A ROOF FRAMING SYSTEM FOR STORY AND ONE-HALF RESIDENCES

Thesis for the Degree of M. S.

MICHIGAN STATE COLLEGE

James Winthrop Goff

1952

This is to certify that the

thesis entitled

A Roof Framing System for Story and one-half Residences

presented by

James W. Goff

has been accepted towards fulfillment of the requirements for

Master of Sci. degree in Wood Technology

A. J. Panshin
Major professor

Date May 23, 1952

O-169

A ROOF FRAMING SYSTEM FOR STORY AND ONE-HALF RESIDENCES

By

James Winthrop Goff

A THESIS

Submitted to the School of Graduate Studies of Michigan

State College of Agriculture and Applied Science

in partial fulfillment of the requirements

for the degree of

MASTER OF SCIENCE

Department of Forest Products

1952

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ACKNOWLEDGEMENTS

The author wishes to express his sincere thanks to Dr. A. J. Panshin, Head of the Department of Forest Products, under whose guidance this project was carried out for his interest and continuing encouragement. Thanks are also due to Mr. William B. Lloyd, Assistant Professor of Forest Products, and Mr. John L. Hill, Instructor in the Department of Forest Products for their helpful suggestions and assistance at various times.

INTRODUCTION

The idea for this design was born because of the curiosity of the writer as to the cause of plaster cracks in the ceilings of story and one-half houses. These cracks seemed to occur almost always directly under the knee-wall, which serves as a short vertical partition in the upper rooms of such houses, as shown in Figure 1. Little investigation was required to ascertain that these defects were caused by excessive deflection in the ceiling joists or second floor joists which, through the knee-wall, were carrying a portion of the roof load not meant to be supported by these joists. Under normal conditions of loading the excess deflection because of the roof load was negligible, but after severe snowstorms or during high winds, the roof load increased to an extent sufficient to cause enough deflection to crack the ceiling plaster on the first floor.

The direct solution to this problem lies in merely increasing the size of the second floor joists sufficiently to carry this additional load without excess deflection. In the analysis of this problem several difficulties are encountered, however. The roof rafter in this type of house is an indeterminate beam resting on four unequally spaced supports. It is a difficult matter to determine the deflection of the rafter alone, not to mention the difficulty of determining the deflection of the rafter and joist in combination as they exist in the story and one-half building. Such a solution is possible, but impractical, since each case would involve different values and, as a consequence, entirely new computations.

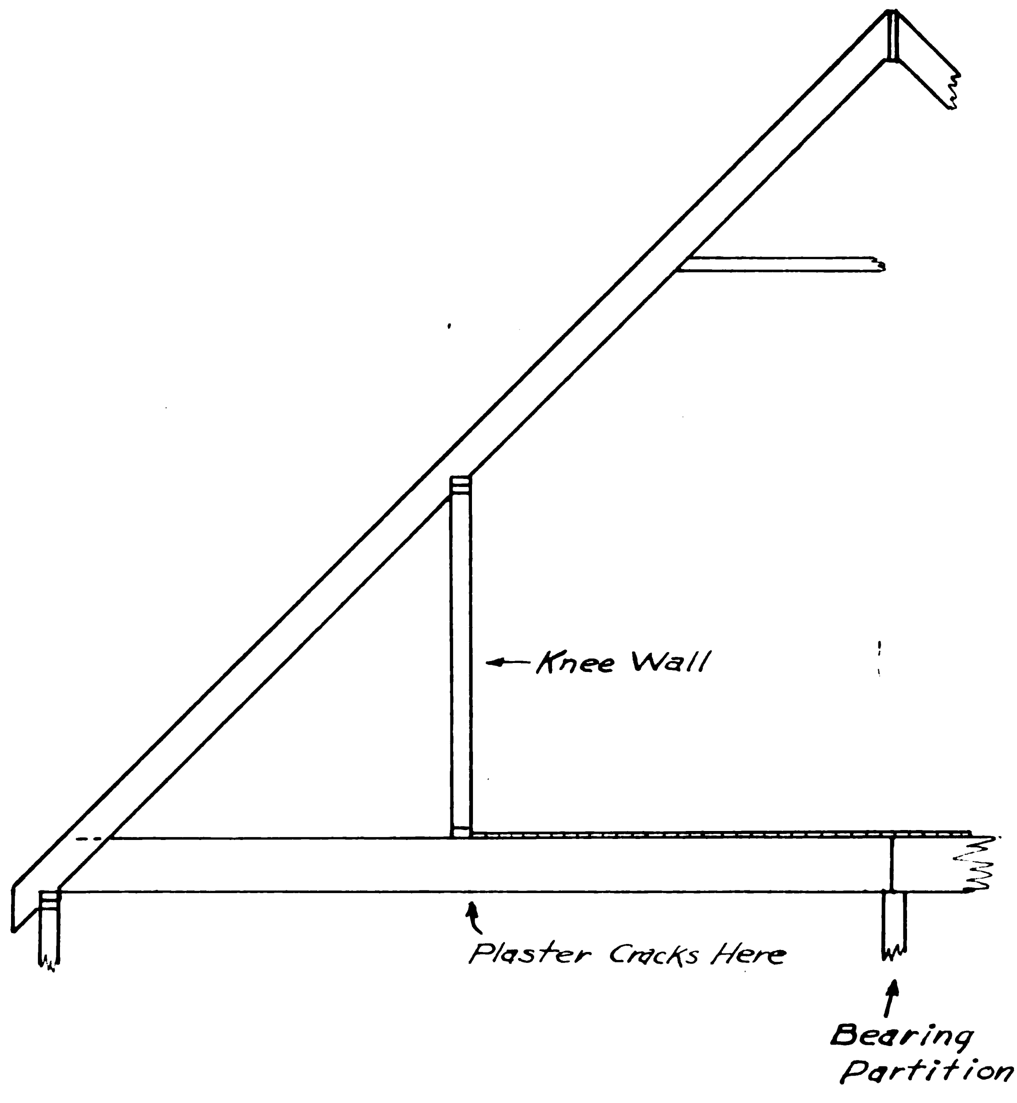

Figure 1.. Conventional $1\frac{1}{2}$ Story Frame

The second approach to this problem was to investigate the design of an indeterminate truss, with the members thereof loaded both axially and in bending. Such a truss would be very similar in appearance to the frame in common use except that each rafter joist and knee-wall member would be parts of a rigid continuous frame which could be prefabricated and erected in place as a unit. Such a design presents the desirable possibility of eliminating bearing partitions in the center of the building such as is currently being done in single story houses with trussed rafters. When this proposition was more closely analyzed several difficulties arose, however. Prominent among these was the difficulty of making suitable joints between the rafter and the knee-wall member, and between the joist and the knee-wall member. The required end distances for timber connector joints prohibited the use of these devices, and other methods did not provide sufficient strength. Another problem was the necessity for either obtaining extra long ceiling joists or splicing the joists between the outer supports. The creation of a moment resisting splice between two joists is difficult but would be necessary in this case because the members are loaded transversly as well as axially. In addition, the indeterminate truss would be highly inflexible in use. If it were desired to install dormers on such a roof frame, great difficulty would be encountered where truss members would have to be severed. It would even be a problem to install a flight of stairs leading to the second floor since this would involve cutting the lower chords of several trusses. The indeterminate truss was dropped as a possible design because of the above difficulties in analysis and in use.

The third solution, which is developed fully in the body of this thesis, lies in the imposition of the entire second floor load, a major portion of the first floor ceiling load, and a major portion of the roof load upon two plywood girders or beams running the full length of the building in the position usually occupied by the knee-wall. In the adoption of such a design it is entirely possible as was mentioned above to do away with the conventional bearing partition in the house, since the entire floor load is carried by the girders to the end walls of the building. The elimination of this central partition allows economies in construction methods as well as gives the designer greater freedom in use of the available space. A schematic cutaway view of this proposed roof framing system is shown in Figure 2.

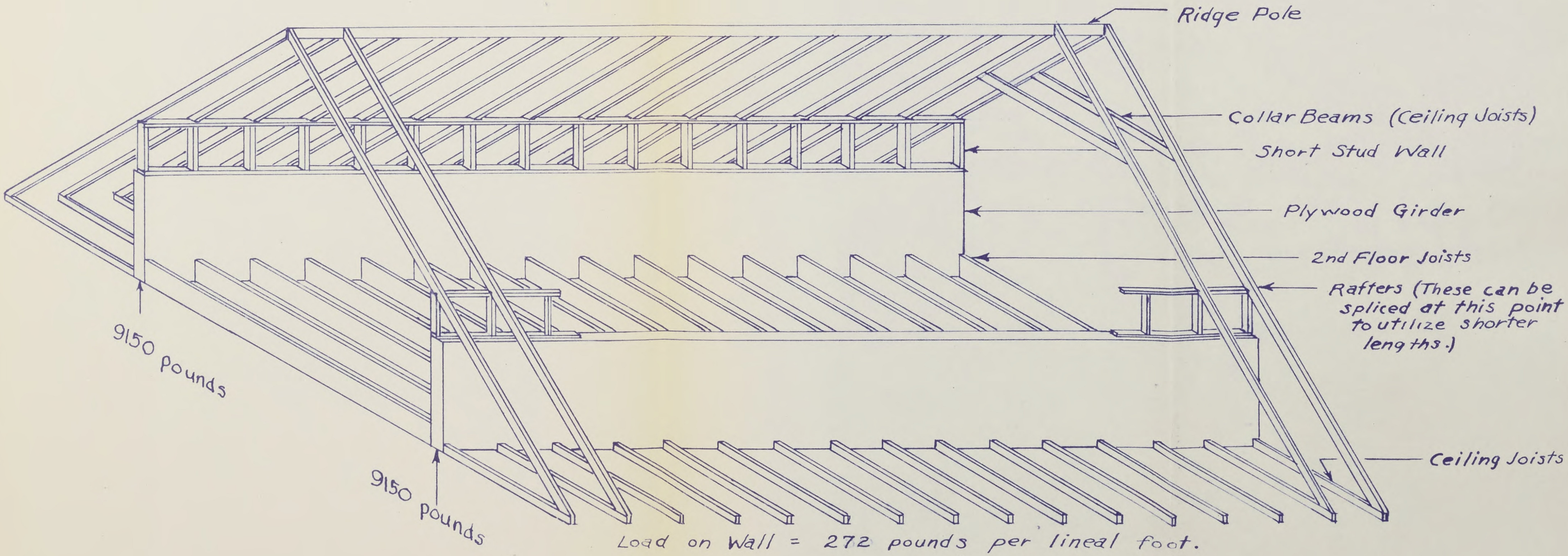

Figure 2. Oblique Cut-away View of One and One Half Story Framing System

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ANALYSIS OF THE PROPOSED SYSTEM OF FRAMING

For purposes of illustration of the method roof and floor loads which are in common acceptance by building authorities were selected for use in the following analysis and the subsequent design of the members. A roof load of 38 pounds per square foot of surface acting normal to the roof surface was chosen. This total roof loading is composed of eight pound dead load combined with a 30 pound live load which is assumed to be either wind or snow or a combination of the two. The second floor load is assumed to be 50 pounds per square foot, a combination of dead and live loads. That portion of the first floor ceiling which is carried by the lighter ceiling joists outside of the girders is assumed to impose a total dead load of 14 pounds per square foot on its supports. It will also be assumed for purposes of analysis and design that the maximum allowable deflection is 1/360th of the shortest span. Since the dimensions of the roof which will be used in the proposed design are 24 feet by 30 feet, the maximum allowable deflection will be 0.80 inches.

Figure 3 is a schematic diagram of the distribution of the roof and floor loads to the girders. It is assumed for purposes of design that the entire vertical component of the load imposed on the upper portion of the roof is carried by the girders. This assumption results in maximum possible load on the girders and is entirely satisfactory for the purposes of design. From the information given in Figure 3, the

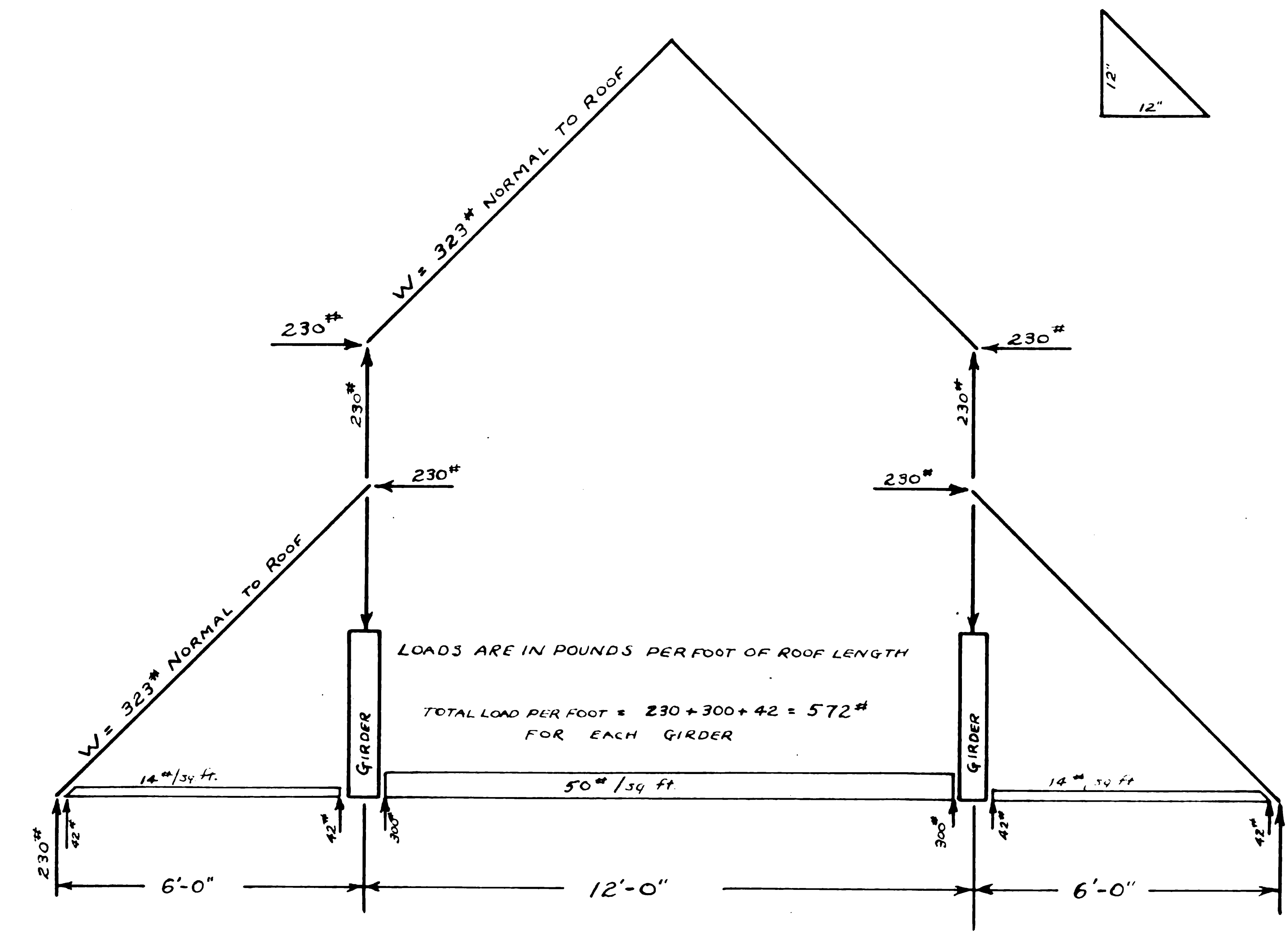

FIGURE 3. TRANSVERSE SECTION SHOWING LOADS CARRIED BY GIRDERS

beam diagram, the shear diagram, and the moment diagram for the girder can be plotted. These diagrams are shown in Figure 4. The weight of the girders has been assumed to be 38 pounds per lineal foot, and this has been considered in plotting the diagrams.

The rafters, as indicated in Figure 2, are of two by four inch material, and are spaced two feet apart as are all other framing members of the proposed system. To utilize standard lengths of lumber, the rafters should be spliced where they bear on the knee-wall, as noted in Figure 2. If this is not done, eighteen foot rafters will be required. The ceiling joists which lie outside of the girders are also of two by four inch stock. It can be readily assumed that neither of these members is critical, but if the reader is curious standard references can be consulted. (4,5) The second floor joists on the other hand are the critical members, not only because they carry the greatest load, but also because they have the greatest span. In addition the deflection of those two joists which straddle the center line of the beam will determine the maximum permissable deflection in the girders themselves. This deflection is found to be 0.255 inches at the center of a two by ten inch joist loaded uniformly at 100 pounds per lineal foot. If this deflection is subtracted from the total allowable deflection of 0.80 inches, the allowable deflection for the plywood girder at midspan is 0.545 inches. In order that the total deflection will not exceed the allowable it will be necessary to limit the deflection in the girder to 1/600th of the span.

As indicated from the diagram of the proposed system, Figure 2, the bearing load at each end of the girder is 9,150 pounds. The actual length of the bearing provided by the two by four plate common in

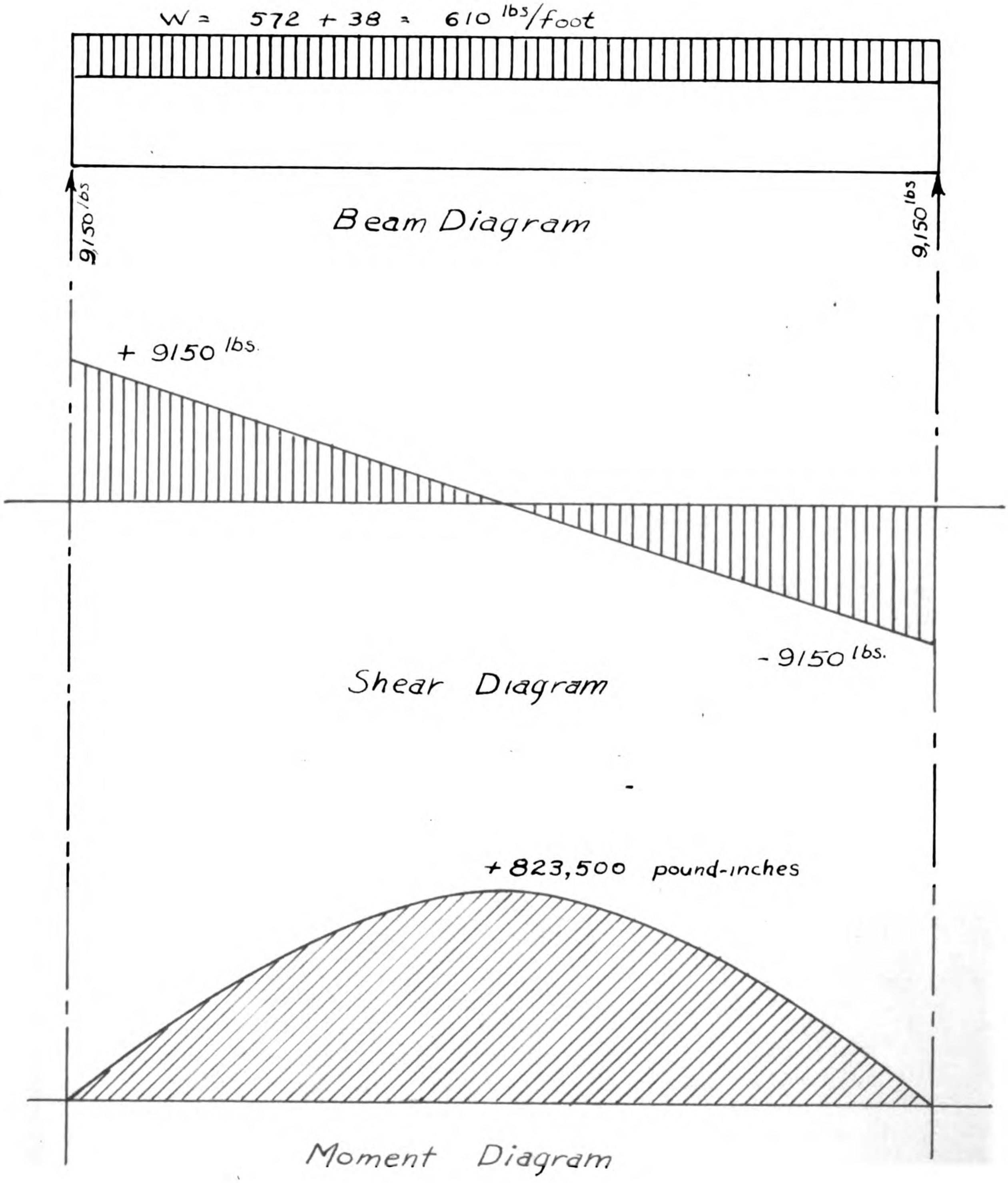

Figure 4. Beam, Shear & Moment Diagrams

residential construction is 3.625 inches, consequently the beam will have to be of sufficient width to create enough bearing area to support the load. In a similar manner the stud wall must be reinforced at the point beneath the bearing points with a timber of sufficiently large cross sectional area to carry the transmitted load. It must also be provided with a large enough bearing area on the underside of the plate. It will be found (4,5,6) that a four by six inch timber framed into the wall at each of these points will possess sufficient column strength to support the beam loads.

DESIGN OF PLYWOOD GIRDER

With the span and the load carried by each of the girders known, the design of the beam can now be undertaken. Several different procedures are involved so the task will be broken down into several steps. The steps will be summarized here and then taken up in their proper order.

The first step is the selection of a trial section for the beam. The shear and moment diagrams are plotted so that the needed information will be at hand. The grade of lumber to be used in the flanges should be chosen at this time, and it is convenient to list all the stress values of that lumber grade for use in the following steps of the calculation. With the maximum bending moment, the maximum shear force, the allowable extreme fiber stress, and the limit of deflection known, a section which satisfies these conditions can be computed.

The second step involves a checking of the trial section against the allowable stresses, and the calculation of the actual deflection from bending and shear. To accomplish this it is necessary to compute the exact form factor and the section constant for shear deflection.

The third and last step is the design of joints and splice plates, and the design of stiffeners. It may also be necessary in this step to design the braces for the top flange to provide lateral stability for the beam.

Design of a Trial Section

The shear and moment diagrams for the beam are plotted in Figure 4, and the allowable deflection is, as calculated previously, 1/600th of the span. The lumber to be used in the flanges is Coastal Douglas Fir, Select Structural Grade No. 1, as graded by the West Coast Bureau of Lumber Grades and Inspection. This grade has the following allowable unit stresses. (5)

Extreme fiber in bending (f)	1450 psi
Horizontal shear (H)	120 psi
Compression perpendicular to grain (c)	390 psi
Modulus of Elasticity (E)	1.6×10^6 psi

The plywood to be used for the webs is unsanded Douglas Fir exterior grade, sound two sides, and the allowable unit stresses of interest here are: (1,2)

Shear, in plane perpendicular to plies and parallel or perpendicular to face grain	210 psi
Shear, rolling, in plane of plies, parallel or perpendicular to face grain	54 psi

With these values established the computation of the trial section can begin. For any beam the flexure formula $\underline{M} = \frac{FfI}{c}$ is applicable, where $\underline{M}$ is the bending moment as found when plotting the moment diagram, $\underline{F}$ is the form factor depending on the section of the beam, $\underline{f}$ is the allowable extreme fiber stress in bending, and $\underline{c}$ is the distance of the extreme fiber from the neutral axis. The form factor is usually neglected in solid beams because it is automatically compensated for in deriving the allowable stresses. (3) The form factor for the trial section will be assumed to be 0.80. (1) The deflection formula, $d = \frac{5WL^3}{384\ E\ I}$, is also applicable. It is necessary, however, in designing the trial section to make allowance for shear deflection by increasing the value of the deflection thus found by 25 percent. Since the deflection is limited to 1/600th of the span it is possible to write the above deflection equation as follows: $\frac{L}{600} = \frac{1.25 \times 5\ W\ L}{384\ E\ I}$. Further, since M in this case is equal to $\frac{WL}{8}$, it is possible to write the flexure formula as $WL = \frac{16FfI}{h}$, where h equals 2c or the height of the beam. If the deflection formula above is also solved for WL, the formula $WL = \frac{384\ EIL}{1.25 \times 5 \times 600 \times L^2}$ is obtained. These two equations can then be set equal and solved for L/h and will give a ratio of beam span to height that will satisfy both flexure requirements and deflection requirements. For the beam in question, the L/h value calculation follows:

$$\frac{L}{h} = \frac{384(1.6 \times 10^6}{60,000\ (0.8)\ (1450)} = 8.8$$

For the span of 30 feet then:

$$h = \frac{30 \times 12}{8.8} = 41 \text{ inch minimum}$$

Standard four-foot wide plywood sheets will be used in the beam and will provide a beam depth of 48 inches.

To determine the thickness of the webs the formula (1,5) $t = \frac{1.25\ V}{hv}$, where V is the shear on the section in pounds, h is the depth of the beam section in inches, and v is the allowable plywood shear stress perpendicular to the plane of the plies in pounds per square inch.

Applying this to the problem:

$$t = \frac{1.25 \times 9,150}{48 \times 210} = 1.132 \text{ inch}$$

It is apparent that two webs of 5/8th inch plywood can be used to make a total web thickness of 1.25 inches. This plywood is found to have a total of 5 plys, each ply 1/8th inch thick. Three of these plies are parallel to the long dimension of the piece for a total area of parallel plies of 0.375 square inches per inch of height.

With the webs selected it is now possible to compute the trial flange dimensions. The required moment of inertia I can be computed from the flexure formula and is found to be:

$$I = \frac{Mc}{Ff} = \frac{823{,}500 \times 24}{0.8 \times 1450} = 17{,}100 \text{ inches}^4$$

It is also necessary to compute a moment of inertia from the deflection formula:

$$I = \frac{1.25(600)5ML^2}{384\ E} = \frac{1.25 \times 5 \times 600 \times 18{,}300 \times 360^2}{384\ (1.6 \times 10^6} = 14{,}500 \text{ inches}^4.$$

It is evident that the I as determined from the maximum bending moment governs the design of the beam. If the I provided by the plywood webs is now calculated, the remaining I, which must be provided by the flanges, can be determined.

$$I_{webs} = \frac{2 \times 0.375 \times 48^3}{12} = 6{,}950 \text{ inches}^4$$

$$I_{fl} = I_b - I_{webs}$$

$$I_{fl} = 17{,}100 - 6{,}950 = 10{,}150 \text{ inches}^4$$

By rearrangment, the formula for the moment of inertia will yield the following:

$h_1^3 = h^3 - \frac{12\ I}{w}$, where h_1 is the clear distance between flanges and w is width of each flange, while I is the moment of inertia which must be provided by the flanges.

At this point it becomes necessary to assume a width (w) for the flanges. Since it will not be possible to obtain scarf jointed flange members in field operation, sufficient laminations should be provided to make staggered butt joints possible. A trial flange width of $4\frac{7}{8}$ inches, which will be laminated from three $1\frac{5}{8}$ inch pieces is selected. If the above equation for h is solved using the values available the following result is obtained.

$$h_1^3 = 48^3 - \frac{12 \times 10{,}150}{4.875}$$

$h_1 = 44.1$ inches between flanges.

Solving for flange depth (d), we find:

$$d = \frac{48 - 44.1}{2} = 1.95 \text{ inches}$$

Since the nearest standard lumber size which meets or exceeds these requirements is the 2 x 4, the flanges will be laminated from three 2 inch x 4 inch nominal pieces. Sixteen foot lengths will be used for the laminations and the joints will be staggered along the beam by at least two feet. The trial section has now been completely designed. A scale drawing of this section is shown in Figure 5. The various ratios and constants indicated thereon are used to find the exact form factor and section constant for shear deflection and will be explained when those functions are used.

Accurate Design Check of the Trial Section

Since the section has been changed in the flanges from that for which the modulus of inertia was first calculated, it is now necessary to calculate the exact I of the beam that has been adopted. The I as computed for the webs has not changed, but the I for the flanges has

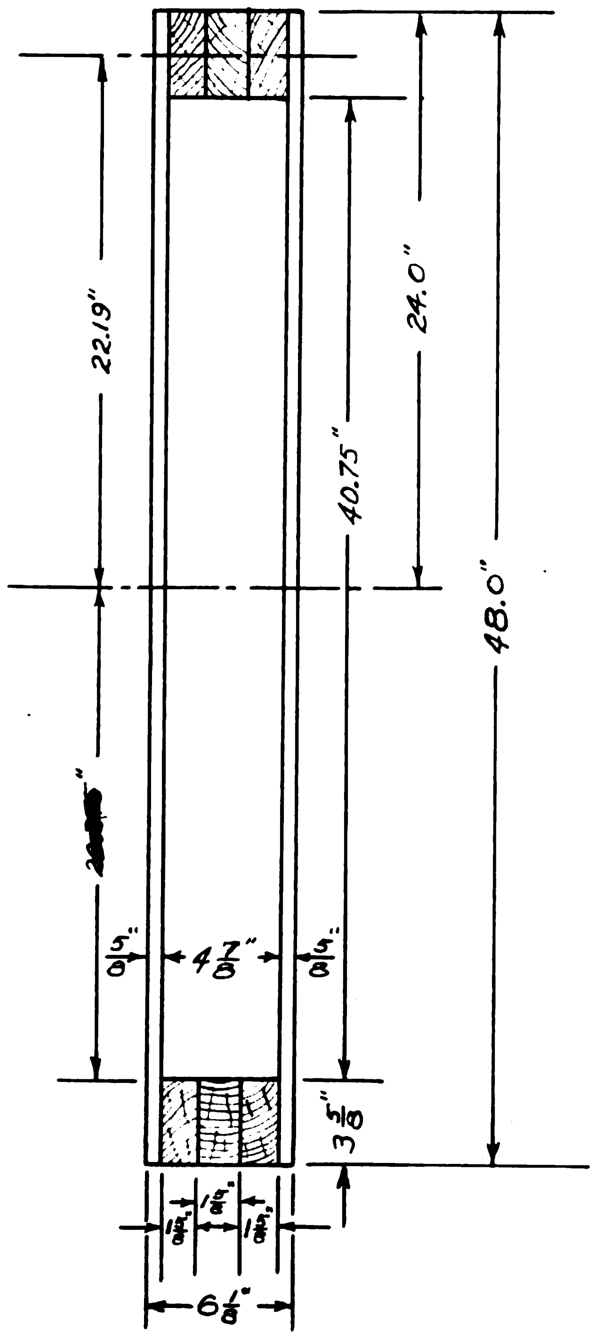

FIGURE 5. SECTION OF GIRDER

because deeper flanges than were needed are being used to correspond with stock sizes of lumber. The dimensions for this calculation are indicated in Figure 5.

$$I = I \text{ webs} + I \text{ flanges}$$
$$= 6{,}950 + \frac{4.875(48^3-40.75^3)}{12}$$
$$= 6{,}950 + 17{,}500 = 24{,}450 \text{ inches}$$

Computing the form factor (6,7)

$$d/h = 3.625/48 = 0.0755$$
$$t_1/t_2 = 1.25/6.125 = 0.204$$
$$F = 0.67 \times 10/9 = 0.74$$

The allowable bending moment on the beam is then:

$$M = \frac{FfI}{c} = \frac{0.74\ (1450)\ (24{,}450)}{24} = 1.09 \times 10^6 \text{ pound inches.}$$

This is well above the applied moment.

It next becomes necessary to check the beam for resistance to horizontal shear. To do this the value of (Q) which is the statical moment of all fibers lying outside the neutral plane must be calculated.

$$Q = 2(0.375)\ (24)\ (12) + 3.625\ (4.875)\ (22.19)$$
$$= 216 + 390 = 606 \text{ inches}^3$$

The maximum allowable shear in the beam can then be calculated from the standard shear formula:

$$V = \frac{vIt}{Q} = \frac{210(24{,}450)\ (1.25)}{606} = 10{,}600 \text{ pounds}$$

This allowable shear value is well above that imposed on the beam.

Since plywood is relatively weak in shear between the plies (rolling shear) the shear developed between the webs and the flanges is of great importance. Because the shear acts on a concentrated area of the web the allowable value will be cut to half of that indicated in the table of values given previously. This gives an allowable rolling shear stress of 27 pounds per square inch in this case. The maximum allowable vertical shear in the beam on the basis of the allowable rolling shear stress is given by the formula:

$$V = \frac{sId}{Q_{fl}} \times \frac{\Sigma t}{t_1} = \frac{27(24,450)\ (3.625)}{390} \times \frac{\Sigma 1.25}{0.625} \quad 12,300 \text{ pounds.}$$

This allowable vertical shear value is also well above that which is applied to the beam under maximum load.

The deflection of the beam must be checked to ascertain if it is within the design limits. Deflection due to the bending moment is computed using the equation:

$$d_b = \frac{5\ W\ L^3}{384\ E\ I} = \frac{5 \times 18,300 \times 360^3}{384 \times 1.1 \times 1.6 \times 10^6 \times 24,450} = 0.245 \text{ inches}$$

The factor 1.1 which appears in the denominator of the equation above is necessary to convert the $\underline{E}$ as obtained from static tests of small specimens to the true modulus of elasticity. It is customary to neglect this conversion because of the fact that in ordinary beam calculations shear deflection is not considered by itself since the value of $\underline{E}$ as used is computed from total deflection of the static specimen and this deflection will include shear deflection. In the case at hand shear deflection will be computed separately and it is therefore necessary to exclude it from the equation for bending deflection.

Shear deflection is computed from the formula:

$d_s = \frac{PlKh^2C}{GI}$, where $\underline{P}$ is the total in pounds,

$\underline{l}$ is the span in inches.

$\underline{K}$ is the section constant for shear deflection. (1)

$\underline{b}$ is the depth of the beam, in inches.

$\underline{G}$ is the shearing modulus of the webs (Douglas Fir Plywood). $\underline{G}$ equals 117,000 psi when the face grain is parallel or perpendicular to the span.

$\underline{C}$ is a constant depending on the manner in which the beam is loaded.

The values of $\underline{K}$ and $\underline{C}$ for various conditions have been computed and plotted and the charts are available in the references. (1) $\underline{K}$ is related directly to the quantities d/h and t_1/t_2, and for the values of these functions found previously K = 0.64. $\underline{C}$ is the load coefficient and for simply supported uniformly loaded beams $\underline{C}$ is equal to 0.05. If this formula is used with the appropriate values as previously given the following shear deflection is obtained.

$$d_s = \frac{FlKh^2C}{GI} = \frac{18,300(360)\ (0.64)\ (48^2)\ (0.05)}{117,000\ (24,450} = 0.169 \text{ inches}$$

Total deflection is then equal to the sum of the two deflections or 0.245 + 0.169 = 0.414 inches. This is well within the allowable deflection of 0.455 inches.

Design of Joints

Joints in the web will necessarily be butt joints because of the difficulty of obtaining scarfed panels of the required length and because of the impracticability of scarfing plywood panels in the field. The stresses will be carried through these butt joints by means of splice plates running the full depth of the girder on both sides of each web. The general considerations governing such splice plates are as follows: (1) the face grain of the splice plates should be parallel with the span, the total thickness of the plies parallel to the face plies in both splice plates should equal or exceed the total thickness of the plies parallel to face plies of the web, and the sum of the thicknesses of two splice plates should at least equal the total thickness of the web.

It was found earlier that the webs would consist of five-eighths inch plywood which has three one-eighth inch plies parallel to the face ply. From this information we may deduce that the splice plates must be no thinner than five-sixteenths of an inch and that each must have at least three-sixteenths of an inch in plies parallel to the face grain. From the tables available on the veneer areas of plywood constructions it is rather easy to select the proper material for this purpose. (1,7) While certain panels of five-sixteenths of an inch in thickness may be adequate, it seems appropriate to use a heavier construction, and three-eighths inch unsanded three-ply splice plates will be used. Since the stress is transmitted from the web to the splice plate through a glue joint, it is necessary to design this joint with an area adequate to transmit the developed stresses. It is assumed that the maximum stress in both bending and shear will act on the same place in the beam and the splice plate length is computed accordingly. The length is given directly by the formula: (1)

$$l = \sqrt{\frac{(Fft)^2 - (vt)^2}{S_r}}$$; where $\underline{l}$ is the splice plate length in inches; $\underline{F}$ is the form factor of the beam as previously computed; $\underline{f}$ is the allowable extreme fiber stress in bending in the

flange lumber; t is the total thickness of the plies in the web which are parallel to the span, in inches; v is the allowable horizontal shear stress in the web in pounds per square inch; t is the total thickness of the web, in inches; and S_r is the allowable stress in rolling shear (not reduced for stress).

If this formula is applied to beam in the problem at hand, the following results:

$$l = \sqrt{\frac{(0.74 \times 1450 \times 0.375)^2 \quad [\text{illegible}] \times 0.625)^2}{54}}$$

$l = 7.85$ inches

Eight inch splice plates of unsanded, sound two sides, three-ply, three-eighths inch thick plywood will be used. These splice plates will run the full depth of the web on the outside of the box beam and will run between the flanges on the inside.

Design of Stiffeners

Stiffeners are necessary in plywood beams for several reasons. These are: to distribute the loads into the beam at the point of application, to separate the flanges when the beam is loaded, to stabilize the webs against buckling near the compression flange, and to facilitate accurate fabrication. For the purpose of destributing loads, bearing stiffeners are used. In the beam under consideration, the only bearing stiffeners needed are those at the reactions at each end. The width of the stiffener is controlled by the beam width and the only variable involved then is the thickness of the stiffener (the dimension in the direction of the span). This thickness will be found to equal: (1)

$X = \frac{P}{wc\perp}$: where $\underline{x}$ is the stiffener thickness in inches; $\underline{P}$ is the reaction in pounds; $\underline{w}$ is the total beam width in inches; and $\underline{c\perp}$ is the allowable bearing strength of the flange lumber in pounds.

Applied to the case under consideration this formula becomes:

$$x = \frac{9150}{6.125 \times (390 \times 1.10)}$$

$$x = 3.47 \text{ inches}$$

The factor 1.10 in the above equation is to correct the allowable bearing stress for continuously dry conditions of use. These stiffeners will be cut from naminal four by six inch material, surfaced on the narrow face to a thickness of four and seven-eighths inches to fit snugly between the webs at each end of the beam. It is also necessary to design intermediate stiffeners, the primary purpose of which is to separate the flanges under load and to prevent buckling of the webs. These stiffeners should be spaced according to certain fundamental considerations, the most important of which are: the plywood thickness, and the clear distance between the flanges. Other important considerations, however, are not fundamental to the design for strength and prevention of buckling, but rather are closely related to the use to which the beam is put. In this particular case it seems expedient to place intermediate stiffeners at two foot intervals to correspond with the spacing of the rafters and joists which the girder will support. These stiffeners will also lend lateral support to the plywood webs for the purpose of providing a rigid knee-wall surface in the upper floor rooms, and will make convenient nailing points for wall finish material. Dietz (1) presents a chart giving basic intermediate stiffener spacing for plywood beams. Other works on the subject either make no mention (3) or by-pass the issue with the statement that the shear strength of the web depends in part on the stiffener spacing. (2) It appears, however, that the spacing as recommended above, at two foot intervals, will be well within the requirements of the beam. Stiffeners of this class should completely fill the open space between the webs, and should be continuous between the flanges. A minimum recommended thickness for such members is three-quarters of an inch, (1) and the ones to be used in this beam will be one and five-eighths inches thick in order that this requirement will be adequately met. Two by six inch material will be used with each member surfaced on the narrow faces to four and seven-eighths inches to fit snugly between the webs.

Since the girder is held in line at the upper edge by the action of the rafters, and at the lower edge by the floor and ceiling joists, the lateral stability of the member is assured and need not be checked. The girder is now completely designed and is shown in Figures 6 and 7.

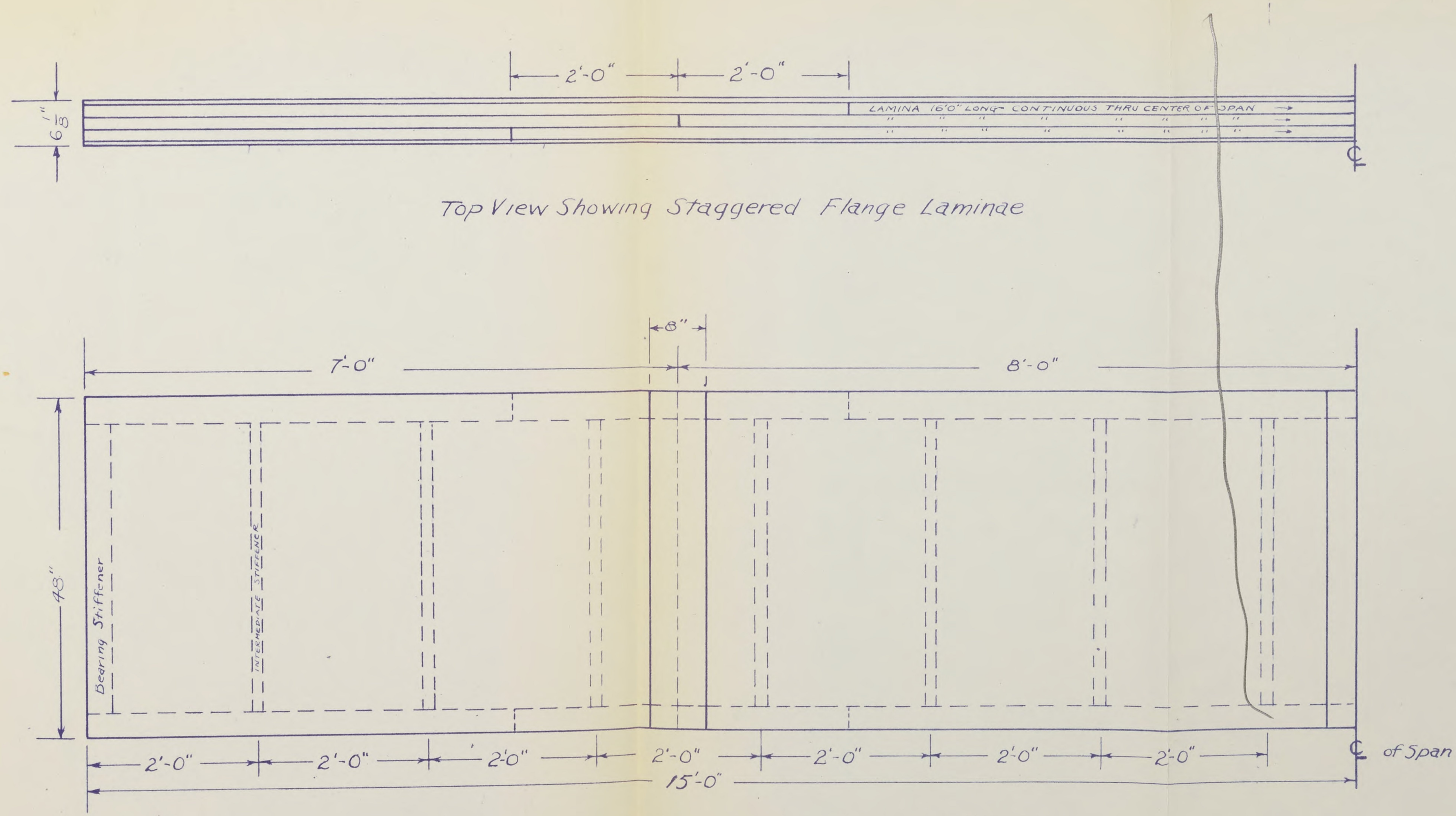

Figure 6. One-Half of Longitudinal Section of Girder Showing Web Panel Layout and Location of Splices, Bearing Stiffeners and Intermediate Stiffeners.

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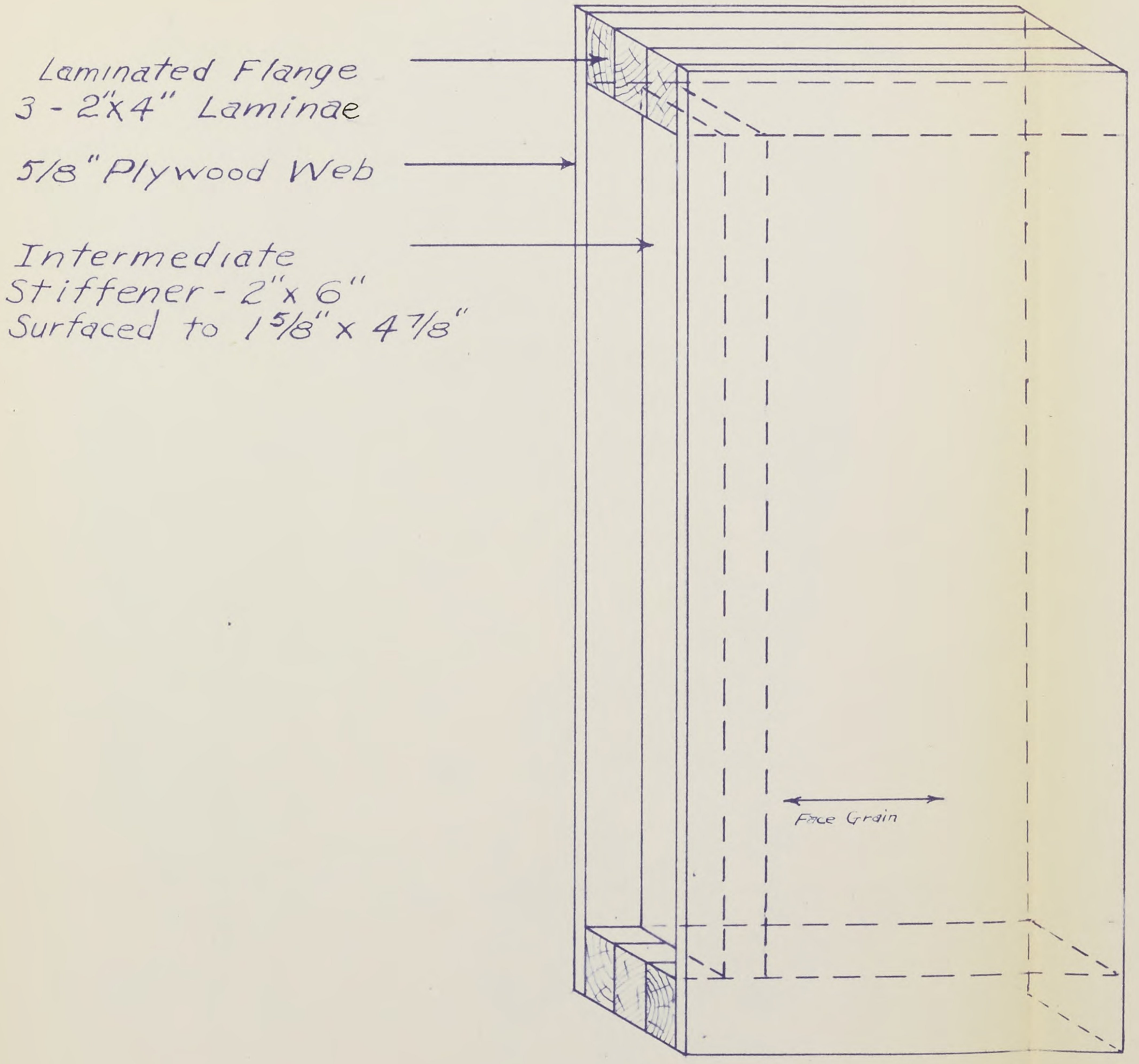

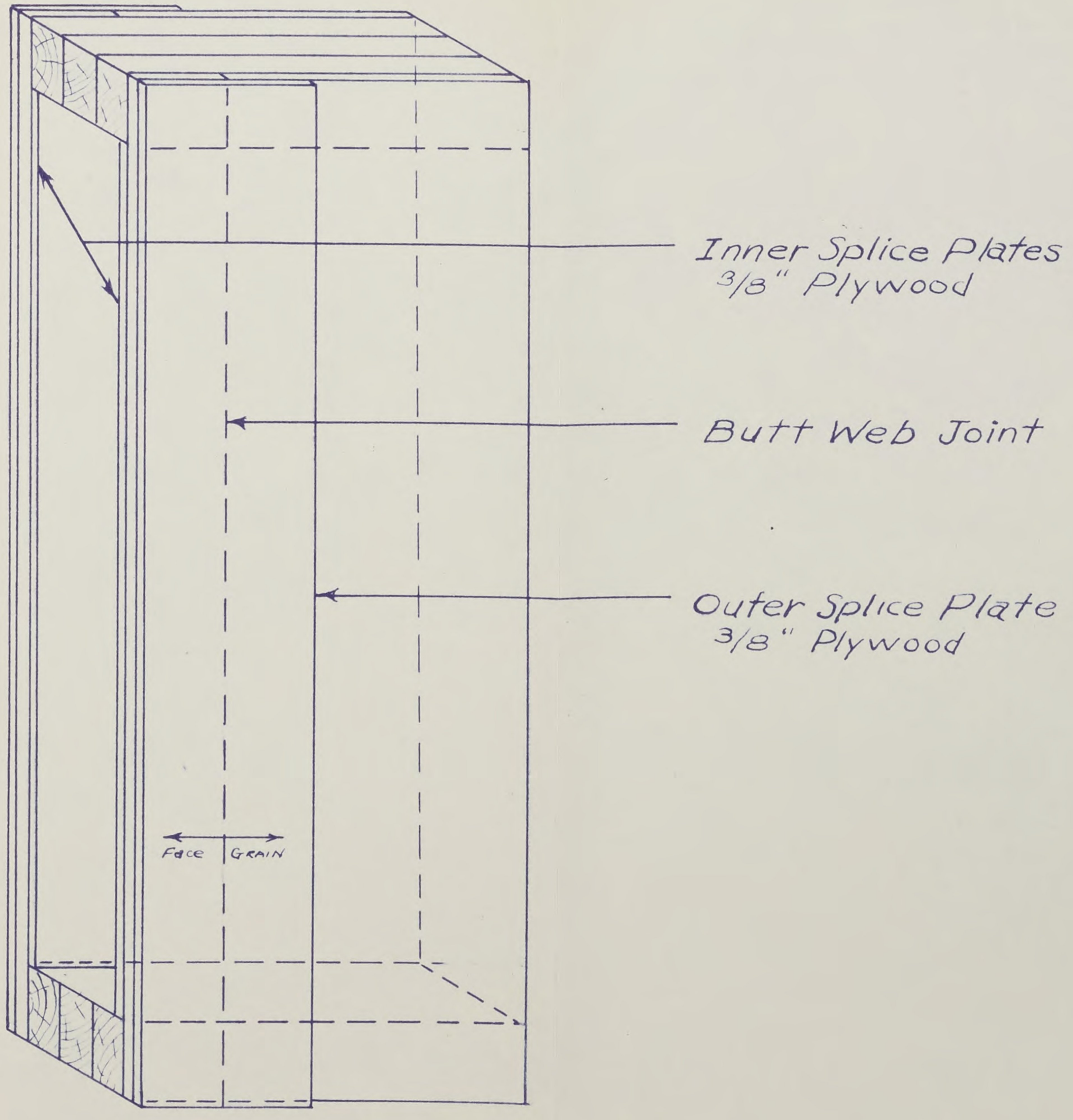

Figure 7.

Oblique Section Indicating Placement of Intermediate Stiffeners.

Oblique Section Indicating Web Splice Plates at a Butt Joint in the Web.

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COST COMPARISON AND ANALYSIS

One important basis for comparison between the proposed system and the conventional system of roof framing lies in the relative cost of the systems. In view of the fact that the proposed system has yet to be tried on a production basis it is difficult to claim that it will result in labor cost savings. It is equally difficult to assess the increase in value of the resultant structure accruing from the added flexibility of design possible with the proposed system. Two factors which enter the cost picture can be definitely established, however. These are the cost of the material needed in each of the systems, and the value of the space saved when the proposed system is used. The first of these is tabulated below.

I Material Cost Analysis --- Conventional Roof Frame

Member	Number and Size	Feet, Board Measure
Joists	32 pcs 2"x10"- 14'	747
Rafters	32 pcs 2"x 6"- 18'	576
Collar Beams	16 pcs 2"x 4"- 8'	86
Knee-wall Studs	32 pcs 2"x 4"- 8'	171
Knee-wall Plate	12 pcs 2"x 4"- 10'	80
Totals	124 pcs	1,660 FBM

If the cost of No. 1 Douglas Fir (Coast Type) is $145 per thousand feet, board measure, the material cost of the above conventional roof frame is 1,660 x 145 or $240.70.

II Material Cost Analysis --- Proposed Roof Frame

Member	Number and Size	Feet, Board Measure
Joists	16 pcs 2"x10"- 12'	320
Joists	32 pcs 2"x 4"- 6'	128
Rafters	64 pcs 2"x 4"- 10'	427
Knee-wall Studs	32 pcs 2"x 4"- 3'	64
Knee-wall Plates	12 pcs 2"x 4"- 10'	80
Collar Beams	16 pcs 2"x 4"- 8'	86
Girder Flanges	24 pcs 2"x 4"- 16'	256
Girder Stiffeners	4 pcs 4"x 6"- 4'	32
Girder Stiffeners	28 pcs 2"x 6"- 4'	112
Total Lumber	244 pcs	1,505 FBM

Webs	8 pcs 5/8"x 4'x 8' So2S, Ext. Plywood, Doug. Fir	256 Square Feet
	7 pcs 5/8"x 4'x 7' So2S, Ext. Plywood, Doug. Fir	224 Square Feet
	Total 5/8" Plywood	480 Square Feet
Splices	1 pc 3/8"x 4' x 8' So2S, Ext. Plywood, Doug. Fir	32 Square Feet

Lumber Cost:	1,505 x 145	$218.23
5/8" Plywood:	480 x 0.38	172.40
3/8" Plywood:	32 x 0.26	8.32
		$409.95

It is evident that the proposed system is more expensive from a material standpoint. The difference is $169.25.

At this point it becomes expedient to consider the second factor of this cost study, the value of the space gained through the elimination of the central bearing partition. An appreciation of this saving can be gained by reference to Figure 8. As indicated thereon the maximum gain which can be realized is 12.4 square feet of floor space. If the completed cost of the conventional house is divided by the square feet of usable floor space a figure can be obtained which will approximate the value of a square foot of floor space. Assuming that the total cost of the house in question was $12,825, and that the available floor space, up and down, is 1,026 square feet, the cost per square foot is $12.50. On the basis of the maximum possible gain, the extra floor area available in the house framed with the proposed system would be worth $155.50. This increase in usable space alone nearly compensates for the excess cost of the proposed system as computed on a material cost basis.

The proposed system is readily adaptable to labor saving techniques, but the monetary saving which could be realized by the use of such methods is, of course, unknown. It is entirely possible with the proposed system to reduce the number of parts to be assembled on the site to a total of 66 pieces. These would be: the upper rafter and collar beam assembly, 16 pieces; the lower rafter and joist assembly, 32 pieces; the floor joists, 16 pieces; and the girders, two pieces. Under this plan, the ridge pole would be omitted and the rafter joined by plywood gusset plates at the radge. All of these different parts or sub-assemblies a could be made up at the builders shop or in a lumber yard and delivered to the site. After the girders were in place, two men could easily handle the balance of the work since the floor joist would be the heaviest piece to be handled. In addition the application of plaster base material to the ceiling can be performed with greater economy in such a clear-span

Bearing Partition is Indicated by Cross-Hatched Area.

Total Area Available on Both Floors With Conventional Frame

$= 2\left(11'\text{-}4\tfrac{13}{16}'' \times 29'\text{-}1\tfrac{3}{4}''\right) + \left(\dfrac{5\tfrac{1}{8} \times 29'\text{-}1\tfrac{3}{4}}{12}\right)$

$+ \left(12 \times 29'\,1\tfrac{3}{4}''\right) = 1{,}026$ sq. ft.

Additional Area with girder frame

$= \dfrac{\left(5\tfrac{1}{8}'' \times 29\text{-}1\tfrac{3}{4}''\right)}{12} = 12.4$ sq. ft.

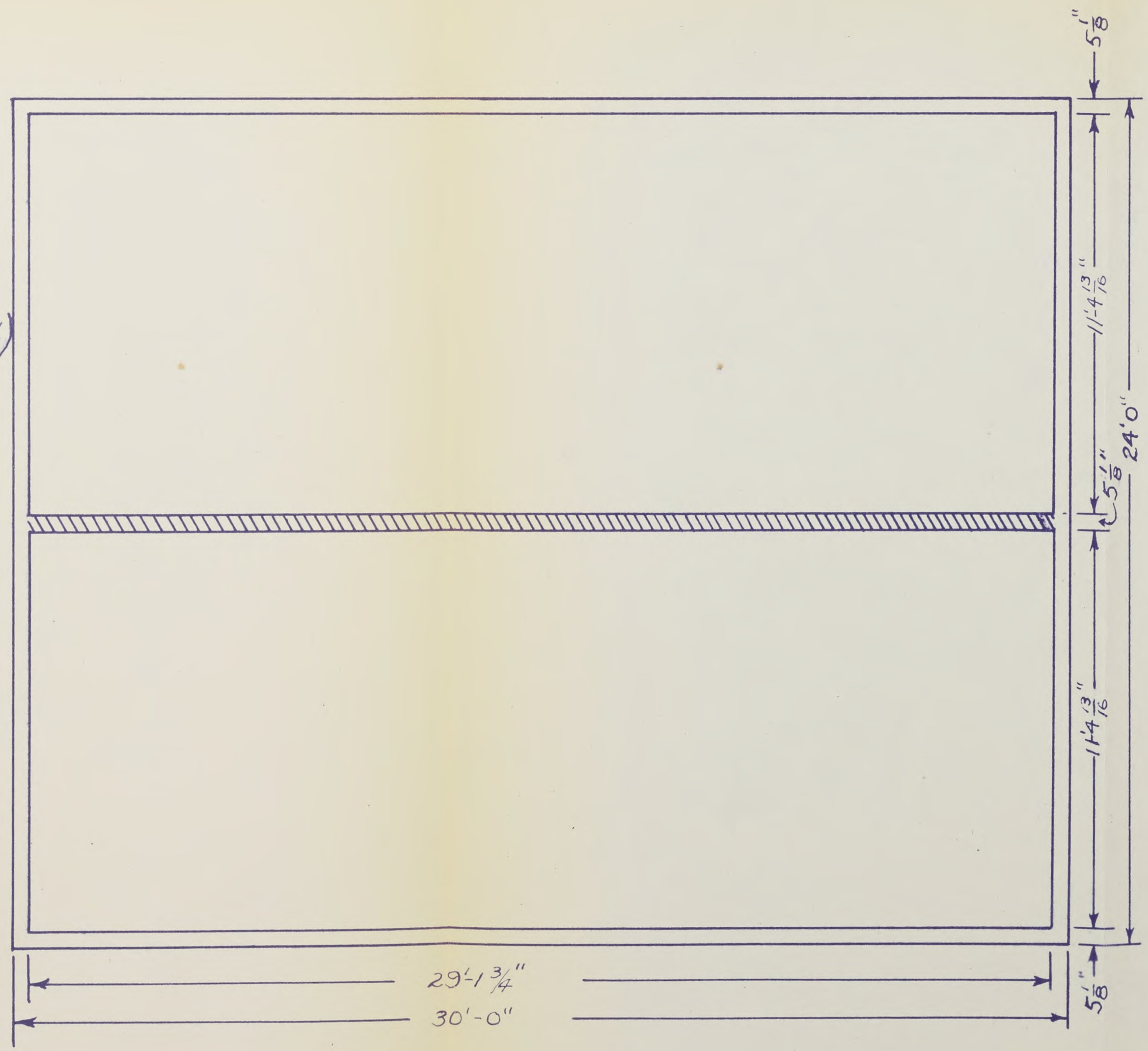

Figure 8. SKETCH SHOWING AREA OCCUPIED BY A BEARING PARTITION

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interior as would be available under the proposed system. Another factor which has not been considered is the elimination of one row of bridging across the building when the proposed system is used. All of the above mentioned savings are small, but could easily result (if known and combined) in bringing the cost of a roof framed with the proposed system below that of a conventional roof.

No attempt has been made to place a value on the intangible factor of greater design flexibility which is possible with the girder supported roof. In certain cases this intangible may be the deciding reason for the use of such a system. In no case should it be neglected.

CONCLUSIONS

Several conclusions can be drawn in regard to the practicality of the proposed design for framing the roof of a one and one-half story residence. In the first place it is evident that excess deflection of the ceiling joists due to transmitted roof loads should not cause plaster cracks because the girders which support the roof, ceiling and floor loads have been designed to carry these loads without excess deflection. This is not true in the conventional frame as commonly used. Furthermore, the system has been designed to support these loads without a central bearing partition. The desirability of eliminating this partition is suitably demonstrated by the increasing number of single story buildings which use trussed rafters to accomplish the same result.

The cost of the proposed system compares favorably with the method in common use, particularly when the value of the space gained under the new system is considered. In addition, the possibility of using labor saving techniques, such as the shop prefabrication of parts prior to delivery, points the way to cost reduction for the proposed design and to an ultimate cost probably below that of the conventional framing system. It has been pointed out that all cost and value considerations completely neglected the factor of increased design flexibility possible without the bearing partition near the center of the building. In a number of cases this single factor may decide the question when the issue involves a choice of systems. This design flexibility becomes increasingly important as building costs increase and maximum space use becomes an economic necessity.

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