Rigid Joint Testing for Gable Roofs

Prepared for:

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Project Implementation Division
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Ottawa, Ontario
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Prepared by:

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DRAFT
FINAL REPORT

RIGID JOINT TESTING

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Housing Technology Incentives Program

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1. INTRODUCTION

It is common practice in, for example, the home construction industry, to use metal gusset plates to fix joints for strength purposes. During the construction of a cottage, the inventor of the gusset plates tested and reported here, Mr. John Irving, realized that a new version could be used. Subsequent to this, a document was prepared to describe the plates and their application, and the Housing Technology Incentives Program of the Canada Mortgage and Housing Corporation was approached to fund a series of tests to evaluate the proposed design. This report thus communicates these findings.

These tests were conducted at the structural laboratories of the Department of Civil Engineering at Ryerson Polytechnical Institute.

1.1 Background

Gable roofs are popular standard forms of structure for wood frames because of their ability to shed rain and snow, their suitability for shingle roofing, and the inherent strength of their triangular shape. The sloping rafters in combination with the ceiling joists form a triangle. Horizontal reactions are taken by the ceiling joists acting as ties. The space above the joist is not utilized and therefore wasted.

A rigid joint at the rafter-wall junction would transfer the horizontal thrust through the wall to the floor, eliminating the need for the ceiling joists. This so called "cathedral" ceiling is architecturally desirable and practical. Walls could be shortened without sacrificing the "open" feeling. By eliminating ceiling joists further economics of labour and material are realized.

A number of configurations have been used to achieve a rigid connection at the roof-wall junction. Most of them have been of plywood glued or nailed to the frame members (see Appendix D, sheet 6 of 13, Figure E). Invariably, they involve the accurate cutting of intricate shapes, careful alignment of the members and location of nails. These have been labour intensive and costly.

The proposed gusset plate can be mass produced from light gauge sheet metal. Hole locations are predrilled, the materials shipped in flat form, and bent to final shape along precreased lines to final shape in the field. Alignment of members to be joined is automatic, eliminating guesswork and assembly errors. This is a definite plus for the do-it-yourselfer for sheds, barns, garages, cottages, etc.

1.2 Scope of Work

The objective of this investigation was to determine the moment capacity and physical behaviour of the proposed gusset plate - joint. The tests simulated a push and pull on the frame by "closing" and "opening" the wall-rafter juncture angle. For this test a partial joint was fabricated and tested prior to the full-frame tests. Two full-scale rigid frames under simulated vertical and horizontal loading were also tested. Partial joints were tested to failure. One frame was also tested to collapse. For all tests, deflections were measured at critical points where possible. Two typical roof slopes were used, 14 and 22 1/2 degrees (3:12, 5:12).

The wood used was SPF#1 2 x 6 dimension lumber. Further testing of 2 x 8 rafters was planned but abandoned because of the limited nature of this test. The results of the first few load tests suggested a revised nailing pattern with less nails and subsequent tests were conducted with the revised plates.

2. TESTING PROCEDURES

2.1 Materials

The lumber used was SPF#1 dimension lumber stored at room temperature for several days and assembled. Assembled and tested dry conditions were assumed. Gusset plates were prefabricated of 20 Ga. galvanized sheet metal. Nails were 1 1/2 inch galvanized roofing nails.

2.2 Apparatus

The assembled haunch units were clampered to a rigid testing frame providing complete fixity at the support points. The cantilever arm was loaded with calibrated dead weights up to rupture. Dial gauges and a surveyors rod, graduated in millimeters, were used to measure deflections. (See Photo 4 of Appendix A).

The full size frames were tested on a loading frame prepared for the purpose as shown in Figure 18 of Appendix A.

2.3 Procedures

For the partial joint assemblies, loads were applied manually in increments of 16.5 kgs and deflection readings recorded, for both "open" and "closed" loading. (See Figure 8 of Appendix A for example).

The full-frames were loaded with the same dead weights at the ridge and fifth points. A separate side load at the ridge was also applied independent of the vertical loads. For both loadings, deflections were recorded using flat scales graduated in millimeters, at the haunch for horizontal movements, and at the ridge and one fifth point for vertical deflections. (Figure 10 of Appendix A).

The first full-frame was loaded to destruction to observe the collapse mechanism. The second frame was loaded to a predetermined deflection of L/180 after which loading was terminated. Deflection was assumed as the "failure" criteria based on the first full frame test results. There was no point in collapsing the frame as deflection became excessive prior to collapse. For both frames, loads equivalent to a ground snow of 1.8 Kpa (40 psf) times 1 1/3 were considered for deflection observations. (As per CSA S307-6.2)

2.4 Limitation of CSA Standard S307 - M1980

Standard S307 covers the procedure for testing "Wood Roof Trusses for Houses and Small Buildings" and as such is not wholly applicable to the frames tested. Reversal of loading is not critical in these frames as it is in a truss. Partial loading was, therefore, not applied in these tests. It was felt that full loading would be more significant. Some clauses were useful as criteria for deflection measurements and loading. Both full frames were loaded for 24 hours with the deflection loads (1 1/3 times ground load x 0.6). Deflection measurements were subsequently read, and the testing resumed. This was done to allow any adjustments in the nailed joints to take place. Relative density of the wood was not considered significant in this series of tests as no failure in the preliminary tests were attributable to the nails.

3. GUSSET PLATE DESIGNS

3.1 Discussion

The geometry of the gusset plate was developed by its inventor Mr. J. Irving. It is intended to provide a rigid joint to the assembly, act as a jig in cutting the wood parts, nail locations, and a nailing surface for the soffit.

Rigid frames provide economy of materials, erection time, stability to the frame, simplicity of construction and versatility of applications. Plates can be manufactured with different roof slopes. One size can accommodate two different rafter sizes. Storage is simple because the gussets are manufactured and shipped flat.

The confining action of the gusset geometry provides good anchorage to the stud end helping to develop its full moment capacity. Nails alone would not be sufficient. The initial plate thickness was selected as 20 Ga. based on its popular use in connectors currently in use on trusses. A thorough scientific analysis of the plate would be impractical due to the high indeterminacy of the stresses within the joint. The bearing stresses in the plate material and the buckling of metal between nails was investigated. Nail values were obtained from the Timber Designs Manual (1974 Edition) for nails bearing perpendicular to grain.

3.2 Proposed Pattern

Figure 1 of Appendix C details the proposed gusset plate. This design was based on Mr. John Irvings proposed design.

3.3 Revised Pattern

Figure 2 of Appendix C details the revised gusset plate, indicating the use of less nails and a shorter soffit.

Figure 3 details the ridge plate.

4. EXPERIMENTAL RESULTS

It was felt that the best way to present the experimental results would be a graphical plot of the numerical values, as opposed to load tables. Reference is therefore made to Figures 1 to 13 of Appendix A, Photos 1 to 22 of Appendix A, and to Table 1 on the page following.

4.1 Partial Joint Tests

Test No. 1 provided the initial view of the behaviour of the proposed gusset under load and collapse. The joint was loaded for a "closed" and an "open" type of loading simulating gravity and wind forces on a rigid frame. In both tests the wood material ruptured at values about four times higher than allowable in bending. Attempts to measure angular change between the two members, using a tool makers protractor, were abandoned because of their small magnitude. This indicated excellent "rigid" behaviour with no measurable slip. No deformation of either the gusset material or nails was observed. Based on these observations the next set of tests used a plate with a revised (less) nail content (see Figure 2 of Appendix C). The 2 x 6 size was chosen for its current popularity as a wall stud. It provides for good wall insulation.

For tests 2 and 3, no deflection measurements were made since they were loaded to failure. Test No. 2 was loaded to failure at 411 Kgs for a stress of 32 MPa. Test Number 3 was loaded to failure at a load of 177 kg and stress of 17.6 MPa.

TABLE 1

SUMMARY OF TEST DATA

Conditions					Test Results	
Test #	**Plate**	**Loading**	**Measured Angle**	**L.A.P.**	**Cracked**	**Failed**
1	5:12 slope for 2 x 6 lumber 20 ga metal	negative	22.0°	83.75 cm	at 240 kg	at 280 kg
2	5:12 slope as above	positive	22.8°	98.5 cm	up to 312 kg did not crack let sit 24 hrs. added 91 kg. and failed immediately	
3	3:12 slope as above	negative	13.6°	125.8 cm	at 167 kg	at 194 kg
4	3:12 slope	negative	14°	81.8 cm	at 176 kg	at 288 kg
4A	3:12 slope	positive	14°	96.3 cm	at 320 kg	at 340 kg
5	5:12 slope	negative	22.5°	100.5 cm	at 208 kg	at 256 kg
5A	5:12 slope	positive	22.5°	110.9 cm	at 224 kg	at 288 kg

L.A.P. = lever arm from plate

Test No. 4 was conducted similarly to No. 1 and the results are plotted in Figure 4 of Appendix A. In this test the failure mode was not initiated by bending distress but by splitting along the grain where nails had penetrated the wood. This was attributed to the increased bearing stress on the nails due to their reduced numbers. Since this only occurred at loads approaching rupture it was not considered a serious reduction of strength. One of the connected members ultimately failed in bending. See Photo 12 of Appendix A.

The results of the remaining tests are shown in Figures 5 to 8 and are self explanatory. No significant reductions in moment capacity of the joint was observed. It was concluded at this stage that the gusset joint is capable of developing the full bending strength of the 2 x 6 members; in an open and closed loading pattern.

4.2 Full-Frame Tests

The final two tests (see Figures 9 to 13 of Appendix A) were conducted on full scale frames, the first one for an 18′-0 (Figures 10 and 11) space and 14 degree slope, the second for a 14′-0 span and 22 1/2 degree slope. The revised span of 14-0 (see Figures 12 and 13) was chosen because of excessive deflections in the first test at low load levels. See Figures 12 and 13. For both frames 2 x 6 lumber was used. Both frames were braced against buckling (as shown in Photos 19 and 20).

Frame No. 1 exhibited excessive deflection at early stages of load. This was expected with such a large span for 2 x 6 joists. The behaviour of the haunch was similar to the "partial" tests conducted earlier, an excellent rigid joint. The frame behaved as a two-hinged frame until buckling of the ridge plate material. Subsequent to this the frame behaved as a three-hinged arch up to collapse. Collapse was simultaneously at the ridge and haunch, indicative of plastic hinge collapse. See Photos 21 and 22.

Frame No. 2 was also loaded as shown in Photo 19 and readings taken as recorded in Figures 12 and 13.

Loading was stopped when signs of buckling at the ridge plate were observed. Most deflection takes place at considerable lower loads than at rupture, therefore, within this range the top gusset plates were providing full rigidity. Consequently, the assumption of full rigid two-hinged frame was justified.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Gusset Plate Performance

The gusset plates demonstrated excellent integrity in developing the full moment capacity of the 2 x 6 members by providing full rigidity. The confining action of the vertical pocket within the gusset is a positive contribution to anchoring the vertical members. These members have a shorter anchorage length, because of the plate geometry, than the sloping members. It was because of this that a reduction in nails was possible. It should be possible to reduce the gauge thickness to 22 gauge and because of this, realize a savings in metal cost and weight. The gussets weighed quite a bit. It took three persons to lift the full trusses into place. The ridge gussets failed in buckling prior to tearing on the tension side. These plates will have to be increased to an 18 gauge or stiffened on the compression side with a lip to prevent buckling. This can be done simply in a jig.

5.2 Cost Analysis and Feasibility

The cost of the gusset material was about $15.00/joint at the haunches and $4.00 for the ridge plates; a total of $20.00 including nails and excluding labour. Labour costs are difficult to ascertain as it is expected these plates will be mass produced. Since mass-production would reduce material cost, it was felt that $40 for a full-frame would be a reasonable estimate of cost. To cut and fabricate one frame would take two men about 1/2 hour, for a total of 1 man hour @ $15.00 bringing the cost of the frame, erected, to $55. This is more than offset by the savings in materials of ceiling joists, reduction of wasted attic space, and speedy erection of a frame.

The tests clearly demonstrate the efficiency of the proposed gusset connection in achieving full rigid frame action. The advantages of rigid frames were mentioned earlier. They are however more flexible than conventional framing used in houses, which relies on partitions and end walls against side sway. Attention should be given to this matter where materials connected to rigid frames are not able to take high deflections (Brick walls or plastered ceilings). Maximum permissible values for this are in the order of L/360, in most codes, for LL. Where deflection is not a problem the system is excellent. Barns, storage facilities, commercial storage, cottages, even areas with low roof loading can benefit from this system.

5.3 Field Erection Procedure

Field erection procedures have been proposed by Mr. J. Irving and are appended to this report (Appendix D).

Both full frames tested were fabricated in the structures lab with two persons and erected with three easily. It is expected that the studs will be toe-nailed to a plate on the supporting structures. This is considered to provide a hinged joint.

APPENDICES

A. Test Results and Photographs

B. CSA Standard S307-M1980

C. Gusset Plate Drawings

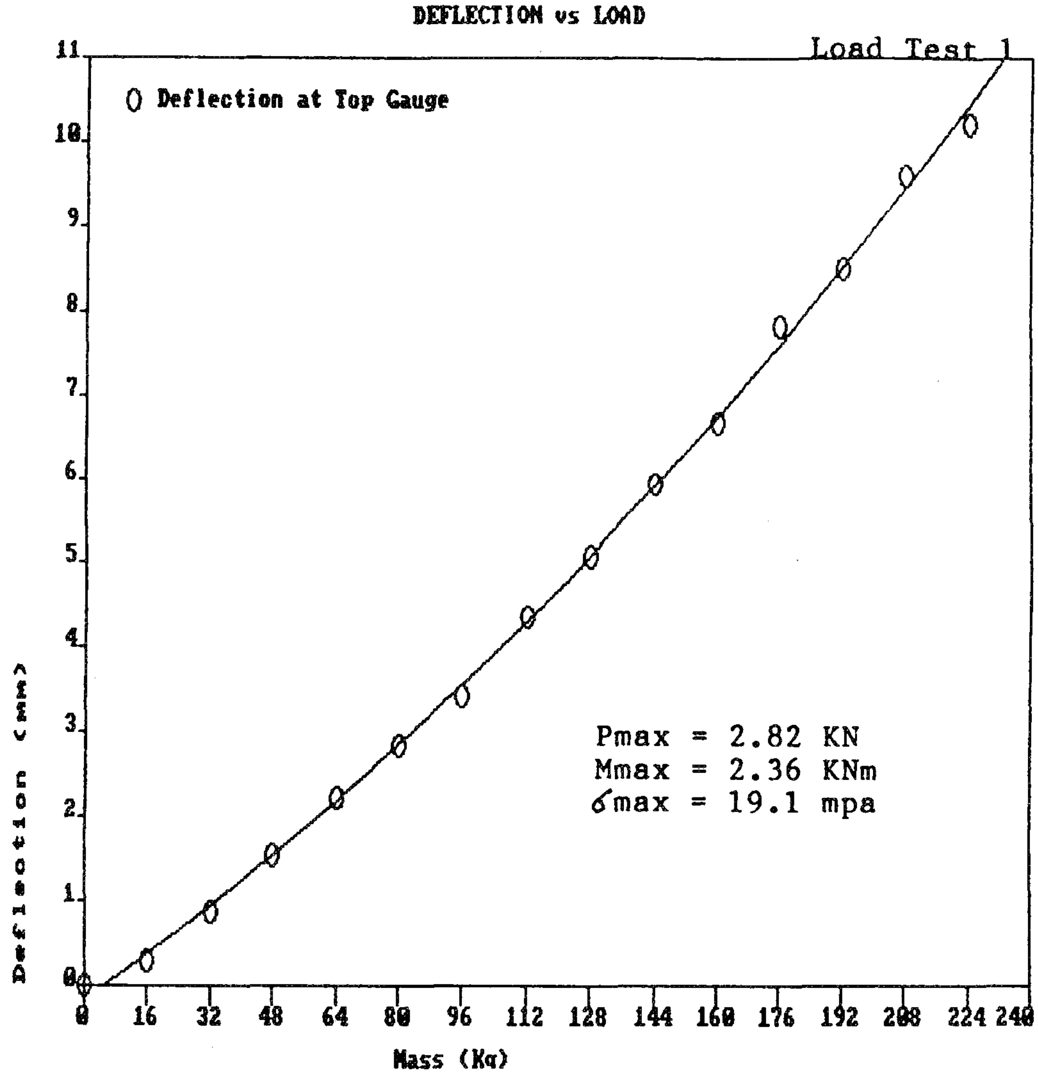
D. Span Tables for Slopes 5:12 and 3:12

E. Field Erection Procedure

APPENDIX A

Test Results and Photographs

(All Test Data and Measurements are in Millimeters)



- Large Plate
- Slope 5:12 (22.0°)

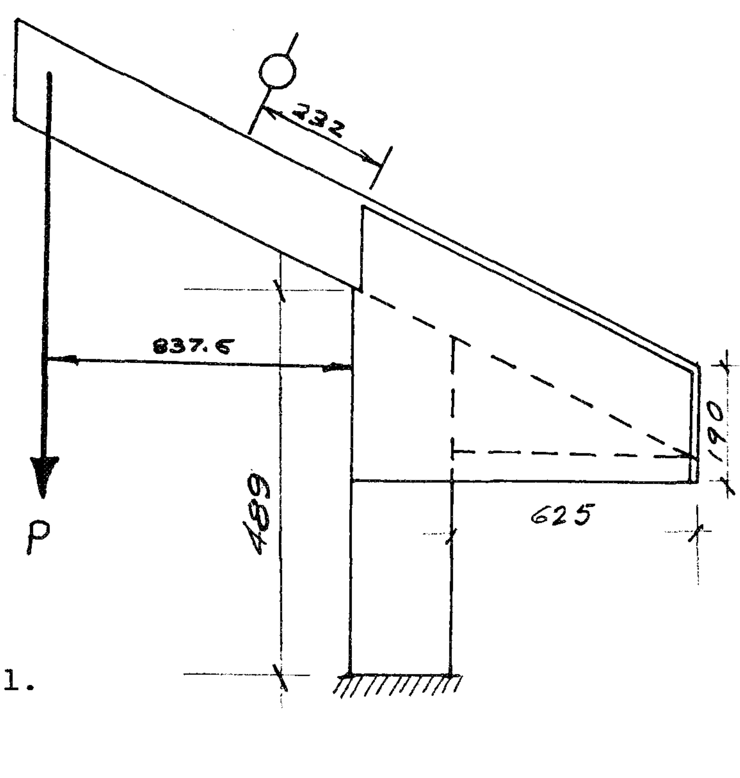
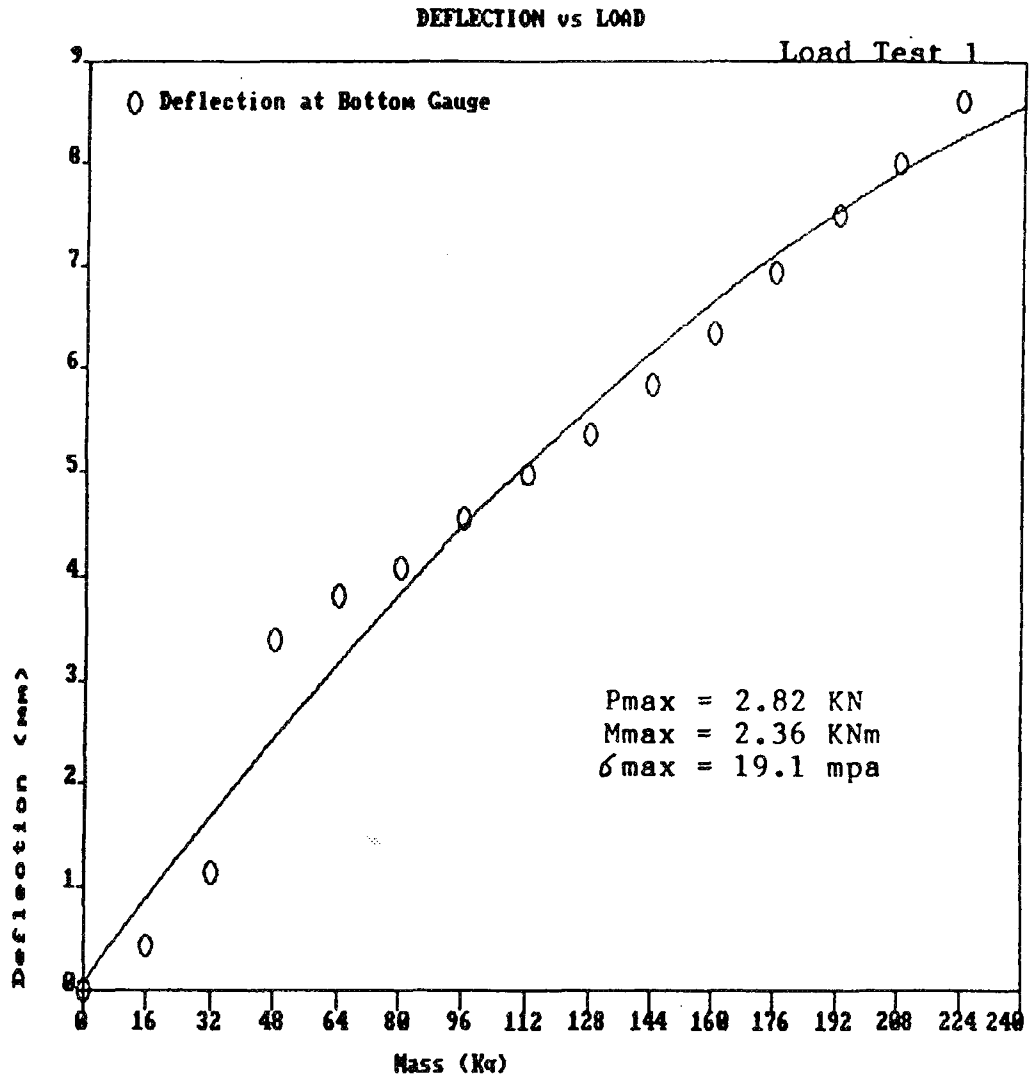


FIGURE 1.

Results For Load Test 1.



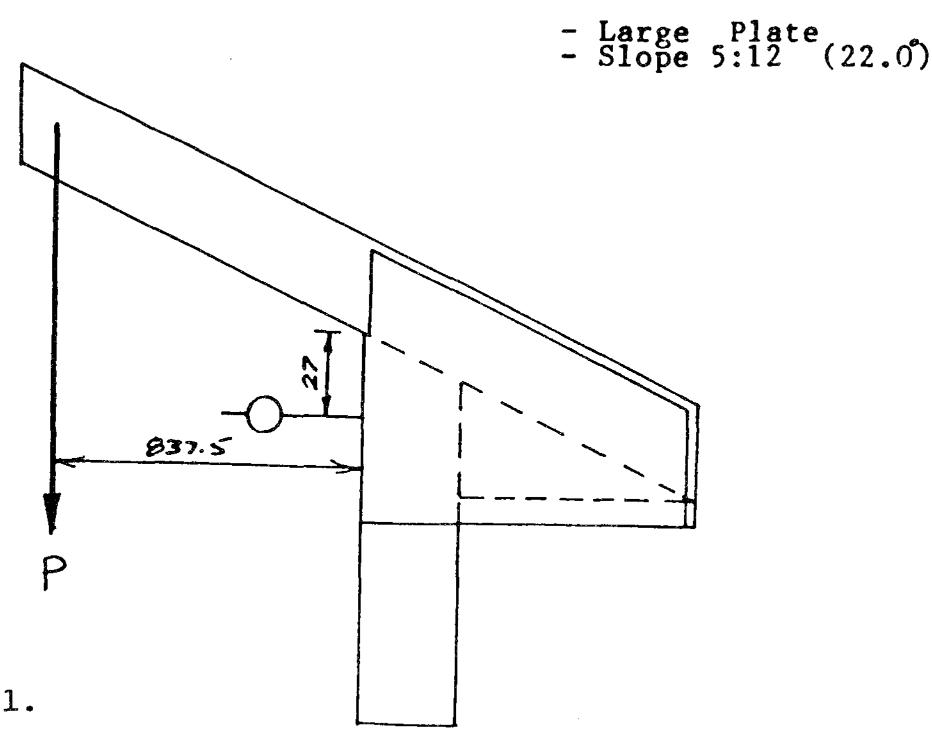
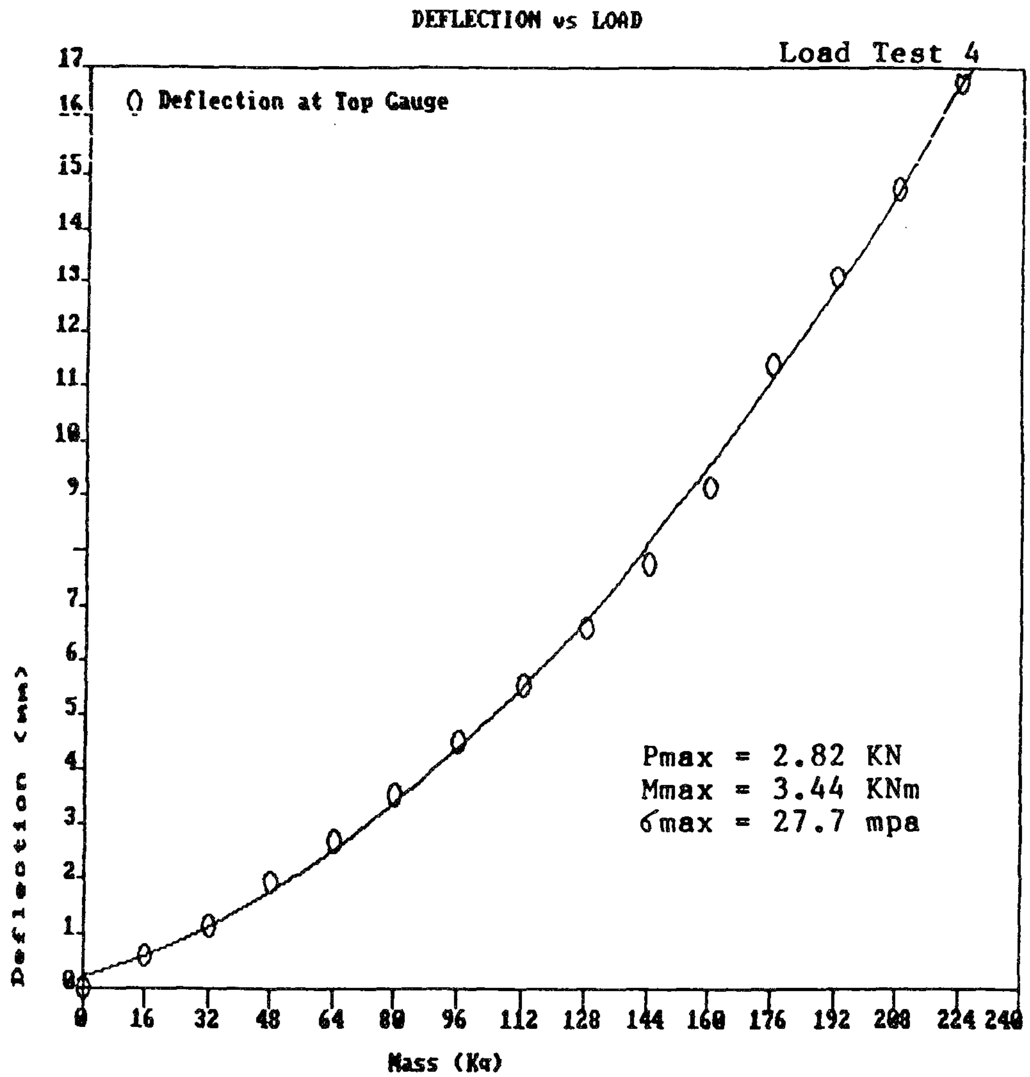


FIGURE 2.

Results For Load Test 1.



- Small Plate
- Slope 3:12 (14.0°)

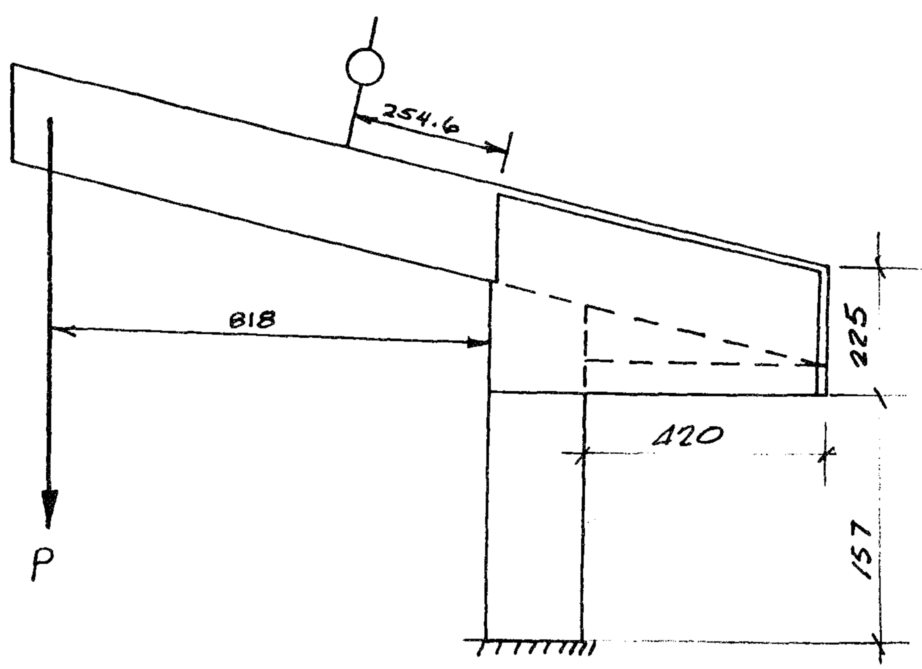
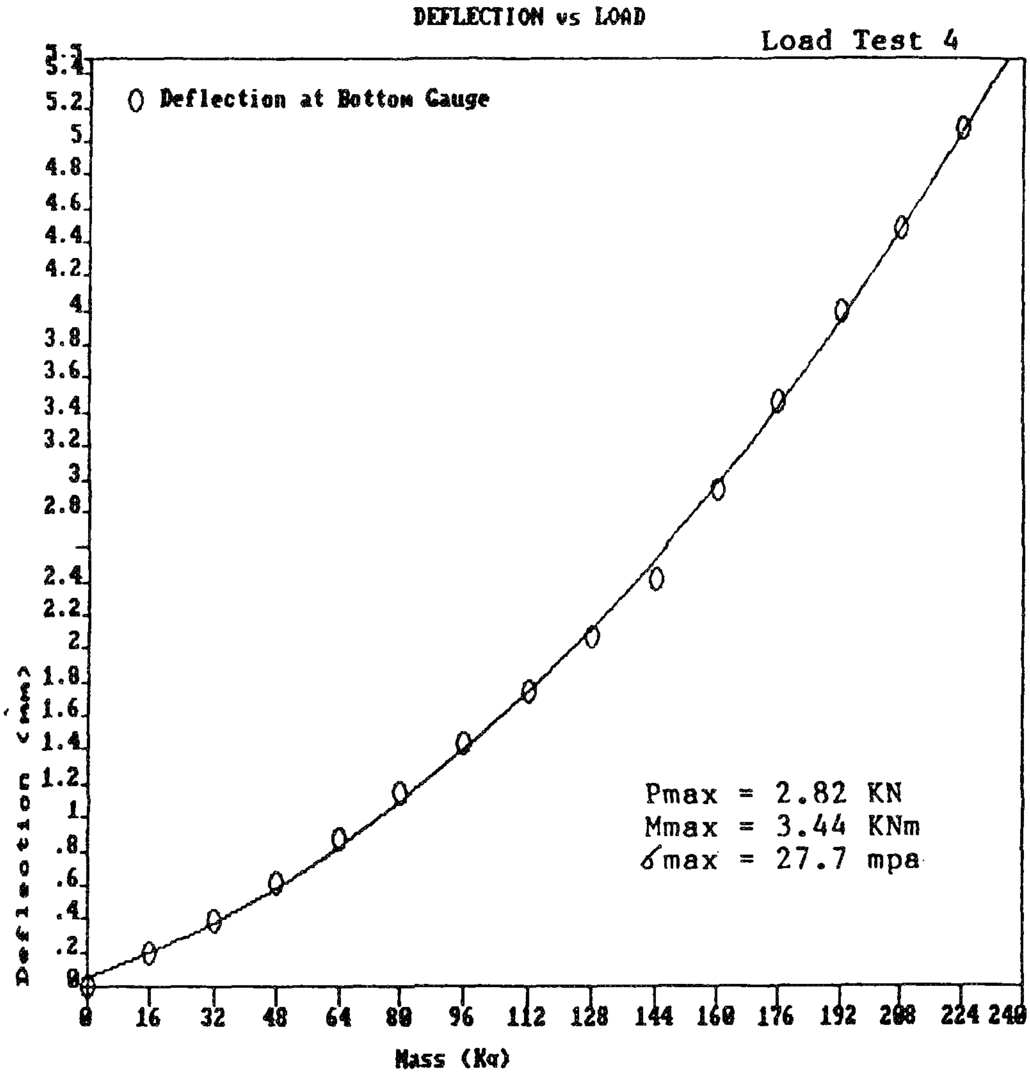


FIGURE 3.

Results For Load Test 4.



- Small Plate
- Slope 3:12 (14.0°)

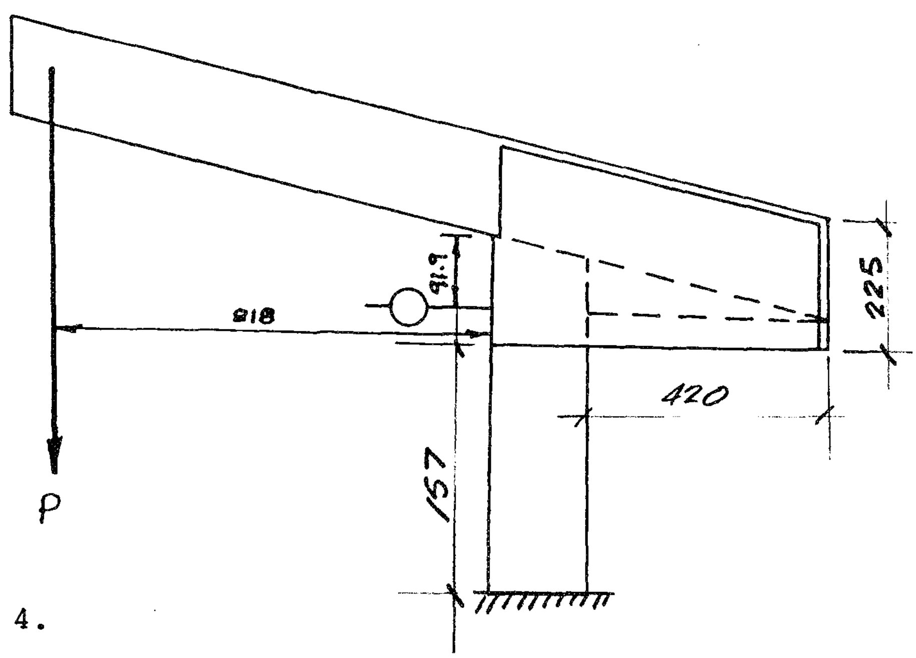
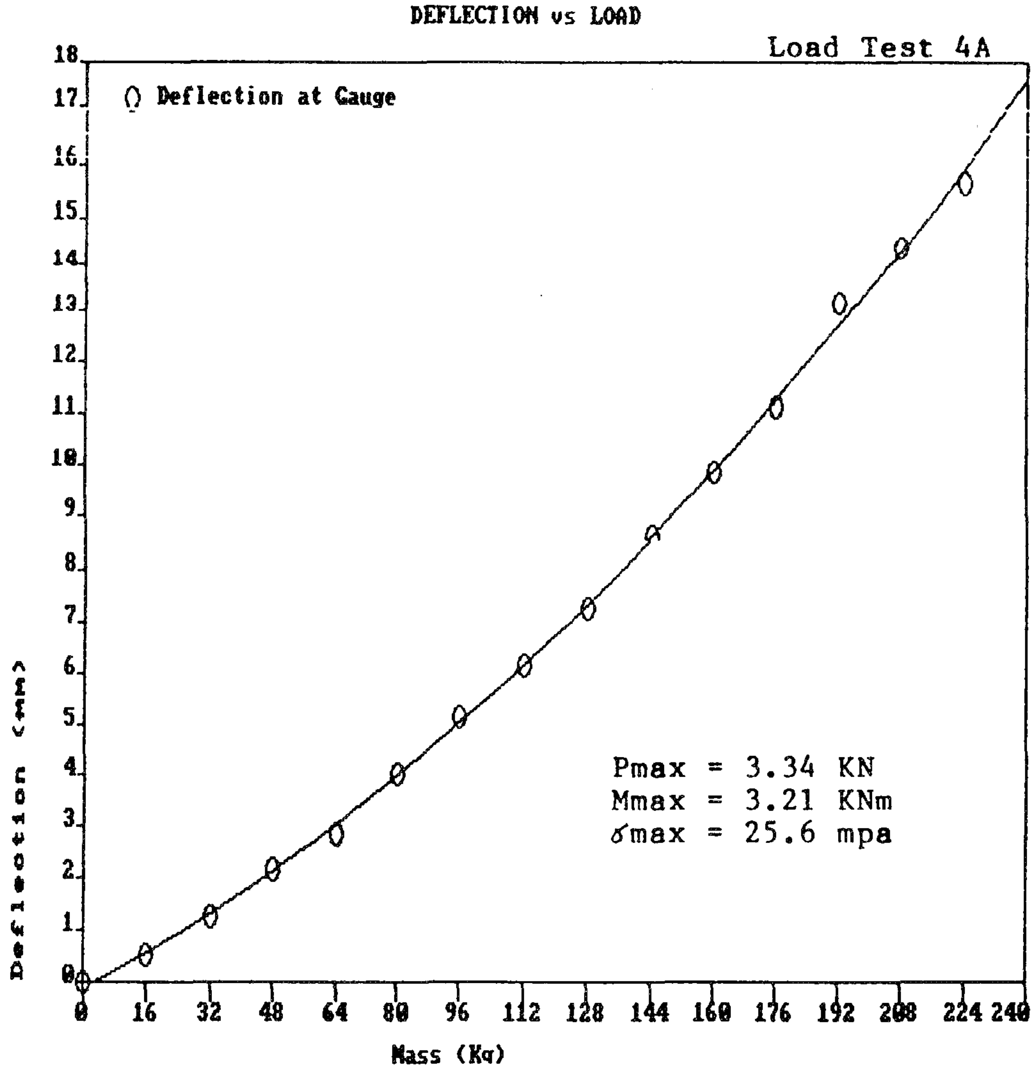


FIGURE 4.

Results For Load Test 4.



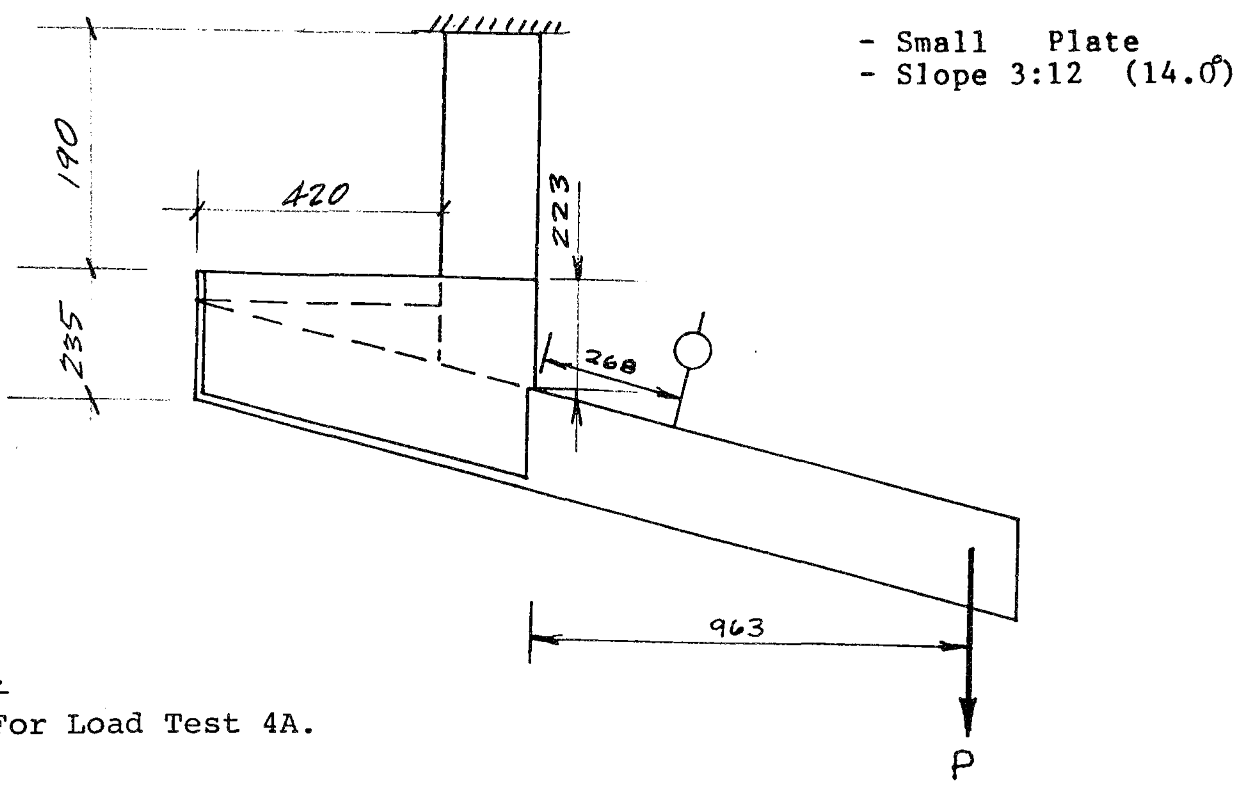
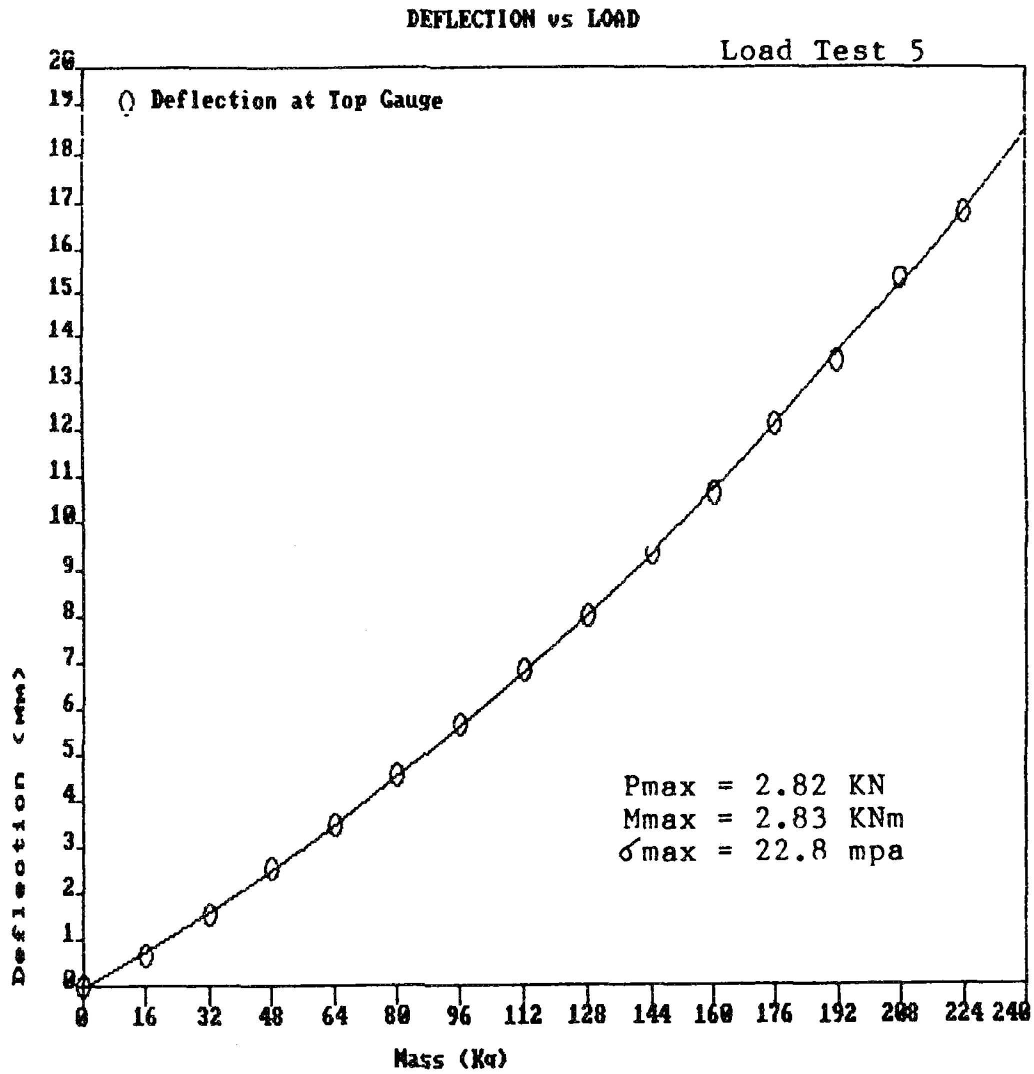


FIGURE 5.

Results For Load Test 4A.



- Small Plate
- Slope 5:12 (22.5°)

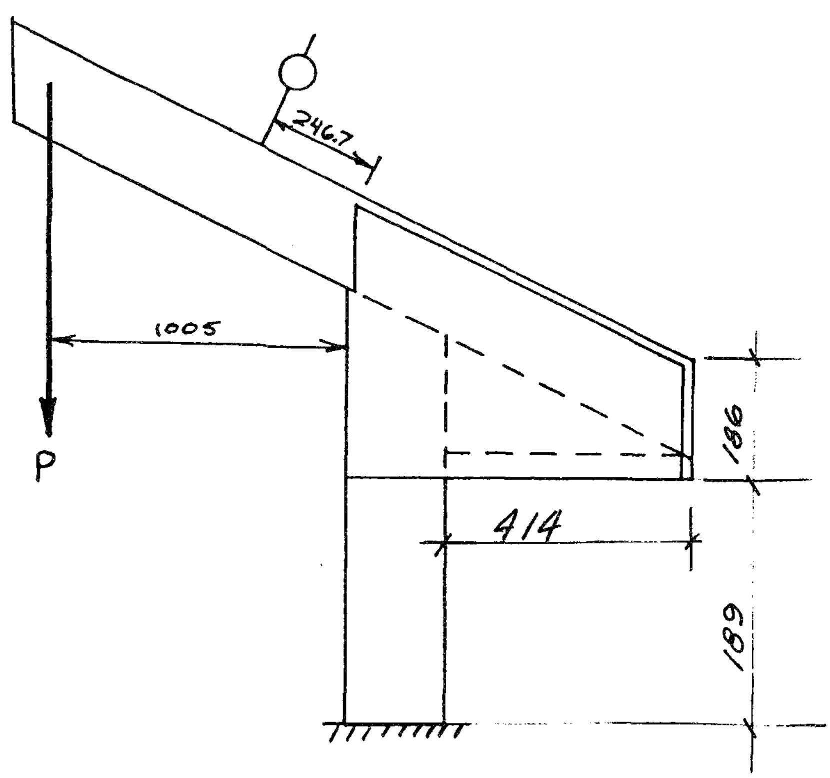
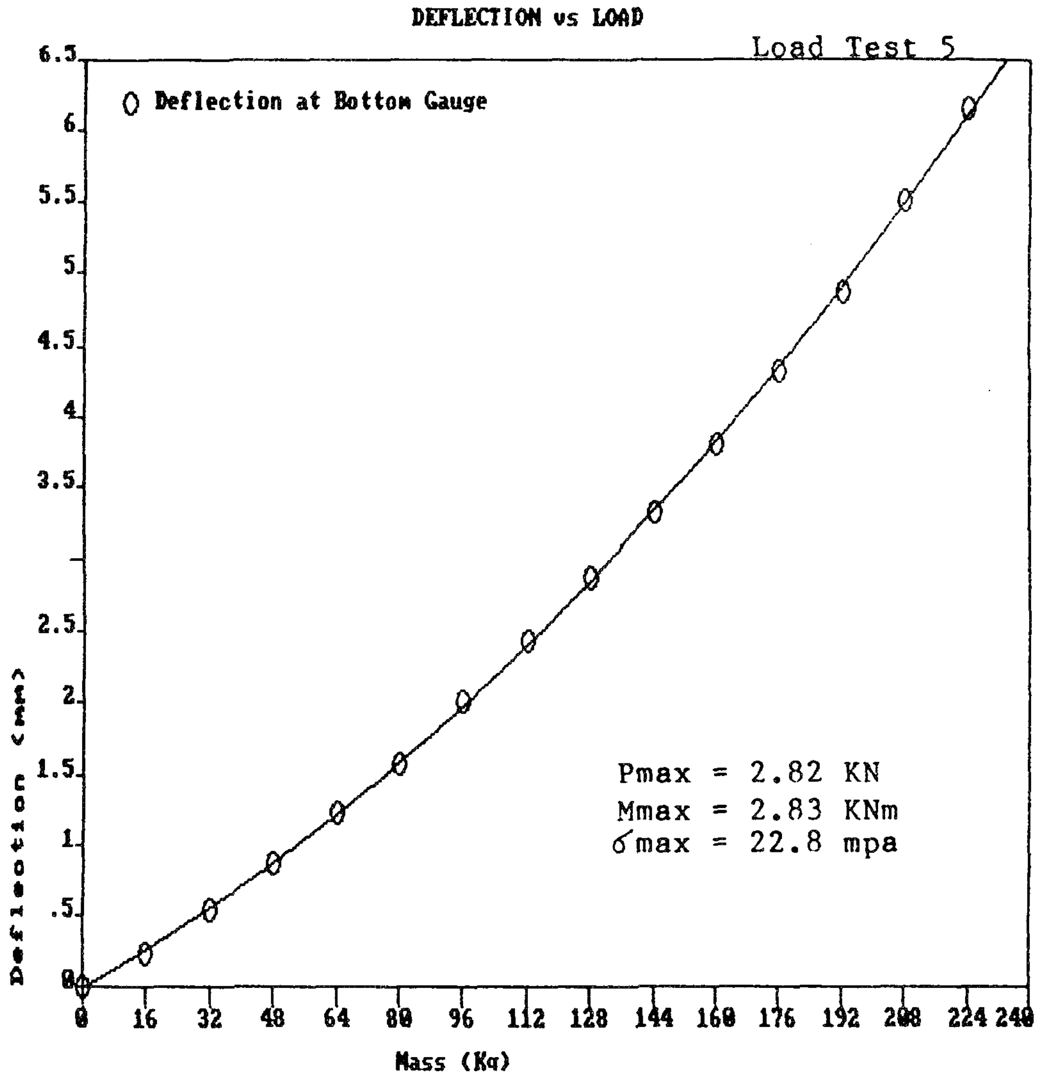


FIGURE 6.

Results For Load Test 5.



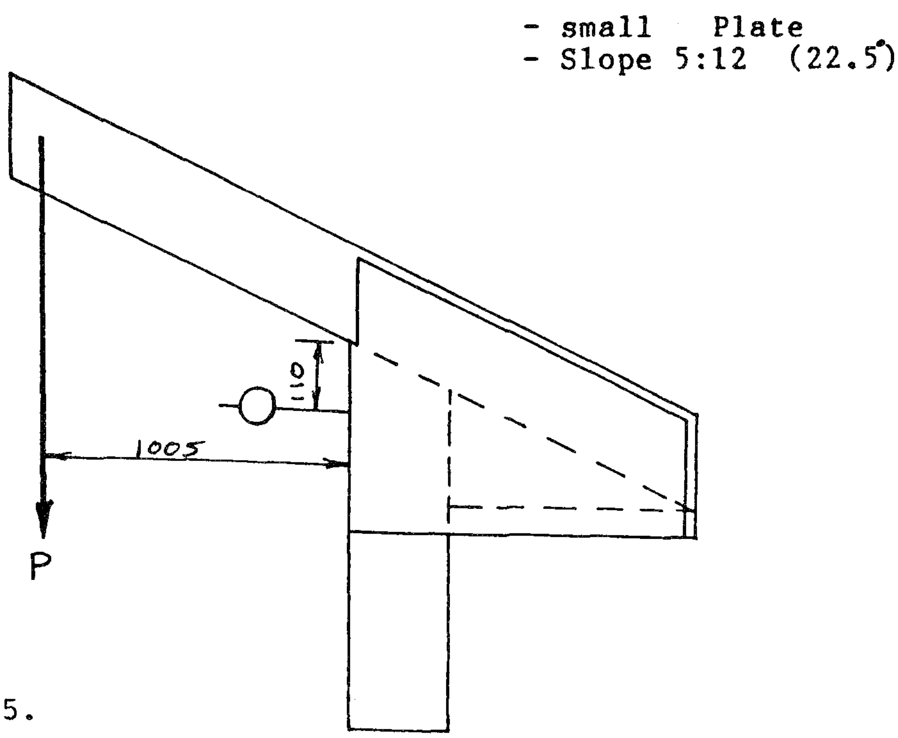
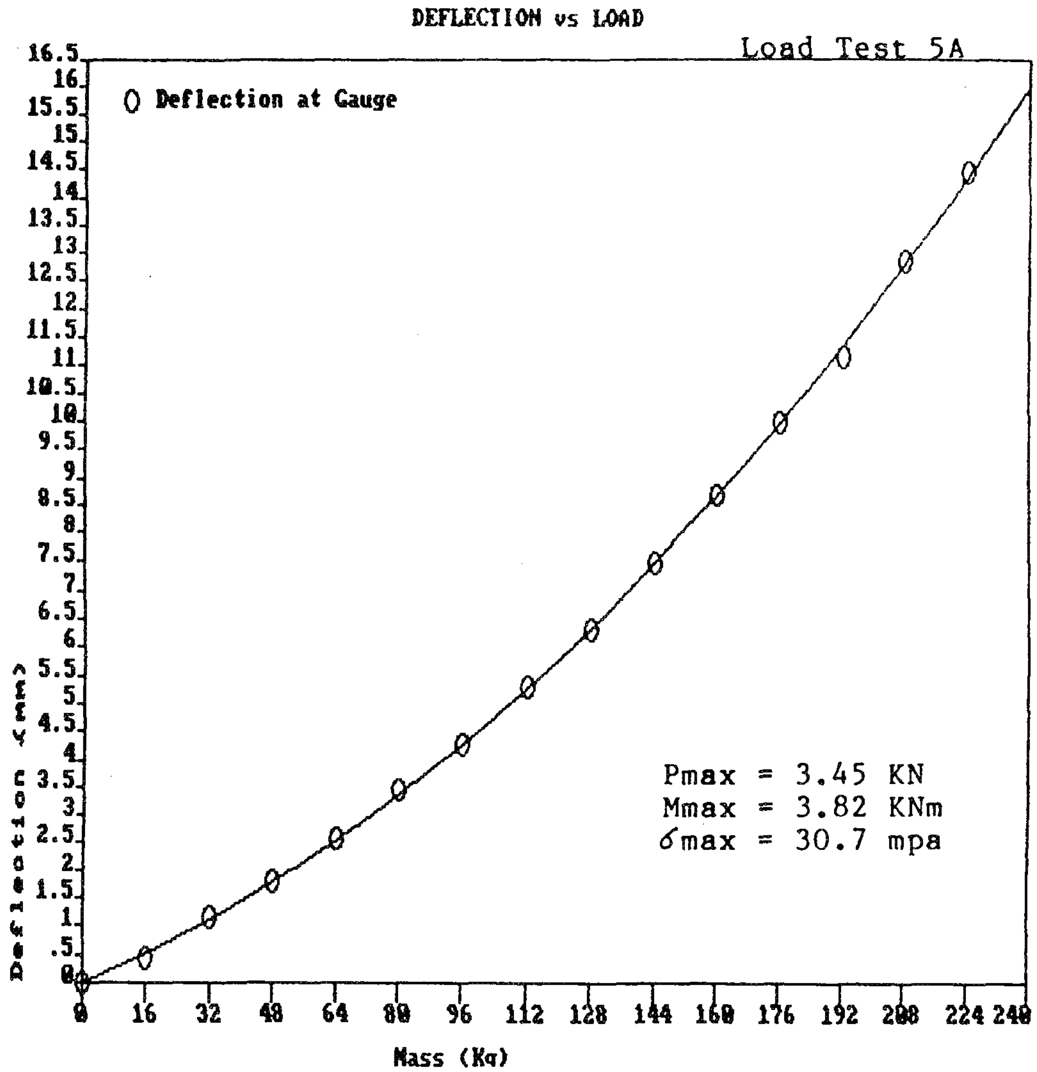
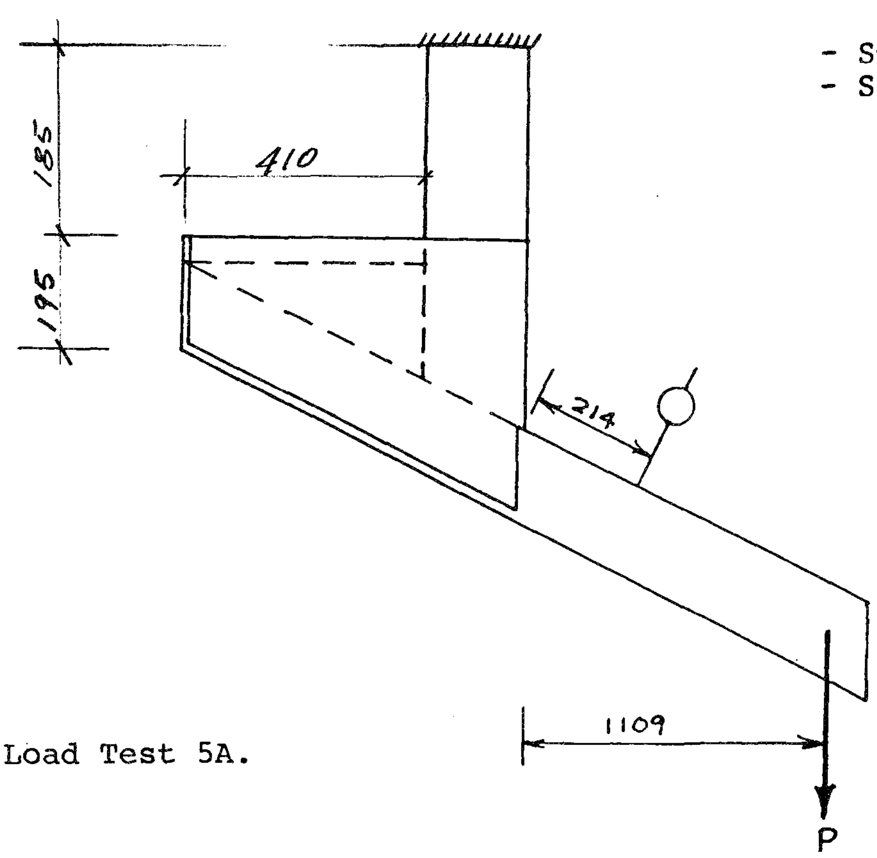


FIGURE 7.
Results For Load Test 5.

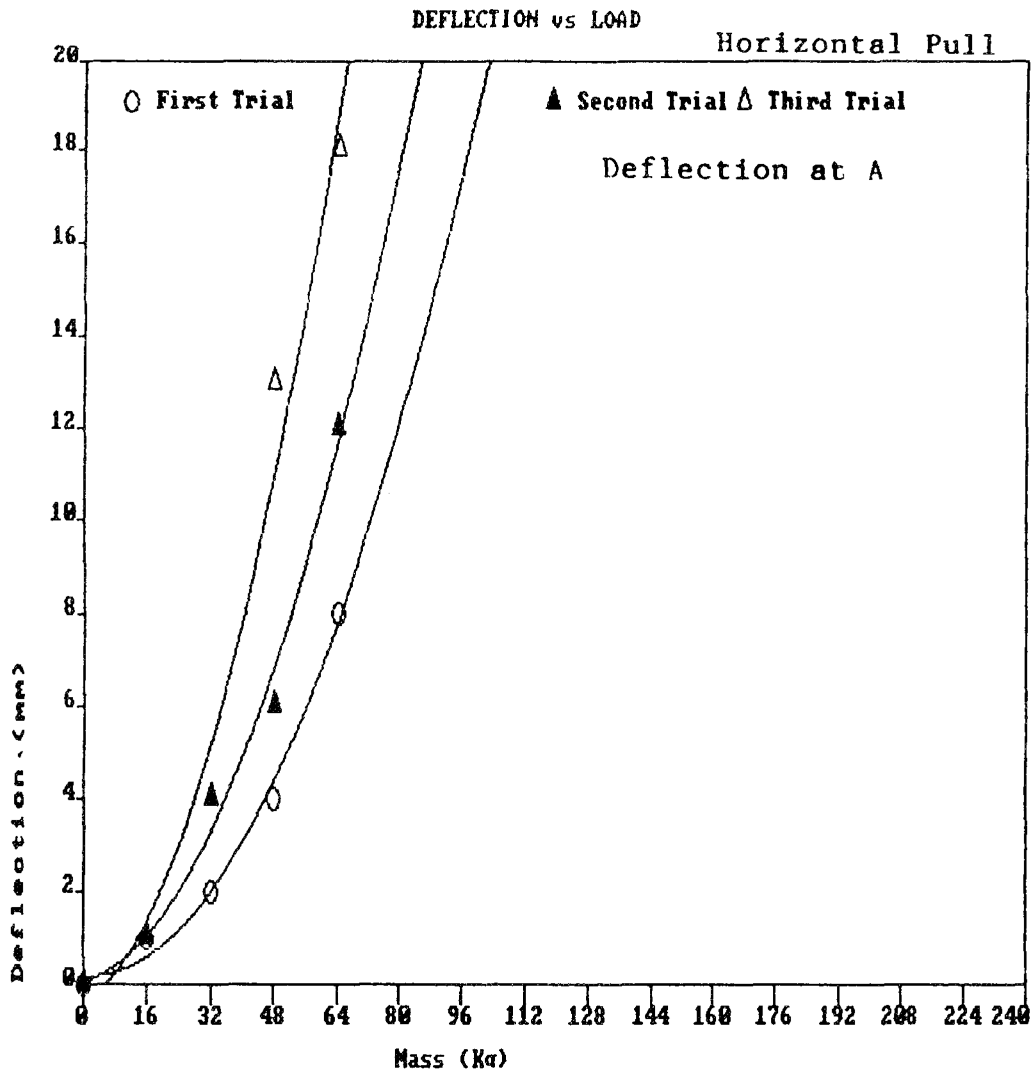




- Small Plate
- Slope 5:12 (22.5°)

FIGURE 8.

Results For Load Test 5A.



- 18 FT. Span
- Slope 3:12 (14.0°)
- Small Plate

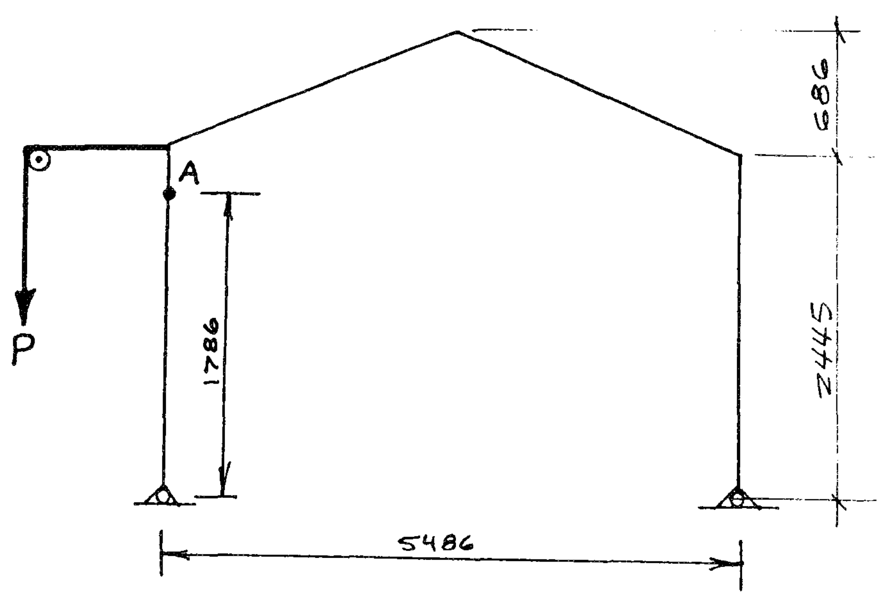
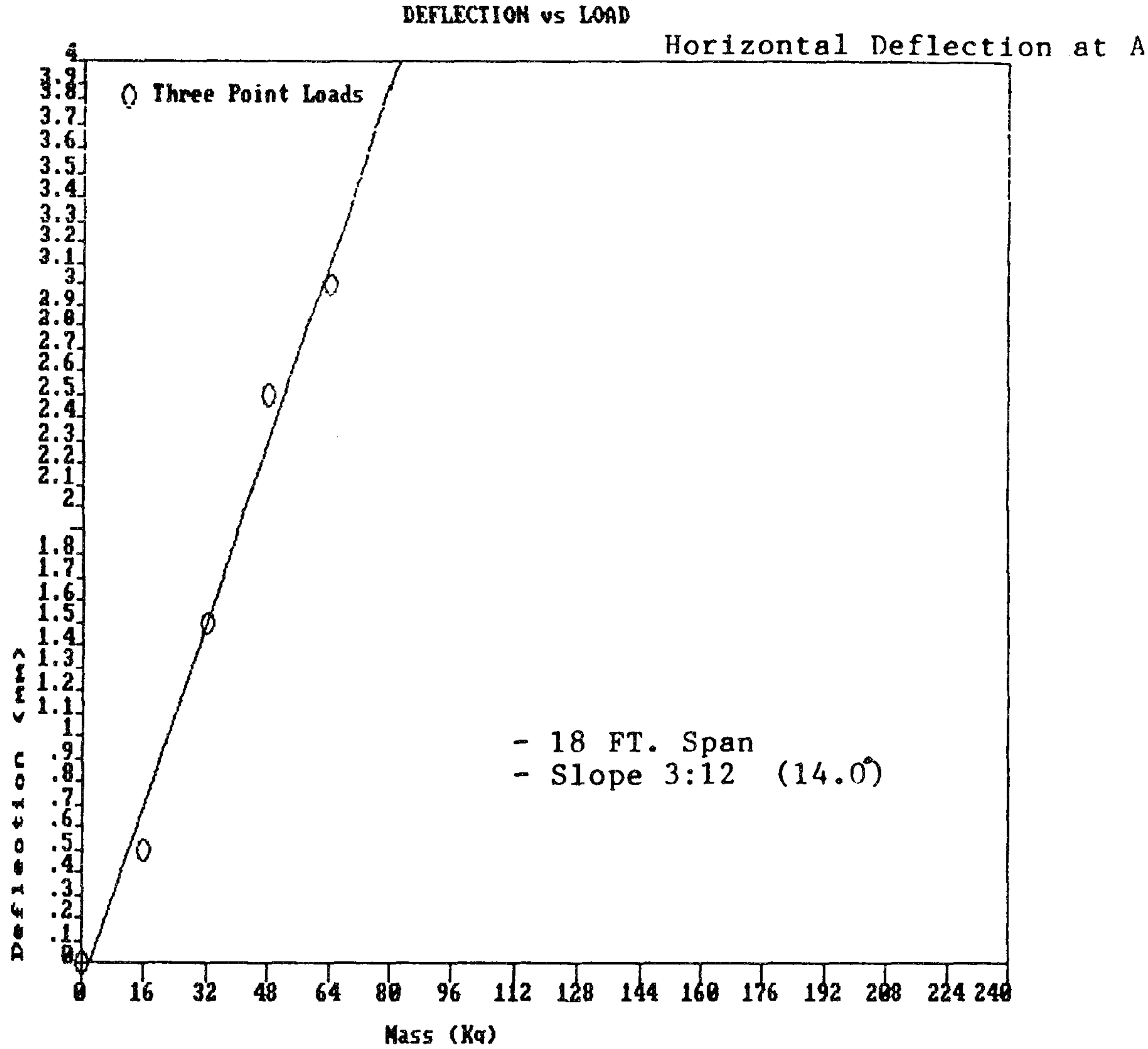


FIGURE 9.

Horizontal Load Deflection Curves For Full Frame #1.



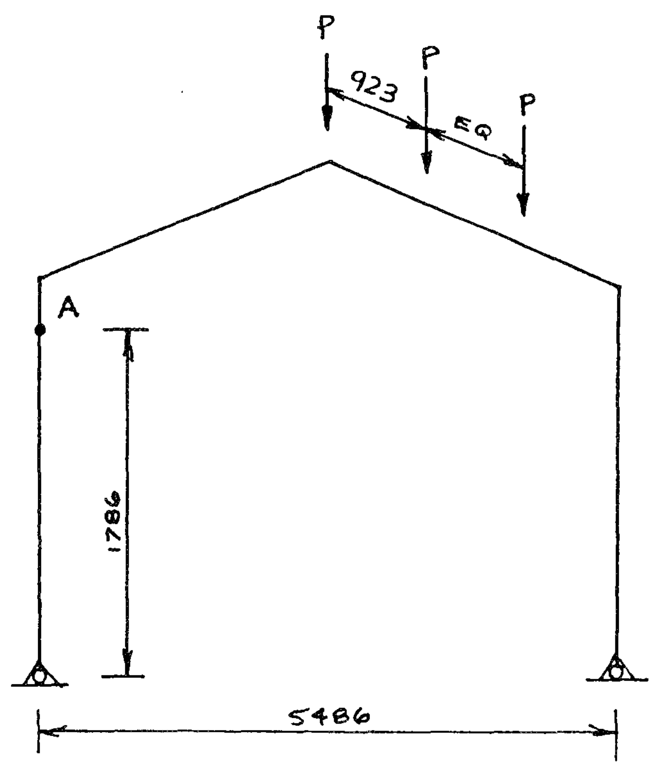


FIGURE 10.

Partial Load Deflection Curve For Frame #1.

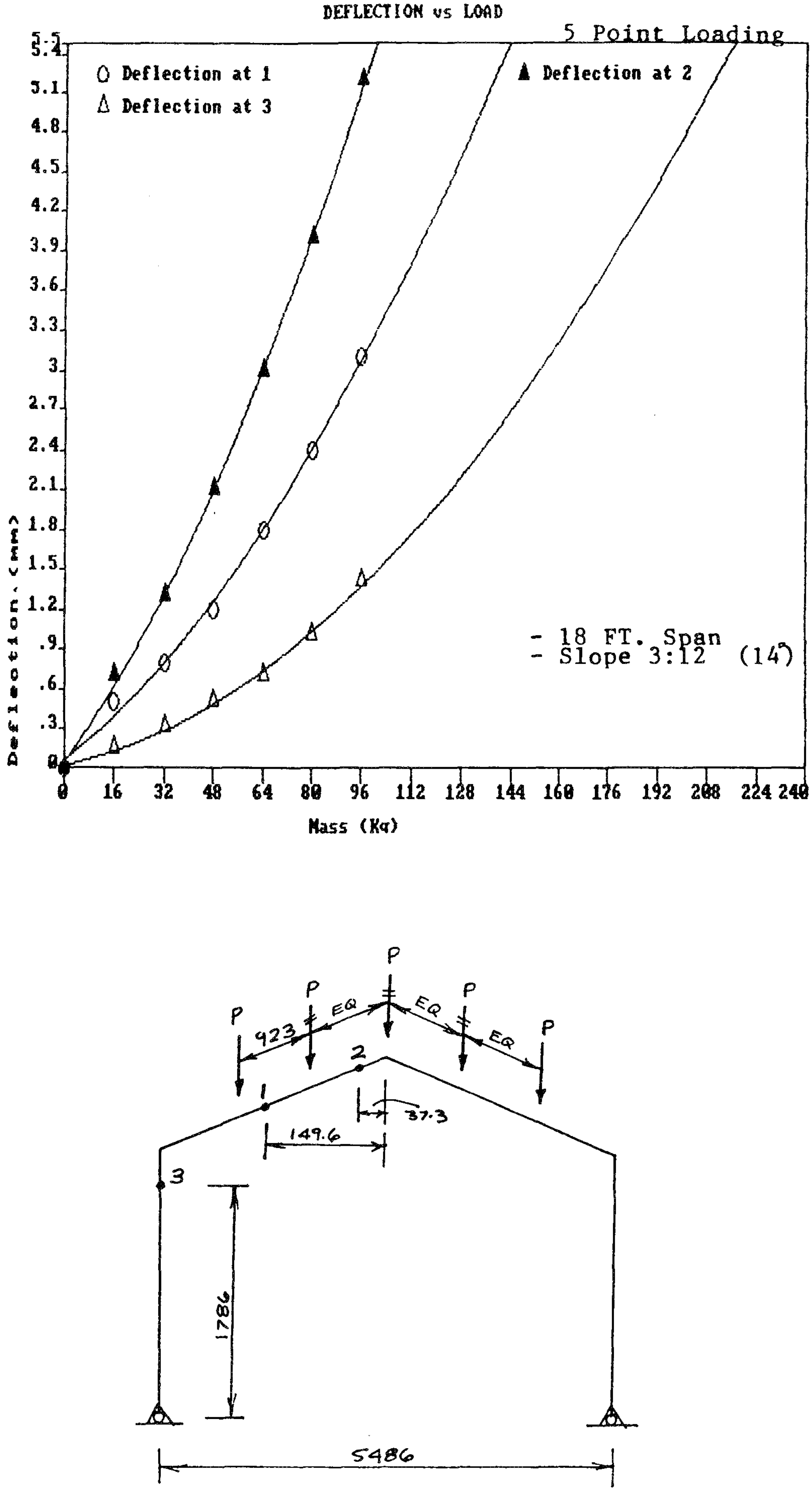
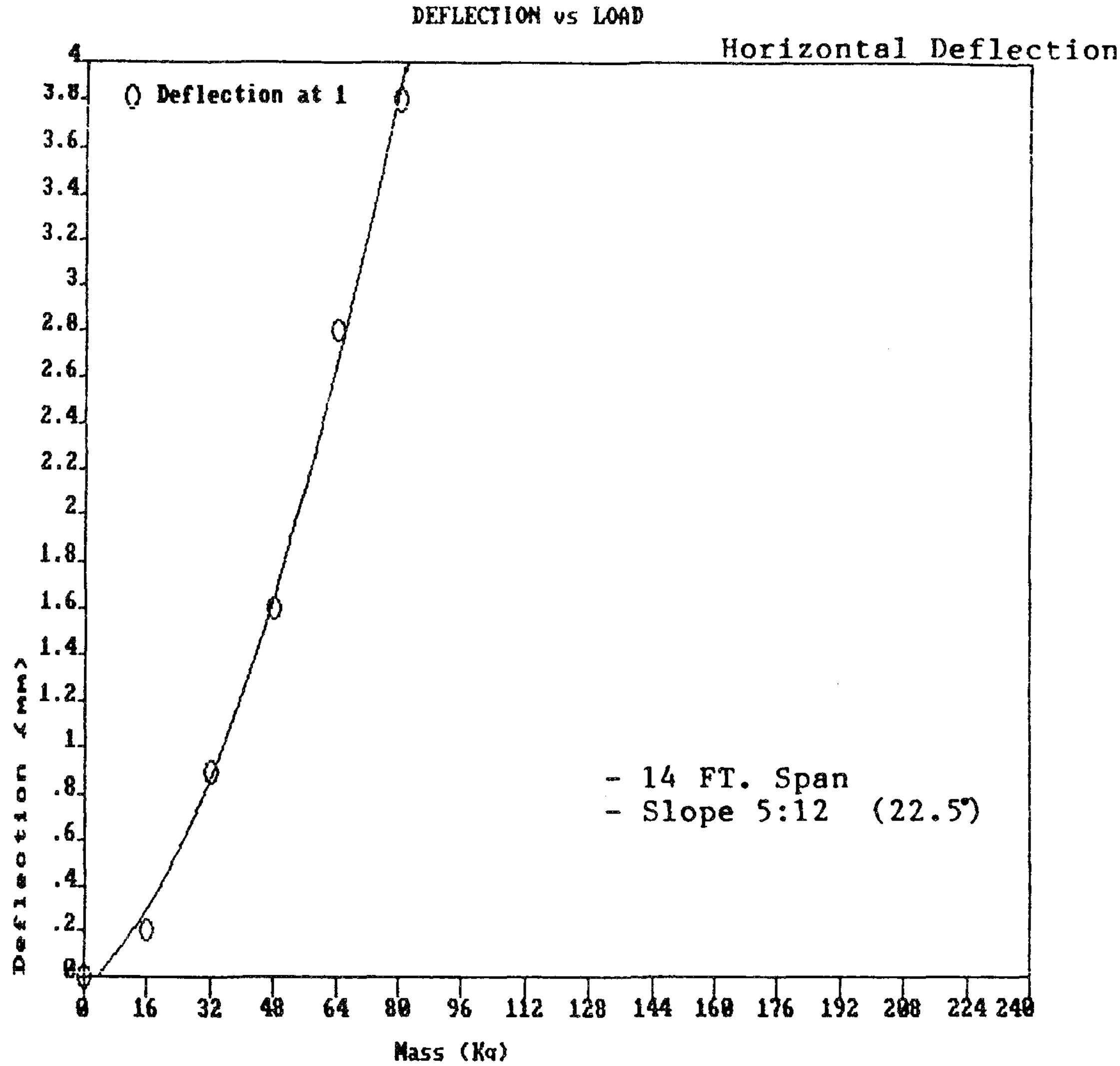


FIGURE 11.

Full Load Deflection Curves For Frame #1.



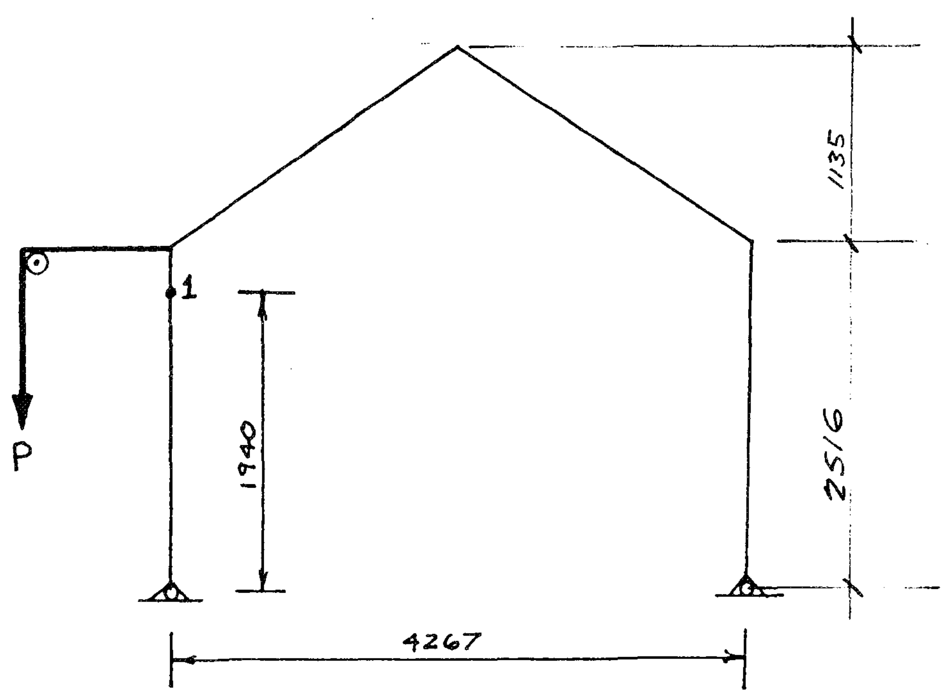
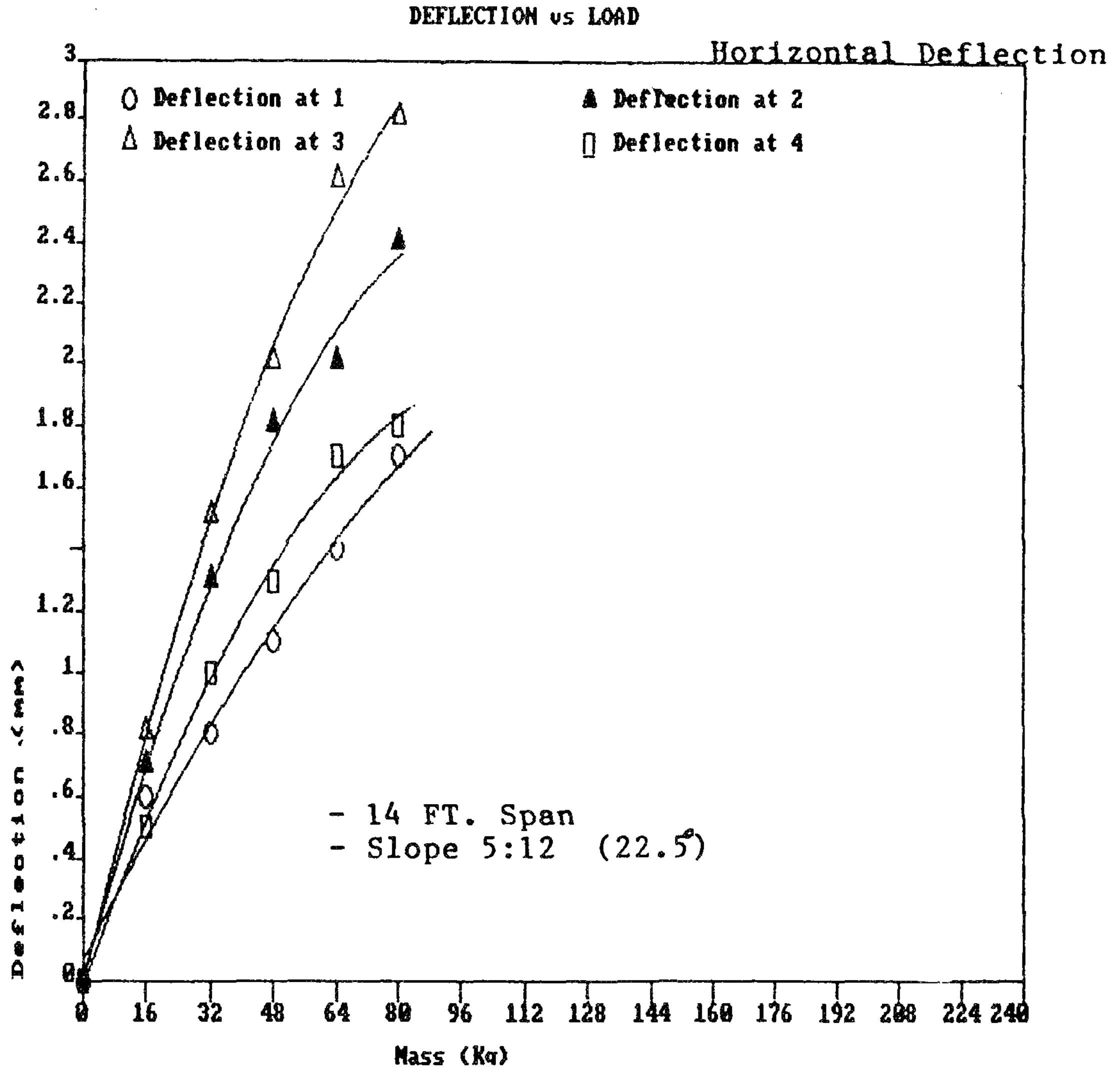


FIGURE 12.

Horizontal Load Deflection Curve For Frame #2.



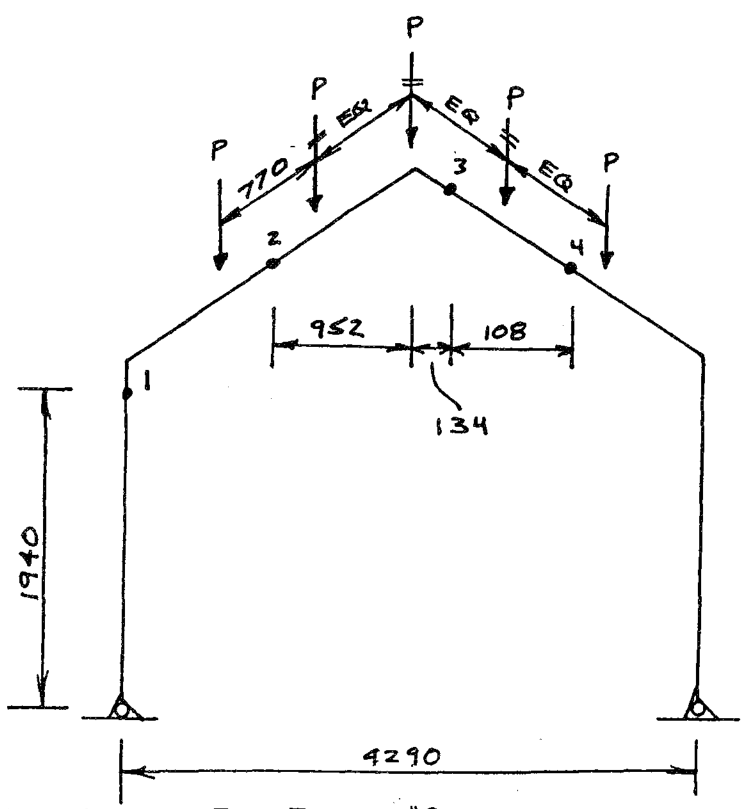


FIGURE 13.

Full Load Deflection Curves For Frame #2.

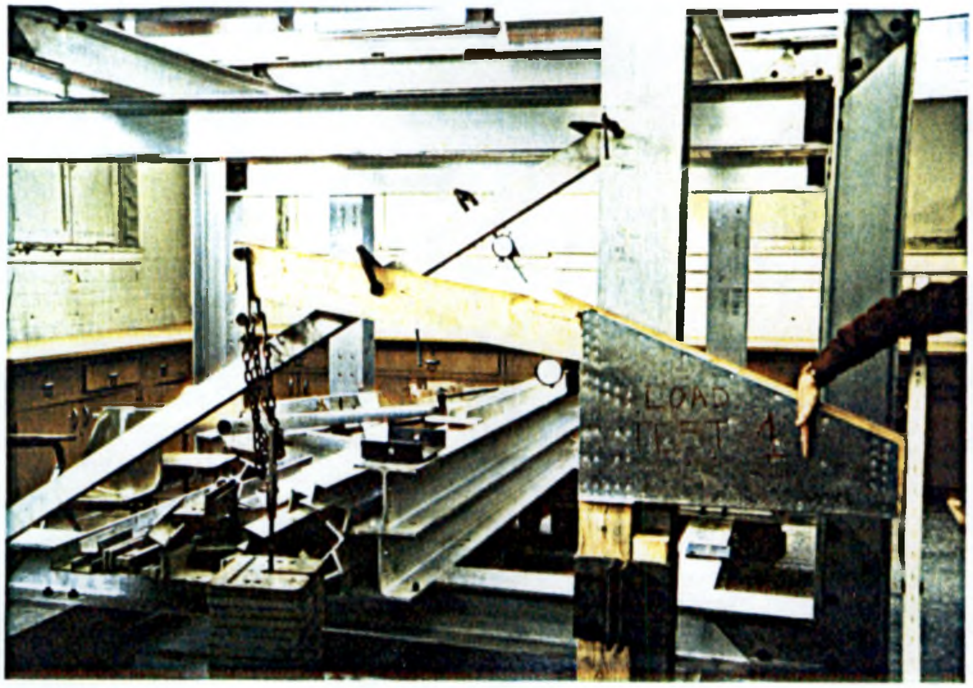


PHOTO 1.
LOAD TEST NO. 1 AT FAILURE.

PHOTO 2.
PHOTO ILLUSTRATING THE DEGREE OF DAMAGE AT FAILURE POINT.

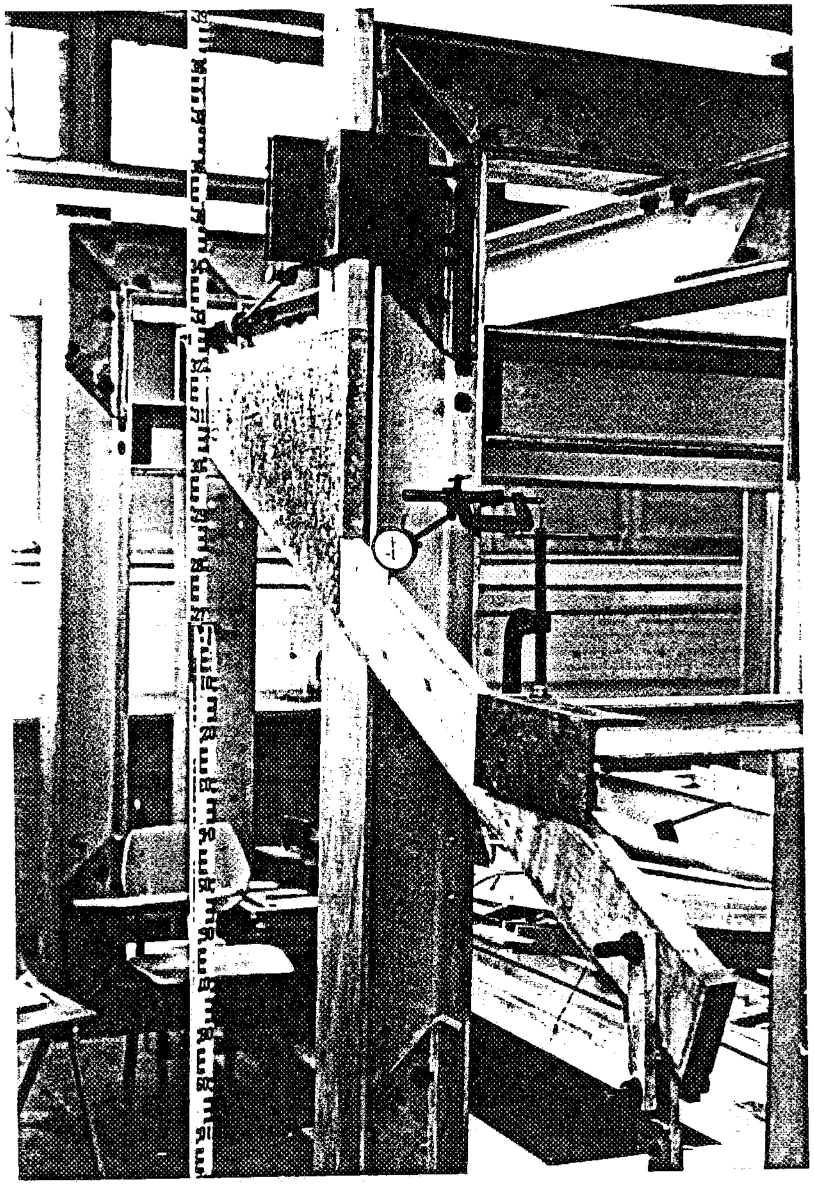


PHOTO 3.

TYPICAL TEST SET-UP.

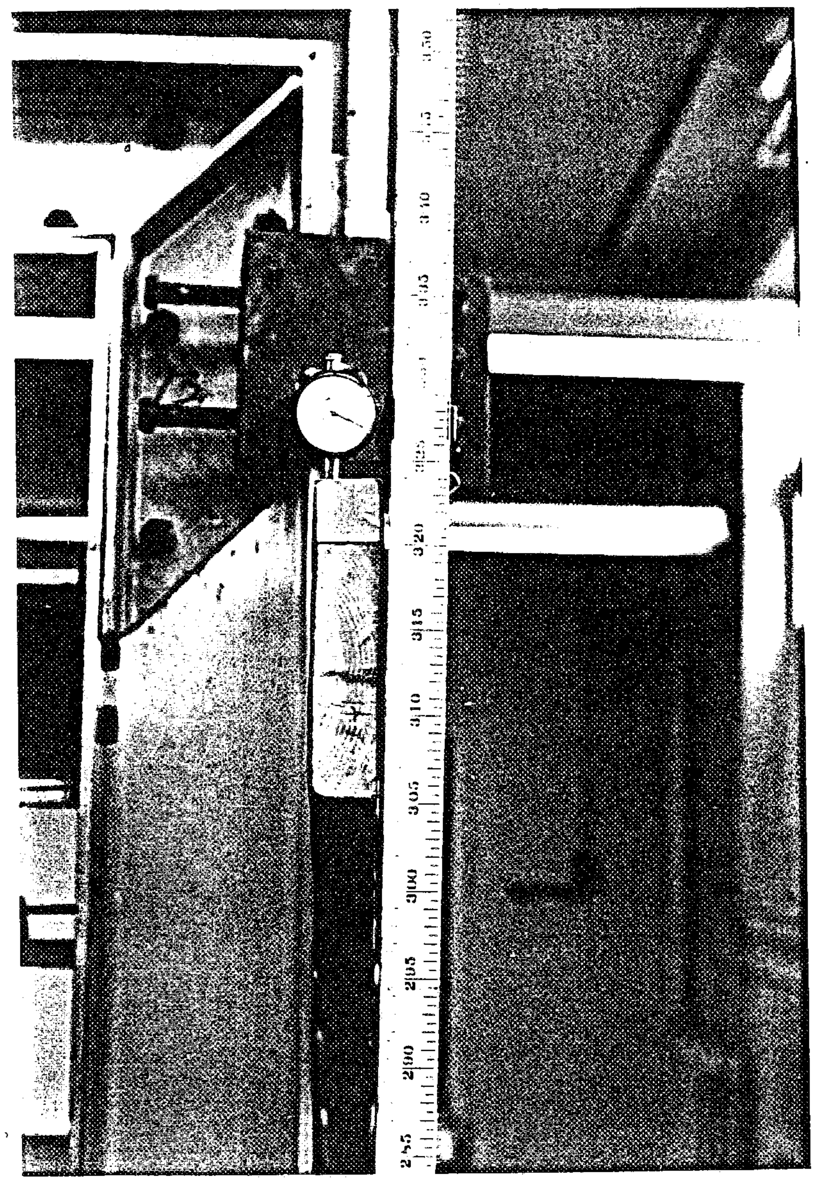


PHOTO 4.

APPARATUS USED TO MEASURE DEFLECTION.

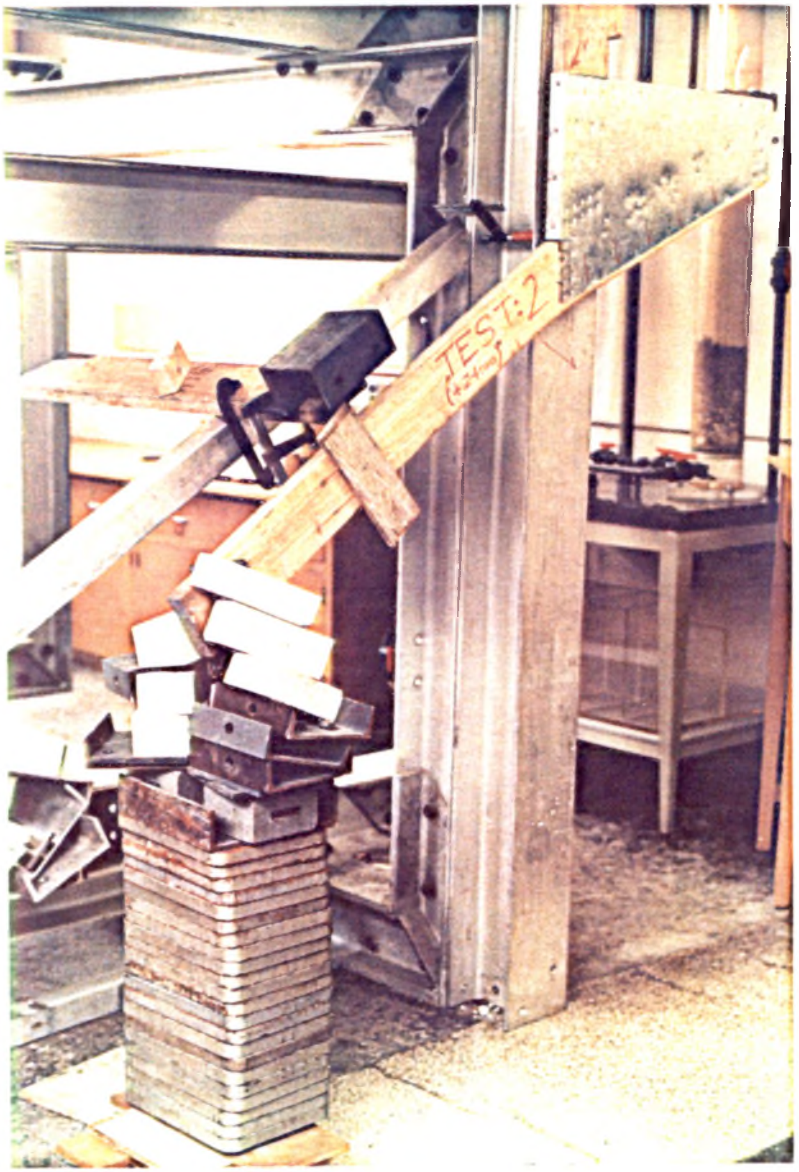


PHOTO 6.

LOAD TEST NO. 2. AT FAILURE.

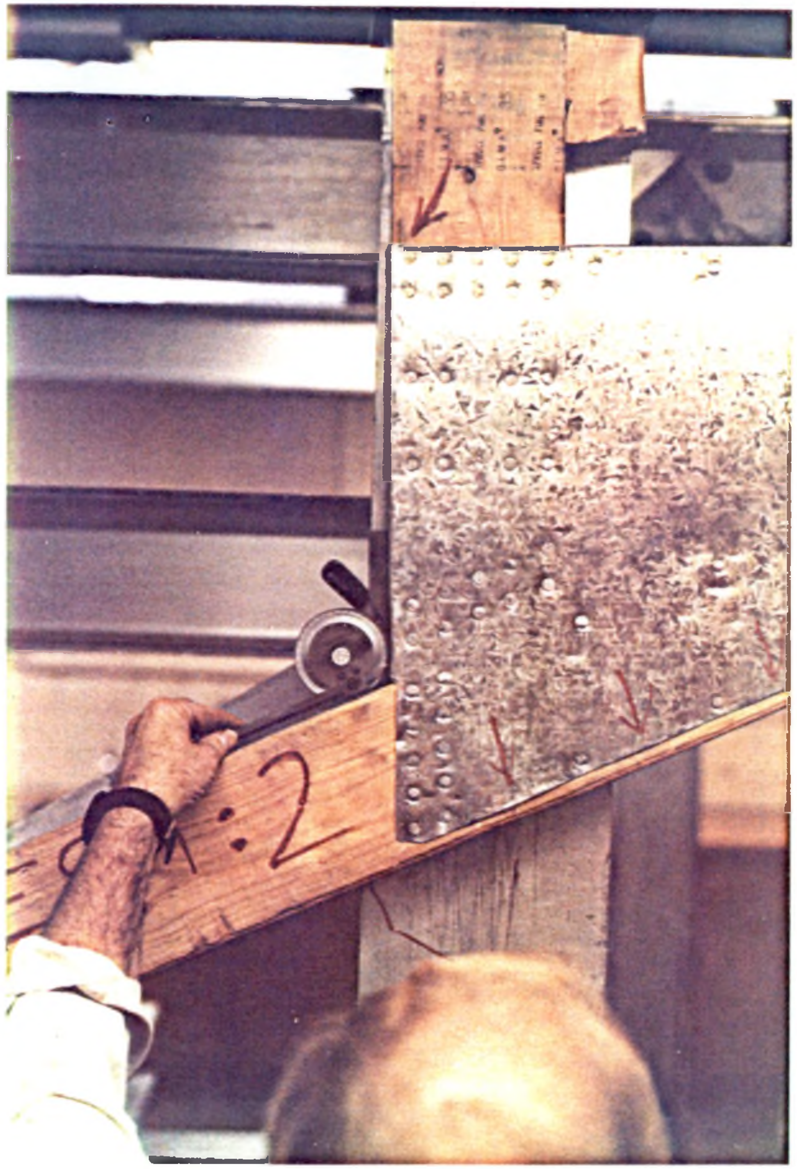


PHOTO 5.

MEASURING THE ANGLE BETWEEN THE ROOF CHORD AND THE GUSSET PLATE.

PHOTO 8.

LOAD TEST NO.3 AT FAILURE.

PHOTO 7.

FAILURE POINT OF LOAD TEST NO. 2.

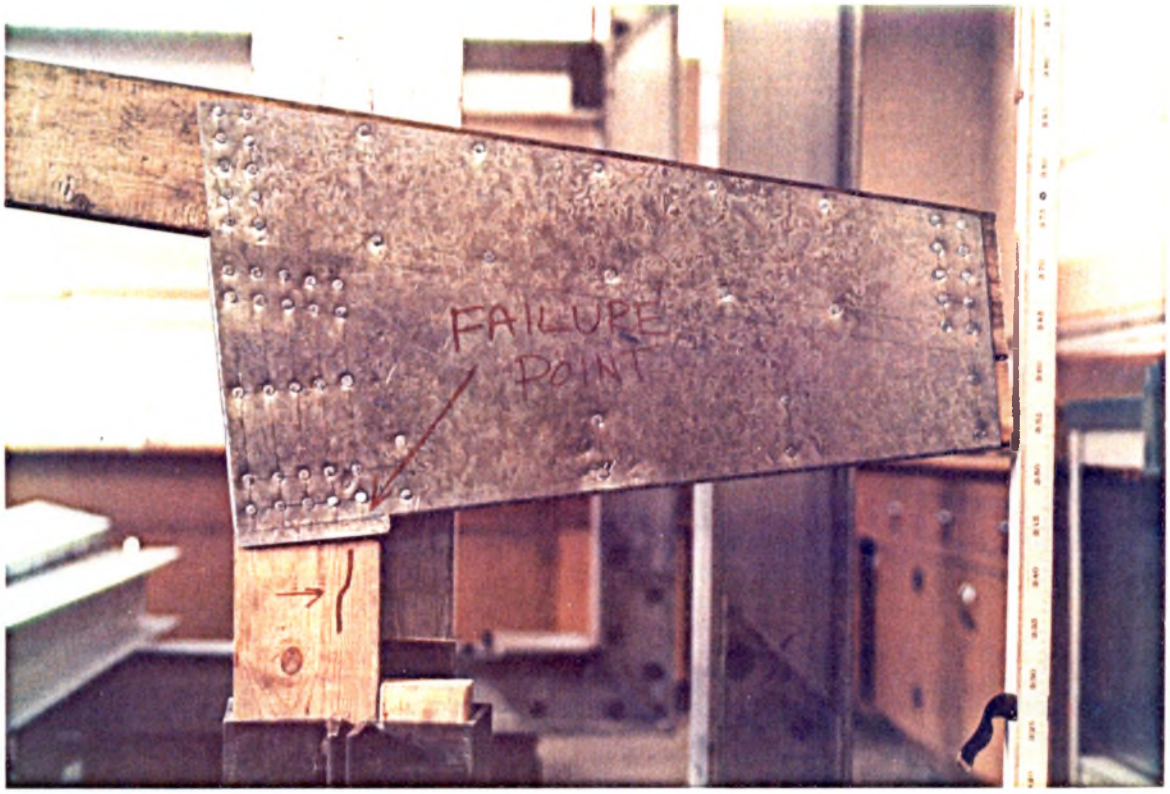


PHOTO 9.
FAILURE POINT OF LOAD TEST NO. 3.

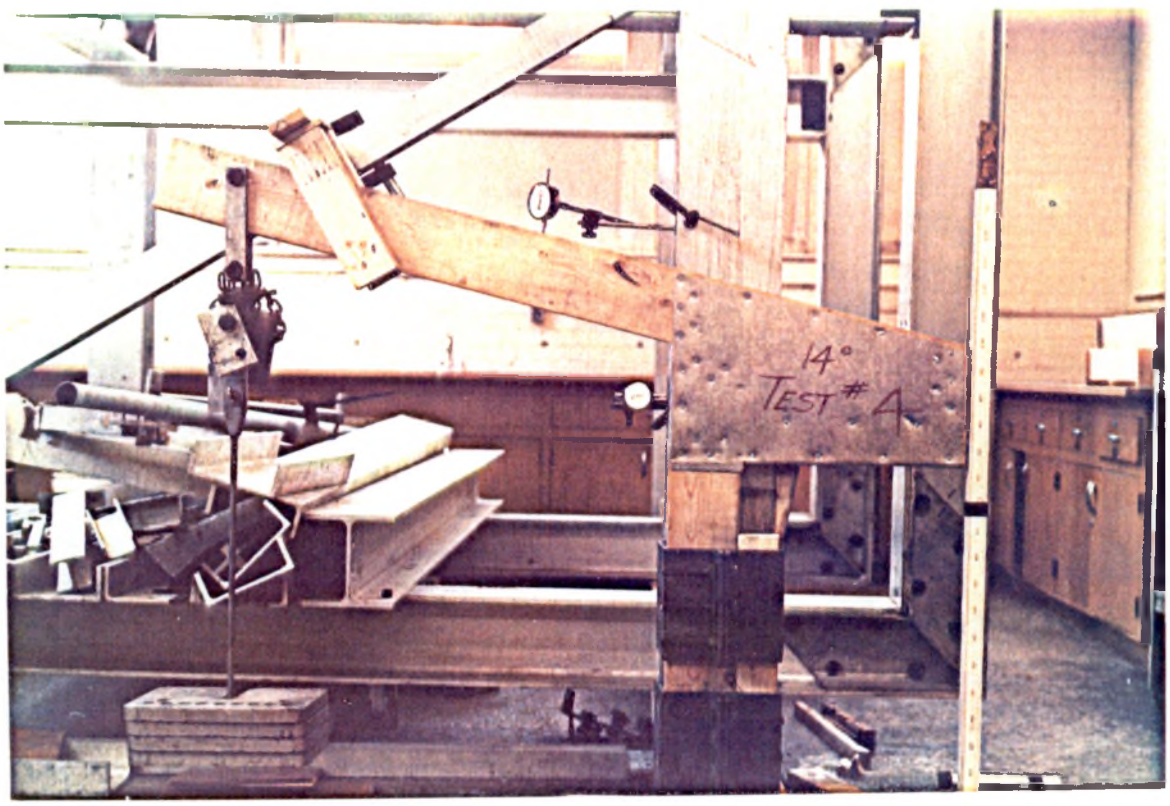


PHOTO 10.
SET UP OF LOAD TEST NO. 4.

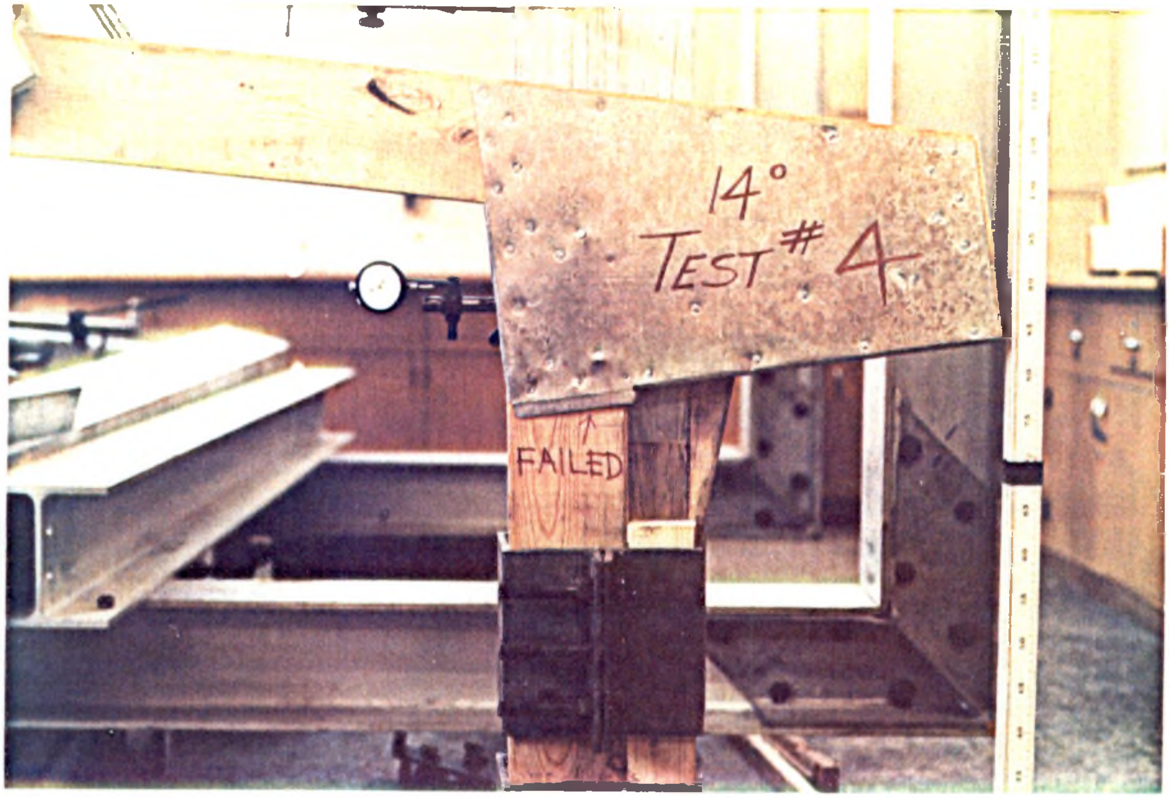


PHOTO 11.
FAILURE POINT OF LAOD TEST NO. 4.

PHOTO 12.
CRACKING PARALLEL TO GRAIN CAUSING FAILURE, TEST NO. 4.

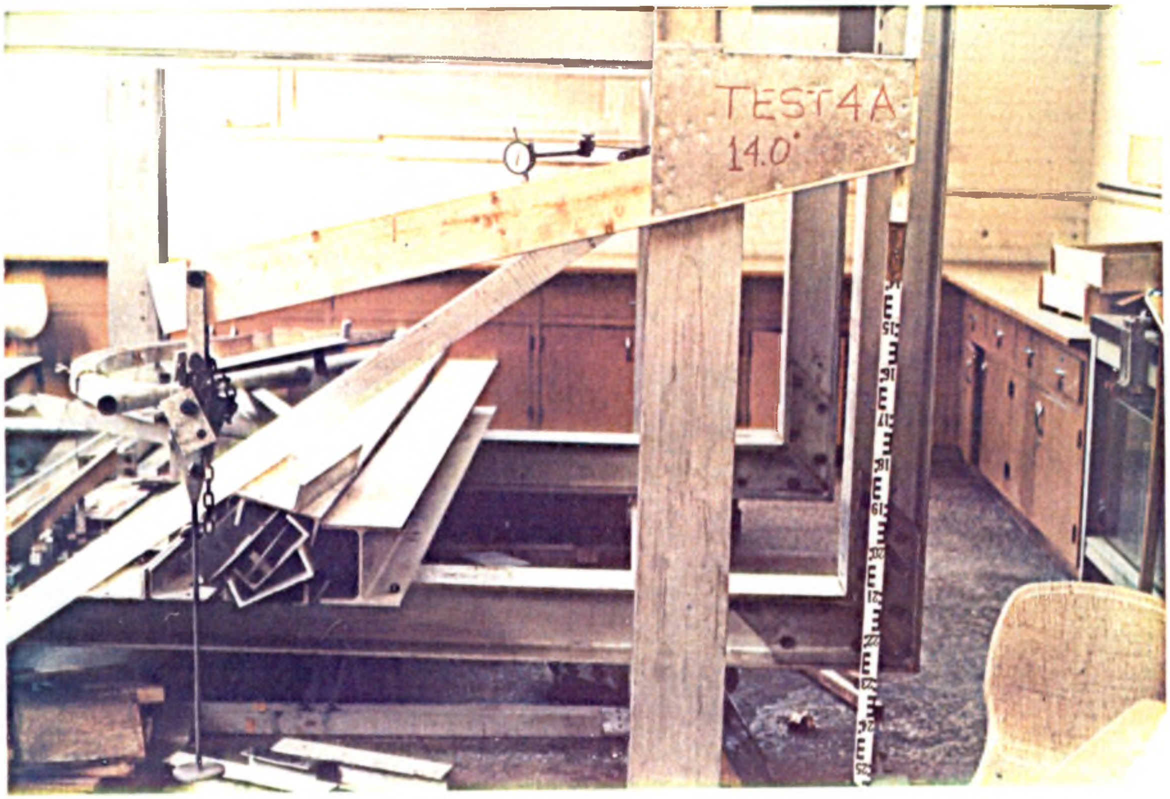


PHOTO 13.
SET-UP OF LOAD TEST NO. 4A.

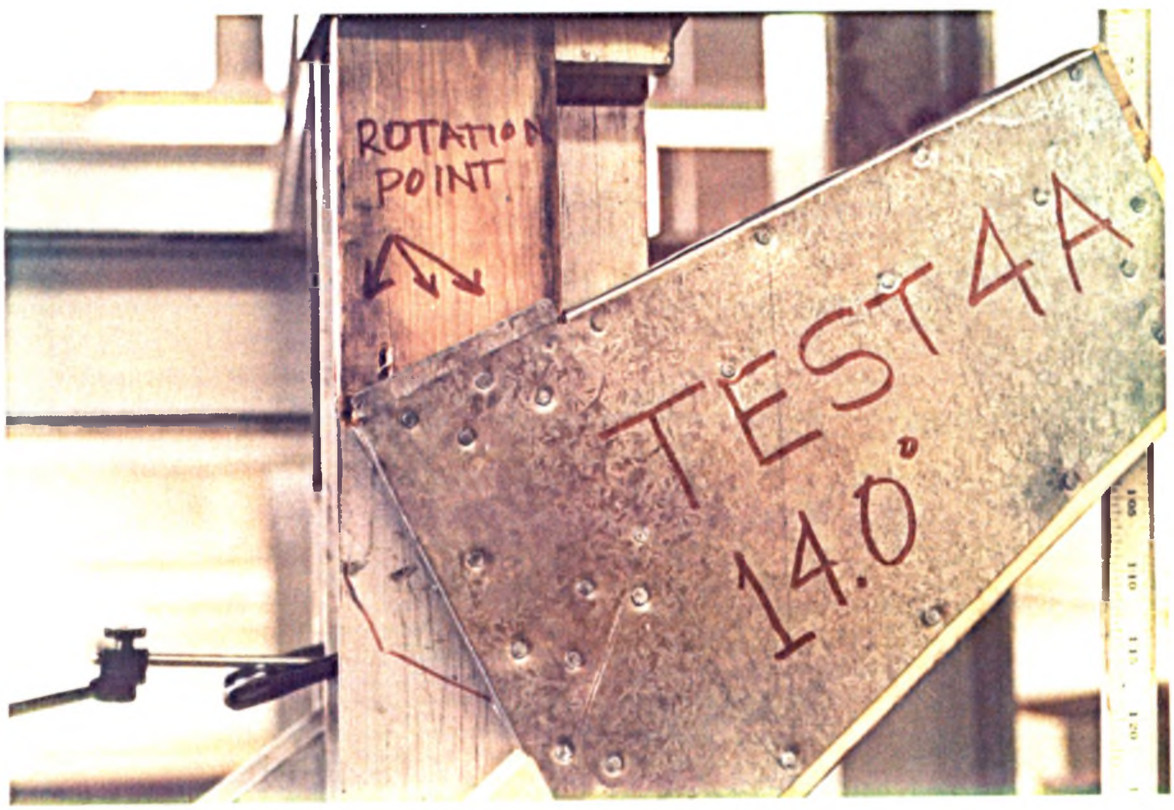


PHOTO 14.
FAILURE POINT OF LOAD TEST NO. 4A.

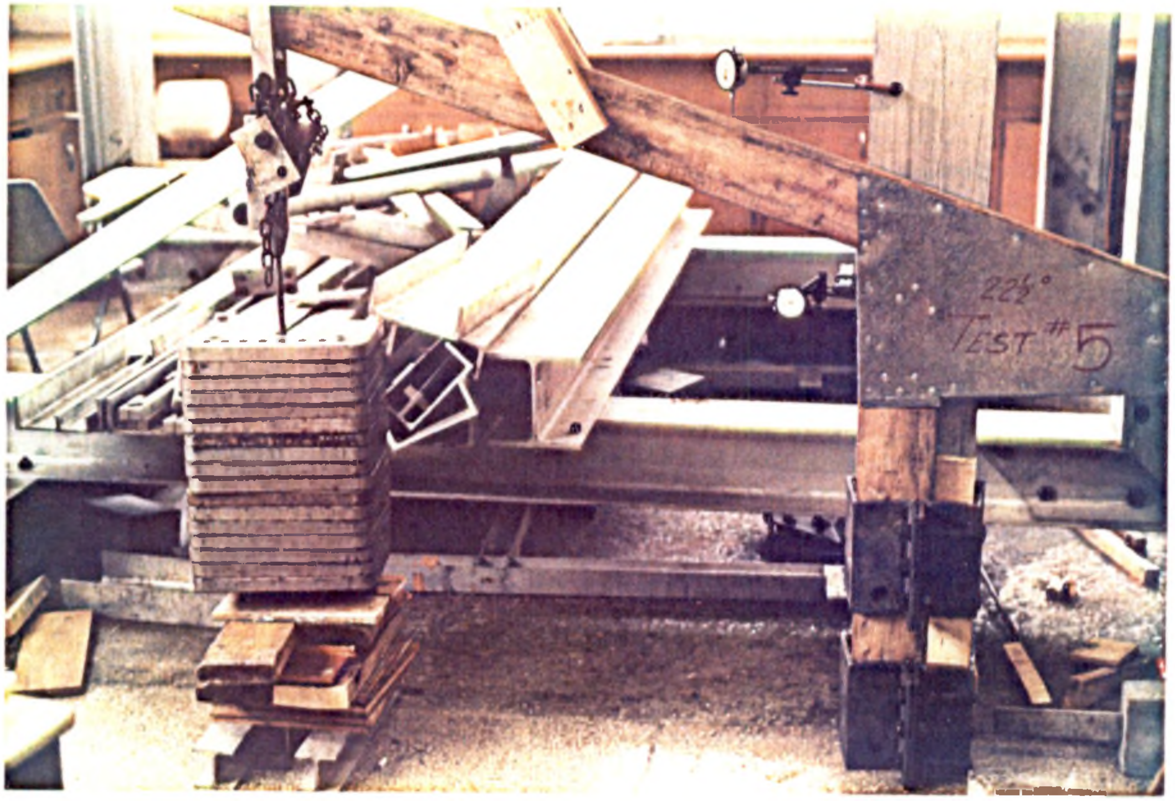


PHOTO 15.
PHOTO OF LOAD TEST NO. 5 PRIOR TO FAILURE.

PHOTO 16.
CRACKING PARALLEL TO GRAIN CAUSING FAILURE IN LOAD TEST NO. 5.

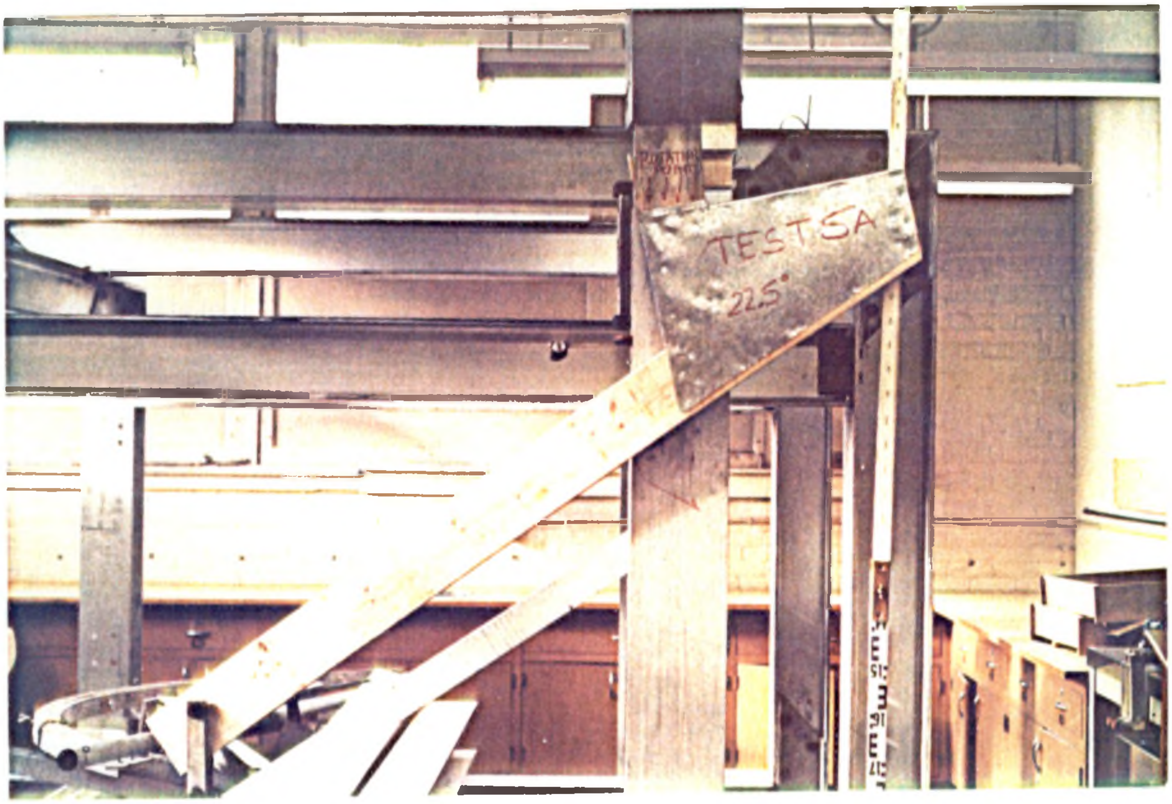


PHOTO 17.
FAILURE OF LOAD TEST NO. 5A.

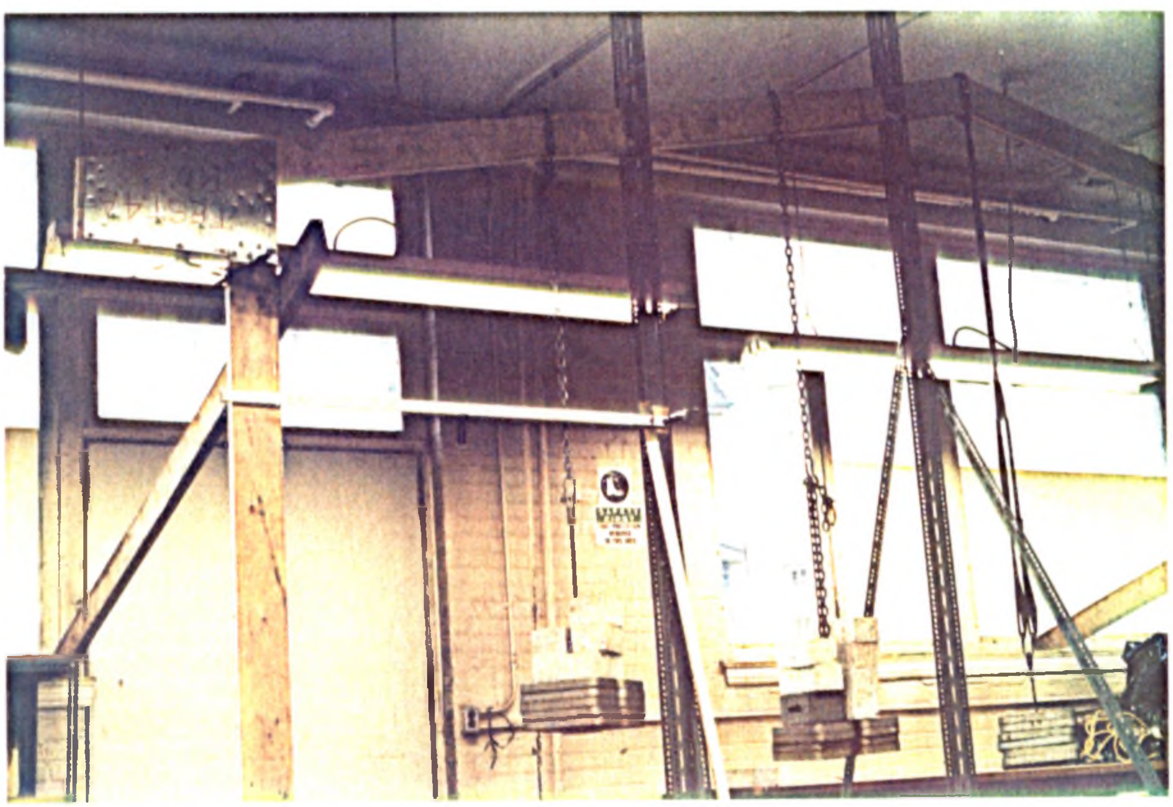


PHOTO 18.
SET-UP OF FRAME NO. 1.

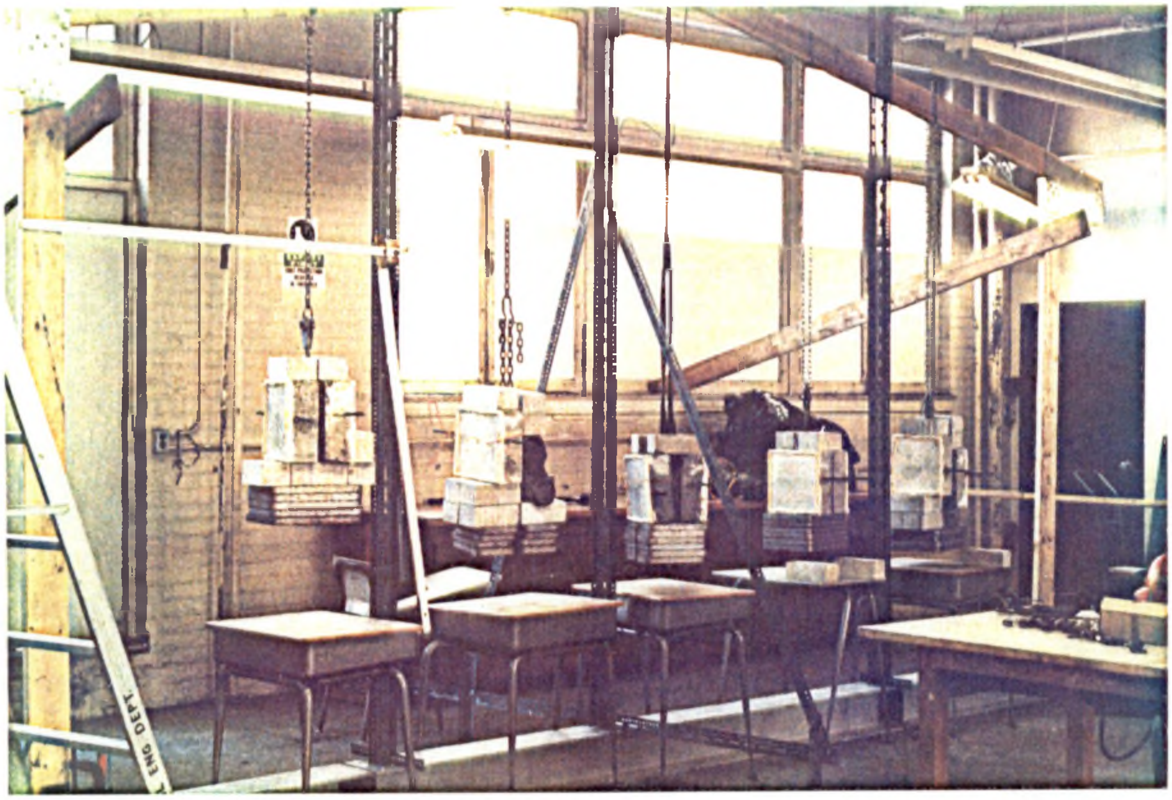


PHOTO 19. (ABOVE)
PHOTO SHOWING FRAME
NO. 1 FULLY LOADED.

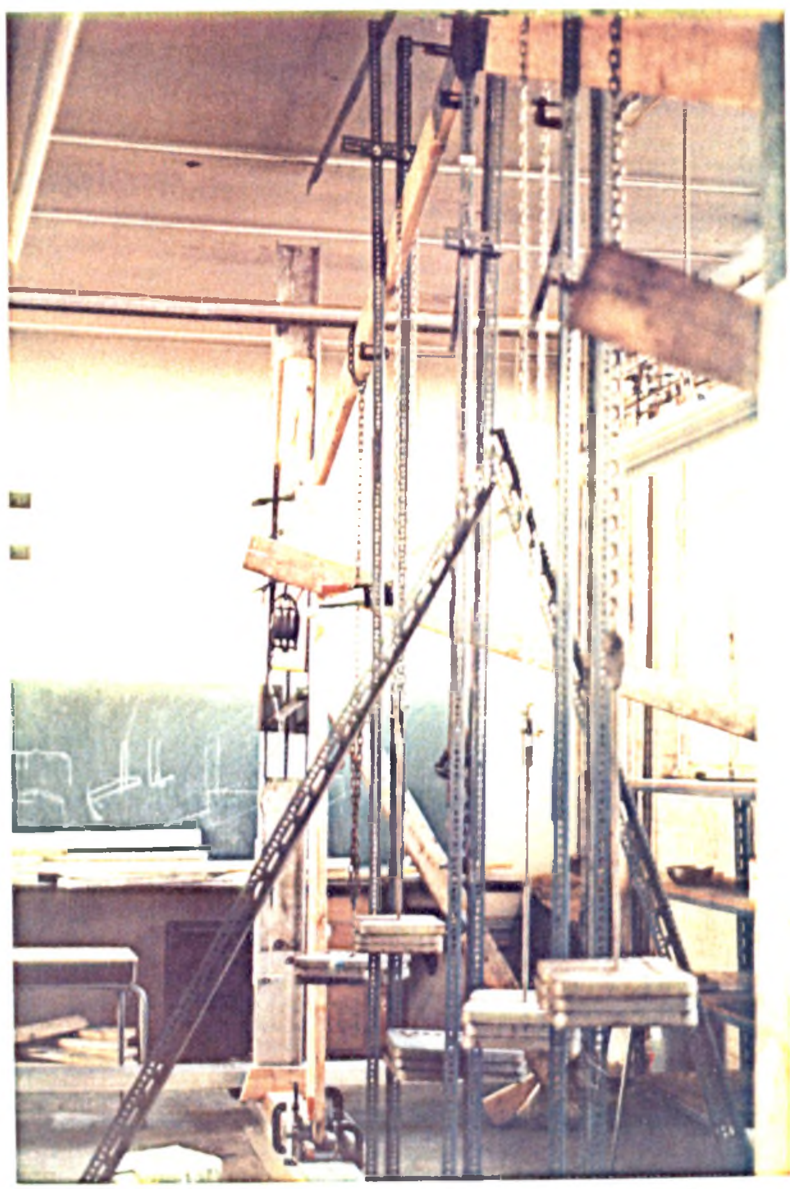


PHOTO 20. (RIGHT)
PHOTO SHOWING FRAME
NO. 2 PARTIALLY LOADED.

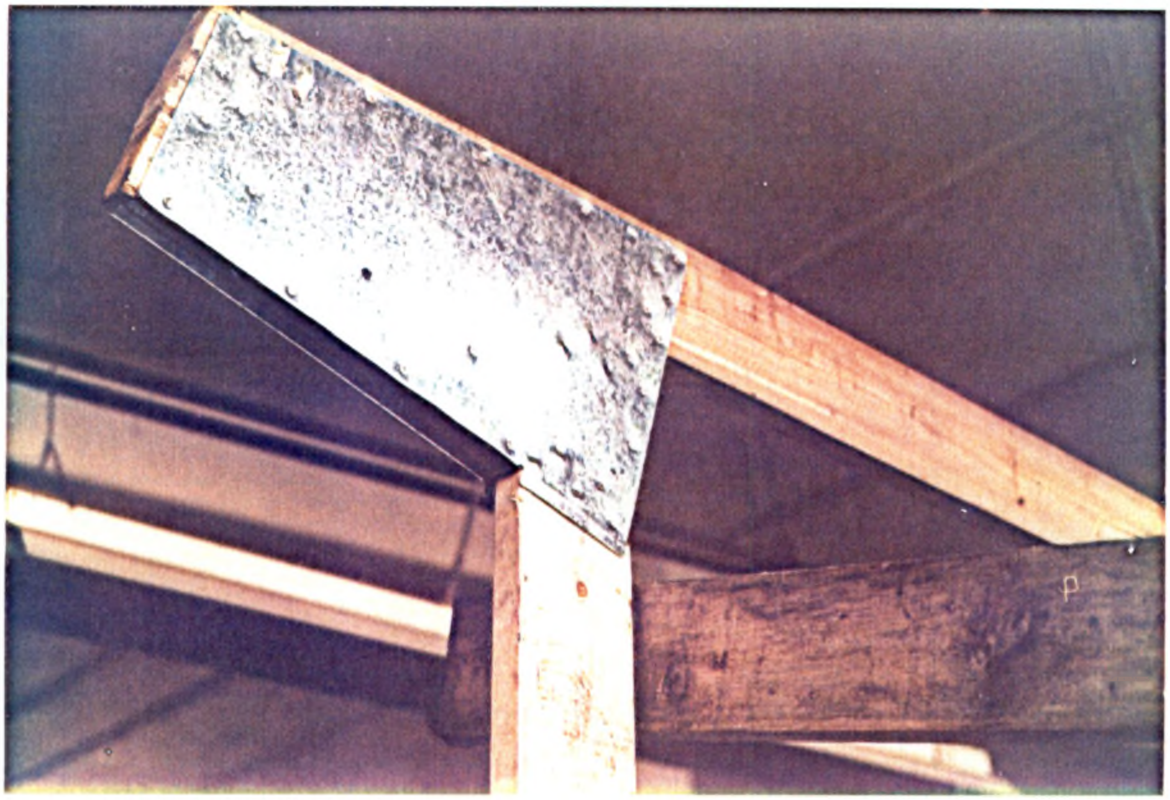


PHOTO 21.

PHOTO SHOWING A FAILURE POINT ON FRAME NO. 1.

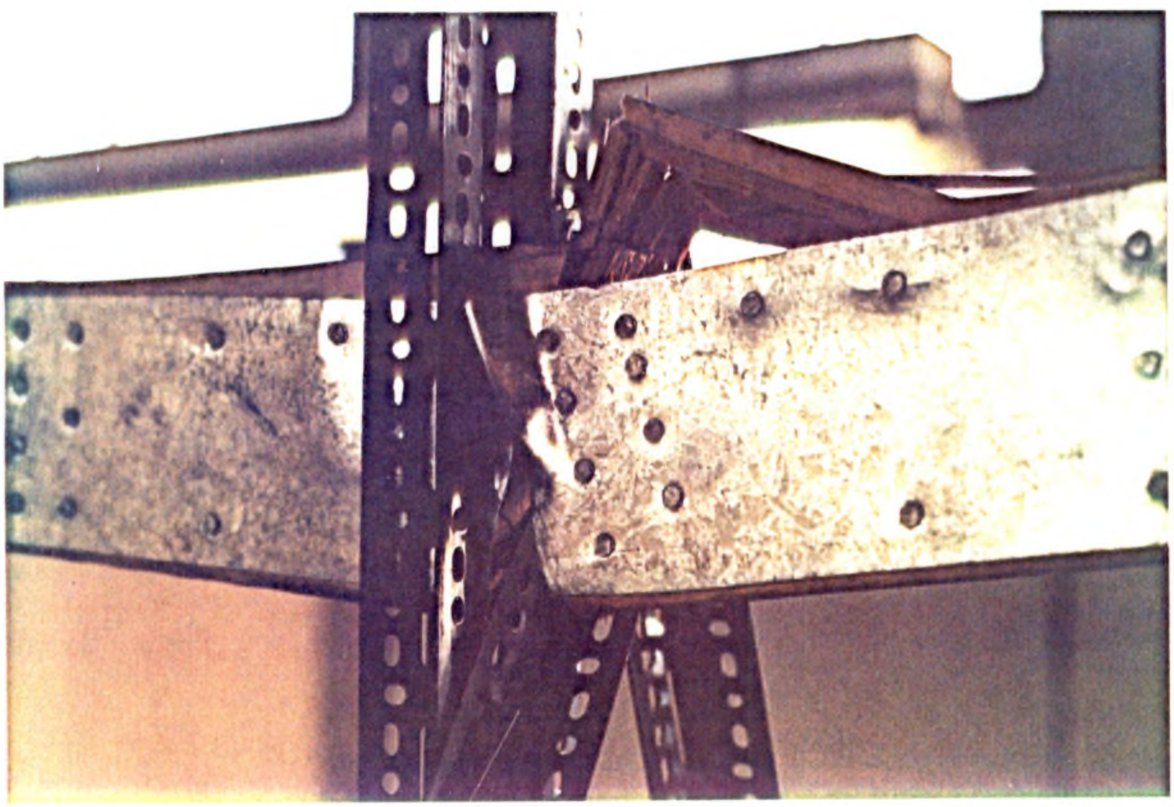


PHOTO 22.

PHOTO SHOWING FAILURE AT PEAK OF FRAME NO. 1.

APPENDIX B

CSA Standard S307-M1980

Canadian Standards Association

CSA Standard
S307-M1980

Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings

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Preface

This is the second edition of CSA Standard S307, Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings. The Standard is written in SI (metric) units and supersedes an earlier edition published in 1977.

In addition to metric conversion of the Standard and editorial changes, two changes have been made in the test procedure itself. A 1-week interval is now required between the time the trusses are assembled and the time they are tested. The procedure also calls for load testing with an unbalanced load.

The Standard was prepared by the Technical Committee on Engineering Design in Wood under the jurisdiction of the Standards Steering Committee on Structures and was formally approved by these Committees.

Rexdale, March, 1980

Note: *All enquiries regarding this Standard should be addressed to Canadian Standards Association, 178 Rexdale Boulevard, Rexdale, Ontario M9W 1R3.*

CSA Standards are subject to periodical review and suggestions for their improvement will be directed to the appropriate committee.

Requests for interpretation will also be accepted by the committee. They should be worded in such a manner as to permit a simple "yes" or "no" answer based on the literal text of the requirement concerned. Formal interpretations are published in "CSA Information Update" (for subscription details and a free sample copy, write to CSA Information Central or telephone (416) 744-4128).

**Reference
Publication**

This Publication refers to the following and the year date shown indicates the latest issue available at the time of printing:

National Building Code of Canada, 1977.

April, 1980

S307-M1980

Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings

1. Scope

1.1 General
This Standard covers the load test procedure for evaluating the strength and deformation performance of wood roof trusses intended for use in housing and small buildings as defined in Part 9 of the National Building Code of Canada.

Note: *Part 9 of the National Building Code of Canada provides detailed requirements for the construction of houses and small buildings up to* 600 m^2 *in building area and 3 storeys in building height and applies to all occupancies except assembly, institutional, and high-hazard industrial.*

1.2 This Standard does not stipulate mandatory performance criteria (see Appendix A for acceptable performance criteria).

1.3 Intent of Test
This load test procedure is intended to evaluate the adequacy of wood roof truss designs and is not intended as a quality control test. The procedure may, however, be used for retesting production run samples in cases where the quality of the truss is in dispute.

Note: *It is recognized that measures to ensure the quality of production of trusses must be maintained for on-going production of designs acceptable under this Standard.*

1.4 Limitations
Assemblies to be tested by this procedure are limited to:

(a) Triangulated trusses having a maximum span of 12 m; or

(b) Bowstring or lenticular trusses having a maximum span of 4.3 m;

whose members are connected using mechanical connectors or nailed gusset plates and which are intended to be installed at a maximum spacing of 600 mm on centres and in such a manner that the top chords are suitably sheathed or braced to prevent lateral buckling.

Note: *Clause 2 has been renumbered A2 and has been relocated in Appendix A, page 18.*

Effective date: November, 1983

3. Selection of Trusses

3.1 The trusses to be tested shall be full-scale and shall be representative of those to be made or being made in regular production and shall be fabricated with the same type of equipment and methods as are used for the regular production. The lumber used in the test trusses shall be grade-stamped and of the same species combinations and grades to be used in the regular production of trusses and shall have relative density within the ranges shown in Table 1 for the species combinations specified.

Table 1
Range of Relative Density of Lumber for Testing Purposes*

Species Combinations	Stamp Identification	Range of Relative Density†
Douglas-Fir-Larch	D. Fir-L (N)	0.39—0.46
Hemlock-Fir	Hem-Fir (N)	0.31—0.37
Eastern Hemlock-Tamarack	Hem-Tam (N)	0.36—0.41
Coast Species	Coast Species	0.30—0.36
Spruce-Pine-Fir	S-P-F	0.31—0.36
Western Cedars	W. Cedar (N)	0.28—0.32
Northern Species	North Species	0.28—0.32
Red Pine or White Pine‡		0.31—0.37
Northern Aspen	N. Aspen	0.33—0.38

(Continued)

Table 1 (Continued)

**In testing trusses to establish the validity of truss designs it is important that the wood used be representative of the species combination and grade specified. As wood strength and stiffness are related to relative density, some control of the relative density of wood can assure that the wood is not unrepresentative. To be conservative, the relative density ranges shown are towards the low end of the scale of relative densities associated with each species combination.*

†*Based on volume saturated, mass oven-dry.*

‡*The Pines are included in the Northern Species species combination but are also sold separately using a variety of stamp identifications, all of which include the word "pine".*

3.2 Joint fitting and connector plate placement shall comply with the tolerances specified in the design.

3.3 When a maximum moisture content is not specified for the production line trusses, the trusses shall be assumed to be manufactured from unseasoned lumber and the test trusses shall be fabricated from lumber with a moisture content in excess of 30 per cent. The maximum moisture content at the time of test shall not exceed 19 per cent.

3.4 A minimum period of 1 week shall elapse between the time of assembly and the testing of wood roof trusses under this Standard.

4. Test Set-Up

4.1 General

Two full-scale trusses shall be tested to evaluate a design, either individually or as a pair. Supports and bracing shall simulate but not exceed those called for in the design drawings.

4.2 Test of Single Trusses

The top chord may be prevented from buckling by lateral restraints, provided these supports do not in any way restrain vertical deflection of the truss or assist in carrying any applied load.

4.3 Test of Pairs of Trusses

Lateral stability during test shall be ensured by spacing the trusses at least 600 mm apart even if they are designed to be more closely spaced in actual use. The top chords shall be sheathed with plywood or may be prevented from buckling by lateral restraints, provided these supports do not in any way restrain vertical deflection of the truss or assist in carrying any applied load. The length or width of the plywood sheets shall not exceed 1200 mm and thickness shall not exceed 9.5 mm. Plywood sheets are to be placed with the face grain at right angles to the top chords. The sheathing shall be nailed to the top chords with 50 mm common nails at 150 mm spacing and a gap of at least 3 mm left between sheets. The top of bottom chords may be sheathed with plywood in the same manner as the top chord, or, alternatively, boards or slats applied at right angles to the bottom chord may be used at least at the panel quarter points.

5. Deflection Measurements

5.1 Unless otherwise approved, deflections shall be measured at the following points of the bottom chord of each truss:

(a) Mid-span and other likely points of maximum deflection;

(b) All splices of bottom chord;

(c) All panel points (joints) between end supports;

(d) The end of the cantilevered bottom chord of cantilevered trusses.

5.2 Deflections shall be measured by means of a taut wire or other line not more than 0.5 mm in diameter. A graduated scale with graduations of 1 mm or finer shall be attached to the bottom chord at points where deflections are measured. Measurements shall be made in such a way as to avoid errors due to parallax. (This may be accomplished by using small mirrors fastened to the member or by using a double line.)

5.3 Other methods for measuring deflections may be used if approved by the authority having jurisdiction.

5.4 When the trusses are not cantilevered the taut wire or line shall be fastened at one end to a screw or nail in the bottom chord member located directly over the centre of the support and held taut by means of an elastic or spring fixed to a nail or screw located directly over the centre of the other support. Alternatively, the line may be kept taut by suspending a weight from one end of the line outside the nails or screws over the supports. Care shall be taken to ensure that the line does not contact the truss supports during the test.

5.5 When trusses are cantilevered, the deflections shall be measured by means of a taut line spanning the entire length of the bottom chord. This line shall be located high enough above the support at the cantilevered end so that it will not contact the support when the cantilevered portion deflects. A scale shall be placed on the bottom chord above this support to measure the movement of the line relative to the scale. The bottom chord deflections shall be determined relative to a line drawn through the support points and extended to the cantilevered end.

6. Loads for Testing

6.1 When the trusses are intended to support roofs where the total dead load due to sheathing and roofing exceeds 0.025 kN/m^2, the excess over 0.25 kN/m^2 shall be added to the design roof snow load and used in the determination of the deflection and strength test loads according to Clauses 6.2 and 6.3.

6.2 The deflection test load applied to the top chord shall be 1-1/3 times design roof snow load.*

6.3 The strength test load applied to the top chord shall be 2-2/3 times design roof snow load.*

**The design roof snow load according to Part 9 of the National Building Code of Canada is not less than 60 per cent of the ground snow load listed in Supplement No. 1 of the NBC or 1 kN/m^2, whichever is greater. However, for flat roofs, particularly those adjacent to higher roofs, the designer will need to take into account factors such as drift loads, ice accumulation, and ponding in assigning appropriate spacing to the trusses.*

6.4 The test load applied to the bottom chord shall be not less than 0.5 kN/m^2 for both the deflection test and the strength test. For trusses where the bottom chord load anticipated in service is more than 0.5 kN/m^2 or includes concentrated loads, the test load shall be not less than that anticipated, subject to the approval of the authority having jurisdiction.

6.5 Loads may be applied by means of weights, hydraulic jacks, or other suitable apparatus.

6.6 The method of measuring the loads shall be accurate to ± 5 per cent.

6.7 When weights are used for loading, provision shall be made to prevent complete collapse or overturning of the trusses, which could result in injury to people close by. Such safety supports shall not be in contact with the trusses during test and should be adjustable in height so that they can be maintained within 50 mm of the members they are intended to catch.

6.8 When weights are used, they shall be distributed uniformly along the truss and sufficient space left between weights to avoid arch action. If concrete blocks or other rigid weights are used, their length in the direction parallel to the truss shall not exceed 380 mm, except that, for bowstring or other curved top trusses, this length shall not exceed 190 mm.

6.9 When a load is applied as concentrated loads produced by hydraulic jacks or other suitable loading apparatus, such loads shall be equal and shall be:

(a) Spaced uniformly at not more than 300 mm on centre; or

(b) Applied to at least two points in each panel, so located that the maximum bending moments and shear forces produced by those loads are the same as would be produced by the required uniformly distributed load, taking into account continuity over panel points; or

(c) Spaced uniformly so that at least three load points fall within each top chord panel.

6.10 When the trusses are to be supported at or near the ends of the bottom chord and have an "ordinary roof overhang" (i.e., 600 mm or less), none of the test load shall be applied outside of the truss span.

6.11 When trusses are the cantilevered type in which the support at one or both ends is located more than 600 mm from the end of the lower chord, the cantilevered overhang, including the eave projection, shall be loaded with the same unit loads as are applied to the rest of the truss.

7. Testing Procedure

7.1 Measure the moisture content of all chord members immediately before the test.

7.2 Read all deflections to the nearest 0.5 mm.

7.3 Take zero deflection readings prior to the application of any load.

7.4 Apply the bottom chord load and read deflections 5 min after the load has been applied.

7.5 Apply the deflection test load to the top chords at a steady rate.

7.6 Read deflections 1 h after the deflection test load has been applied.

7.7 Increase the top chord load gradually on one-half of the span or, in the case of unsymmetrical peaked trusses, on the longer side to the strength test load and maintain this load for 1 h. Increase the top chord load on the remainder of the span to the strength test load and maintain this load on the full span for 24 h. The total strength test load shall be in place within not more than 5 h of the start of the application of the bottom chord load.

7.8 Determine the relative density of the lumber used in the top and bottom chord as required in Clause 3 and re-determine the moisture content immediately after the test by the oven-dry method.

8. Report

8.1 The report submitted by the testing authority shall include:

(a) Detailed design drawings of the trusses tested showing the dimensions of the trusses, the member sizes, the lumber grades and species, joint details including type of connector, size, location, and tolerances;

(b) Photographs of the trusses under the deflection test load and under the strength test load at the time of determining acceptable performance;

(c) A brief description of the fabrication method employed;

(d) The moisture content and relative density of each chord member at the time of testing (and a description of the method used to measure the moisture content and relative density);

(e) A description of the loading method used;

(f) The total loads applied to the top and to the bottom chords at both the deflection test load and the strength test load;

(g) The deflections of the bottom chord under the bottom chord load and after carrying for 1 h the bottom chord load plus the deflection test load;

(h) A statement of whether or not the strength test load was successfully carried for the required duration of total load;

(i) A statement of the ground snow load and the truss spacing for which the design was tested;

(j) A description of any abnormal or unusual behaviour of the trusses under load.

9. Acceptable Performance

9.1 The minimum performance criteria shall be specified by the authority having jurisdiction.

Note: *The strength and deflection criteria as specified in Part 9 of the National Building Code of Canada are shown in Appendix A.*

9.2 The truss design is deemed acceptable if both test trusses meet the minimum performance criteria.

9.3 If the trusses are tested singly according to Clause 4.2 and both trusses fail to meet the minimum performance criteria, the design is unacceptable. If only one of the two test trusses meets the criteria, a retest of two more trusses is permitted. If either of the two trusses in the retest fails to meet the criteria, the design is unacceptable.

9.4 If the trusses are tested as a pair according to Clause 4.3 and if only one truss fails to meet the minimum deflection criterion or if collapse occurs, a retest of two more trusses is permitted. If one or both of the two trusses in a retest fails to meet the deflection criterion or if collapse occurs, the design is unacceptable.

Appendix A

Acceptable Performance Criteria

Note: *This Appendix is not a mandatory part of this Standard.*

A1. Table A1 reproduces the strength and deflection criteria for trusses from Part 9 of the National Building Code of Canada.

Table A1
Deflection and Strength Requirements

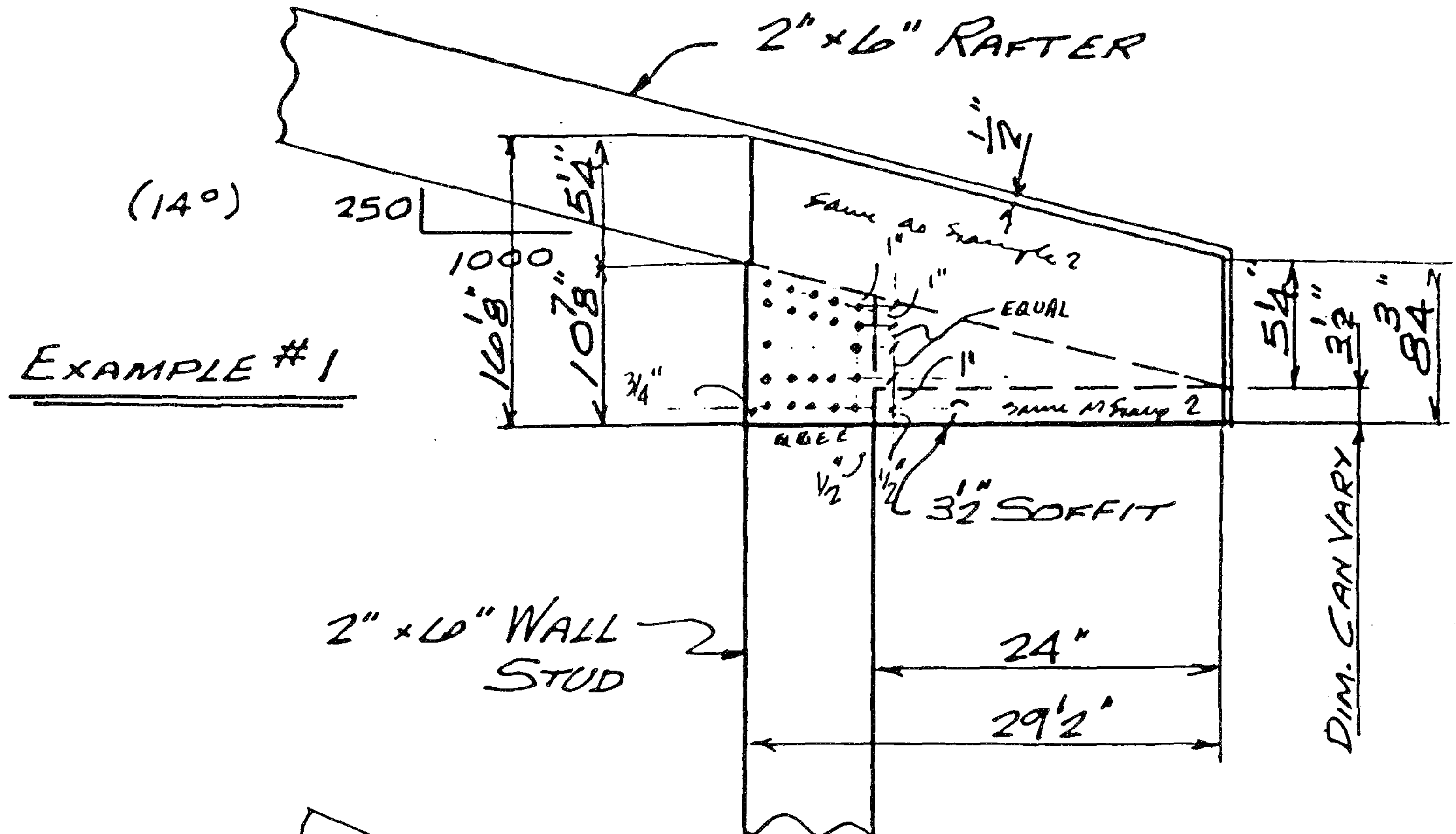
Type of Test	Load and Duration	Acceptable Performance
Deflection	Deflection test load according to Clause 6 applied for 1 h.	Deflection not greater than SPAN/360 for trusses intended to support plaster or gypsum board ceilings.
		Deflection not greater than SPAN/240 for trusses intended to support other types of ceiling finishes.
		Deflection not greater than SPAN/180 for trusses spanning not more than 4.3 m and intended to support ceiling finishes other than plaster or gypsum board.
		Deflection not greater than SPAN/120 for cantilevered portions of trusses where SPAN is taken as the length of the cantilever.
Strength	Strength test load according to Clause 6 applied for 24 h.	No collapse.

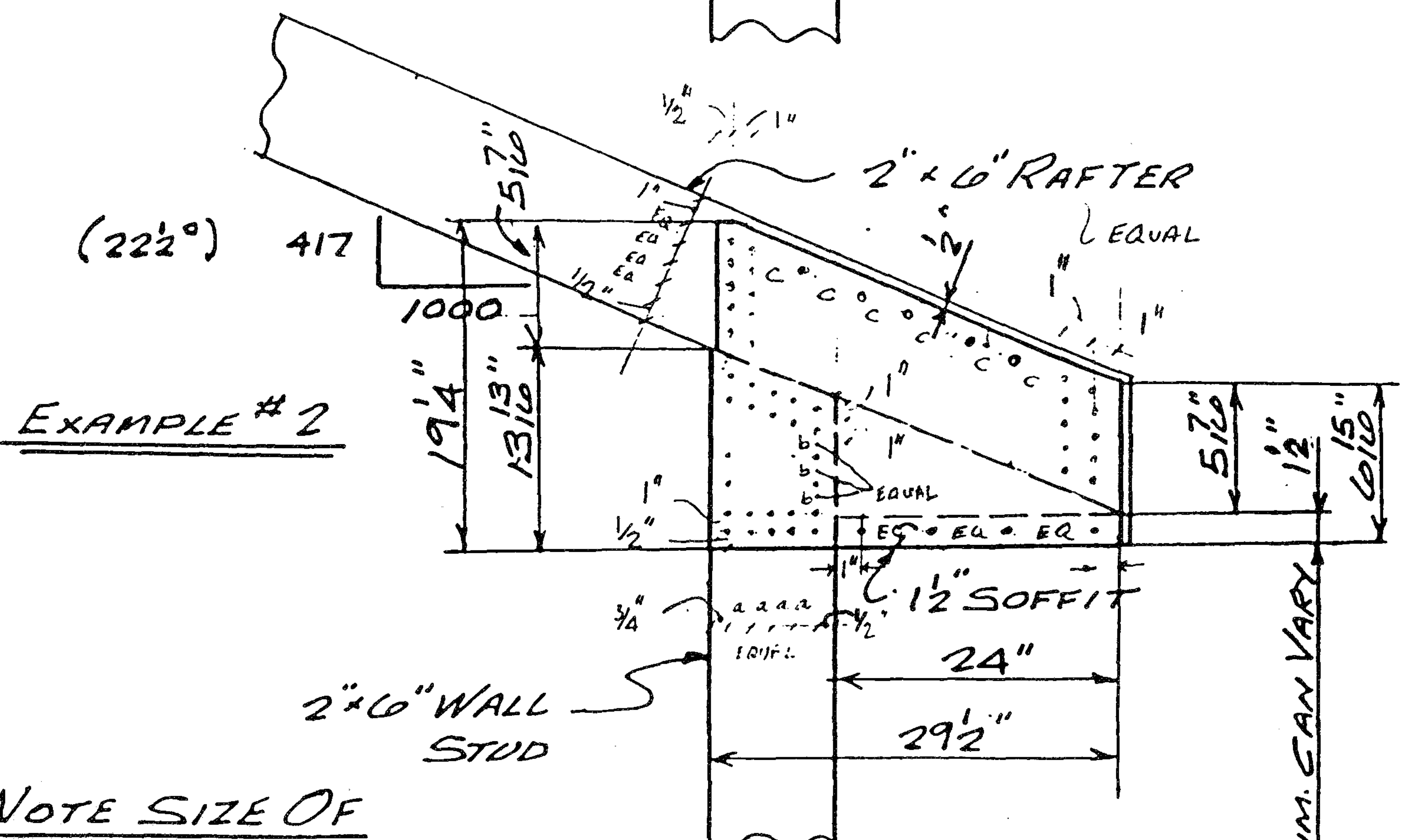
A2. Supervision

A2.1 The selection and testing of trusses shall be supervised, witnessed, and reported on by a registered professional engineer representing a testing organization acceptable to the regulatory authority.

APPENDIX C

Gusset Plate Drawings





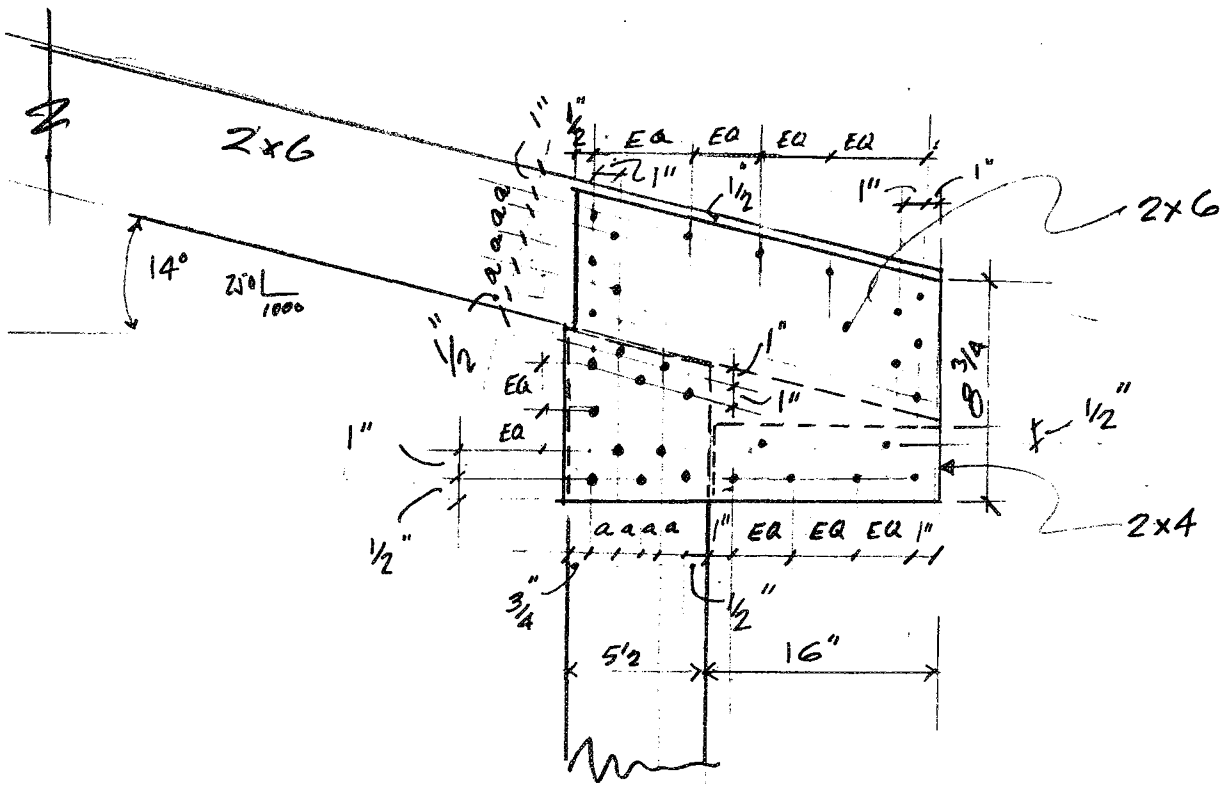
NOTE SIZE OF ALL LUMBER

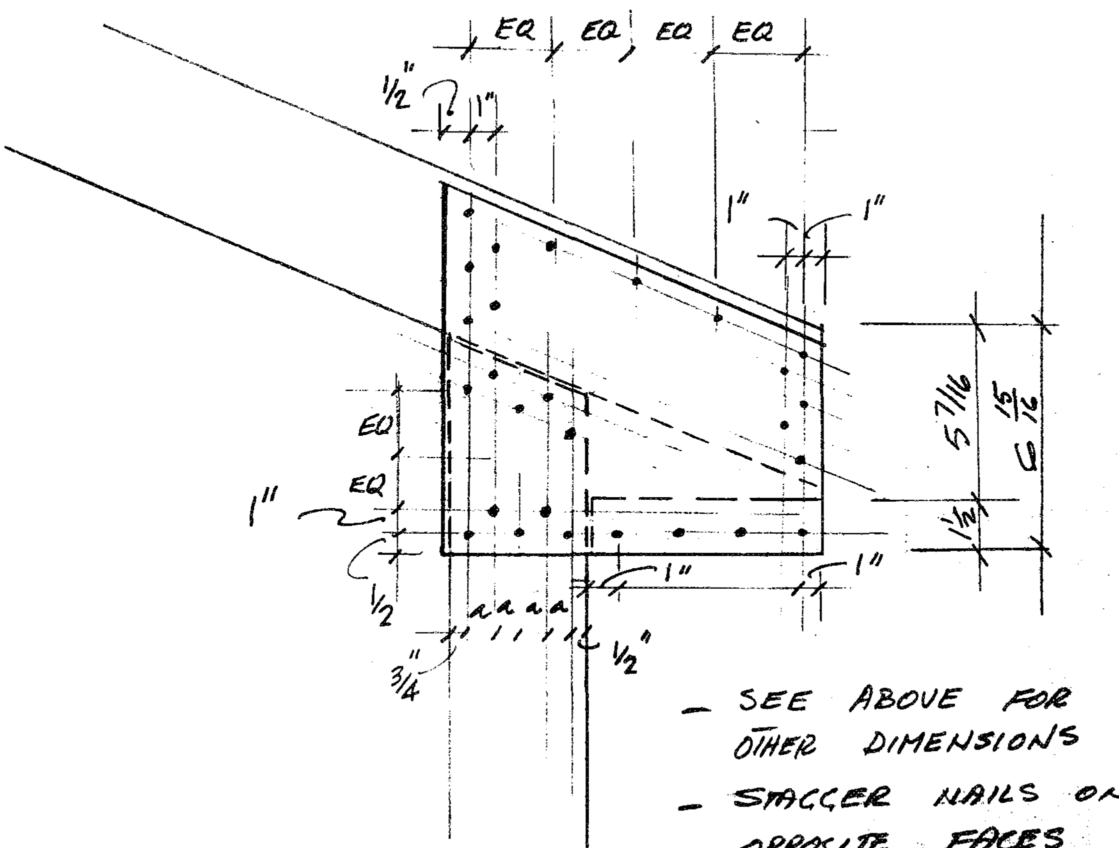
1½" × 5½"

38 × 184 LUMBER

K[illegible] 12/1/86

FIGURE 1.



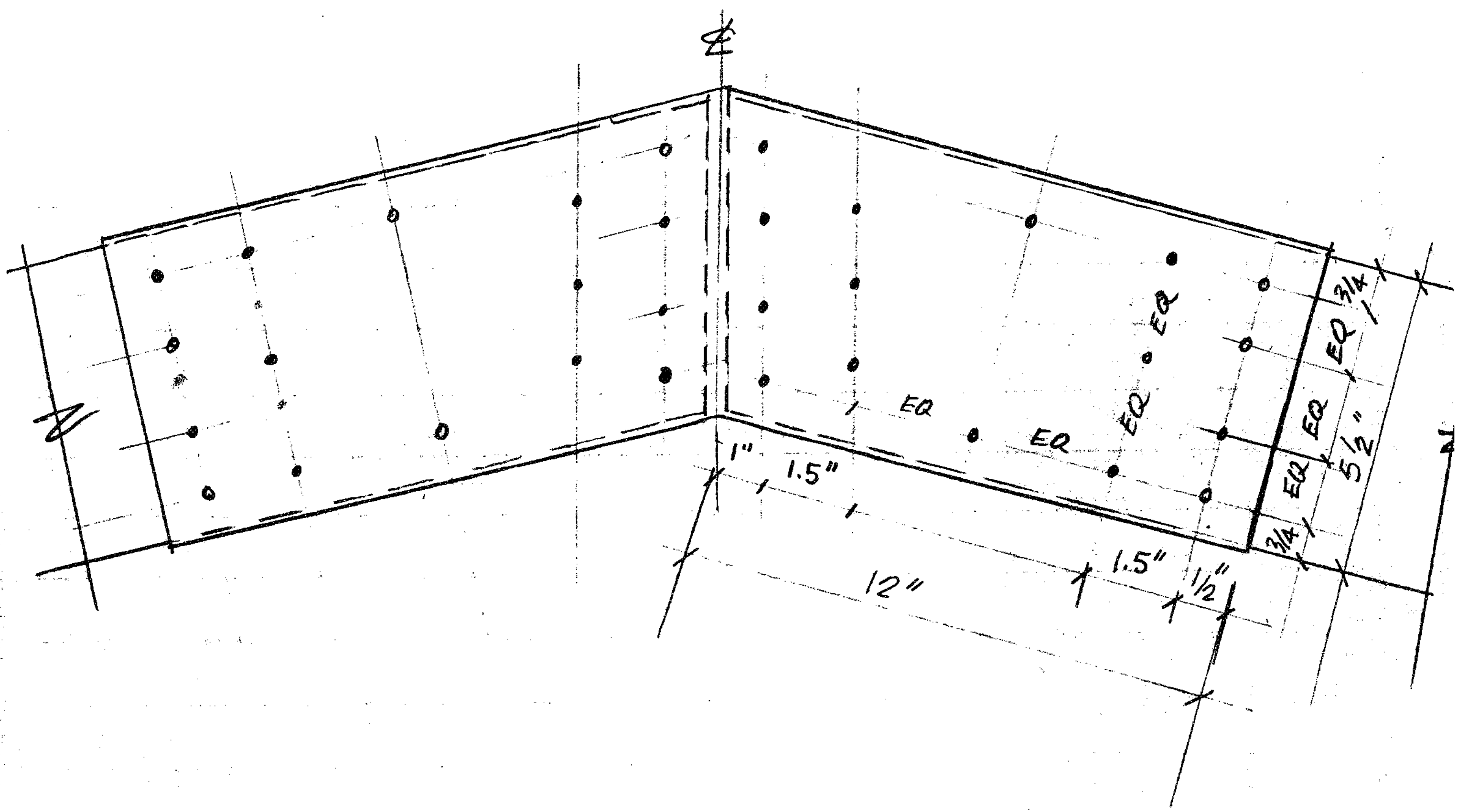


- SEE ABOVE FOR OTHER DIMENSIONS
- STAGGER NAILS ON OPPOSITE FACES

MODIFIED KNEE PLATE

FIGURE 2.

R KURKJIAN 1/20/87



DETAIL OF RIDGE PLATE

RK 2/17/87

FIGURE 3.

APPENDIX D

Span Tables For Slopes 5:12 and 3:12

SPAN TABLES FOR SLOPE 5:12 USING 38X140

W(KPa)	L(m)	M(KNm)
2.82	3.05	-1.47
2.28	3.36	-1.37
1.88	3.66	-1.27
1.58	3.97	-1.20
1.35	4.27	-1.13
1.17	4.58	-1.06
1.02	4.88	-1.01
0.90	5.19	-0.96
0.80	5.49	-0.91
0.72	5.80	-0.87
0.65	6.10	-0.83
0.59	6.41	-0.80
0.53	6.71	-0.76
0.49	7.02	-0.73
0.45	7.32	-0.70
0.42	7.63	-0.67
0.39	7.93	-0.65
0.36	8.24	-0.62
0.34	8.54	-0.60
0.31	8.85	-0.57
0.30	9.15	-0.55

SPAN TABLES FOR SLOPE 3:12 USING 38X140

W(KPa)	L(m)	M(KNm)
2.83	3.05	-1.63
2.27	3.36	-1.53
1.87	3.66	-1.43
1.56	3.97	-1.35
1.33	4.27	-1.29
1.14	4.58	-1.22
0.99	4.88	-1.17
0.87	5.19	-1.12
0.77	5.49	-1.08
0.69	5.80	-1.04
0.62	6.10	-1.00
0.56	6.41	-0.97
0.51	6.71	-0.94
0.46	7.02	-0.91
0.42	7.32	-0.88
0.39	7.63	-0.85
0.36	7.93	-0.83
0.33	8.24	-0.81
0.31	8.54	-0.79
0.29	8.85	-0.76
0.27	9.15	-0.75

SPAN TABLES FOR SLOPE 5:12 USING 38X184

W(KPa)	L(m)	M(KNm)
4.36	3.05	-2.28
3.52	3.36	-2.12
2.91	3.66	-1.97
2.45	3.97	-1.85
2.09	4.27	-1.74
1.80	4.58	-1.65
1.58	4.88	-1.56
1.39	5.19	-1.48
1.24	5.49	-1.41
1.11	5.80	-1.35
1.00	6.10	-1.29
0.91	6.41	-1.23
0.83	6.71	-1.18
0.76	7.02	-1.13
0.70	7.32	-1.09
0.65	7.63	-1.04
0.60	7.93	-1.00
0.56	8.24	-0.96
0.52	8.54	-0.92
0.49	8.85	-0.89
0.46	9.15	-0.85

SPAN TABLES FOR SLOPE 3:12 USING 38X184

W(KPa)	L(m)	M(KNm)
4.37	3.05	-2.53
3.52	3.36	-2.36
2.89	3.66	-2.22
2.42	3.97	-2.10
2.06	4.27	-1.99
1.77	4.58	-1.90
1.54	4.88	-1.81
1.35	5.19	-1.74
1.19	5.49	-1.67
1.07	5.80	-1.61
0.96	6.10	-1.55
0.86	6.41	-1.50
0.79	6.71	-1.45
0.72	7.02	-1.40
0.66	7.32	-1.36
0.60	7.63	-1.32
0.56	7.93	-1.29
0.52	8.24	-1.25
0.48	8.54	-1.22
0.45	8.85	-1.18
0.42	9.15	-1.15

APPENDIX E

Field Erection Procedure

PROPRIETARY NOTICE

All information herein is considered confidential.

Its contents may not be used by, nor disclosed to any other party without prior written consent from Mr. John R. Irving.

THE SHEET METAL GUSSET HAUNCH AND RIDGE GUSSETS FOR RIDGID FRAME CONSTRUCTION

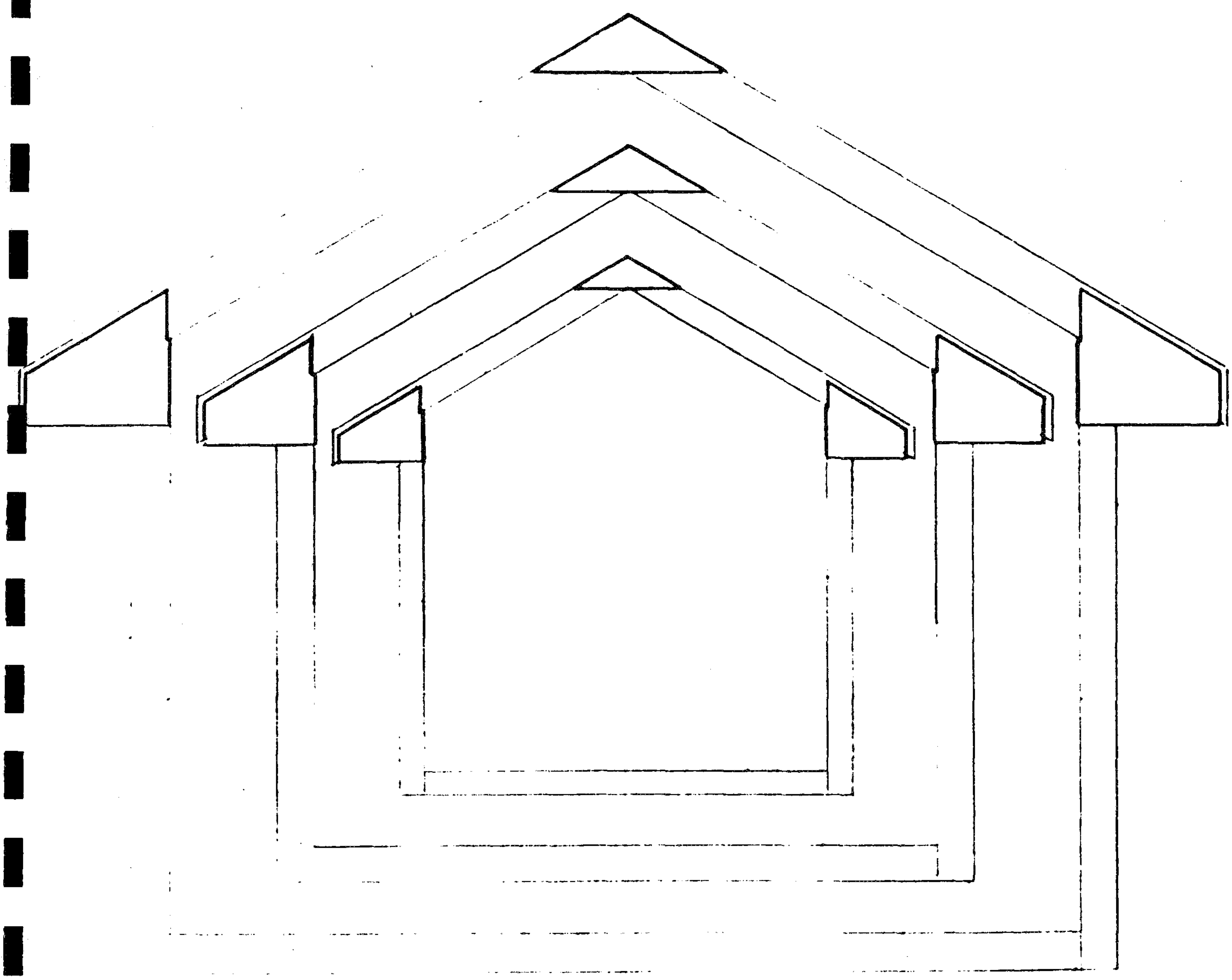


TABLE OF CONTENTS

INTRODUCTION

Prior to the 1940's, there were relatively few sheet metal connectors manufactured for joining wood in the construction industry. All such connectors known to applicant utilized othogonal nailing. As labor and lumber became more expensive, the use of sheet metal connectors such as joist hangers, angles, anchors and bracing became more commonly accepted. In the past twenty years, several major companies have begun producing metal connectors in huge volumes and today, hardly a wood frame building is constructed without the extensive use of these lightweight connectors.

When the architect would incorporate a cathedral ceiling into his design, the builders "Standard Construction Practices" changed considerably. Eliminating the ceiling joists also eliminated the means to tie oppositly opposed walls from being forced outward as the roof would collapse inward. To restrain these forces, heavy timber beams and posts became the main structual members within this type of structure. (See Fig "D" Sheet 5 of 13)

Other types of building practices used for this type of design are very limited. Builtup timber arched trusses is the one other method but again, high labor cost, erection time, heavy large main members and hazardously high working conditions still exist. Once erected, these builtup trusses tend to rob valuable interior floor and wallspace and leaving the structure with a devalued esthetic value. (See Fig "E" Sheet 6 of 13)

What I propose, to overcome these problems is virtuously new concept in cathedral ceiling construction:- Allowing a totally open ceiling, yet conforming completely to conventional appearance.

With the use of sheet metal haunches to join the wall studs to the roof rafters in a manner which provides a complete ridgid connection by wraping the splice using a single piece of sheet metal, thus providing a three sided metal lamination. Pre-drilled holes provide the exact sequence and number of nails to be used at each connection insuring a sound design and eliminating faulty nailing. Once the building structure is completed and finished inside and outside, these metal connectors are totally hidden from view giving an overall picture of open space and clean lines. "There are no protruding members to be boxed in with roof and wall finishing materials." (See Fig "F" Sheet 8 of 13)

ADVANTAGES OF RIDGID FRAME CONSTRUCTION USING
THE SHEET METAL GUSSET HAUNCH AND RIDGE GUSSETS

ECONOMY

Incorporating the sheet metal haunch into the construction of a ridgid frame building, makes the total job extremely economical. Because these haunches essentially form arches, less material is required to span a given distance.

SIMPLICITY

Easy to fabricate and simple to erect these two factors allows simplicity to the layman. With the sheet metal haunch being totally pre-designed, holes for nailing prepunched or marked, and pre-bent, to form a three sided gusset, relieves the builder from these problems and assures him of a sound structure. (See Fig "A" Sheet 1 of 13) and (See Fig "B" Sheet 2 of 13)

VERSATILITY

The end walls in a ridgid frame building using the sheet metal haunch are non-structural and may be glazed, omitted, set back to provide entry overhang, or positioned wherever the appearance or function of the building dictates. Ridgid frame structures are particularly well-suited for buildings in which large end-wall openings are required.

SPEED OF CONSTRUCTION

Framing time is substantially reduced because frames can be built in advance on the ground or in the shop where labour is usually more efficient. When assembled, the ridgid frames can be easily erected and quickly sheathed due to the sheet metal haunches forming a ridgid frame the two side walls and the roof are erected in one operation. (See Fig "G" Sheet 9 of 13)

STABILITY

A building must be designed to withstand three different loads, the roof load, the wind load on its sides, and the wind load on its end walls. The first two of these forces are resisted by the use of the sheet metal haunches.

The third, the wind load, on the buildings end walls is resisted by the inherent diaphram strength of the wall and roof sheathing.

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ADVANTAGES OF RIDGID FRAME CONSTRUCTION USING
THE SHEET METAL GUSSET HAUNCH AND RIDGE GUSSETS

STRENGTH

Sheet metals sturdy construction makes it exceptionally strong. Its diaphram action adds strength and stiffness to the entire building ensuring stability during high winds or earth tremors.

ADVANTAGES OFFERED

Made in a number of different sizes, each size of metal haunch could be used for several building spans. From a small storage shed to large warehouses, making the haunch available from the handyman up to the large contractor.

Example:-

Haunch Size "A" 8 ft to 10 ft span
Size "B" 12 ft to 16 ft span
Size "C" 20 ft to 24 ft span
(See cover)

These same haunches would also be available in several different roof slopes.

TYPES OF BUILDINGS

1. Owing to simplicity and substantially reduced labour costs, the architect could find these metal haunches an asset in designing homes and churches with cathedral ceilings.

2. Storage sheds for the homeowner, cottager and farmer.

3. Warehouses when overhanging structural members can be omitted to gain height.

4. Cottages, chalets and boathouses as an architectural feature.

5. Greenhouses, where a minimum amount of structural members are required to allow the maximum amount of sunlight to enter.

6. Schools, cabins, park shelters and light industrial plants.

MANUFACTURING BENEFITS

1. Low cost for research and development due to simplicity.

2. Simple and inexpensive to manufacture.

3. Excellent stacking for storage and shipping("One inside the other").

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MANUFACTURING BENEFITS (continued)

4. Large quantities of both sheet metal haunches and ridge gussets are required for each complete building.
Example:-

"One garage 12 ft. wide and 20 ft. long"

Each complete ridgid frame 12 ft. wide, consisting of two wooden roof rafters, two wooden wall studs, two metal haunch gussets, and two metal ridge gussets will be placed at sixteen inch centers one behind the other, for a distance of 20 ft. To accomplish this, the builder would need to prefabricate sixteen ridgid frames, therefore thirty-two metal haunches and thirty-two metal ridge gussets would be needed.

5. Haunches and ridge gussets are stamped out of a single piece of sheet metal. No seams or spotwelding needed.

6. Due to geometric configuration of both haunch and ridge gusset, very little waste material is produced during manufacturing.

On the job site, the builder need only to cut his lumber to the proper lengths ("A full size jig is not required") to assemble the ridgid frames. This is accomplished due to the fact that each sheet metal haunch and ridge gusset have pre-designed built in positioning stops that ensures proper alignment and the exact bevel or roof slope required.

THE SHEET METAL RIDGE GUSSET

Designed and constructed in the same manner as the sheet metal haunch, these metal connectors aquire all the advantages of the haunch to tie the apex of the roof rafters together. (See Fig "C" Sheet 4 of 13)

The ridgid frame can be made up using two different types of engineering designs. The first design is known as a "three hinge ridgid frame". This means that out of the five connections needed to complete the frame "Including the floor joist, the two haunch positions are designed to resist three different loads transmitted through them". The remaining three connections are fastened to resist bearing loads only. (See Fig "H" Sheet 10 of 13)

The second design is known as a "two hinge ridged frame". This means that besides the two haunch connections having a ridgid construction the ridge gusset would also be connected in this manner, leaving the two wall studs to floor joists as a hinged connection. Using this method allows the total roof loads to be distrubuted between three ridged connections which in turn lends to overall smaller gusset plate dimensions. (See Fig "H" Sheet 10 of 13)

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ADVANTAGES OF RIDGID FRAME CONSTRUCTION USING
THE SHEET METAL GUSSET HAUNCH AND RIDGE GUSSETS

BRIEF SUMMARY

As inflation, high labour and material costs drives the price of housing and other types of structures higher, the builders and contractors are continuously searching for means and ways to keep their cost down. With the introduction of this novel building timber system to the market opens a whole new field in building construction.

Once assembled, the building becomes a ridgid load-carrying structural unit. When erected and sheathed the result is one of the most economical clear span structures available.

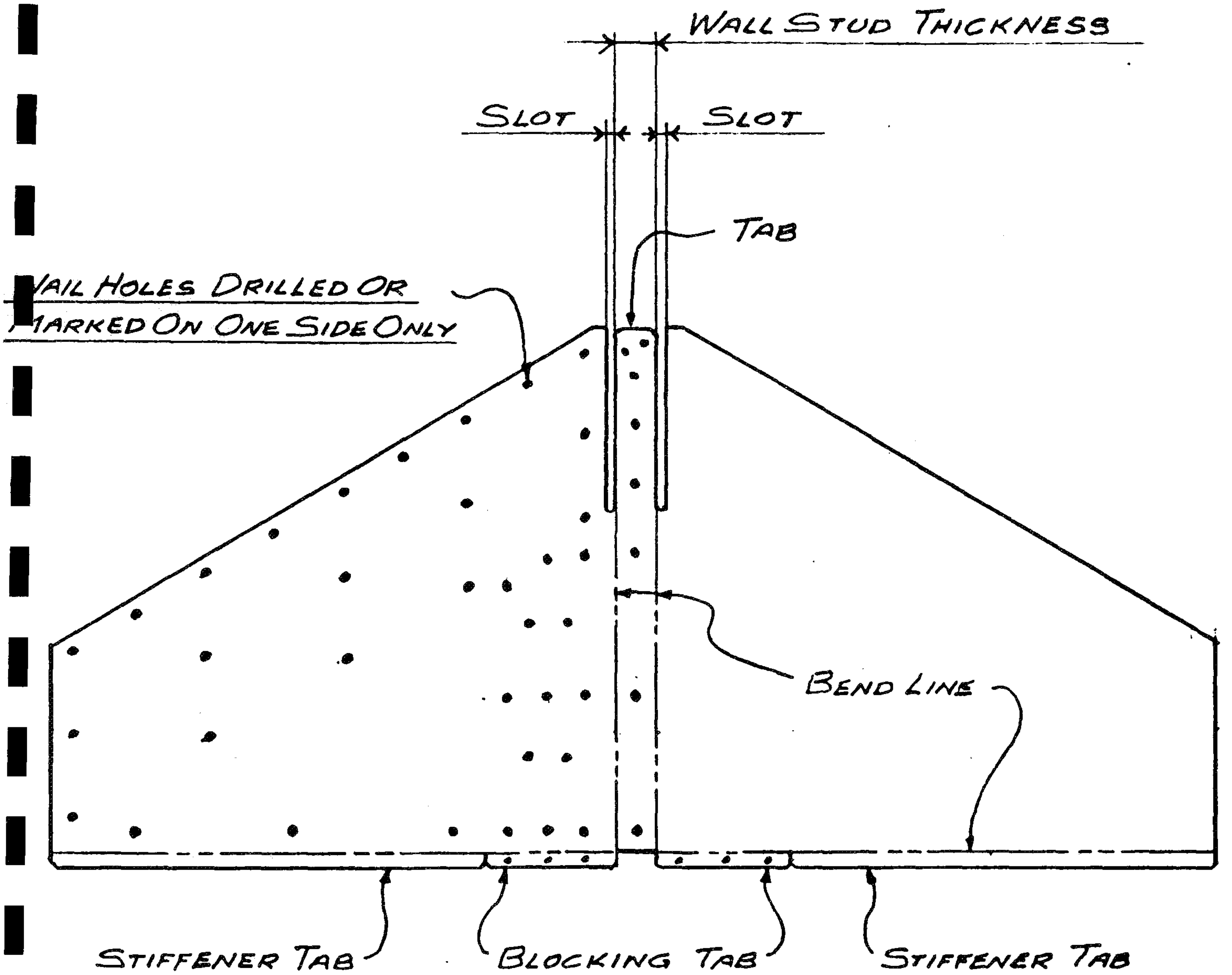
With the use of this building system less lumber is needed, which in turn brings lower material and labour costs. To complement these two savings, the builder can erect his structural shell in at least fifty percent less time than using today's conventional building practices.

Given all these very important saving advantages, in the inevitable advent for far more lower cost smaller housing, an architectural cathedral ceiling is introduced giving smaller rooms an appearance of spaciousness leaving esthetic value usually preserved for more expensive homes.

Yours very truly,

John R. Irving

JRI/mm



LAYOUT OF SHEETMETAL GUSSET HAUNCH

FIGURE "A"

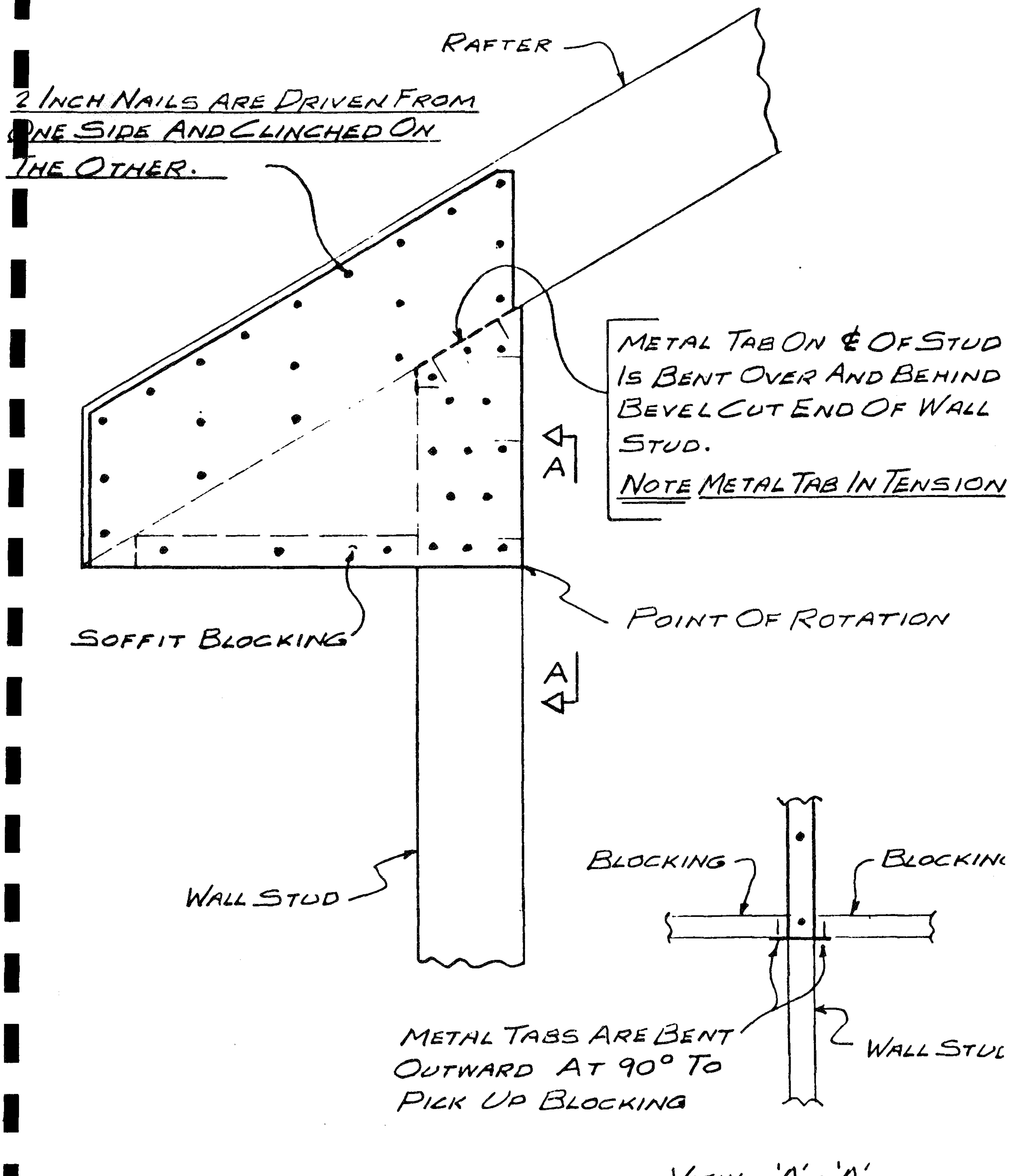
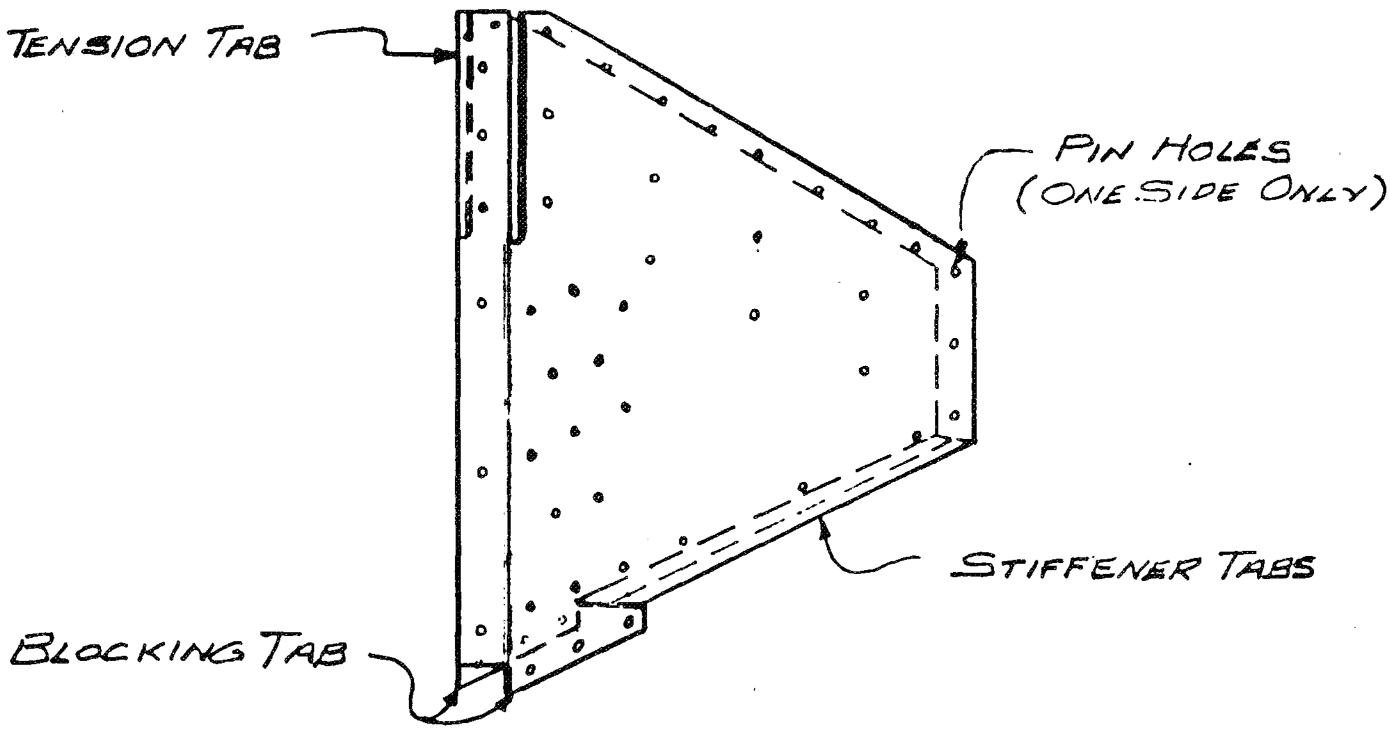
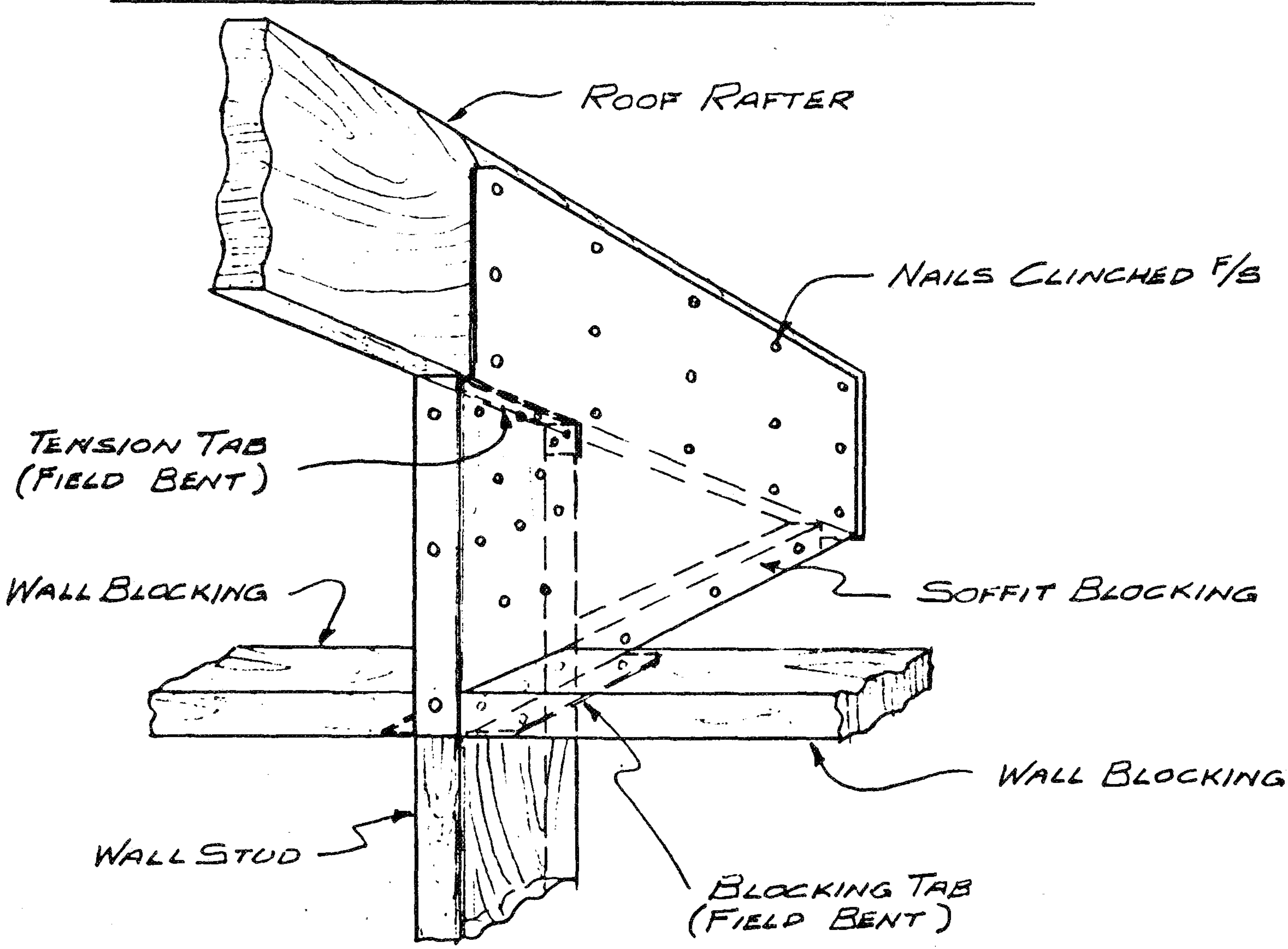


FIGURE "R"

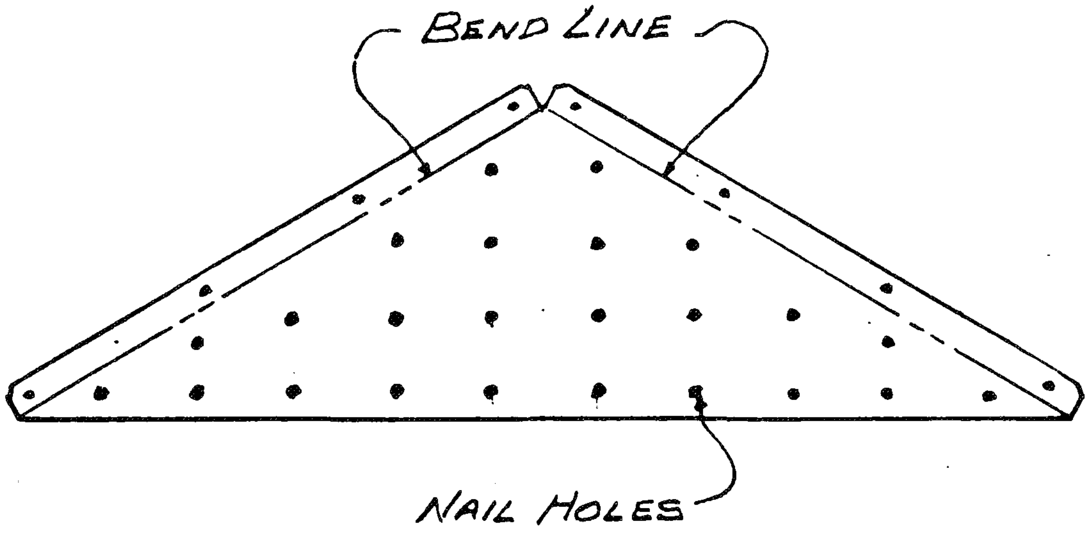
SHEET 2 OF 13



PRE-FABRICATED METAL HAUNCH



INSTALLED METAL HAUNCH



LAYOUT OF SHEETMETAL RIDGE GUSSET

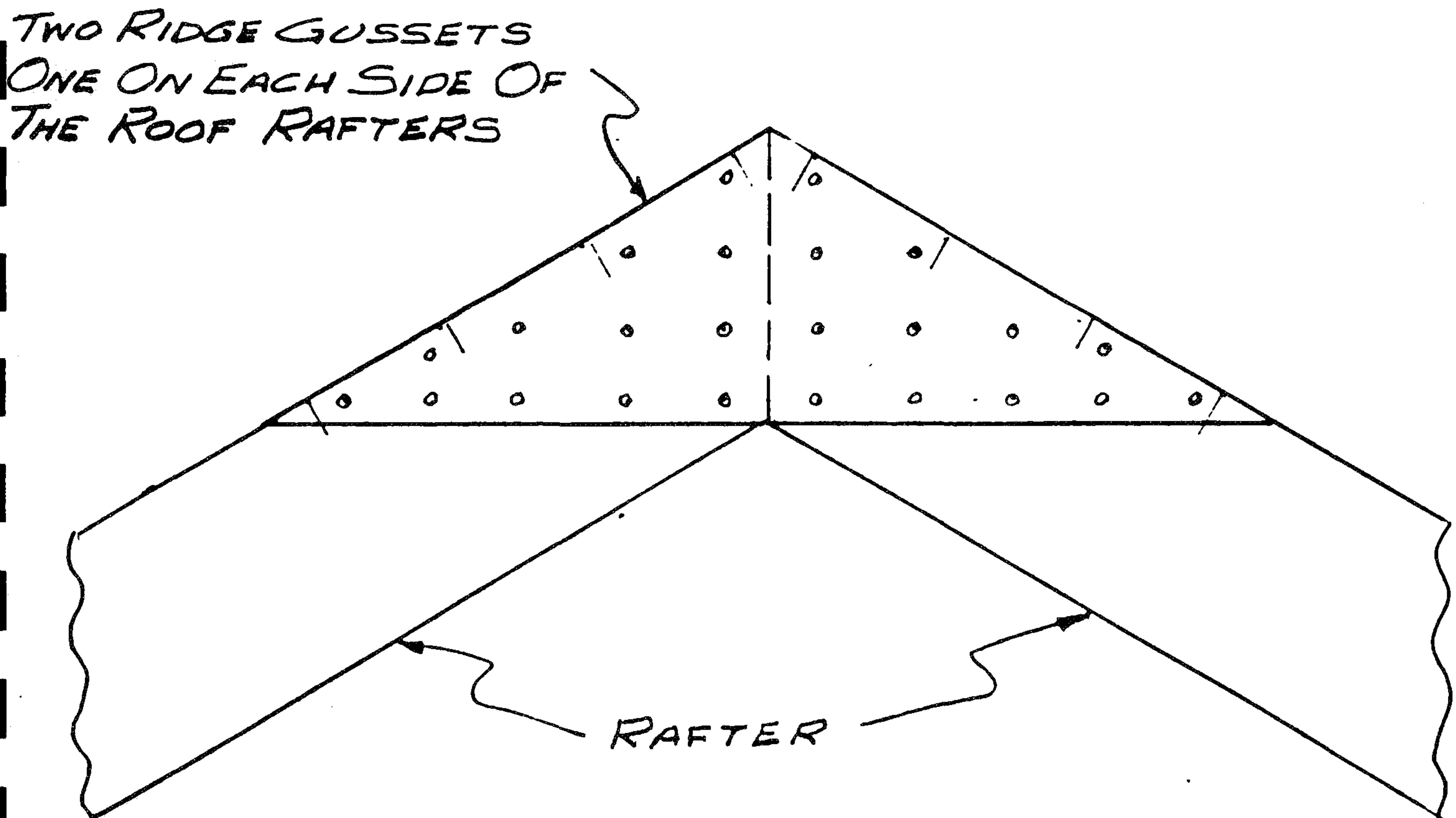
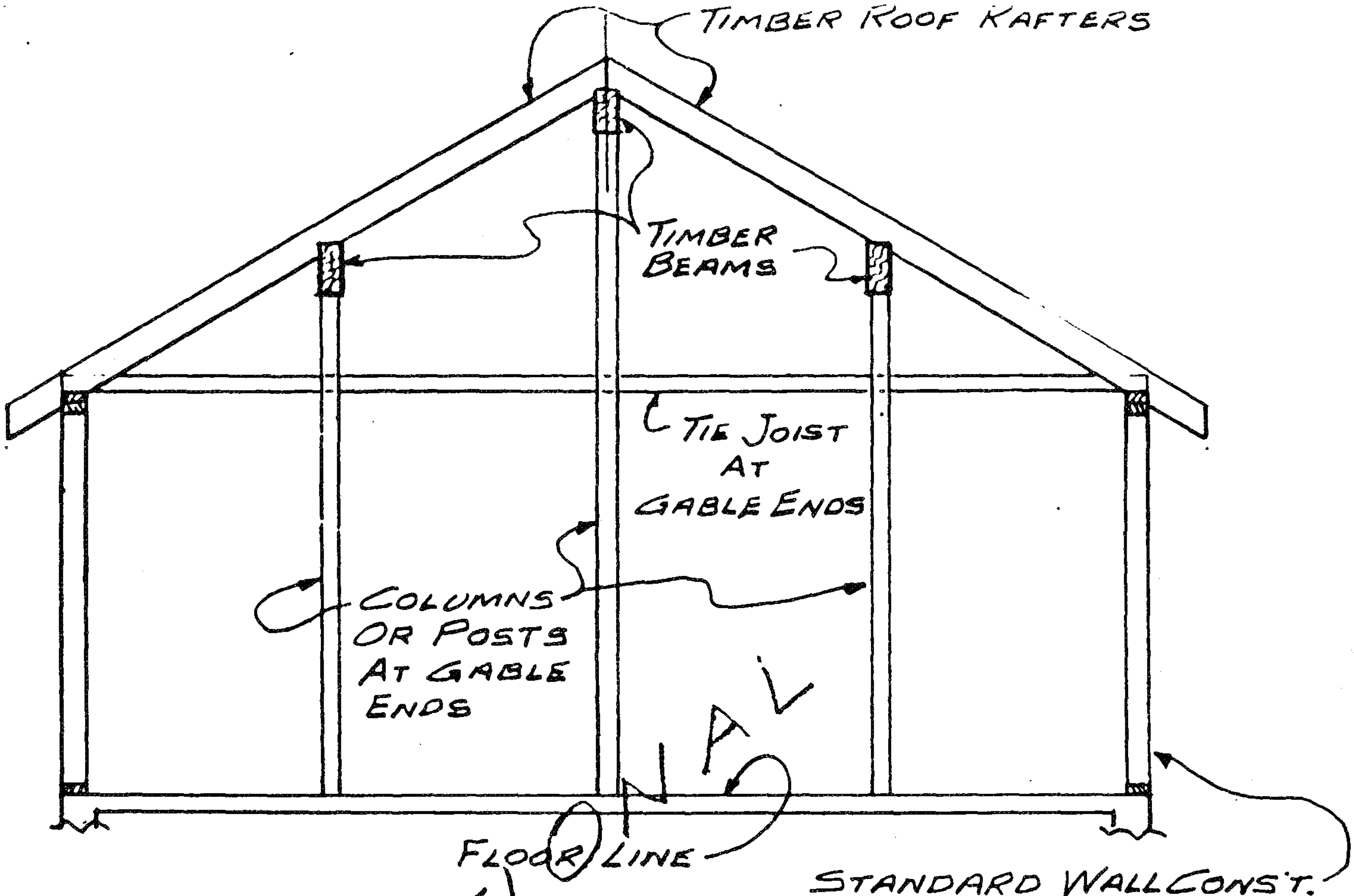


FIGURE "C"



BEAM AND POST CONSTRUCTION

CONVENTIONAL

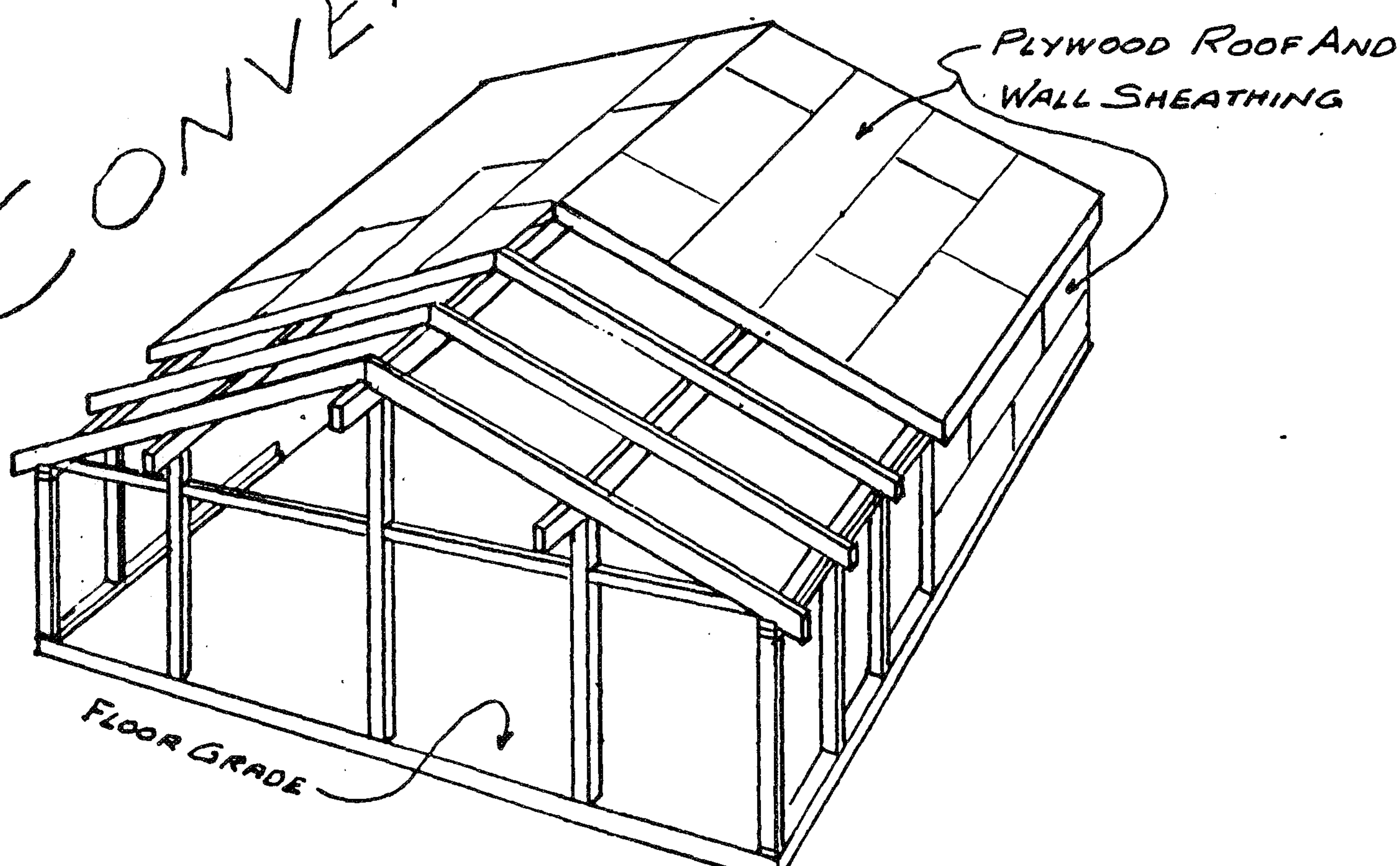
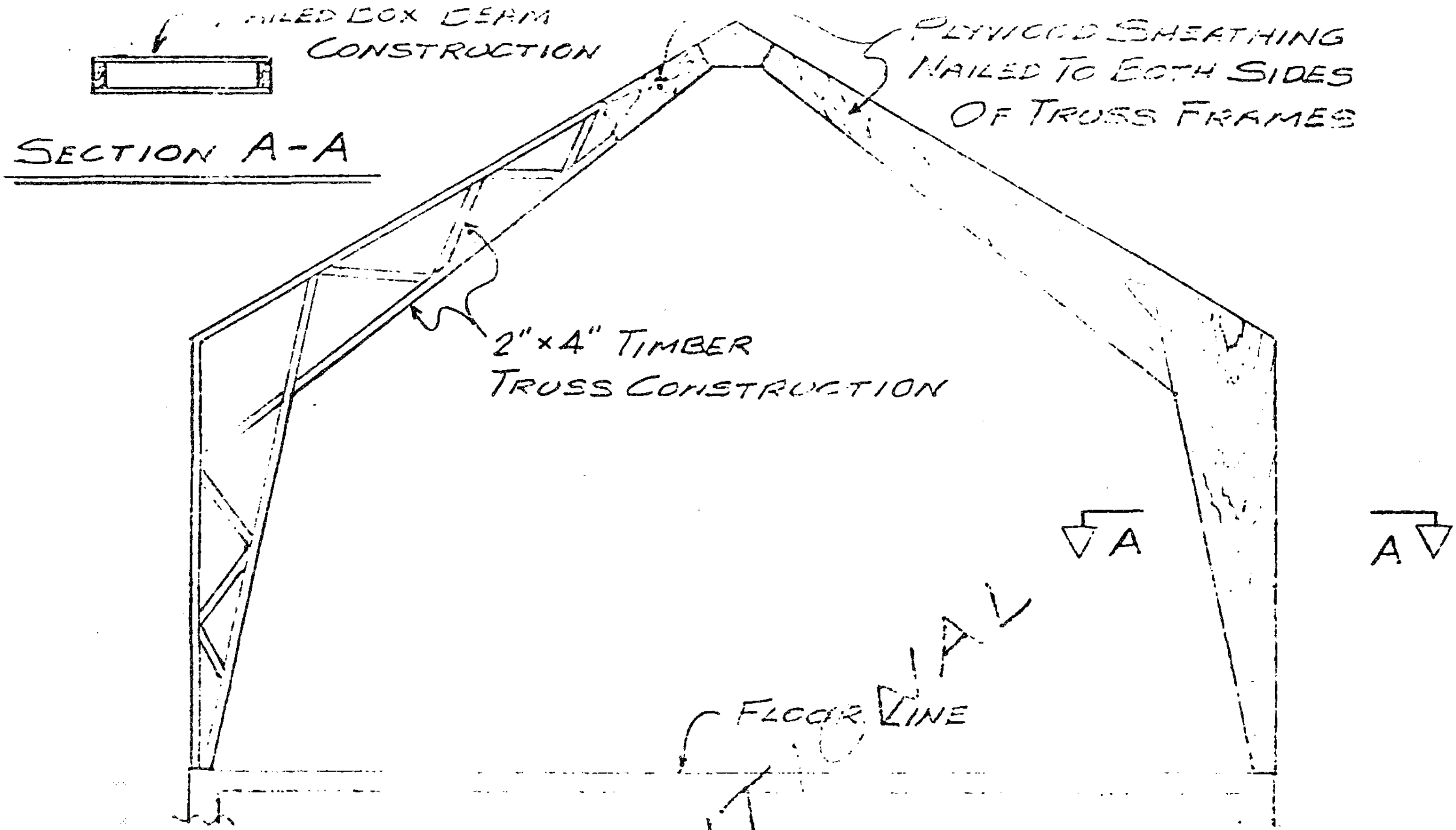


FIGURE "D"



BUILTUP TIMBER ARCHED TRUSSES

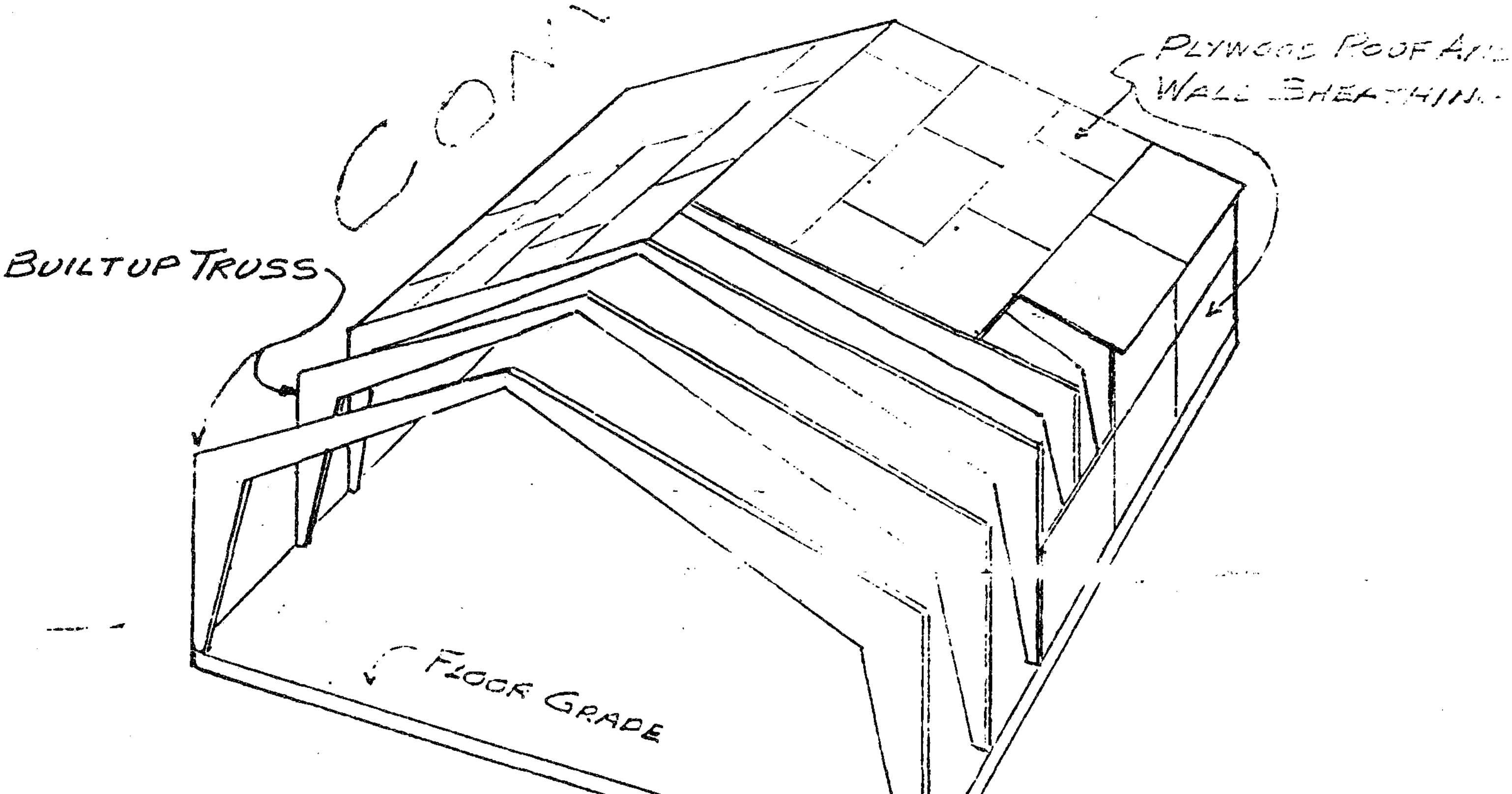
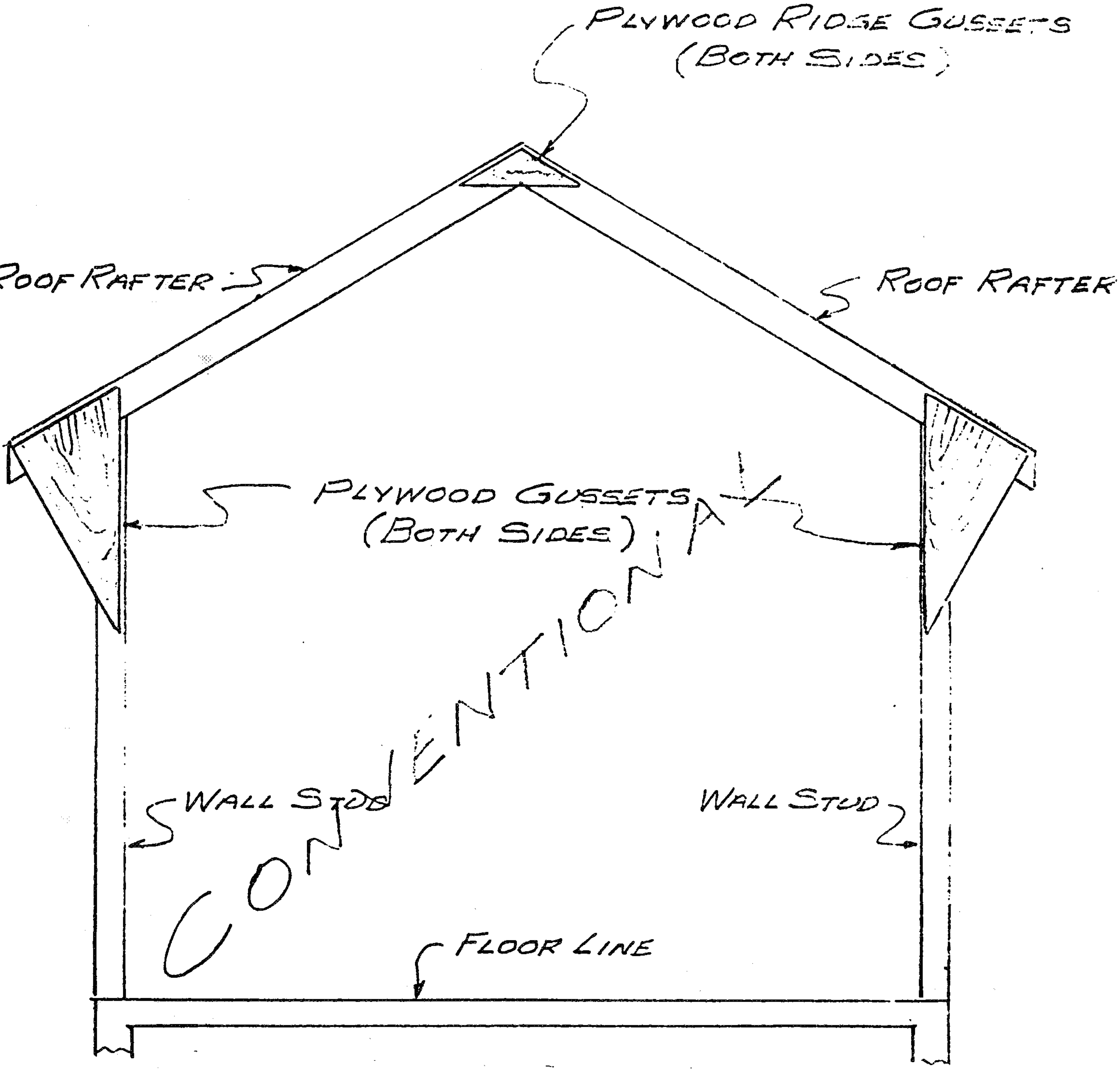
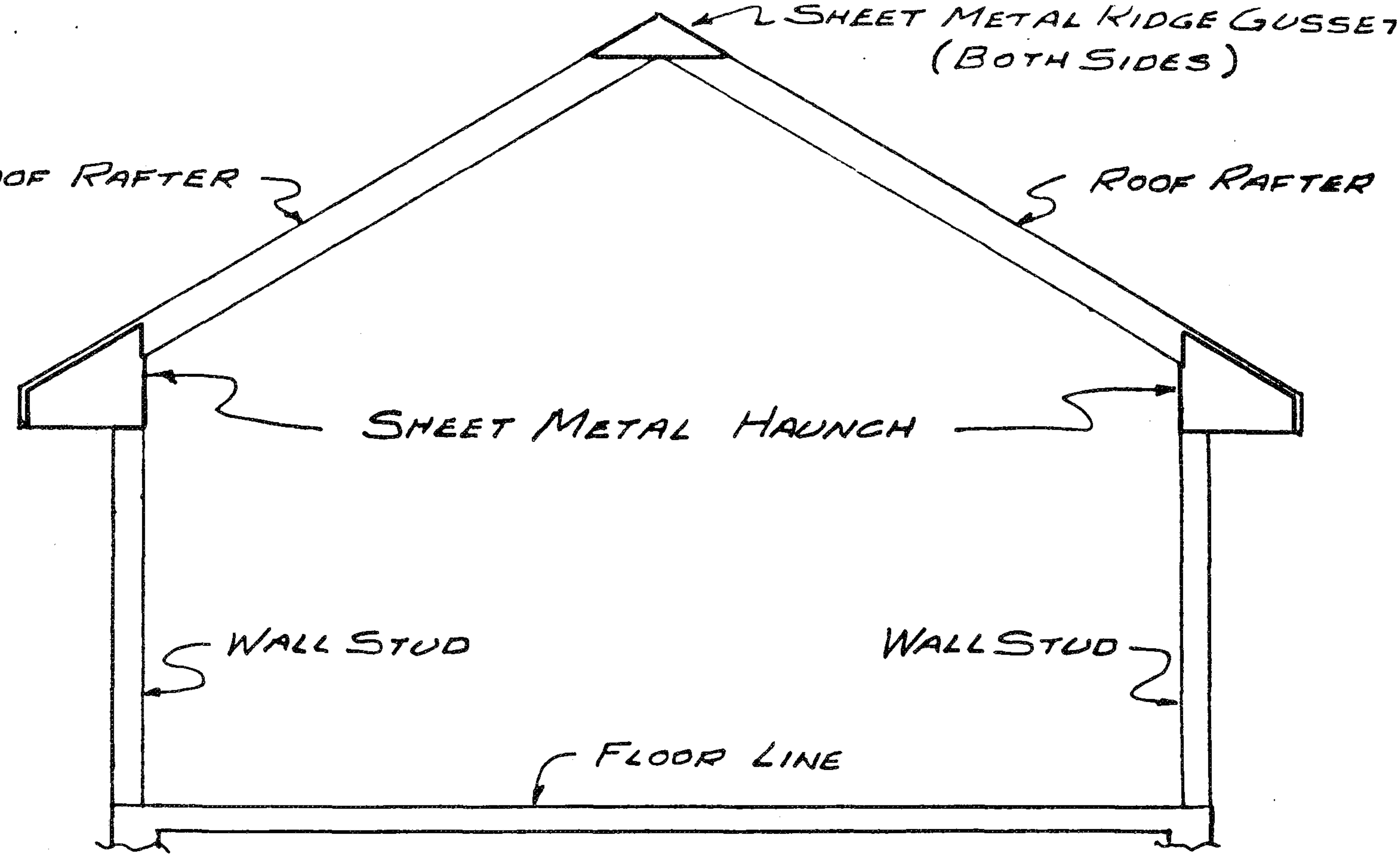


FIGURE "E"

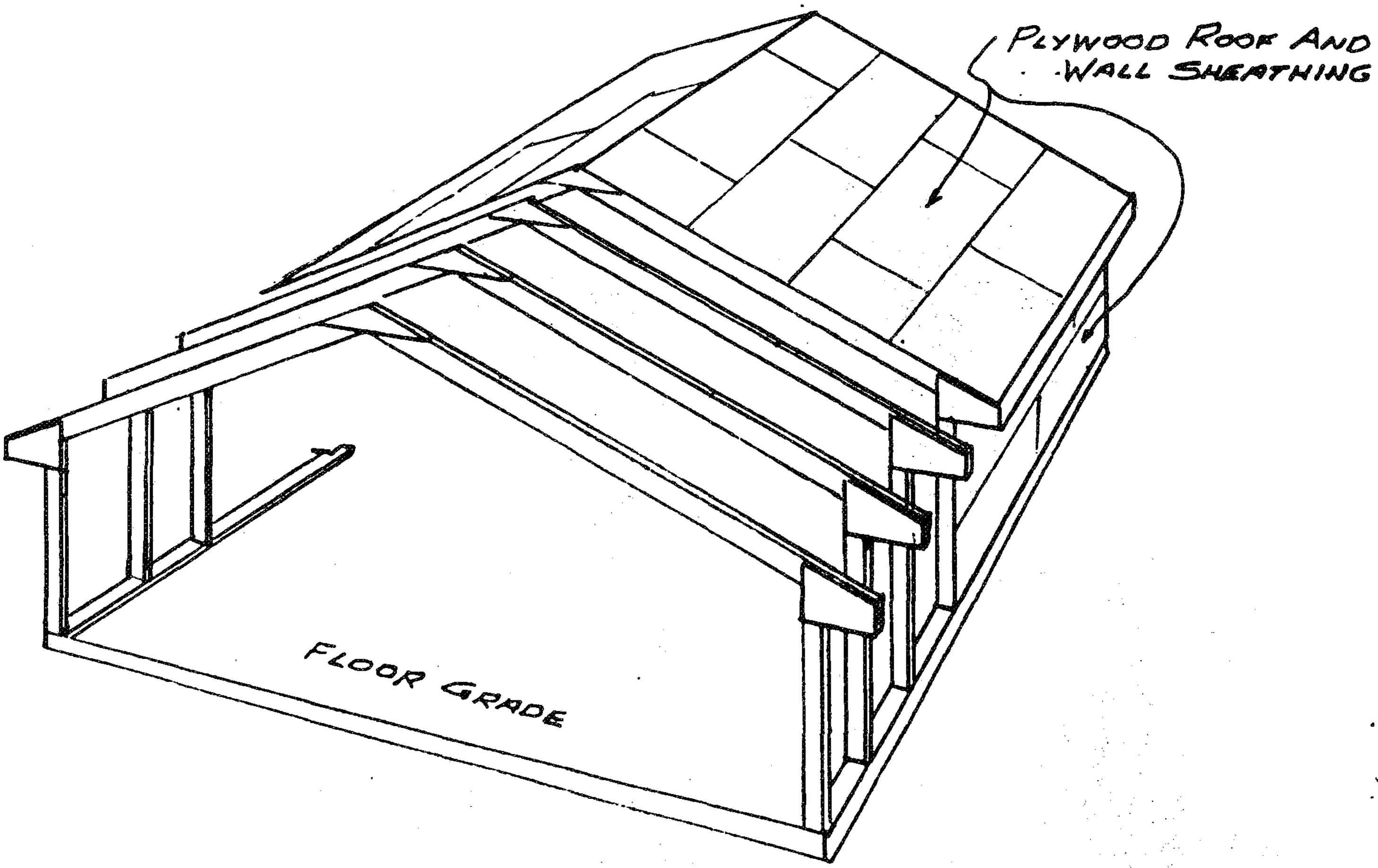


RIDGID FRAME CONSTRUCTION USING PLYWOOD GUSSETS

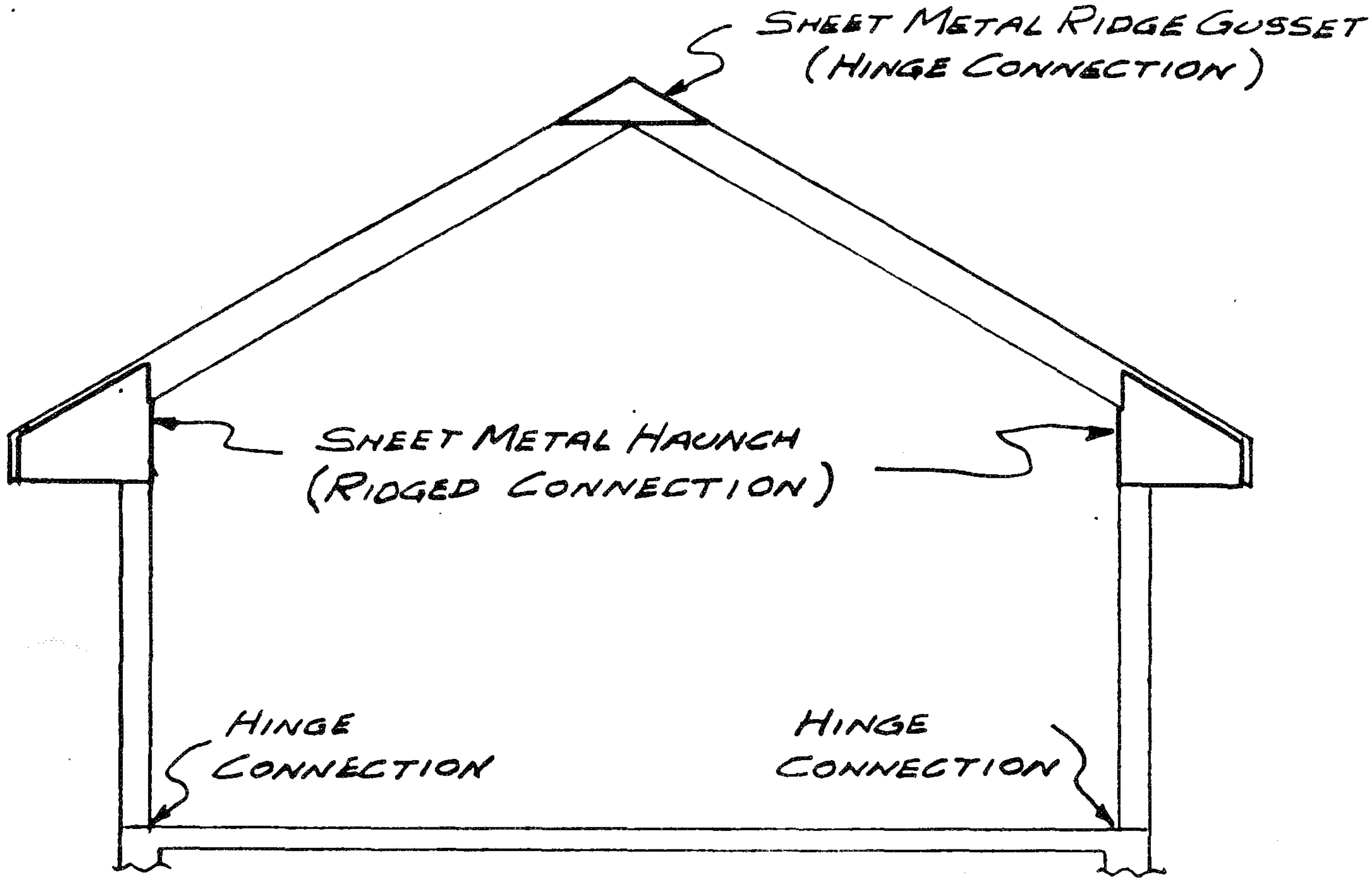
FIGURE "H"



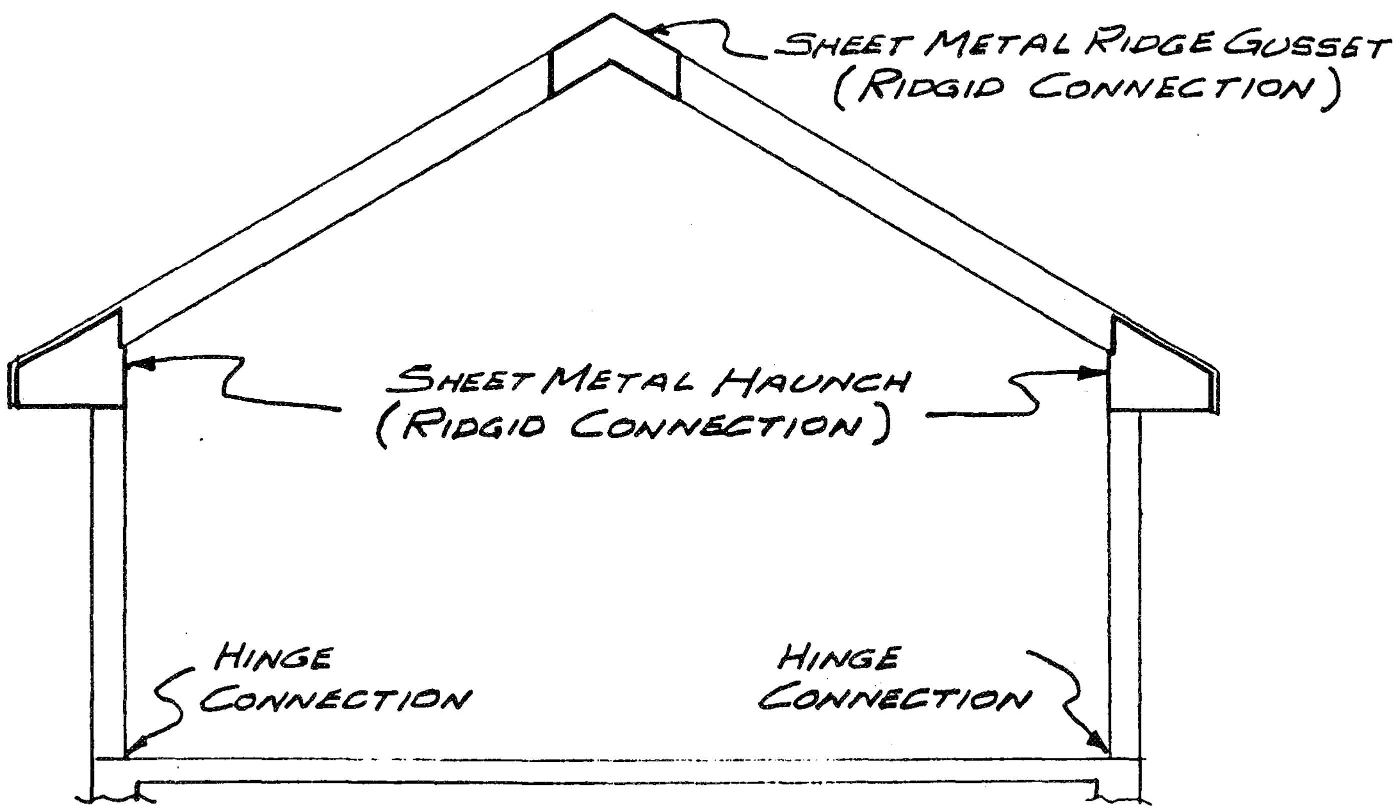
RIDGID FRAME CONSTRUTION USING METAL HAUNCH AND RIDGE GUSSET



"F"

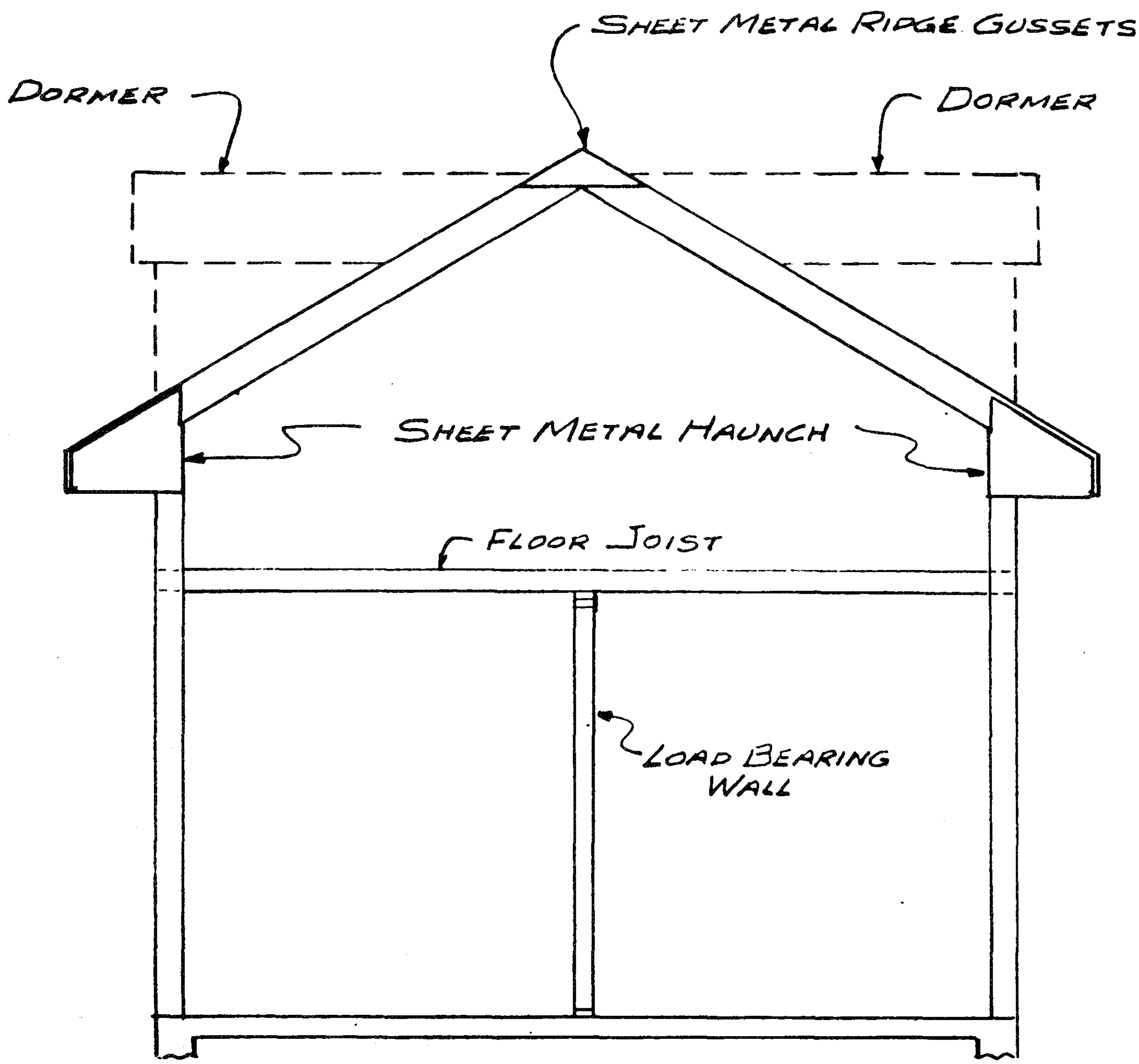


THREE HINGE RIDGID FRAME

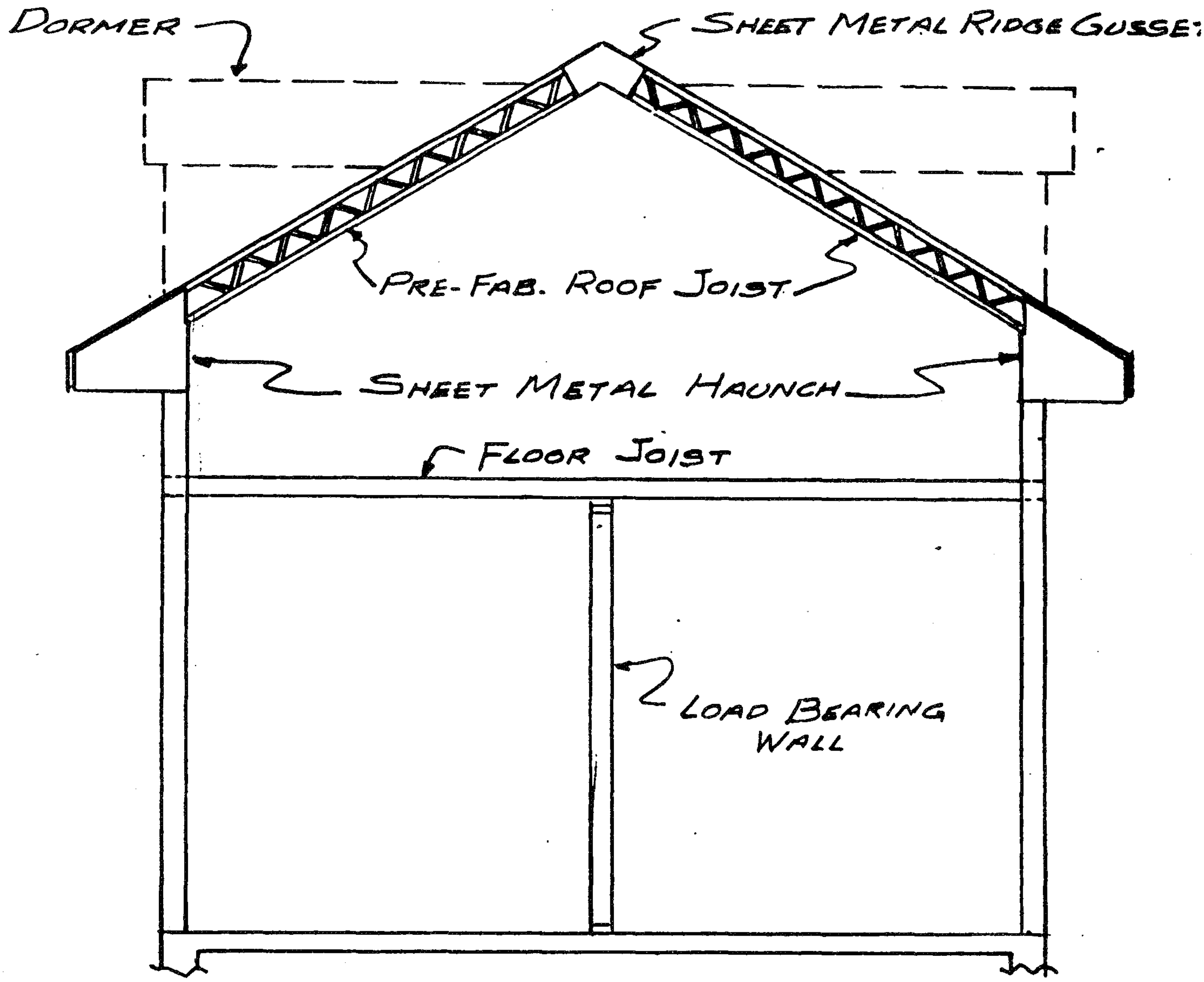


TWO HINGE RIDGID FRAME

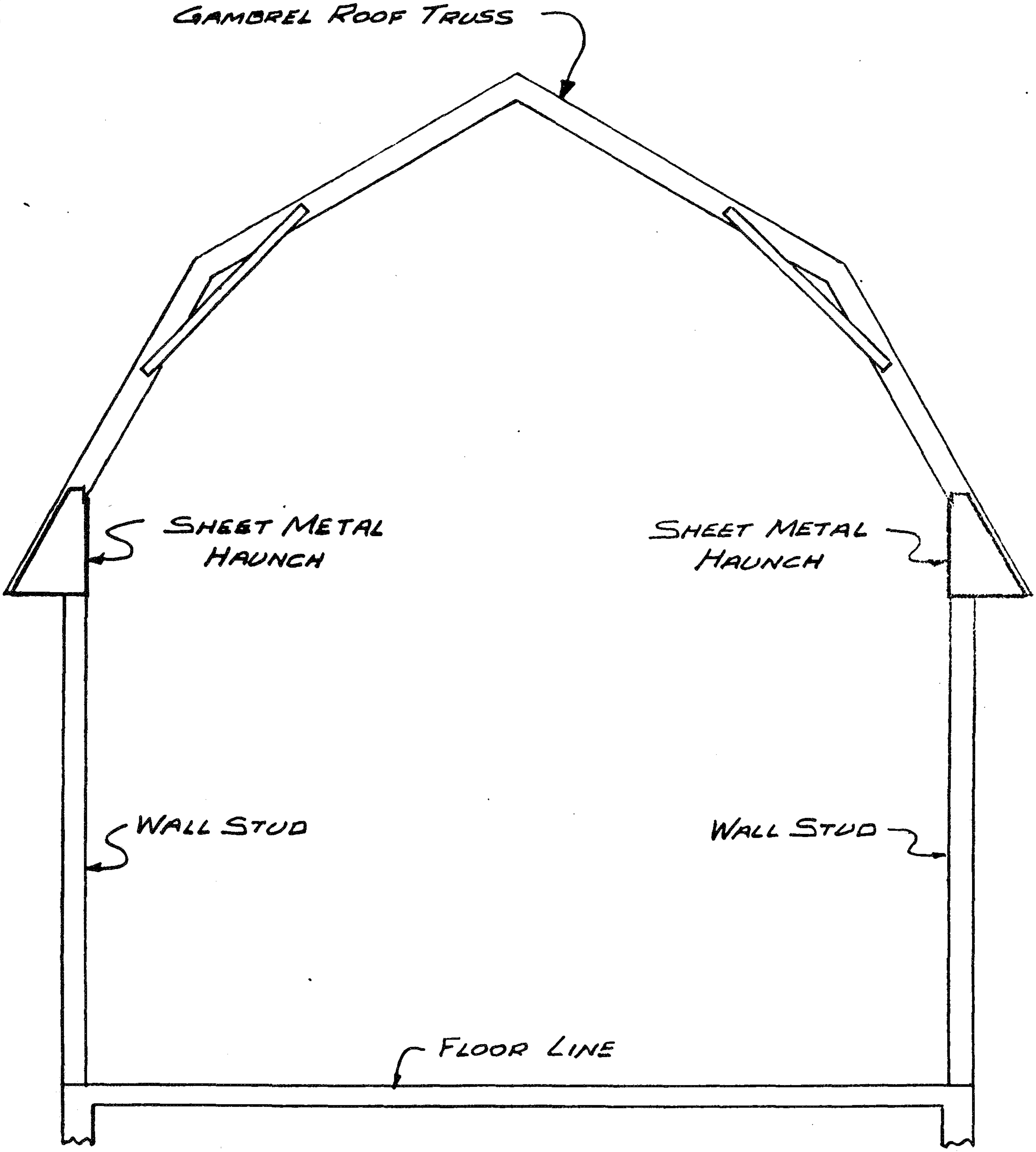
FIGURE "H"



TRUE PRE-FABRICATED ATIC TRUSS



TRUE PRE-FABRICATED ATIC TRUSS



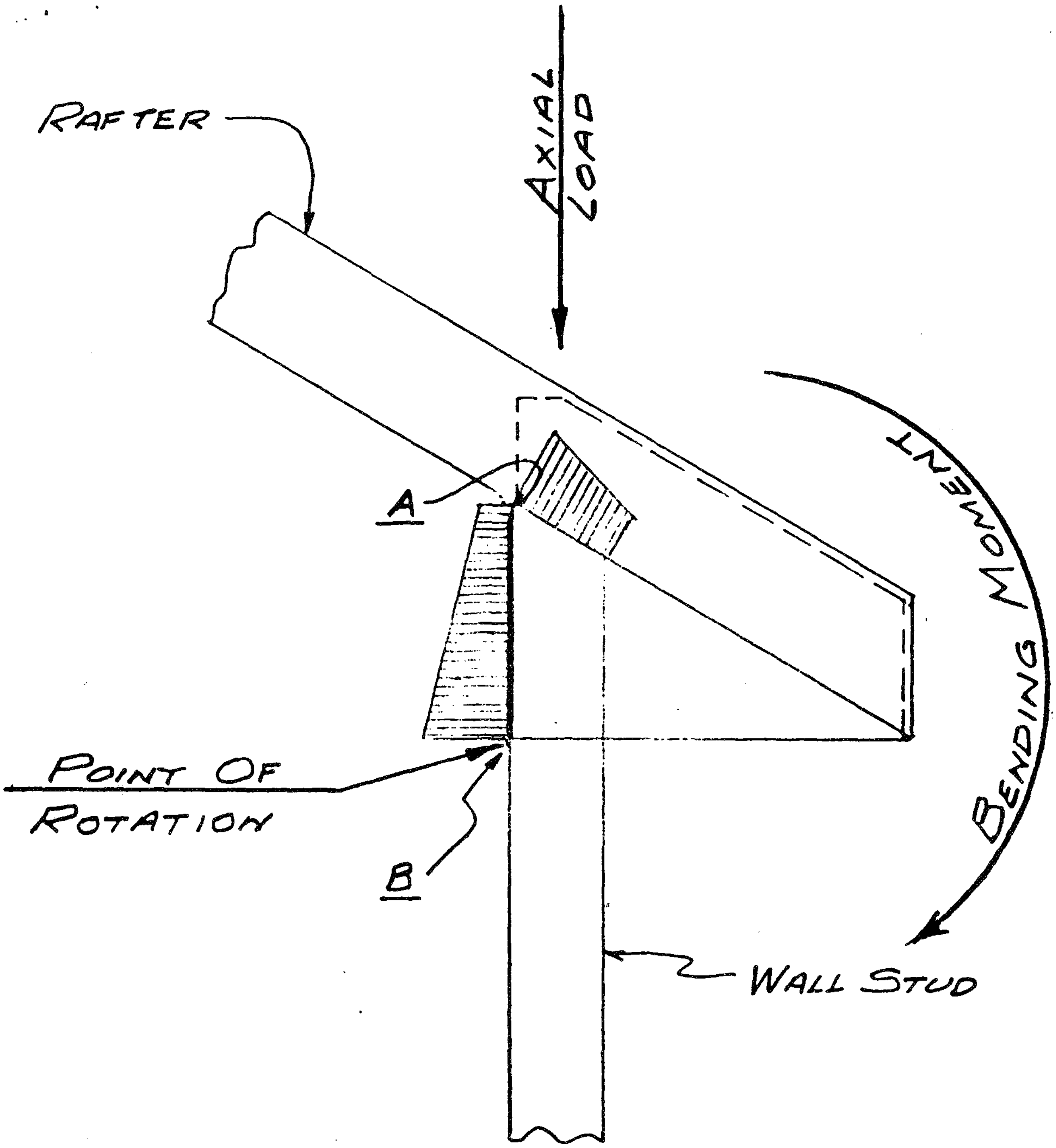
CLEAR SPAN GAMBREL ROOF ARCH

June 25th, 1982

In wraping the splice using a single piece of sheet metal, thus providing a three sided metal lamination, a pocket is formed around the wall stud. As the roof loads are transmitted through the haunch and into the wall stud, the axial load forces the wall stud up the sloping rafter to seat tightly between the underside of the roof rafter and the wrapped around metal frange of the haunch at point (A).

The bending moment produced would in turn be resisted by the same wrap around metal flange, placing the point of rotation at the bottom of the flange at point (B).

Using this method to form a structural connection at the roof haunch would greatly reduce the number of nails needed to connect the wall stud to the metal haunch by interlocking the two timers.



WRAPED HAUNCH POCKET FOR WALL STUD

LOAD DWG.