTERS
OVER PORTION OF FLAT ROOF - UNIT B-22

DEFLECTIONS
(IN 100ths OF FEET)

DEFLECTIONS
(IN 100ths OF FEET)



TAKEN ON CEILING ACROSS RAFTERS

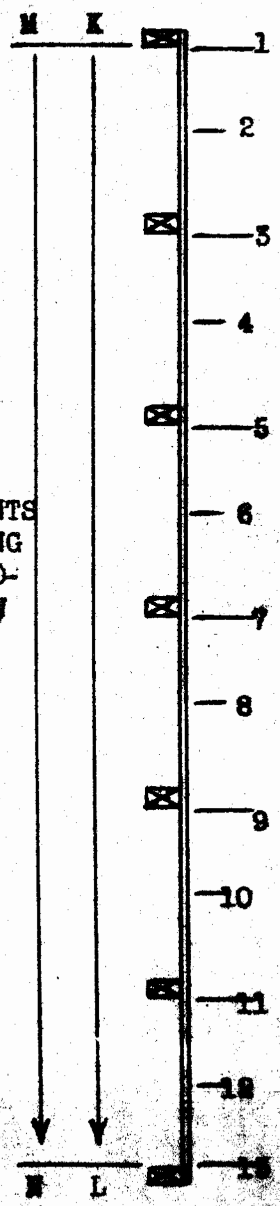


LOCATION OF STATIONS
AT WHICH CEILING AND RAFTER DEFLECTION
READINGS WERE TAKEN

UNIT B-22

SECTION
OF CEILING

INDICATING POINTS
AT WHICH CEILING
DEFLECTION READ-
INGS WERE TAKEN



WALL DEFLECTION READINGS
UNIT B-22

The following three graphs indicate the deflections present in the walls employed in test Unit B-22, 20'0" x 24'0".

The wall construction of this unit employs the materials and construction developed from wall test #144.

Readings were taken vertically along the battens both inside and outside the unit, and the results recorded in 100ths of a foot.

The small plan indicates the points or stations at which these readings were taken.

Elevations of readings indicate the distances above the base of the wall at which readings were taken, given in inches.

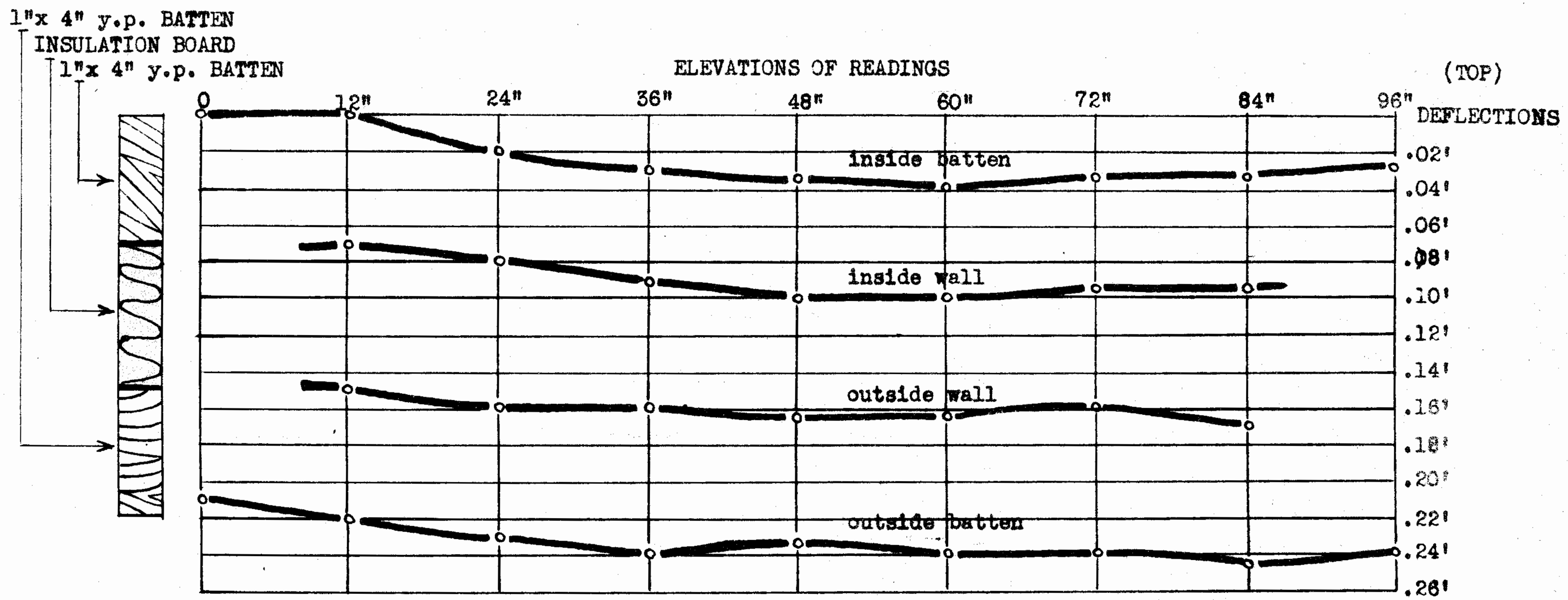
Deflections provide the scale for the deflection readings, given in 100ths of a foot.

Each graph is marked to tell the station to which it applies.

Section graphs indicate the position of the materials in the wall in relation to the larger graph. These smaller graphs also indicate the materials employed in the wall.

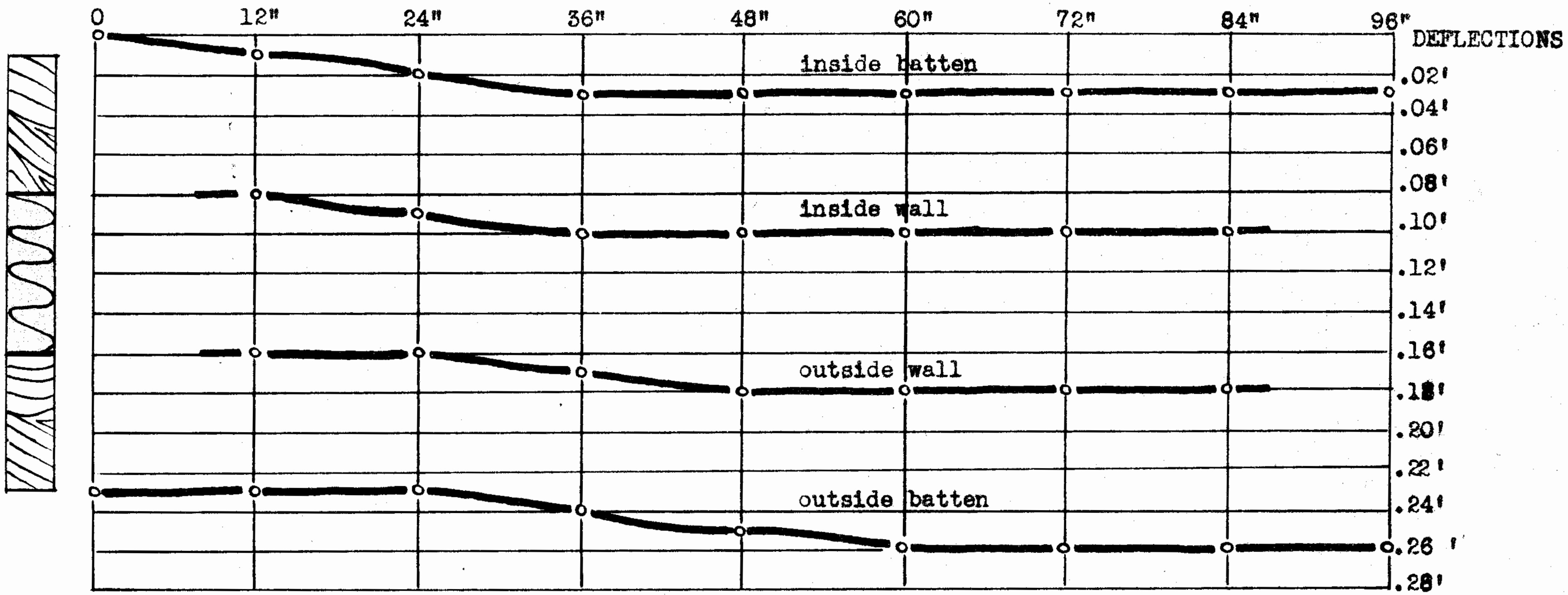
On the graph taken at Station X, the wall materials are described; on that taken at Station Z, their thickness in 100ths of a foot are given.

The lower case notes within the body of the graphs indicate the planes along which the readings were given.



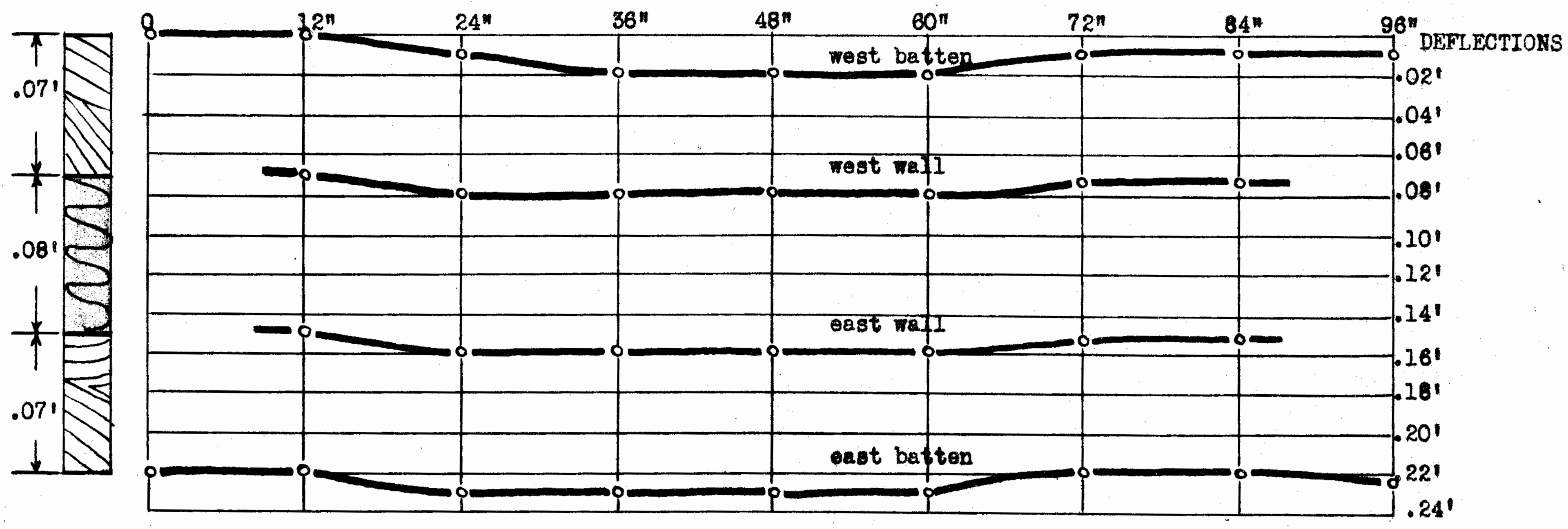
READINGS TAKEN AT STATION X

ELEVATIONS OF READINGS

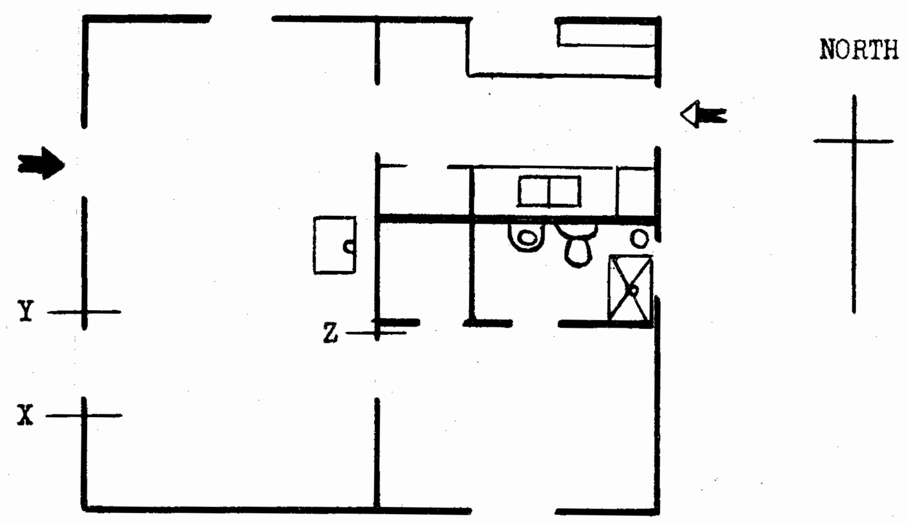


READINGS TAKEN AT STATION Y

ELEVATIONS OF READINGS



READINGS TAKEN AT STATION Z

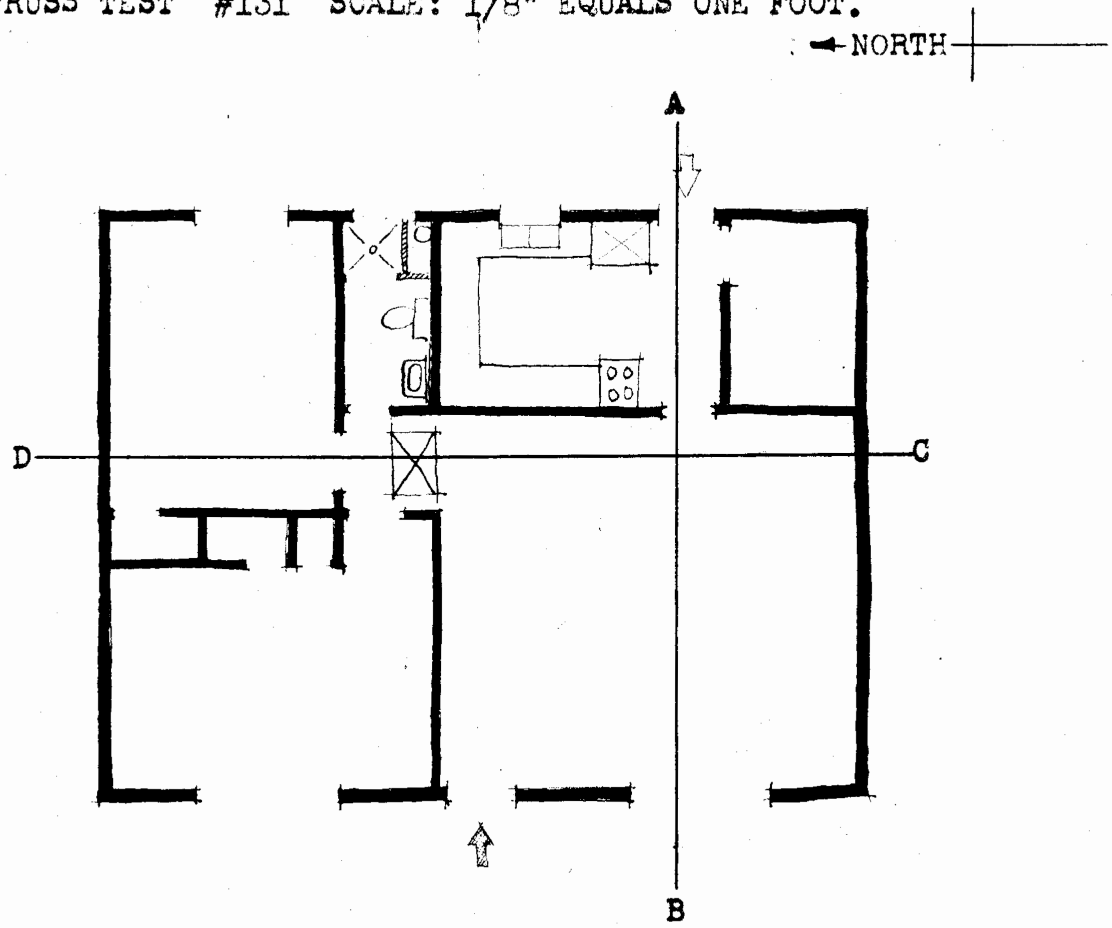


LOCATION OF STATIONS
AT WHICH WALL DEFLECTION READINGS
WERE TAKEN

UNIT - B-22

REFERENCE SKETCH
FLOOR TRUSS TEST #125
ROOF TRUSS TEST #131

IDENTIFYING LINES ALONG WHICH DEFLECTION READINGS WERE TAKEN:
SCALE: 1/8" EQUALS ONE FOOT.



UNIT B-24

24'0" x 32'0"

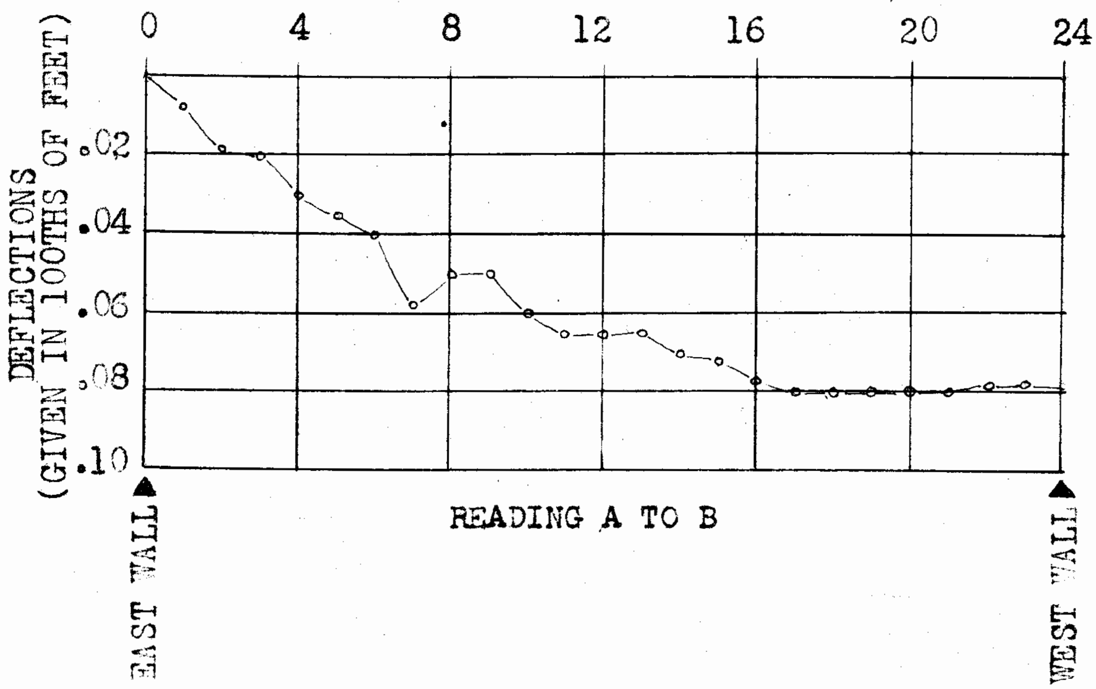
DEFLECTION TESTS
ROOF TRUSS, TEST #131

TRUSSES SUPPORTED AT ENDS.

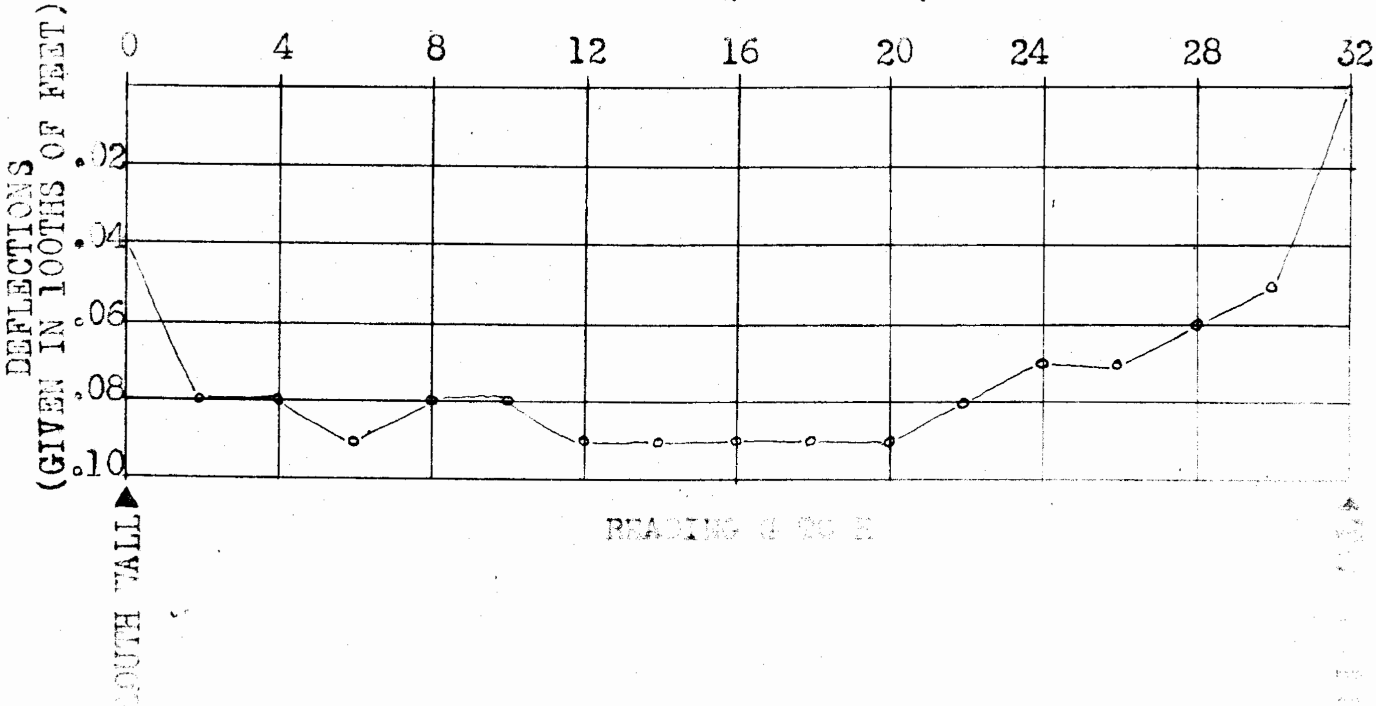
FOR DESCRIPTION OF TRUSS SEE FIG., P.48

CARRYING FULL SPAN CEILING, INSULATION AND ROOF LOADS.

CEILING DEFLECTION
SPAN (IN FEET)



CEILING DEFLECTION
SPAN (IN FEET)

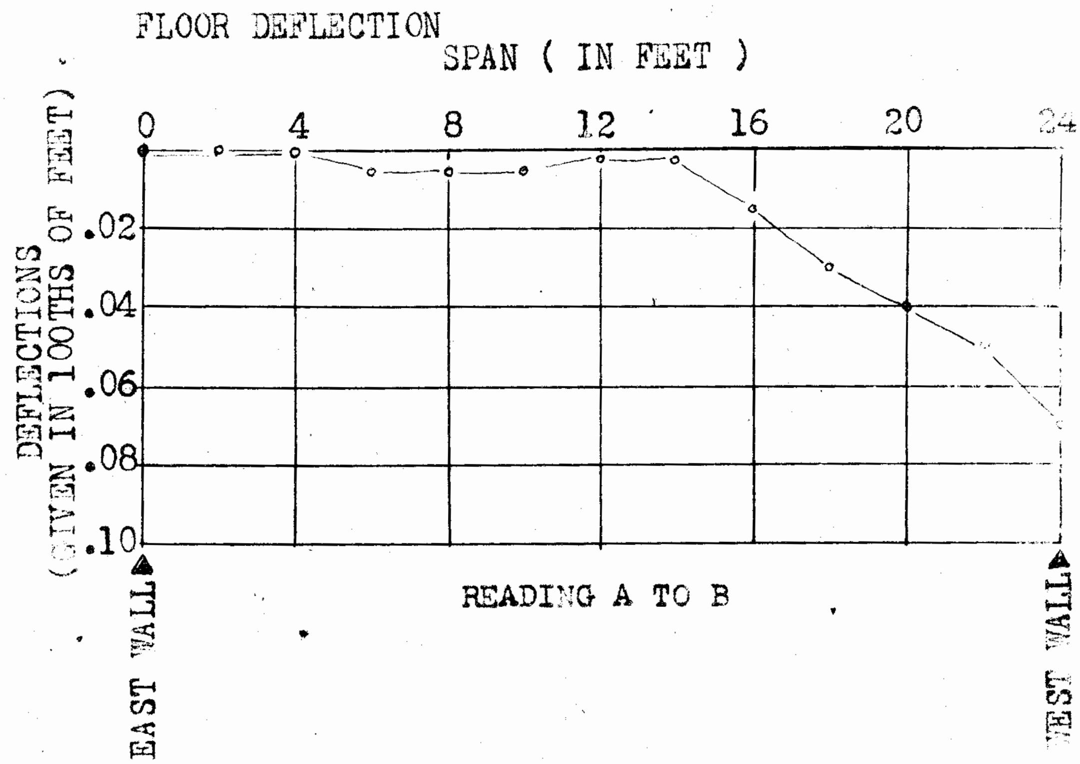


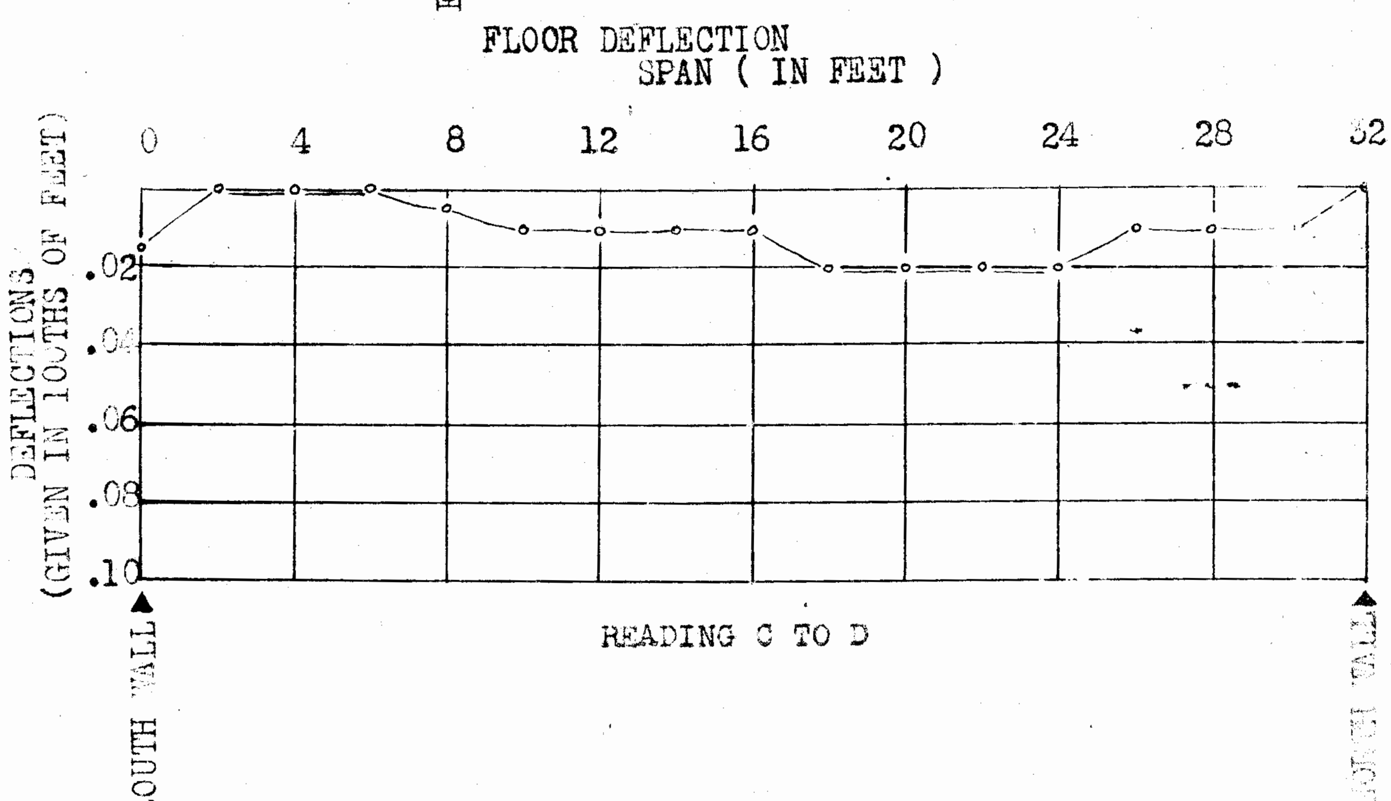
DEFLECTION TESTS
FLOOR TRUSS TEST # 125

FOR DESCRIPTION OF TRUSS SEE FIG., P.35

TRUSSES SUPPORTED AT [illegible]

NO ROOF OR CEILING LOADS. CARRYING 2" THICKNESS OF GYPLAP.





Reference sketch for Unit-B-24 identifies lines along which readings were taken.