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HISTORIC STRUCTURES REPORT

ENGINEERING STUDY
OF
CENTRAL BLOCK ROOF SUPPORT SYSTEM

ARLINGTON HOUSE

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prepared by

U.S. DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
DENVER SERVICE CENTER
NATIONAL CAPITAL TEAM

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ENGINEERING STUDY - ARLINGTON HOUSE

INTRODUCTION

The intention of this report is to assess the overall long and short-range capabilities of the roof's support system in the central block of the Arlington House, describe problems which might affect its performance, and propose an approach to correct these shortcomings and ensure a sound structure capable of protecting this resource. The remarks are based on findings culminating from both a visual investigation of the various structural elements and an analysis of its design. Recommended corrective actions would not adversely affect this structure, which is listed on the National Register of Historic Places.

Concurrent related studies undertaken by the National Capital Team of the Denver Service Center to be published in FY 1983 are as follows:

A. Historical Data Section for Arlington House (The Custis-Lee Mansion), Virginia, by Charles W. Snell;

B. A Summary of the Physical History of Arlington House, 1802-1933, by Charles W. Snell;

C. Arlington House, Historic Structures Report, Architectural Data Section (Phase I), by H. Lee Arnest III;

D. Historic Structures Report, Architectural Data Section (Phase II), The Arlington House, by H. Lee Arnest III and John D. Sligh;

E. Archeological Excavations at Arlington House, The Robert E. Lee Memorial, Arlington County, Virginia, by John F. Pousson.

ANALYSIS

The structural analysis of the roof support system (Appendix A) was based on BOCA Basic Building Code (1978) snow and wind design load

TABLE I STRUCTURAL ANALYSIS SUMMARY OF FINDINGS

MEMBER	SIZE USED IN ANALYSIS	DETERMINATION
TRUSS "A"		
Top Chord, Upper Section	4 1/2" x 9"	o.k.
Top Chord, Lower Section	4 1/2" x 9"	o.k.
Bottom Chord	6" x 9"	o.k.
King Post	6" x 9"	o.k.
Diagonal Brace	5 1/2" x 5 1/2"	o.k.
Vertical Braces	3" x 6" to 6" x 6"	o.k.
TRUSSES "B" THROUGH "E"		
Top Chord, Upper Section	4 1/2" x 9"	Seriously Undersized
Top Chord, Lower Section	4 1/2" x 9"	Moderately Undersized
Bottom Chord	6" x 9"	Seriously Undersized
King Post	6" x 9"	o.k.
Diagonal Brace	5 1/2" x 5 1/2"	o.k.
Vertical Brace	6" x 7"	o.k.
TRUSS "F"		
Top Chord, Upper Section	3 1/2" x 8"	
Top Chord, Lower Section	3 1/2" x 8"	Marginally Undersized
Bottom Chord	4 3/4" x 8 5/8"	Seriously Undersized
Brace	2" x 6 1/2"	o.k.
ARMY TRUSSES		
Top Chord	6" x 6"	o.k.
Bottom Chord	6" x 8"	o.k.
Mtl. Rod	1"Ø	o.k.
PURLINS	5 1/2" x 6"	o.k. for max. spacing = 5' o.c.
RAFTERS	2 1/4" x 3 1/4"	o.k. for max. span = 5' o.c.
CEILING JOISTS	3" x 3 1/2"	o.k.

recommendations for 100-year mean recurrence interval, and on material strength values for Eastern White Pine, No. 1 (Grade) for beams and stringers. Listed in Table 1 is a summary of the results of this structural analysis. As shown, the results derived from this analysis indicate a condition of high overstress in the members. This problem is particularly serious in the case of the bottom chords of all trusses, as these members are significantly overloaded when supporting just the dead loads. The problem is aggravated by the location of interior bearing walls which act as added truss supports. Instead of coinciding with the truss joints, these walls support the bottom chords at several intermediate points between the joints. This misalignment of forces induces excessive bending stresses in these members, which has resulted in extreme undulation and deflection along these members. Although this system has existed with only localized failures, it must not be assumed that the roof structure is therefore safe. In summation, the existing trusses lack deeply in any real engineering design. Not only are individual members of too small a cross-section to safely carry design loads, but their weak methods of interconnection and overall poor truss geometry result in an ill-defined support system through which load transfer is very ambiguous.

Other considerations which should be taken into account for structural systems built with wood and which probably apply to the attic of the Arlington House are:

1. prolonged heating to temperatures in excess of 150°F can cause permanent loss of strength, and

2. when wooden members are fully stressed to the maximum design stress or a joint is fully loaded to the maximum design load for a total of more than ten years, the allowable design load is reduced by 10 percent.

The visual inspection phase of this study produced other indications that the roof support system is in a borderline category as regards structural stability. Following is an outline of specific failures or types of conditions which exist and illustrate the need for repair and reinforcement.

1. Heel joint on south side of truss "B" (Photo 1).

The failure of this connection was the result of a design characteristic unique to this truss - it spanned the entire width of the portico/main block. All the other trusses were, in addition to bearing on the end (N & S) walls, supported by either interior load bearing walls approximately 6 feet on each side of the king post or by a series of brick columns as exist along the east side of the Arlington House. To correct this problem, the Corps of Engineers installed two additional trusses (running east to west) to provide support to truss "B" at interior points. In addition, the failed heel joint was stabilized by inserting a metal pin through the top and bottom chords at the mortise and tenon type connection and by wrapping a heavy link chain around the members at this point to hold them together. Although this stabilization effort has functioned since its completion in the 1930s, its design strength cannot be accurately calculated. It is safe to assume, though, that it does not exceed the capacity of the other heel joints which were determined to be underdesigned.

2. Checks, shakes, knots, and twists.

Natural wood defects such as checks, shakes, knots, or twists are fairly prevalent in the roof support structure, particularly the truss members, resulting in an unquantifiable reduction of its load-carrying capabilities. Examples of such defects are shown in Photos 2 through 4. While bolting, strapping, and other strenghtening procedures might be employed on the existing trusses and other framing members, this alone would be insufficient to insure structural safety.

3. Insect and moisture damage.

Isolated or low level cases of moisture and insect damage are visible in various members of the roof support system. In general, those members which experienced serious deterioraton due to these elements have been replaced, all infested areas made insect-free, and all leaks sealed. One area where a problem still exists, however, is at the south side chimney through which truss "D" passes. Faulty flashing around the chimney allowed water to accumulate around the top and bottom chords of this truss, creating a suitable habitat for powder post bettle colonies and contributing to brown rot decay. Although the roof no longer leaks in this area, the deterioration and subsequent loss of section that has already occurred has significantly reduced the strength of this truss. (See Appendix B and Photos 5 and 6.)

4. Structural failure.

Isolated member or joint failure due to overstressing has occurred primarily in the truss members where the grain is twisted or contains knots. (See Photos 2 and 3). Other cases include several purlins which have split at their end bearing points between trusses "C" and "D" (Photo 7), the heel joint of truss "C" (Photo 8) and several purlin tenons which have been damaged and lost their pegs (Photo 9).

Two other conditions which, although not affecting the roof's load-supporting capabilities, do warrant discussion are:

a. the lack of bearing support for the ceiling joists, and

b. the eastward lean of the trusses.

The frequent lack of support provided to the ceiling joists in this attic was diagnosed and partially treated during the investigative phase of the Historic Structure Report. Shrinkage and sagging of the joists had caused their notched ends to move away from the small ledger strip to the point where many joists had lost contact with their support system. To remedy this problem, bent strap connectors were fastened from the bottom chord of the adjacent truss to the tops of the joists which appeared to have insufficient bearing contacts with a ledger strip. (Photos 10 and 11.) A similar type of treatment should be used to secure support for the remaining joists, with the only modification being that the straps be fastened to the sides of the joists instead of their tops.

The second condition mentioned involves the horizontal movement of the peaks of all the roof trusses 6" to 8" to the east. By all visible indications, this movement occurred prior to the installation of the existing sheathing and roof cover and has since ceased. It therefore represents no threat to the soundness of the roof structure.

RECOMMENDED ARLINGTON HOUSE IMPROVEMENTS:

1. Roof Truss Reinforcement.

Based on the observed member and joint failures and the loading conditions described in the previous section of this report, it is judged that the roof support system should be regarded as in a borderline category in terms of its stability and safety. Considering the historical and architectural importance of this building, such a precarious position is not acceptable. A supplemental roof support system is therefore needed to provide adequate and safe load-carrying capability.

The primary components of the proposed reinforcement system are double-angle "Fink" trusses to be installed adjacent to the existing wood trusses, as well as lateral bracing between them to prevent further horizontal movement. These trusses would be installed to pick up any live loads while allowing existing trusses to carry the dead loads. However, the proposed trusses are designed so that, should the wood trusses fail, the new steel trusses could support both the dead and live roof loads. The rationale behind this approach is:

a. since the trusses have supported both dead and live loads for some time now, an acceptable margin of safety could be achieved by removing the live load they carry;

b. installation of the proposed trusses could be accomplished with a minimal amount of damage or alteration to the historic members as the loads would be transferred directly to the four support points; and

c. as each existing truss has its own unique features and problems, it is impractical to calculate their supporting capacity based on actual individual conditions. Therefore, for simplicity and safety, the proposed trusses were designed to support both dead and live roof loads.

The design of the proposed trusses (Drawings 4 thru 16) reflects the most typical conditions encountered. Although this general approach can be used in all cases, the member lengths and certain details will need to be modified to account for the differences between the trusses.

Installation of the proposed reinforcement systems will require extensive field measuring to determine member lengths at each truss location and some adaptation of the design details to account for these varying conditions. Examples of the anomalies in the existing roof support system which will require adjustment of the proposed design are: the masonry chimneys through which truss "D" passes; the metal link chain at the south-end of truss "B"; and the presence of headers at the heel joints which support the outriggers. In these situations, isolated modification of the proposed trusses will be necessary to accommodate installation and fit. For the most part, these adjustments will only entail notching non-essential portions of wood members or modifying the lengths of members and locations of joints.

Of the two end trusses ("A" and "F"), only truss "F" requires reinforcement. Because a significant portion of truss "F" is bordered by the "finished" interior of the stairwell, the supplemental support system should be installed directly under the top chord and over the bottom chord. This will therefore require the temporary removal of the existing diagonal braces, removal of several floor boards, and the notching of wall studs to accommodate installation of the new truss.

2. Attic Ventilation

In the case of truss "D" (whose ends pass through the brick chimneys), partial disassembly of the masonry will be required. As ventilation in the attic is poor, causing possibly damaging temperatures in this area during summer months, air exhange/circulation should be increased. This could best be accomplished in conjunction with the reinforcement of this truss by capping the chimneys when disassembled and installing fans within the flues. This should be an integral facet of any project undertaken in the attic.

3. Energy Conservation

In compliance with NPS energy conservation guidelines, it is also recommended that Kraft unfaced fiberglass roll type insulation of thermal resistance R=19 and 6 inches thick, be installed following the completion of all other elements of the roof stabilization effort.

4. Wood Member Repair/Replacement

Miscellaneous areas where wood repair or replacement is required are as follows:

a. $5\frac{1}{2}$" x 6" purlins to be added at the lower sections of roof to ensure a maximum span by the rafters of 5 feet;

b. split purlins to be repaired by use of straps or through bolts and reinforced by the introduction of new members.

c. the deteriorated portion of truss "D" in the vicinity of the (south-side) chimney to be repaired as suggested in Appendix B.

5. Attic Use

Use of the attic area should be restricted to small-scale activities, such as light object storage and controlled demonstrations for children.

6. Method of Completion

The completion of this work should be accomplished by means of day labor forces instead of by contract. This approach would more easily accommodate the time and labor intensive requirements of this project, provide greater flexibility in addressing the known and unknown differences between the trusses, and enable the government to avoid unnecessary extra costs due to likely "change-in-conditions" claims.

7. Statement of Effects

The goal of the recommended treatment is to stabilize the attic structure of the Arlington House main block. The results will increase the life of the building and alleviate the possibility of danger to park visitors.

In applying the criteria of effect, 36 CFR Park 800.3(a), the National Park Service has determined that the proposed undertaking will have an effect on the Arlington House.

In applying the criteria of adverse effect, 36 CFR 800.3(b), the National Park Service has found the effect not to be adverse because:

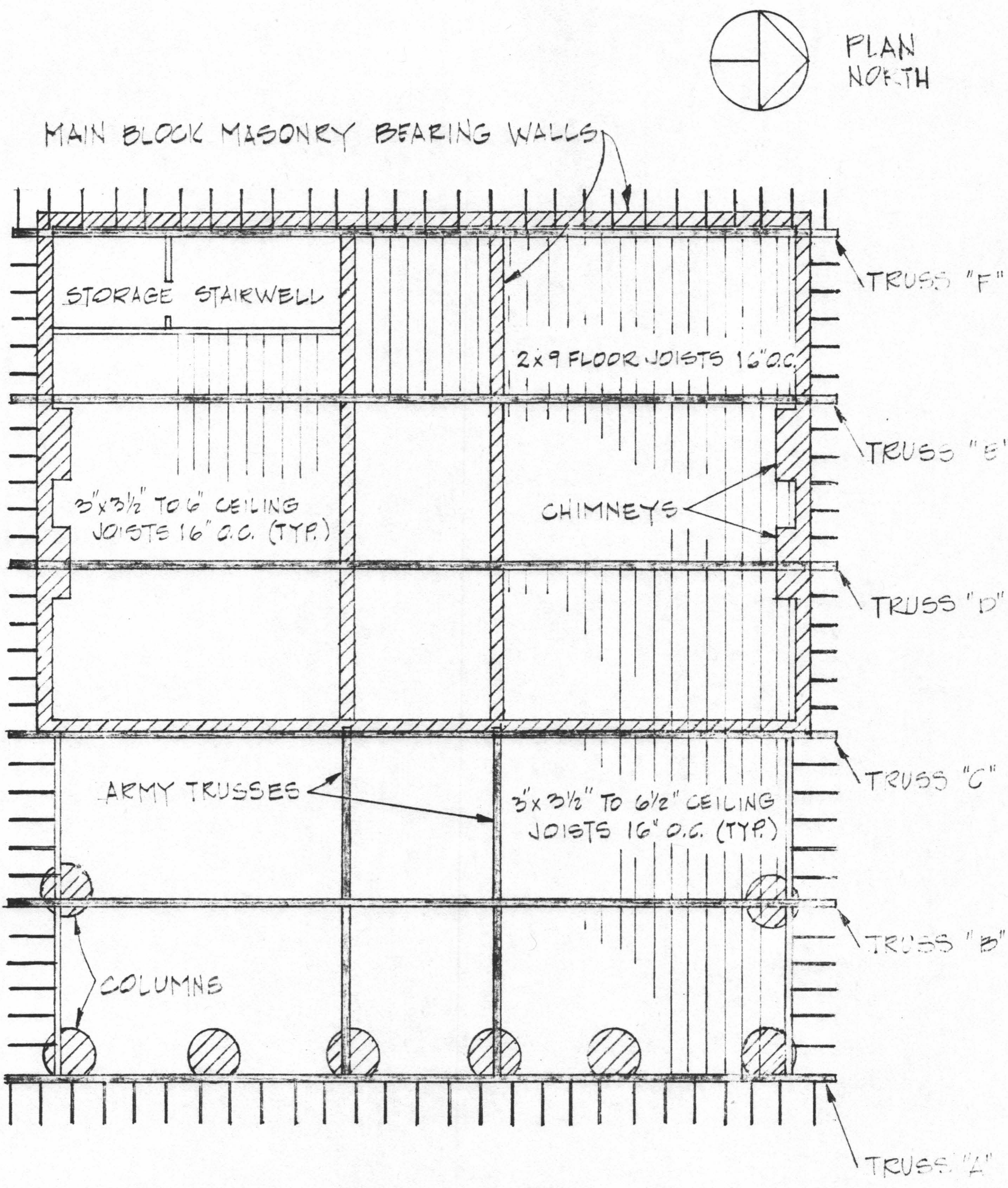
a. The proposed undertaking will not result in the destruction of the significant features of the property, even though substantial alterations are planned. No historic fabric will be removed or irreversibly modified.

b. The proposed undertaking will not result in the isolation of the property from, or alteration of, its surrounding environment.

c. The proposed undertaking will introduce visual elements that are out of character with the historic fabric of the attic. However, the attic is off-limits to the public, so the visual impact will be insignificant.

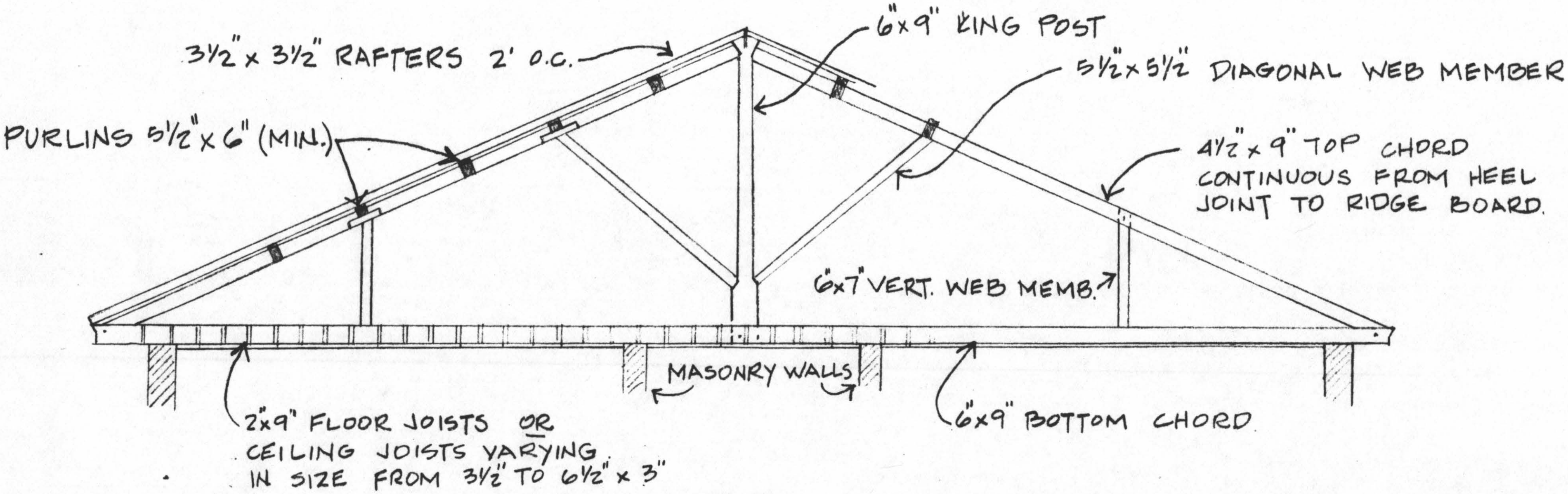
d. The proposed undertaking will not result in the transfer, sale, deterioration, or destruction of federally owned property.

DRAWINGS



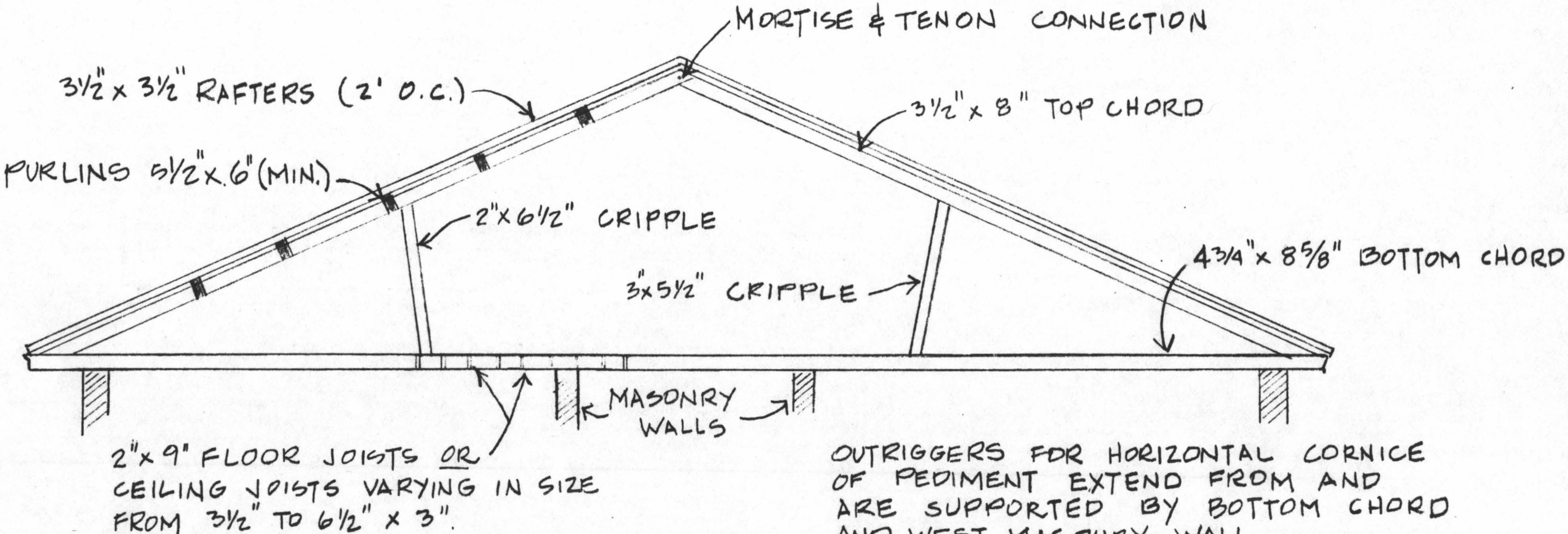
TRUSS AND ATTIC FLOOR FRAMING PLAN
NO SCALE

SHEET 1 OF 16



ELEVATION: MAIN TRUSS TYPE & ROOF FRAMING

SCALE: 1/8" = 1'-0"



ELEVATION - TRUSS "F" & ROOF FRAMING

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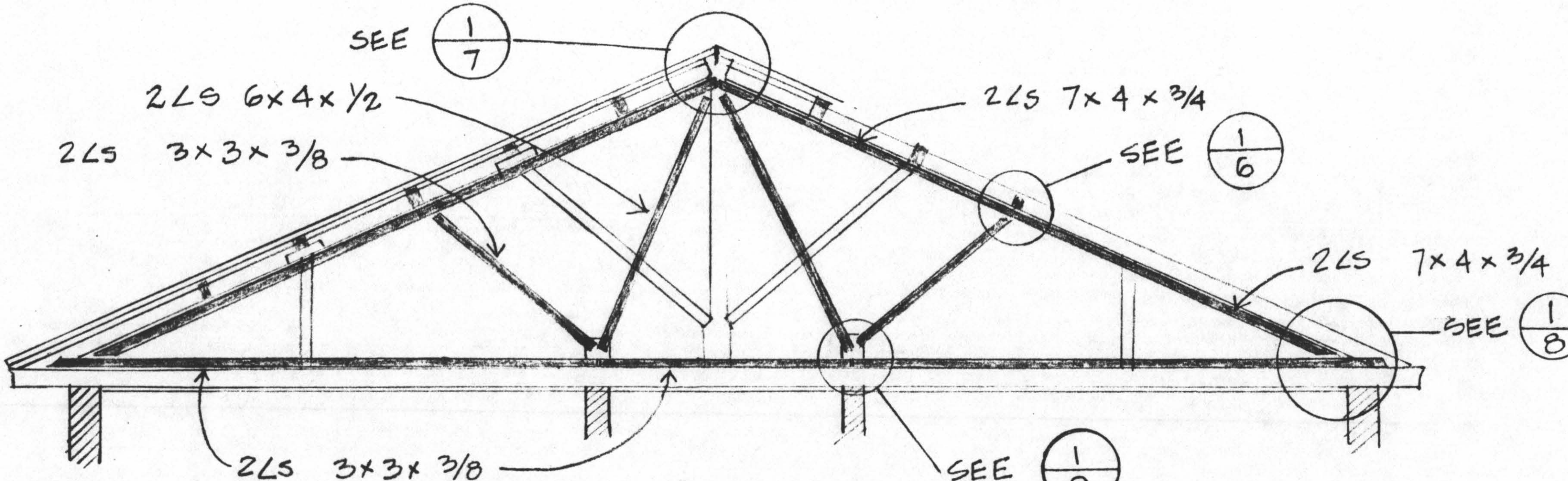
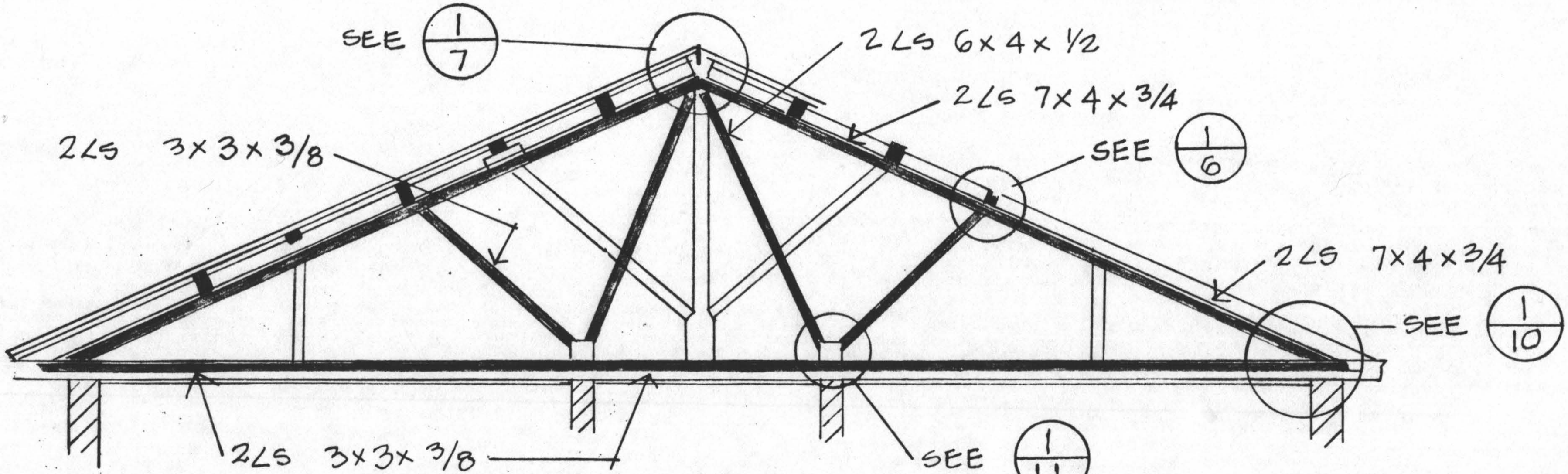
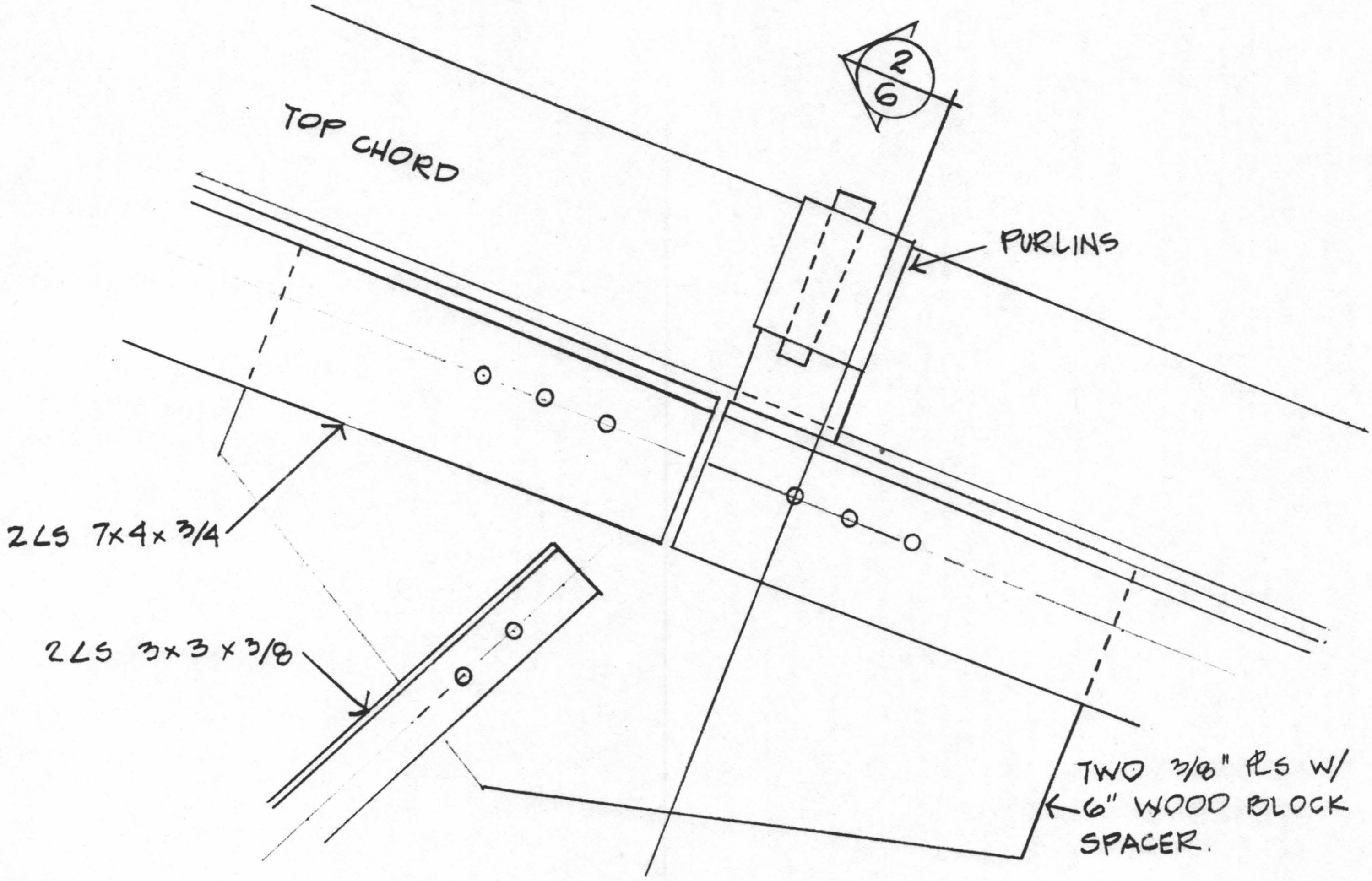


ELEVATION - PROPOSAL FOR STABILIZATION OF TRUSSES "D" & "E"

SCALE: 1/8" = 1'-0"

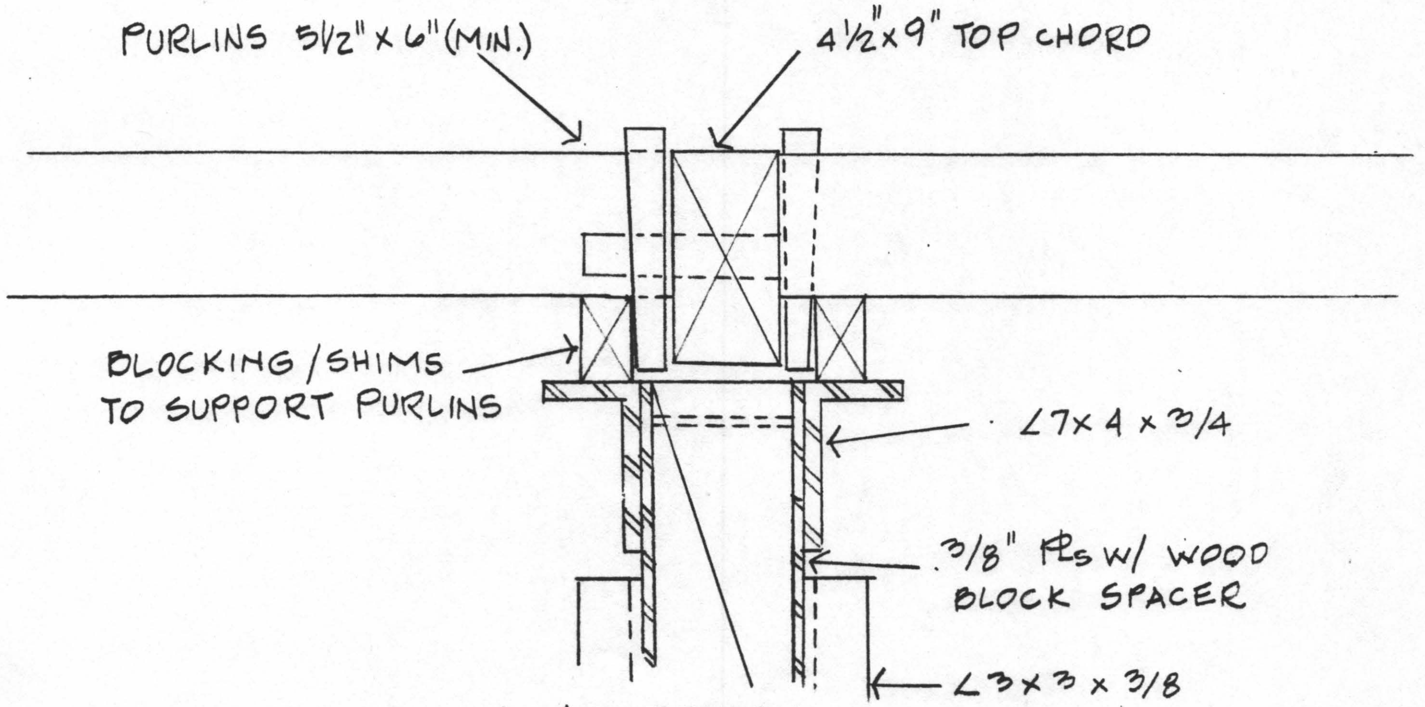


ELEVATION - PROPOSAL FOR STABILIZATION OF TRUSSES "B" & "C"
SCALE: 1/8" = 1'-0"



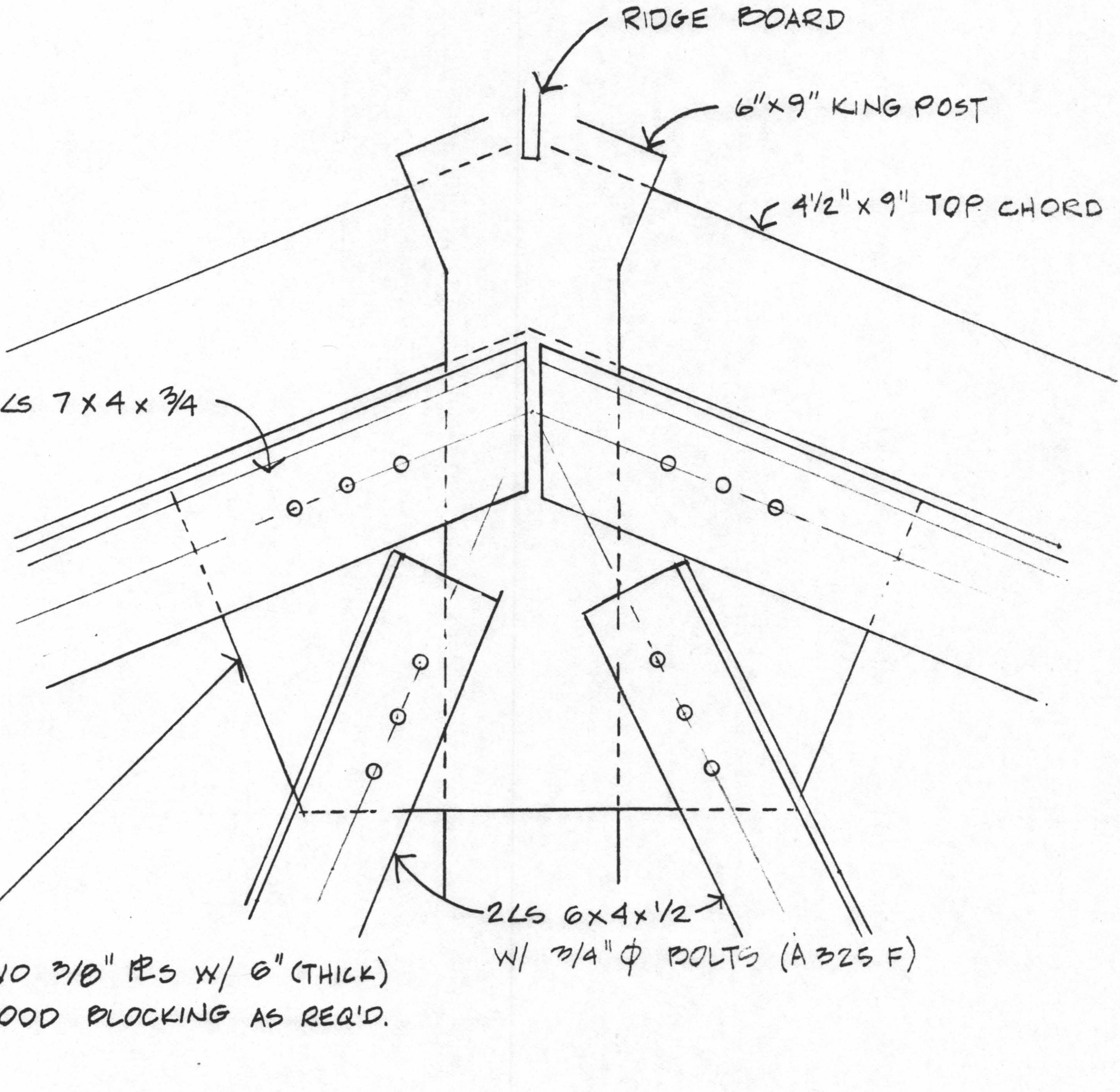
1/6 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSSES

NOT TO SCALE



2/6 SECTION - JOINT DETAIL

NOT TO SCALE



1/7 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSSES
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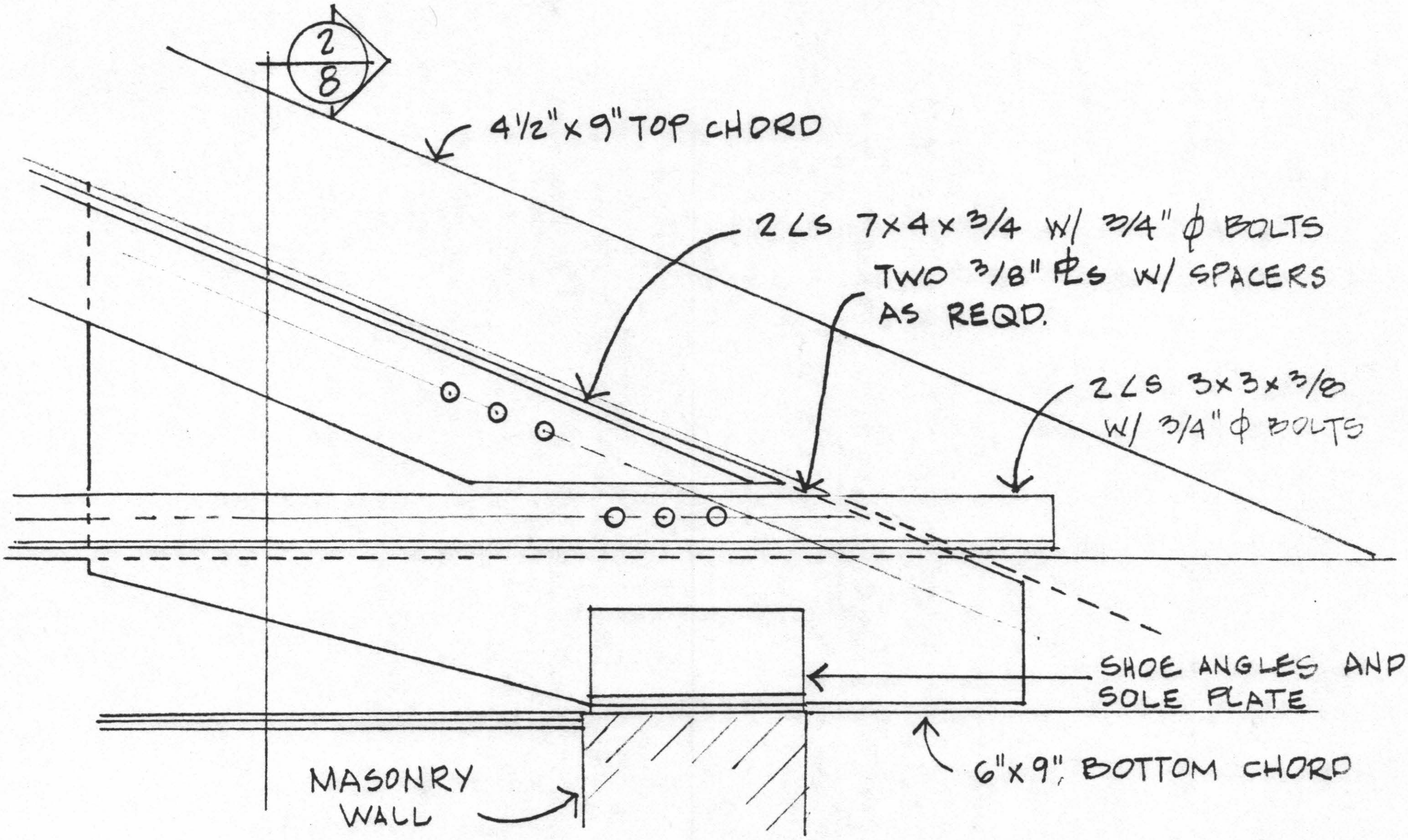

1/8 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSSES

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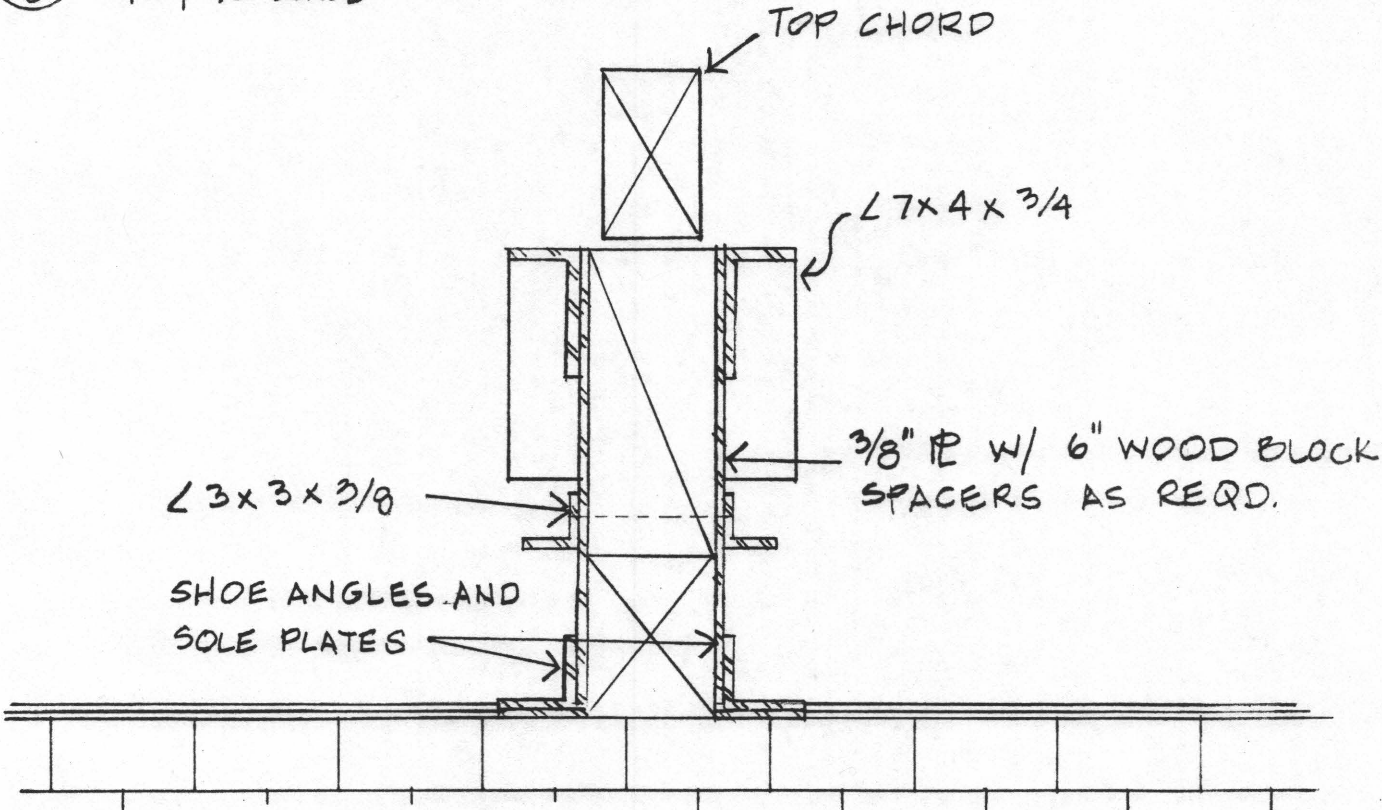

2/8 SECTION - JOINT DETAIL

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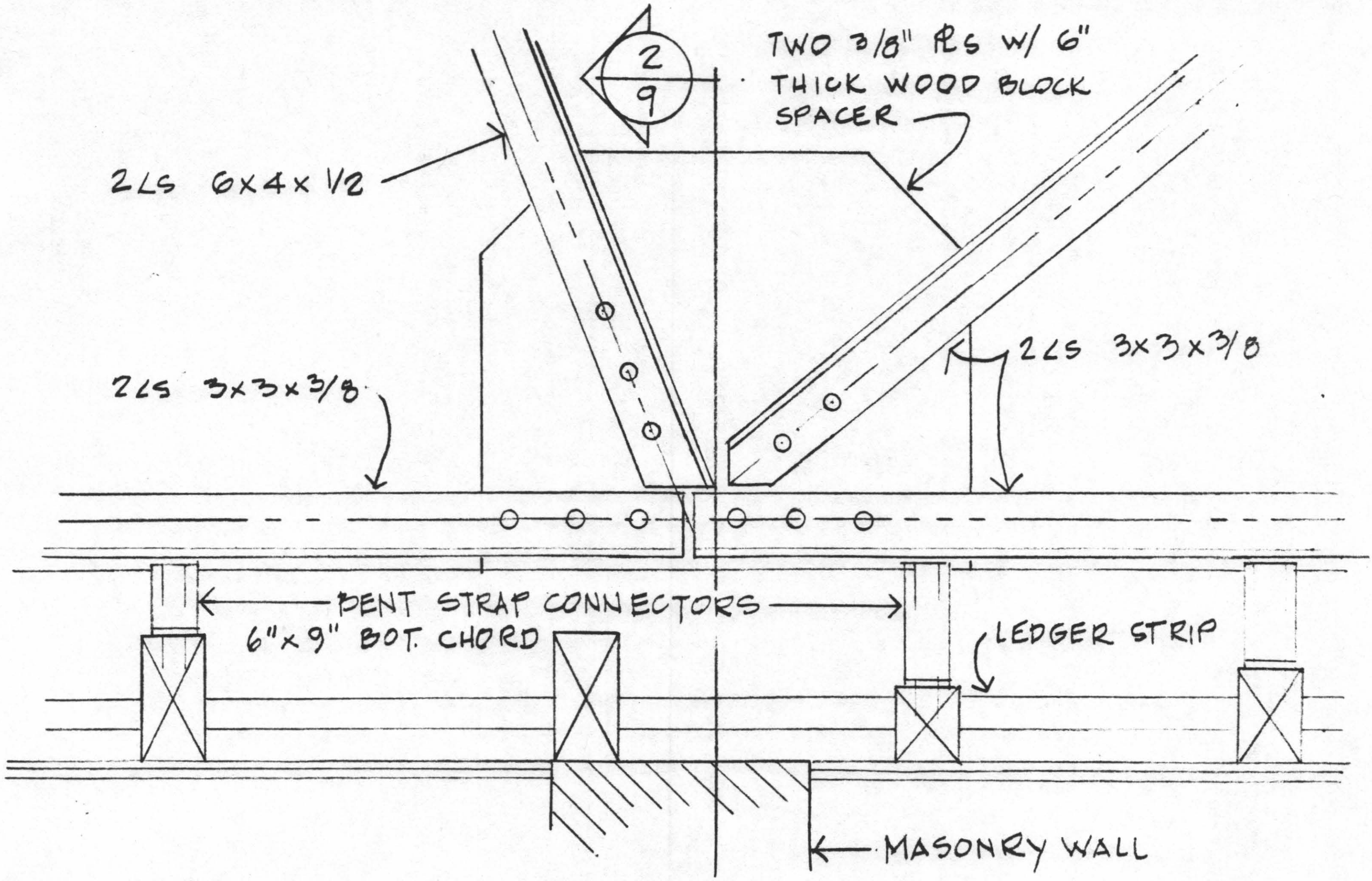
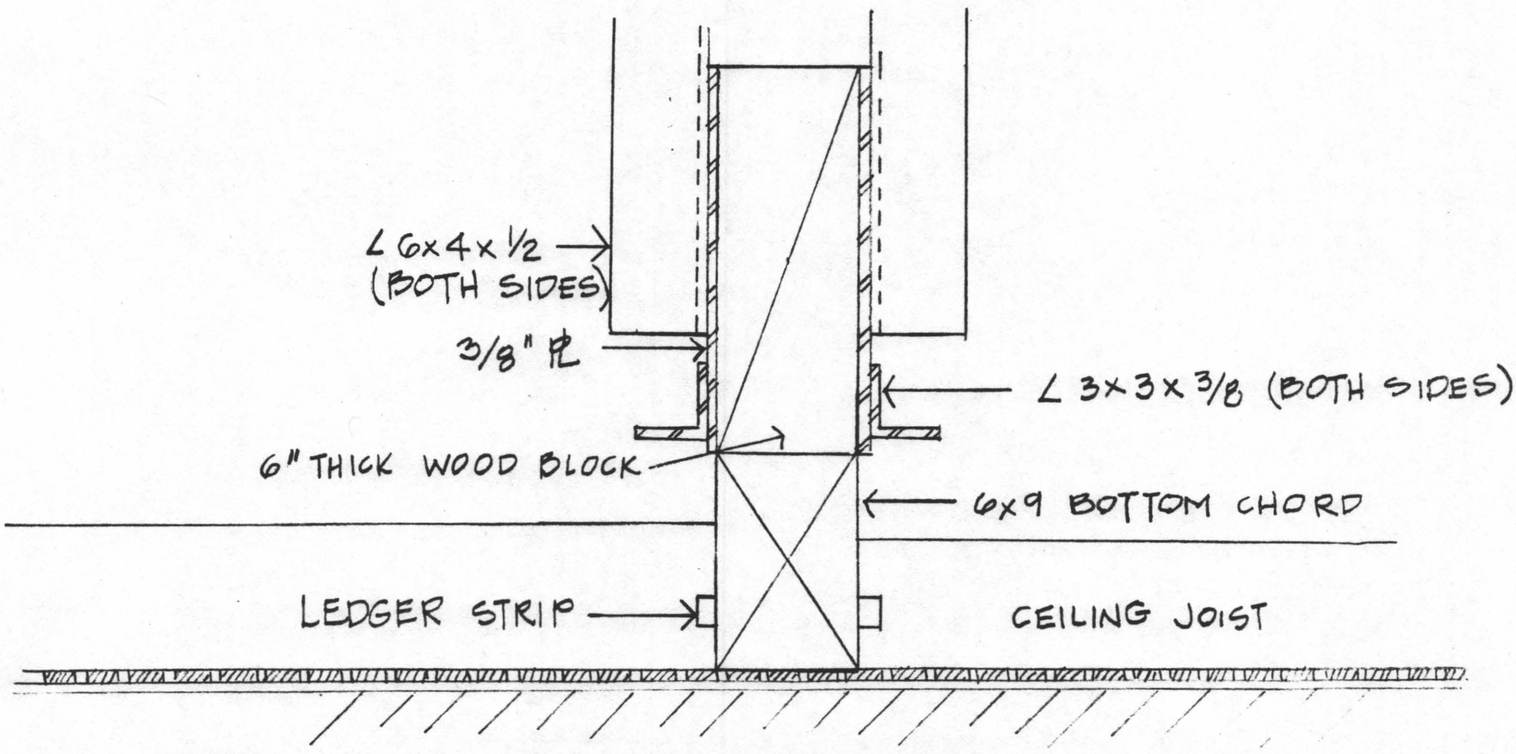
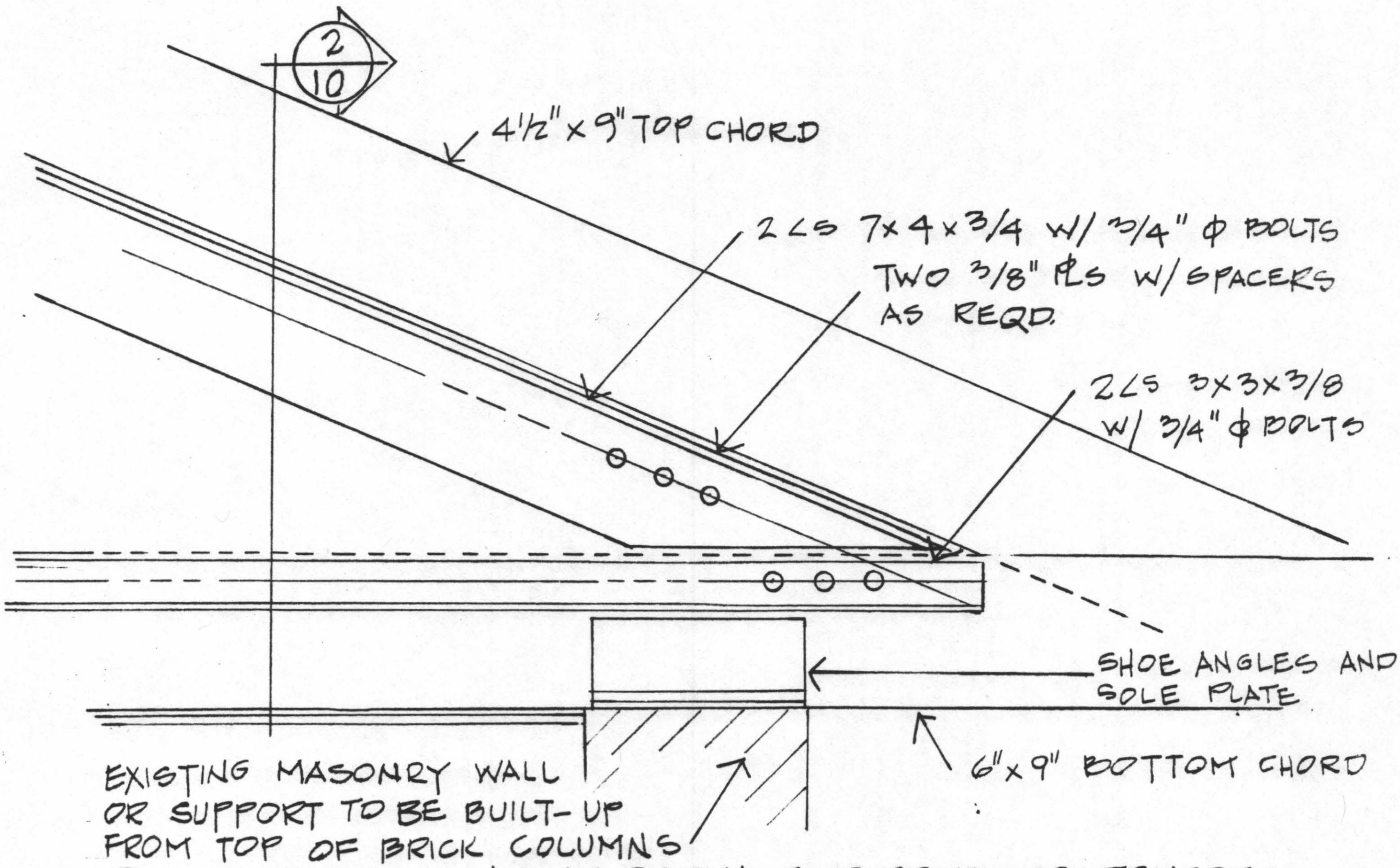


1/9 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSSES
NOT TO SCALE



2/9 SECTION-JOINT DETAIL
NOT TO SCALE



1/10 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSSES

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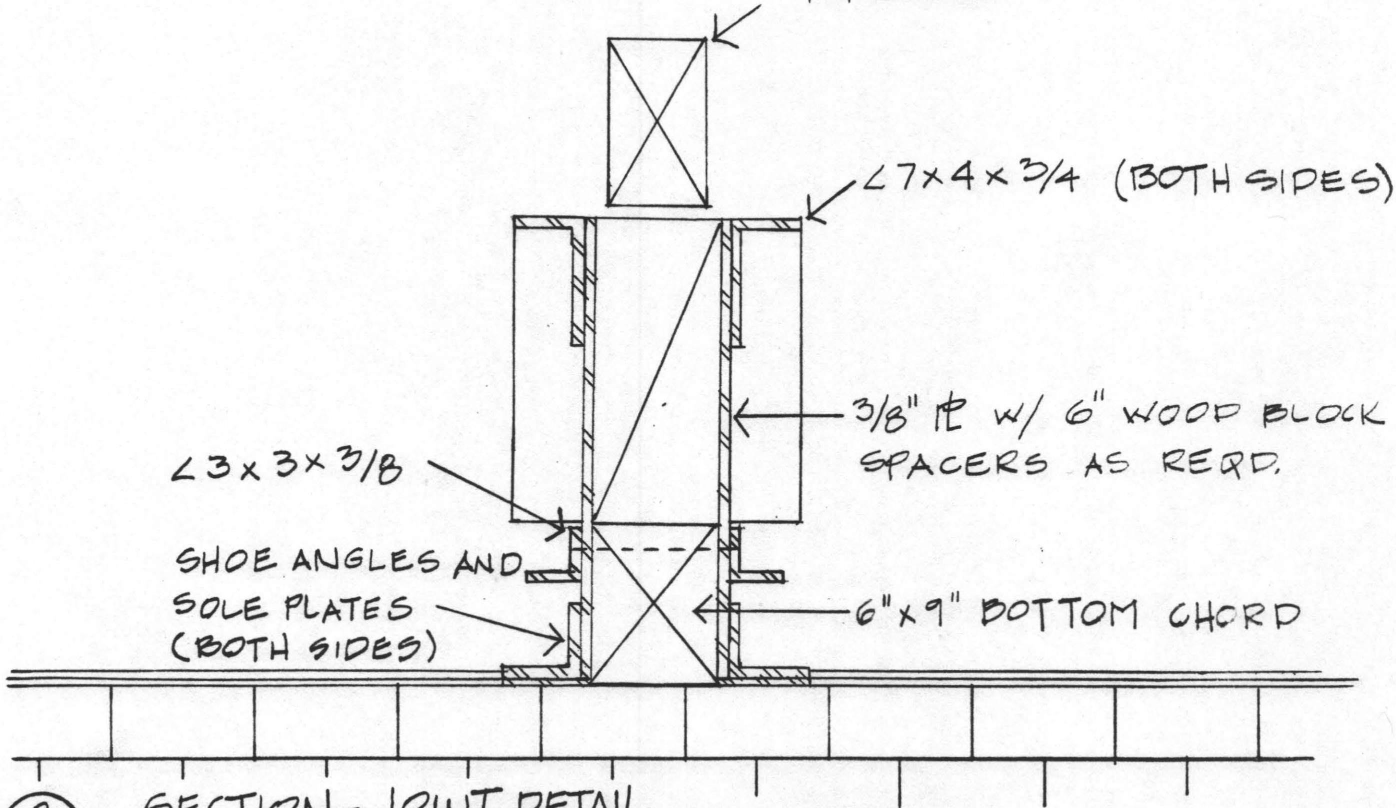

2/10 SECTION - JOINT DETAIL

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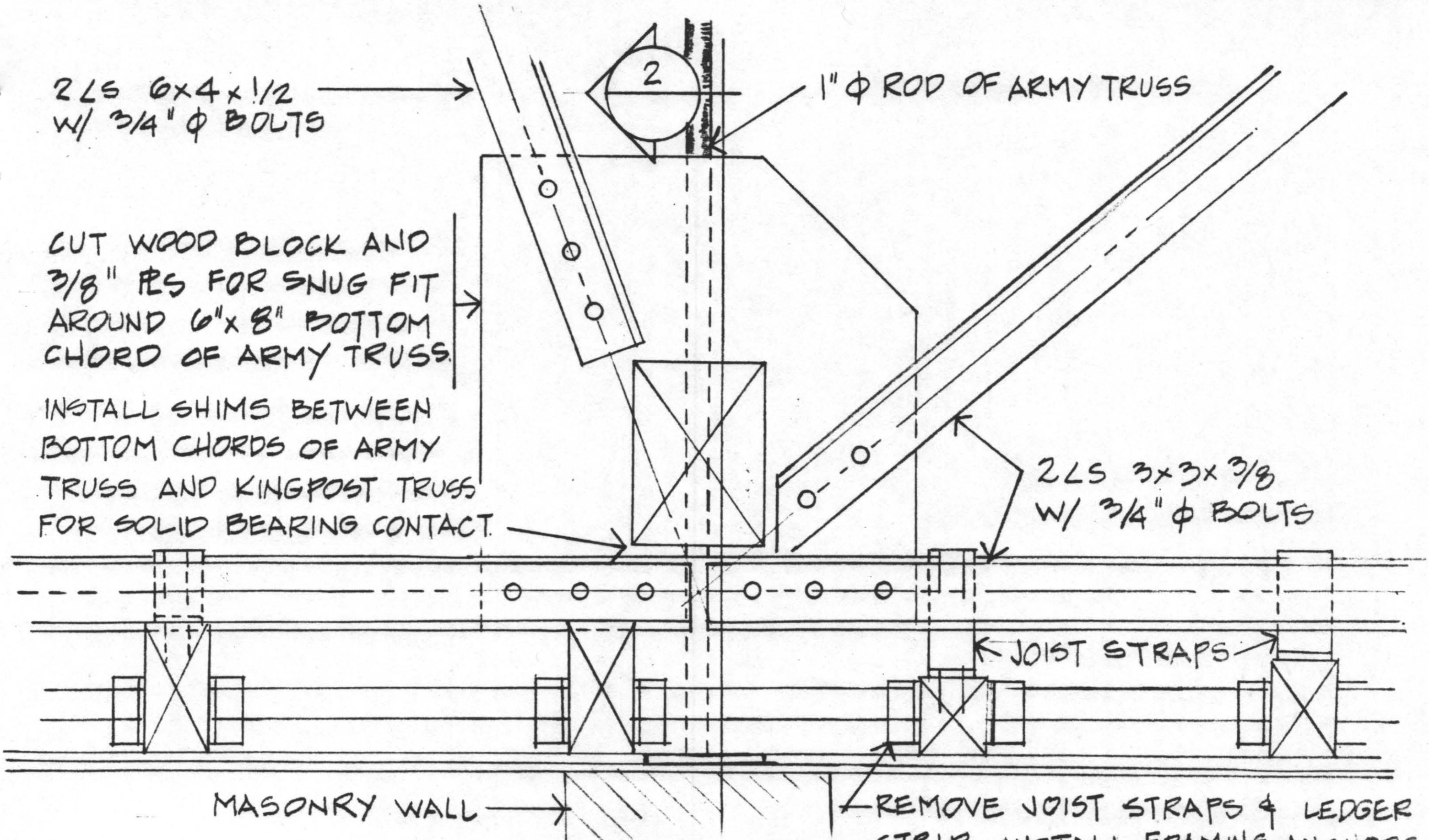

1/11 ELEVATION: JOINT DETAIL
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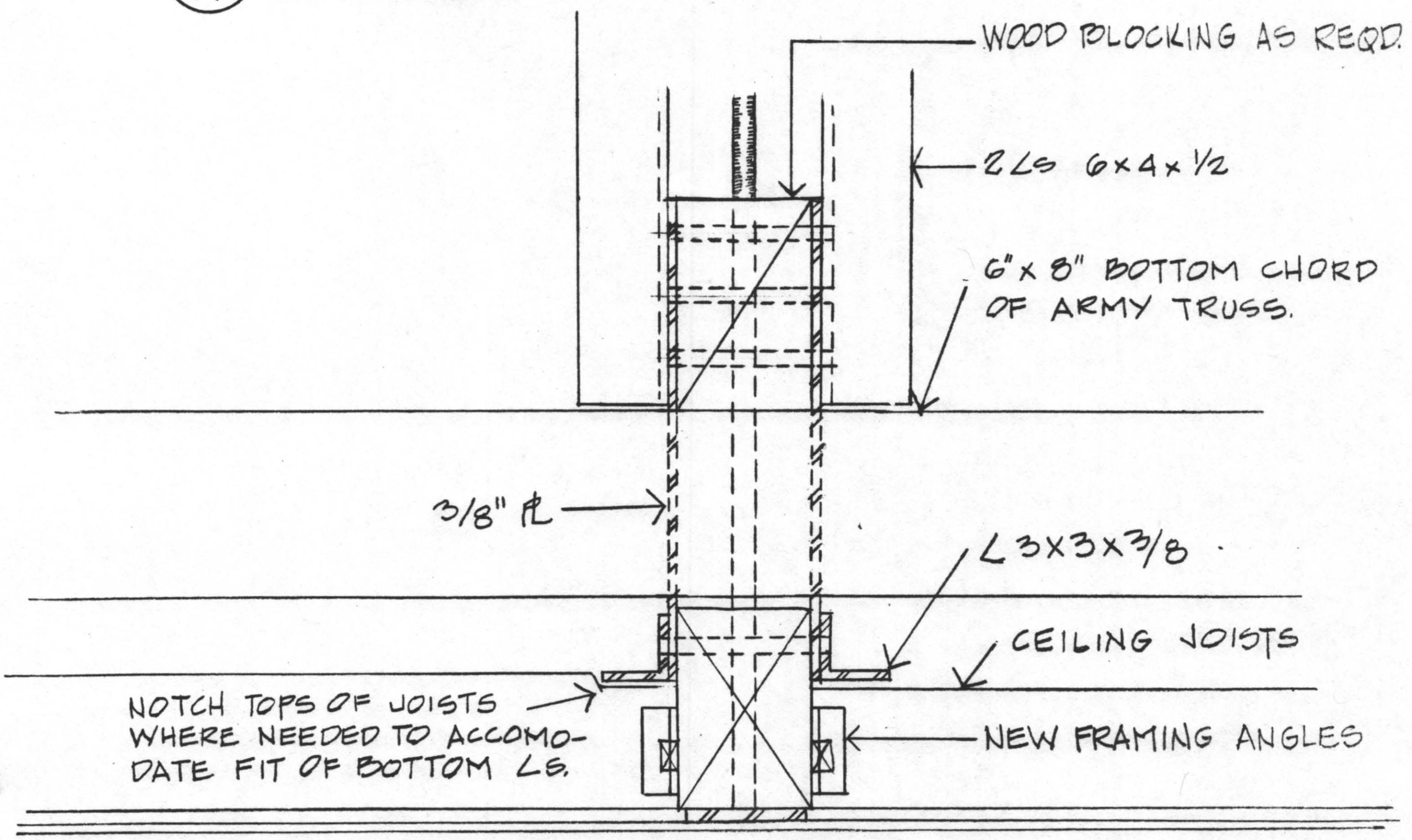

2/11 SECTION - JOINT DETAIL
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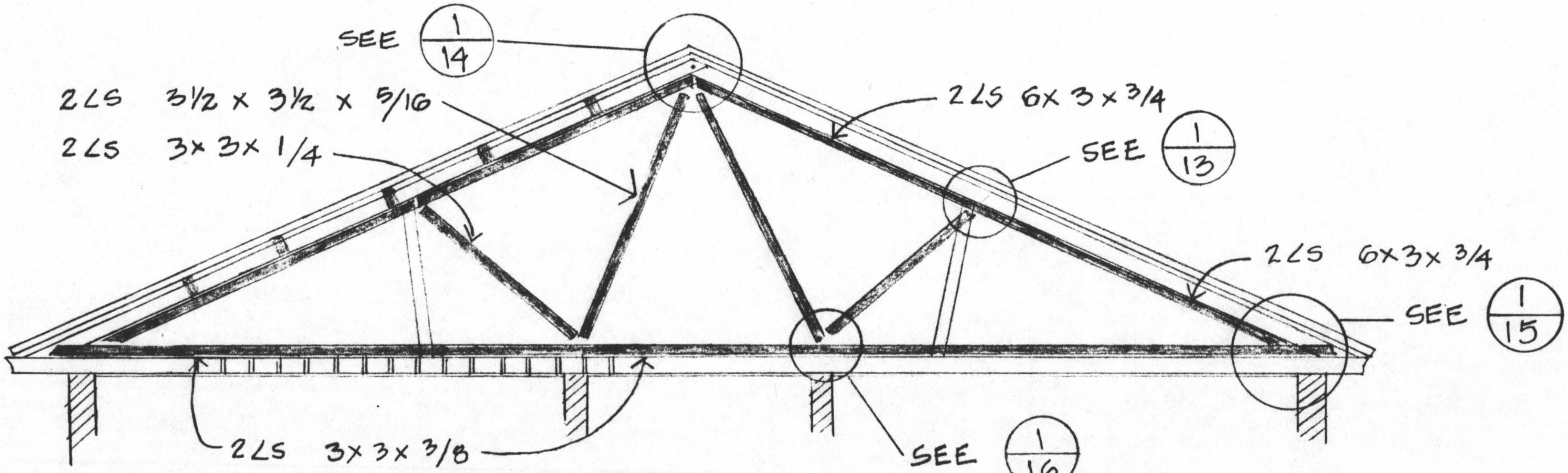
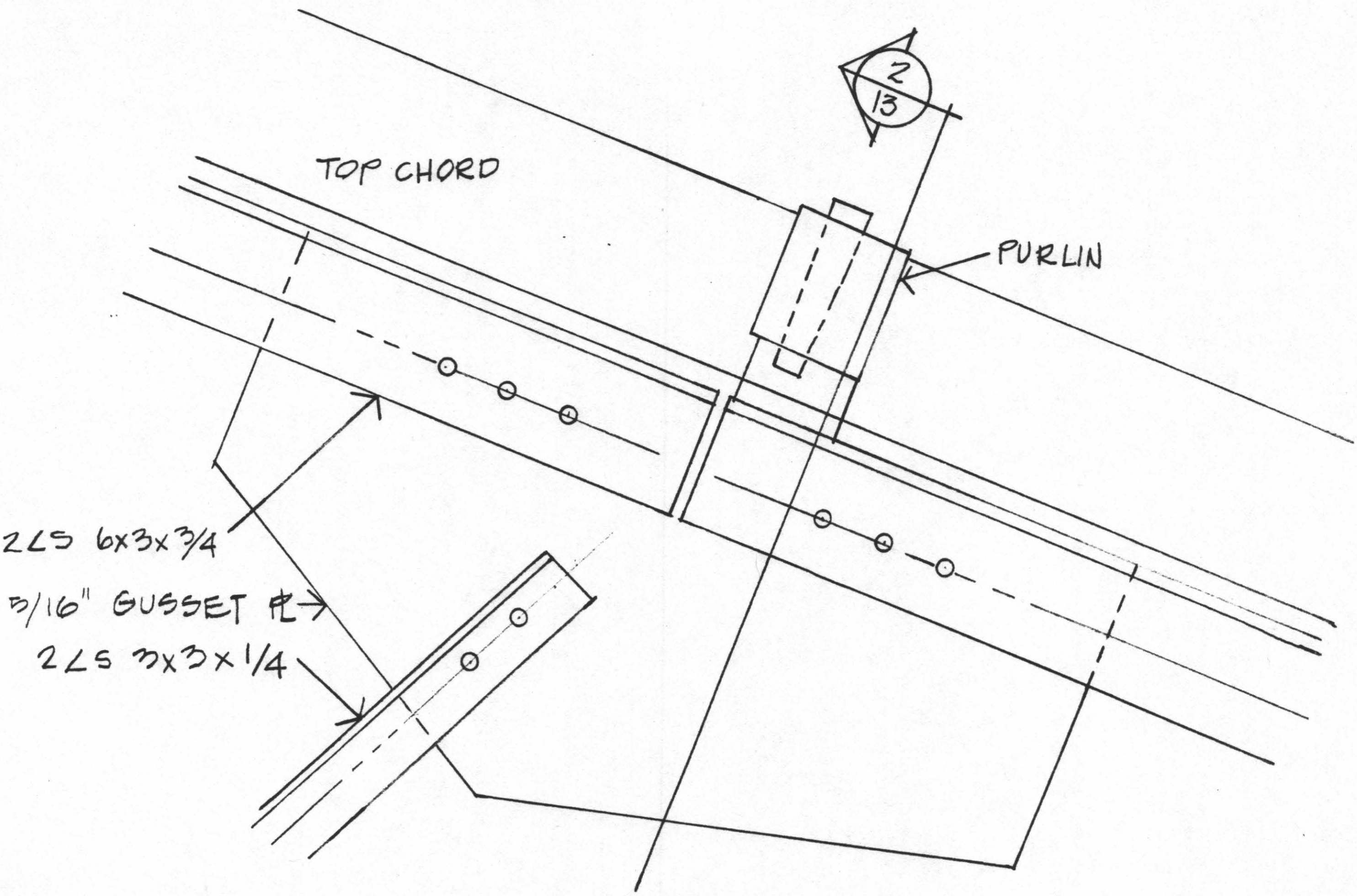


ELEVATION: PROPOSAL FOR STABILIZATION OF TRUSS "F"

SCALE: 1/8" = 1'-0"



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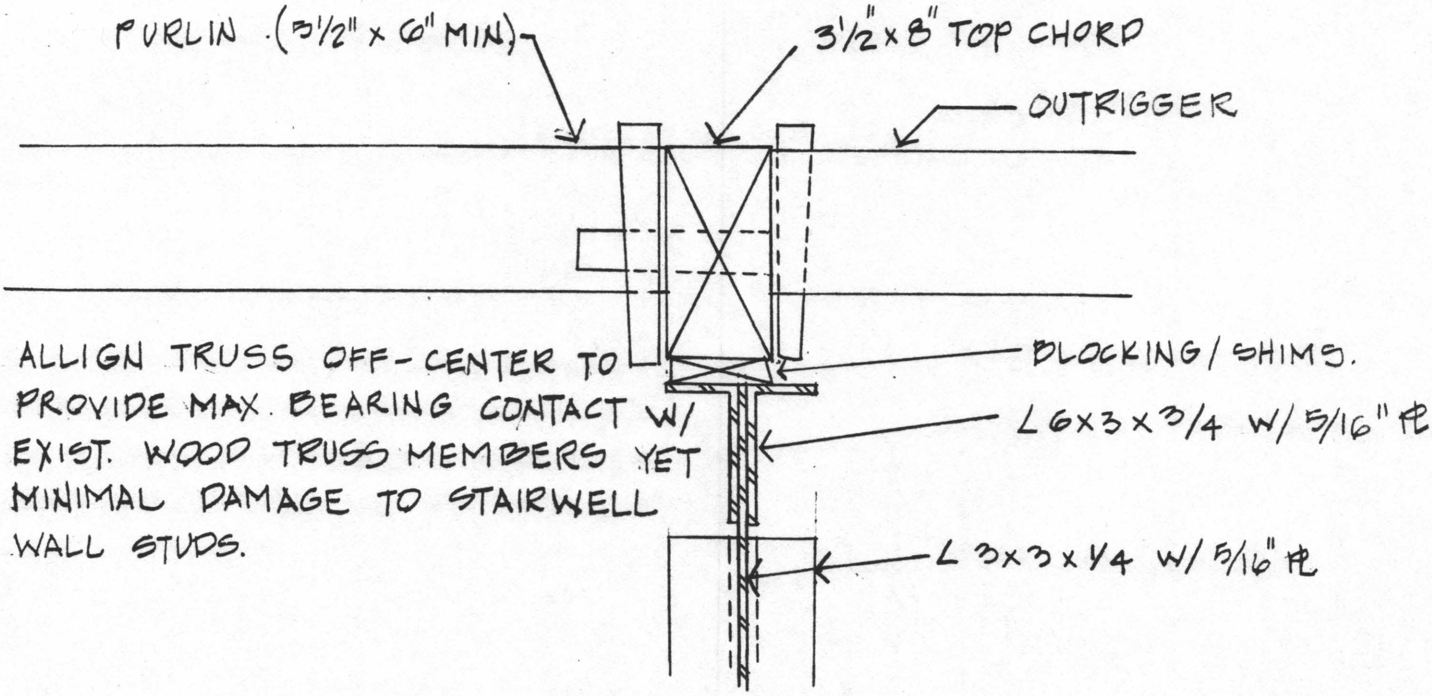
13
ELEVATION: JOINT DETAIL FOR PROPOSED TRUSS
NOT TO SCALE



2
13
SECTION - JOINT DETAIL
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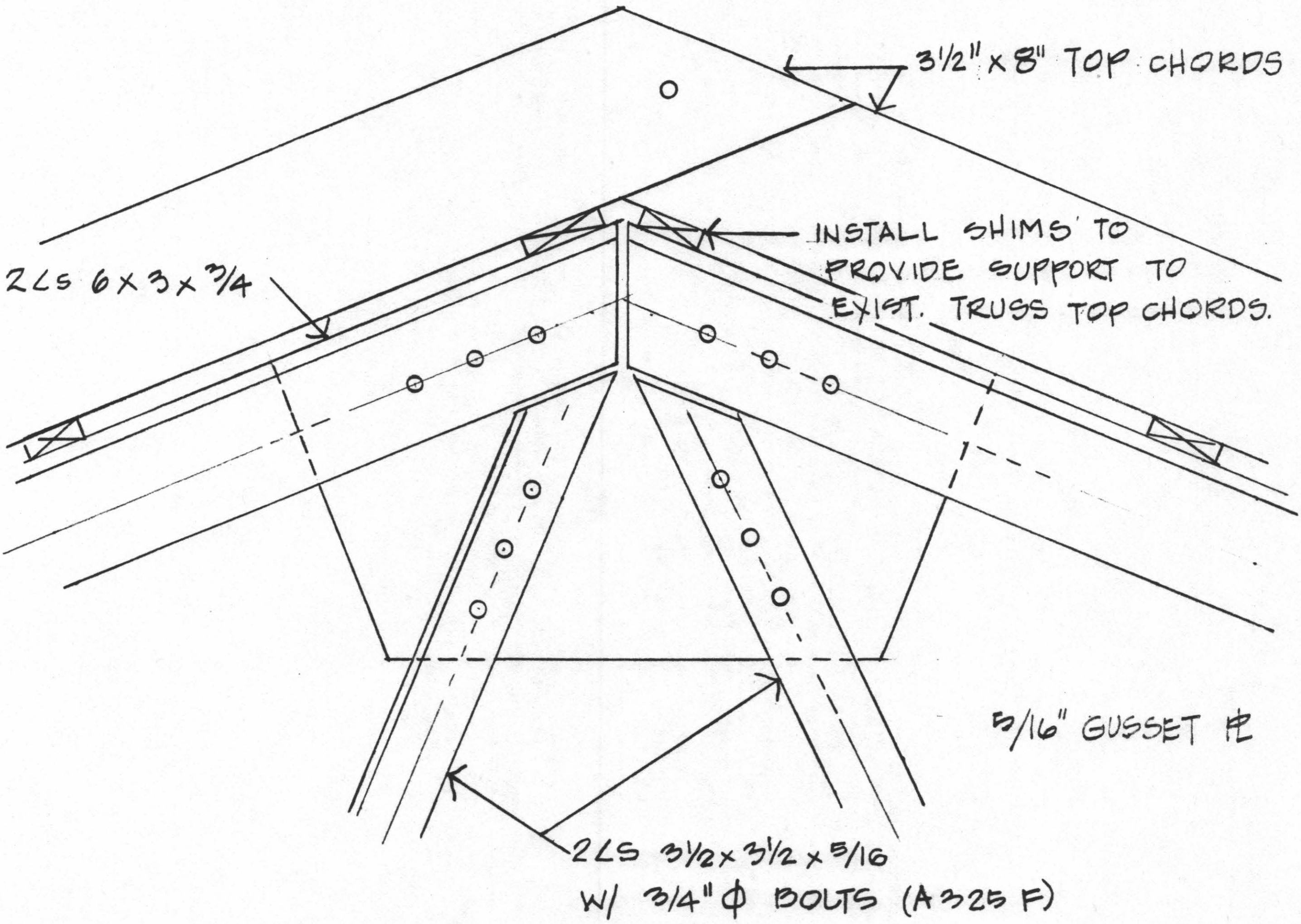

1/14 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSS

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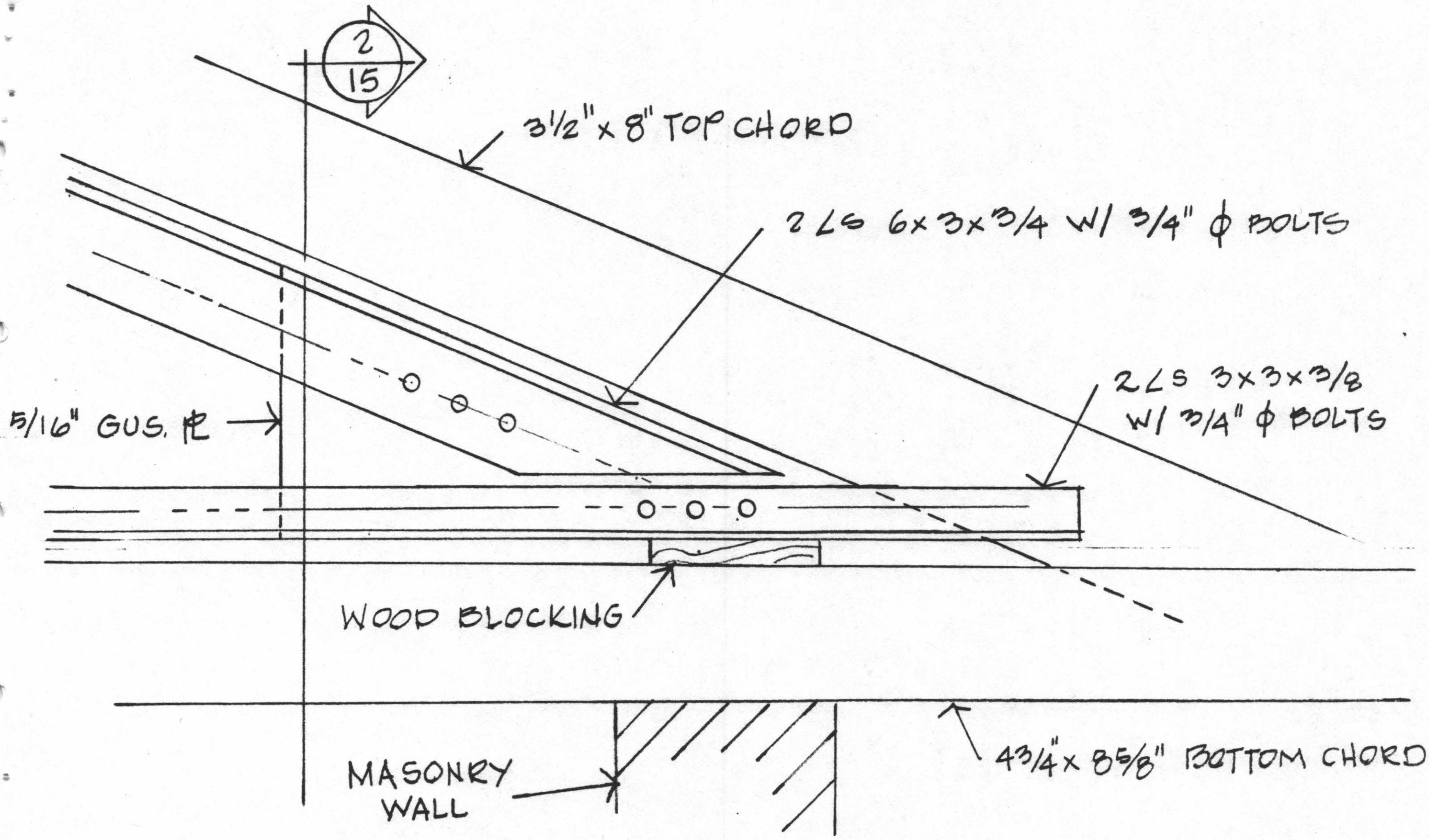

1/15 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSS

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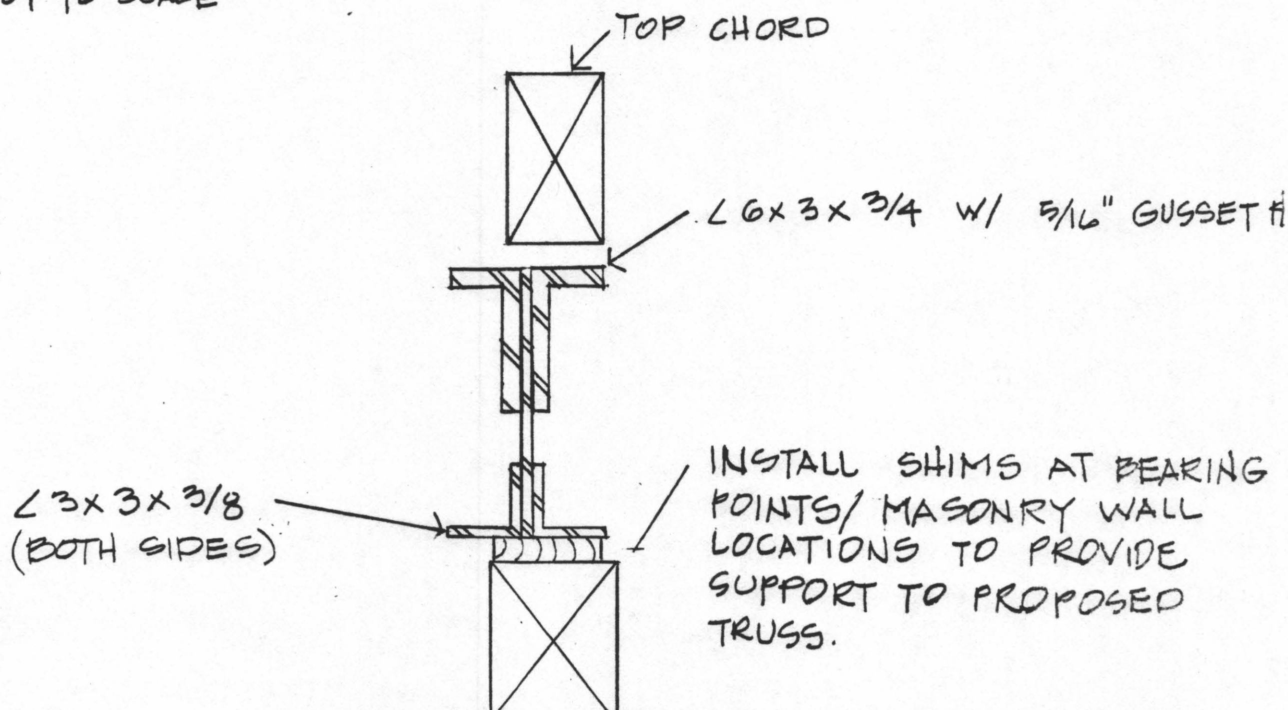

2/15 SECTION- JOINT DETAIL

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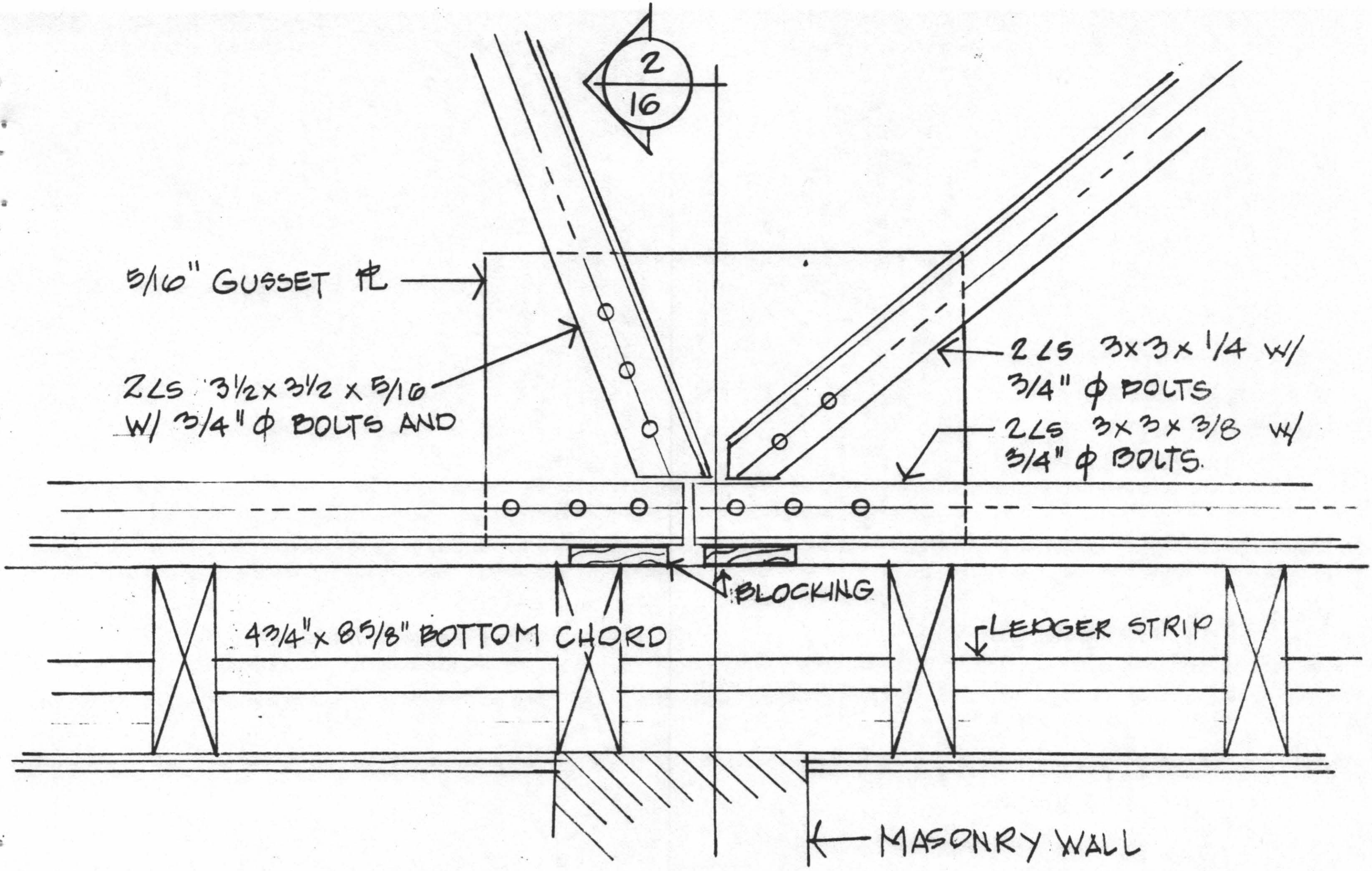

1/16 ELEVATION: JOINT DETAIL FOR PROPOSED TRUSS

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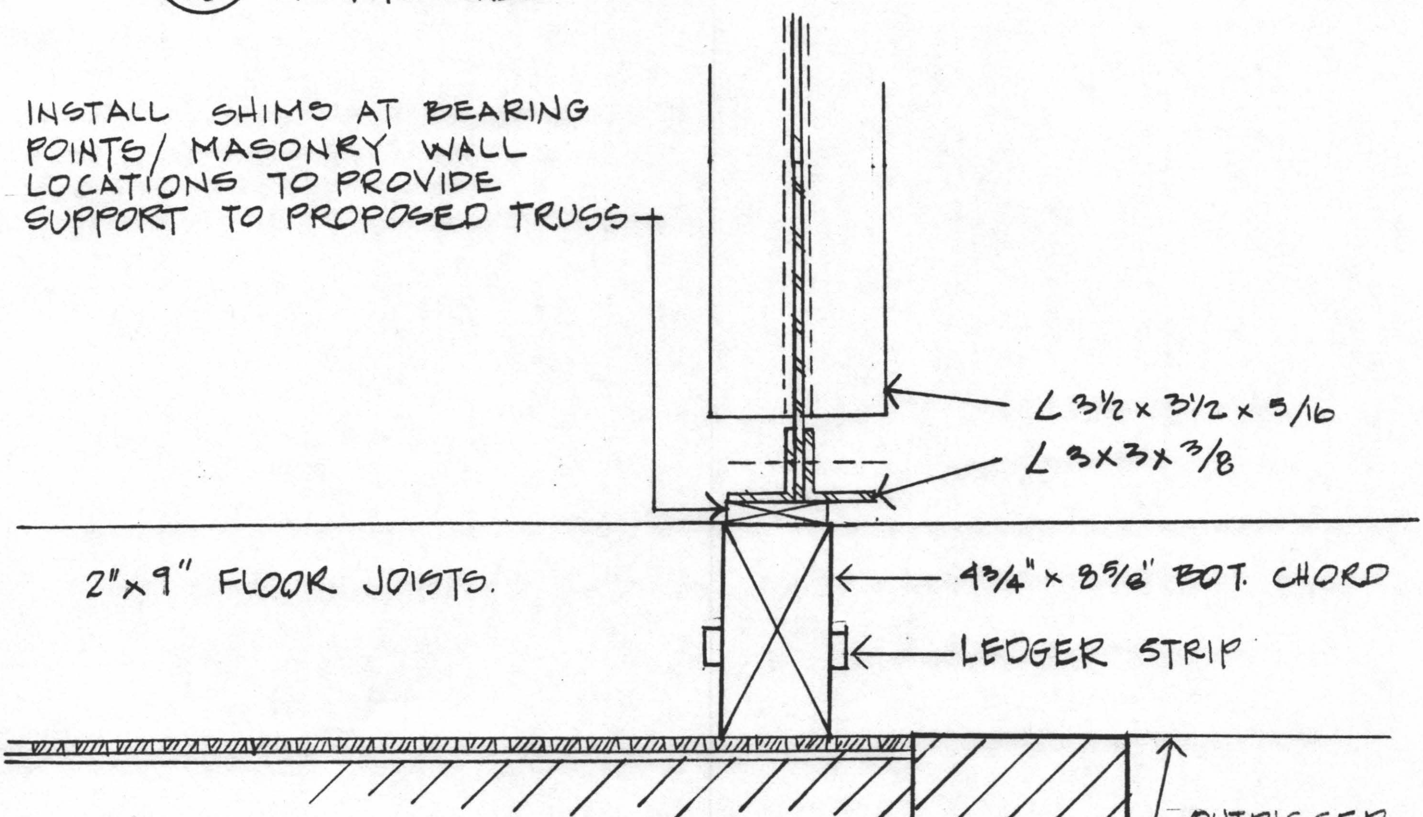


2/16 SECTION - JOINT DETAIL

NOT TO SCALE

PHOTOGRAPHS

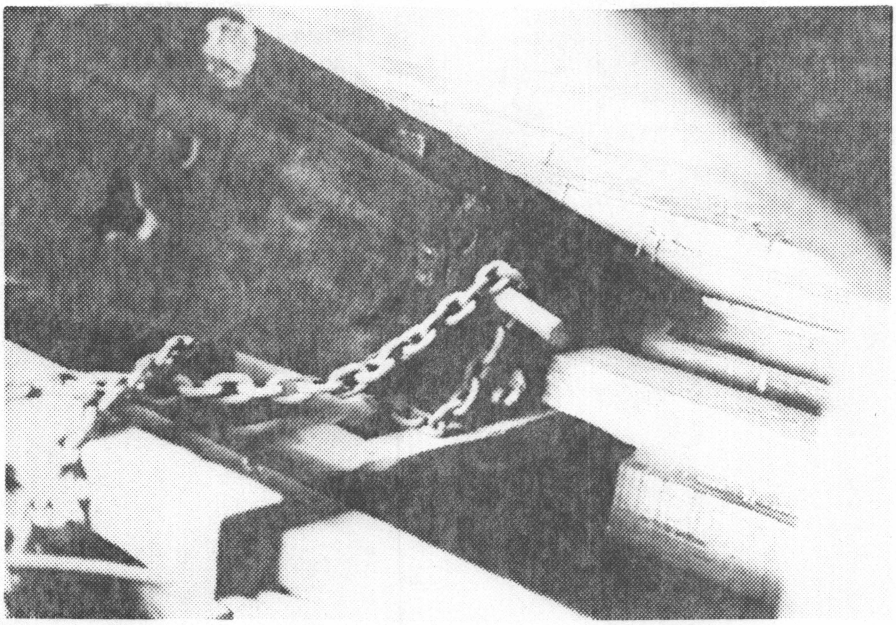


Photo 1. Truss heel joint, south end of truss "B".

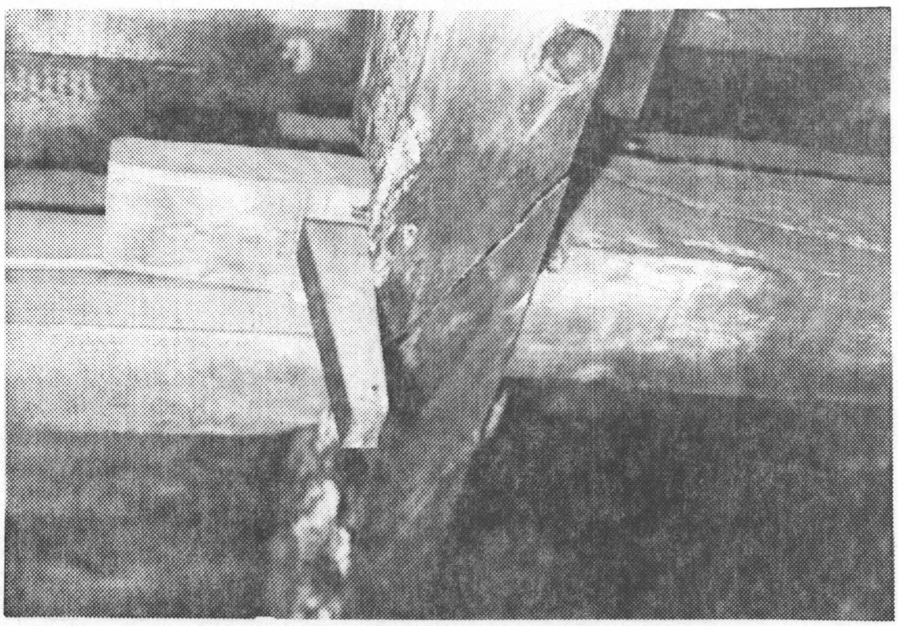


Photo 2. Partial failure of top chord of truss along twisted grain.

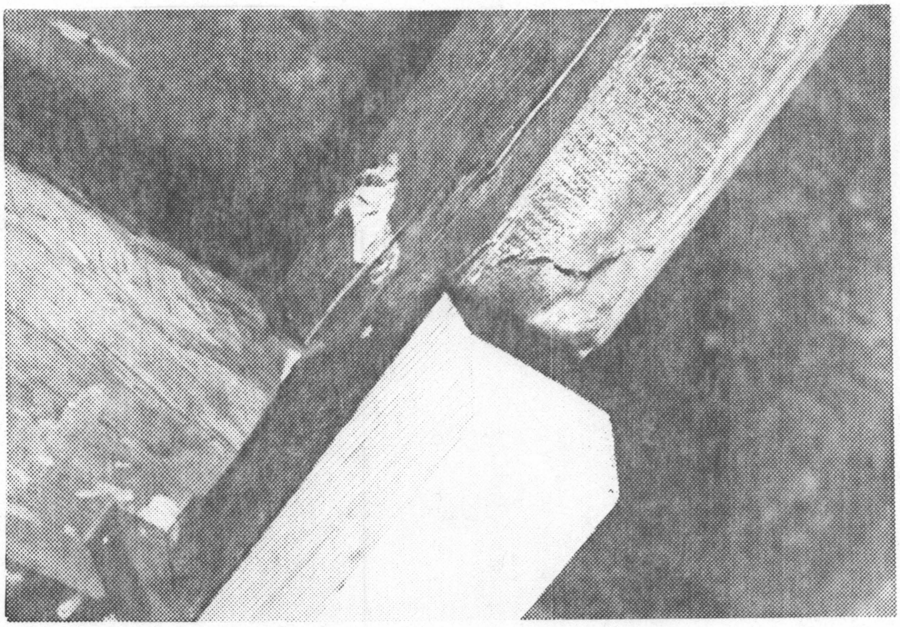


Photo 3. Failure of extreme fibers in bending, top chord, truss "E".

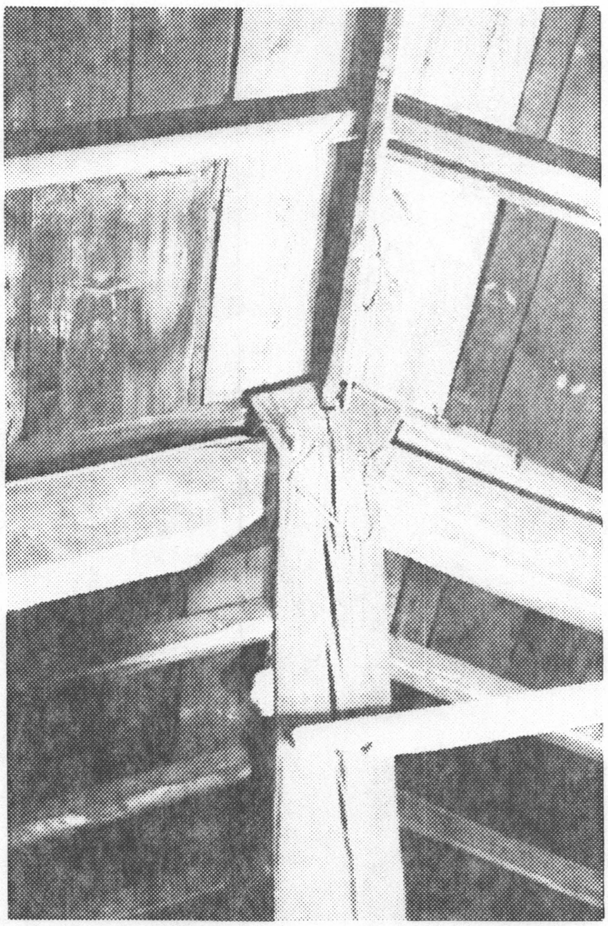


Photo 4. Example of checking in truss members (kingpost) due to seasoning.

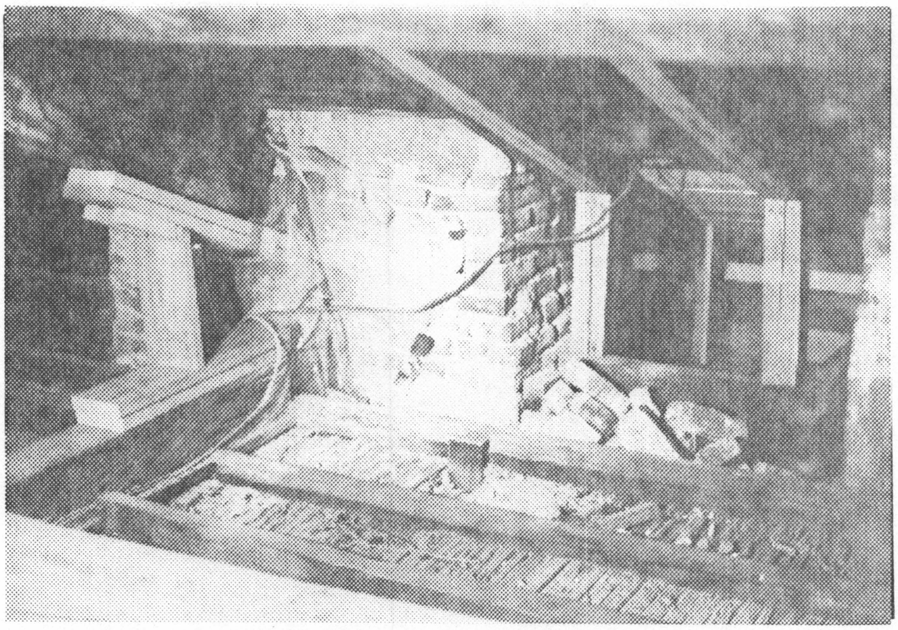


Photo 5. South end of truss "D" on north side of masonry chinmey.

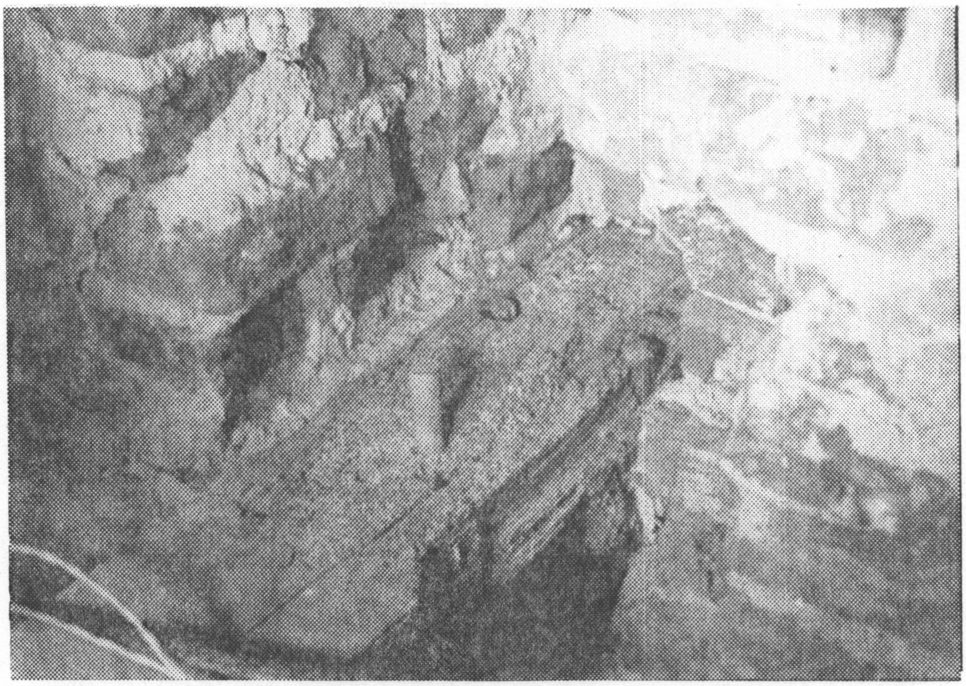


Photo 6. Portion of bottom chord of truss "D" exposed by partial dismantling of masonry chimney. Note extensive level of deterioration due to powder post beetle and brown rot.

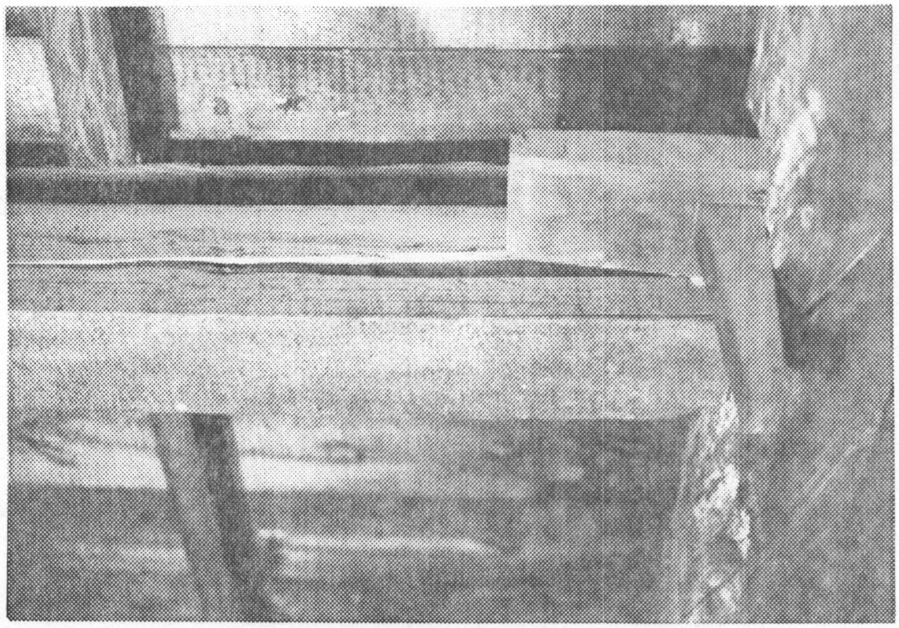


Photo 7. Purlin failure due to excessive loading. Recommended action - repair using metal straps or through bolts.

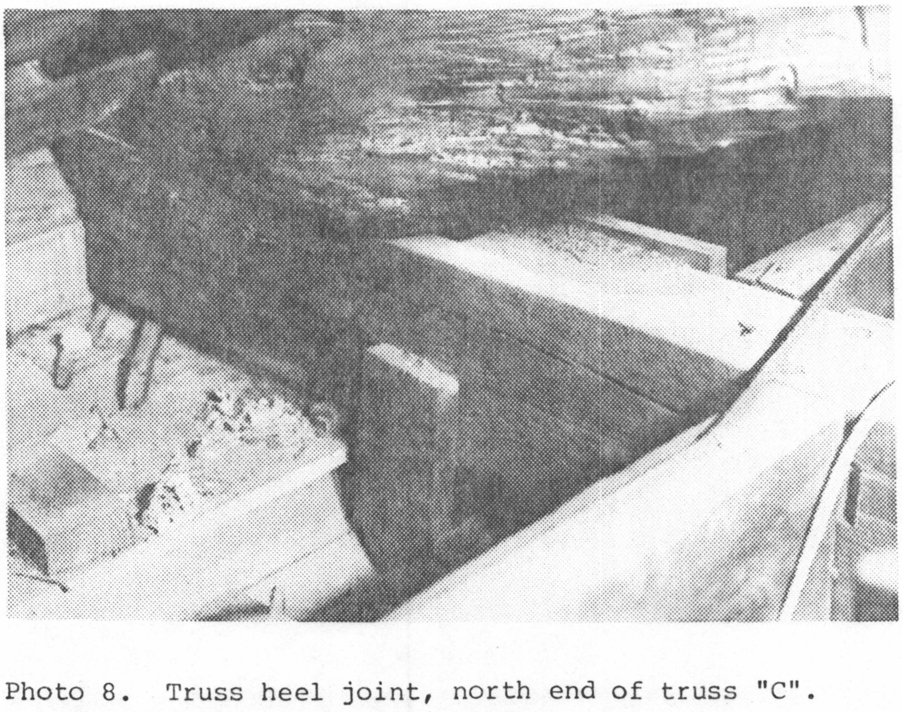


Photo 8. Truss heel joint, north end of truss "C".

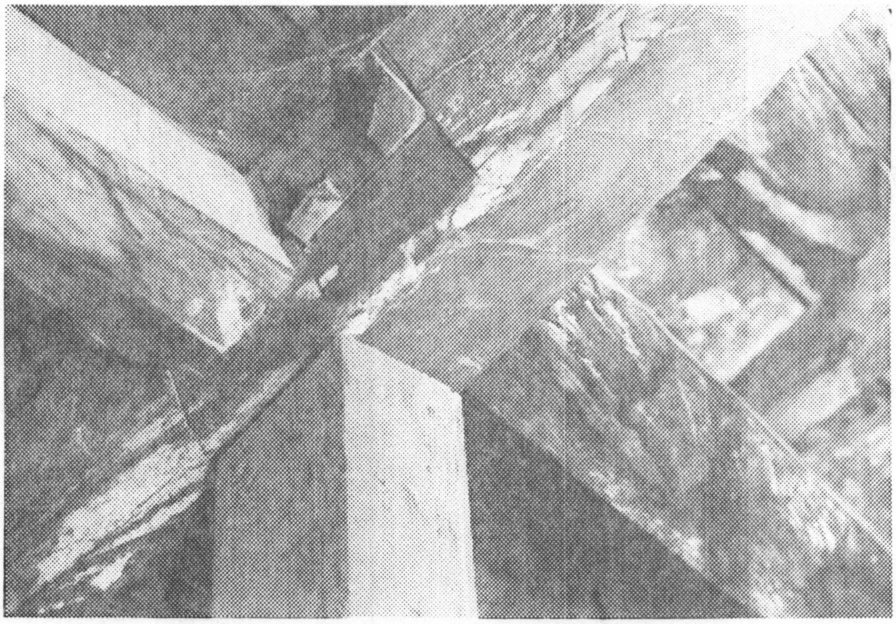


Photo 9. Tenon of purlin on right damaged and peg is missing, reducing ability of roof to resist lateral movement.

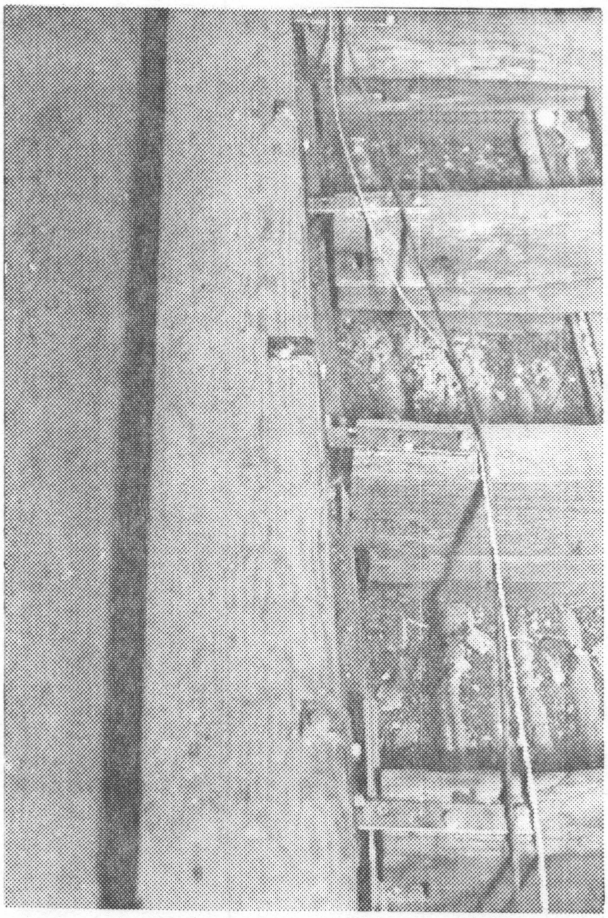


Photo 10. View of ceiling joists fastened to bottom chord by bent bar connectors.

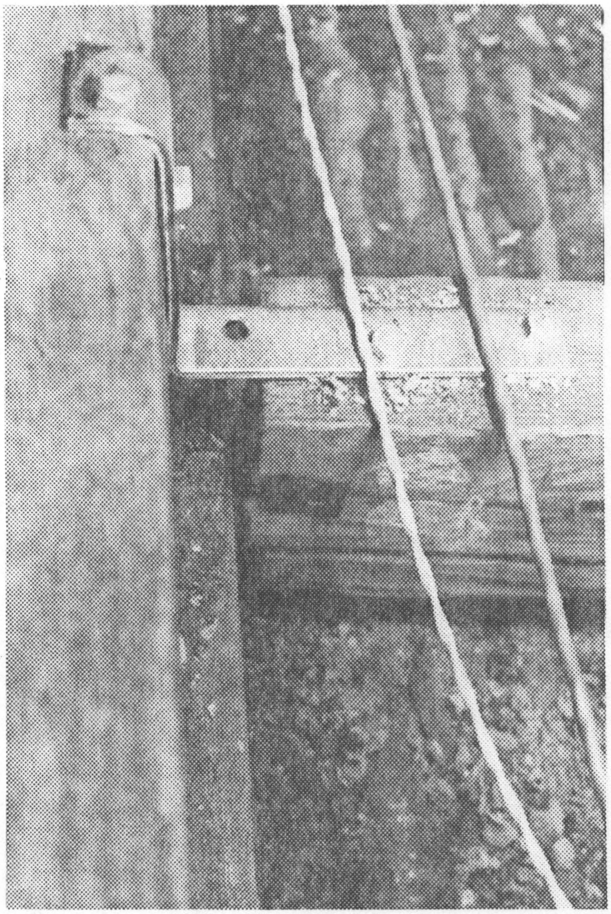


Photo 11. Close-up of ceiling joist showing loss of bearing support/contact w/ledger strip.

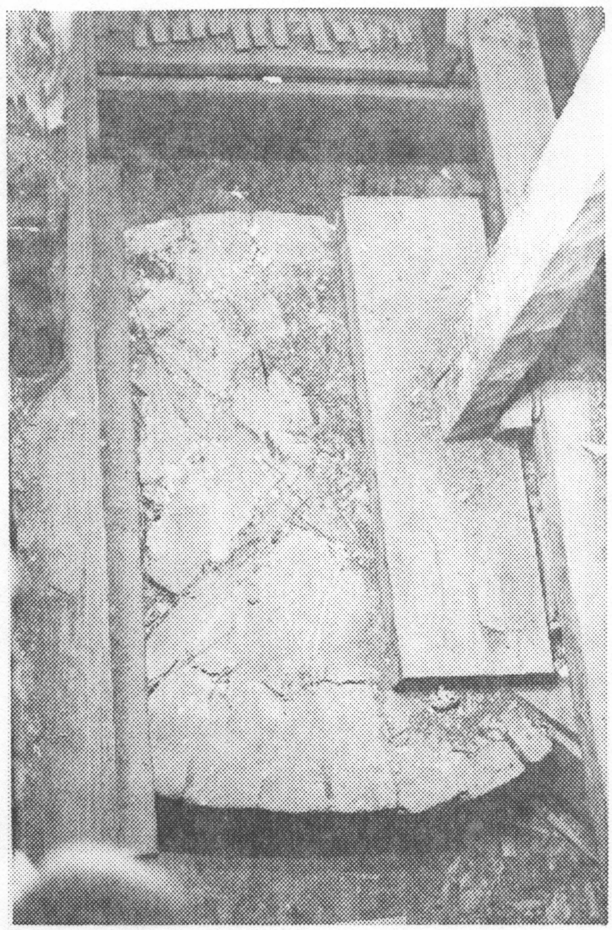


Photo 12. Top of brick column under south side of truss "B".

COST ESTIMATE

COST ESTIMATE FOR NEEDED REPAIRS AND
IMPROVEMENTS IN ATTIC OF CENTRAL BLOCK
OF ARLINGTON HOUSE

Labor costs to transport and erect steel angles, plates, etc. for trusses (No O&P costs included).

TRANSPORTATION/DELIVERY

1 Foreman	1 Day	@ $164.75/day	$164.75
1 Truck Driver	1 Day	@ $163.60/day	$163.60
1 Truck (w/power equipment)	1 Day	@ $109.60/day	$109.60
2 Laborers/Steel Workers	1 Day	@ $295.20/day	$295.20
			$733.15

MOBILIZATION/SETTING UP LABOR COSTS

1 Foreman	5 Days	@ $163.60/day	$818.00
2 Carpenters	5 Days	@ $272.00/day	1,360.00
1 Laborer	5 Days	@ $108.40/day	$542.00
			$2,720.00

ERECTING STEEL TRUSSES LABOR COSTS
(ONE PER WEEK)

1 Foreman	25 Days	@ $163.60/day	$4090.00
2 Steel Workers	25 Days	@ $295.20/day	$7380.00
1 Carpenter	25 Days	@ $136.00/day	$3400.00
1 Laborer	25 Days	@ $108.40/day	$2700.00
			$17,570.00

MATERIAL COSTS FOR FIVE TRUSSES
PROPOSED TRUSS MEMBERS

2 LS	7X4 X 3/4	31.5 ft.	@ 52.4 lbs/ft. X 2 =	3301.2 lbs.
2 LS	6X4 X 1/2	13.25 ft.	@ 32.4 lbs/ft. X 2 =	858.6 lbs.
2 LS	3X3 X 3/8	10.00 ft.	@ 14.4 lbs/ft. X 2 =	288.0 lbs.
		25.5 ft.	@ 14.4 lbs/ft. X 2 =	734.4 lbs.
		12.0 ft.	@ 14.4 lbs/ft. X 1 =	172.9 lbs.

WITH PLATES AND BOLTS, ETC., CALL IT 3 TONS

COST OF MAT'LS (BARE COST ONLY) FOR STEEL TRUSSES
= 3 Tons X $1,255/ton X 5 = $18,825.00

LABOR COSTS TO INSTALL VENTILATING EQUIPMENT IN MASONRY
CHIMNEYS (No O&P Costs Included)

1 Brick Mason	5 Days	@ $140.80/day	$704.00
1 Electrician	3 Days	@ $155.20/day	$465.60
1 Laborer	5 Days	@ $108.40/day	$542.00
			$1,711.60

MATERIAL COSTS FOR VENTILATING EQUIPMENT

	4 Units	@ $250/unit	$1,000.00

LABOR COSTS TO INSTALL ADDITIONAL PURLINS, CLIPS TO SUPPORT CEILING JOISTS, STRAPS, THROUGH BOLTS, PLATES, ETC.

1 Foreman	8 Days	@	$163.60/day	$1,308.80
2 Carpenters	8 Days	@	$272.00/day	$2,176.00
2 Laborers	8 Days	@	$216.80/day	$1,734.40
				$5,219.20

MATERIAL COSTS FOR MISCELLANEOUS REPAIRS TO ROOF FRAMING — $500.00

LABOR AND MATERIAL COST TO INSTALL 6" INSULATION IN ATTIC
2300 SF @ $0.36/SF — $828.00

PLUS COST TO REMOVE AND REINSTALL FLOOR BOARDS BETWEEN TRUSSES "E" & "F" - LABOR COSTS ONLY

2 Carpenters	2 Days	@	$272.00/day	$544.00

LABOR AND MATERIAL COSTS TO REPAIR DECAYED TIMBER ENDS USING EPOXY WHERE PASSING THROUGH CHIMNEYS — $3,000.00

SUB-TOTAL	$52,650.95
Plus 25% For O&P	$13,162.74
TOTAL	$65,813.69

APPENDICES

CALCULATIONS

Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 1
Area ARLINGTON HOUSE			of
Project ENGINEERING STUDY	By MACDONALD	Checked	Pkg. 506
Feature ROOF TRUSS ANALYSIS	Date 11/30/81	Date	Account

LOADS :-

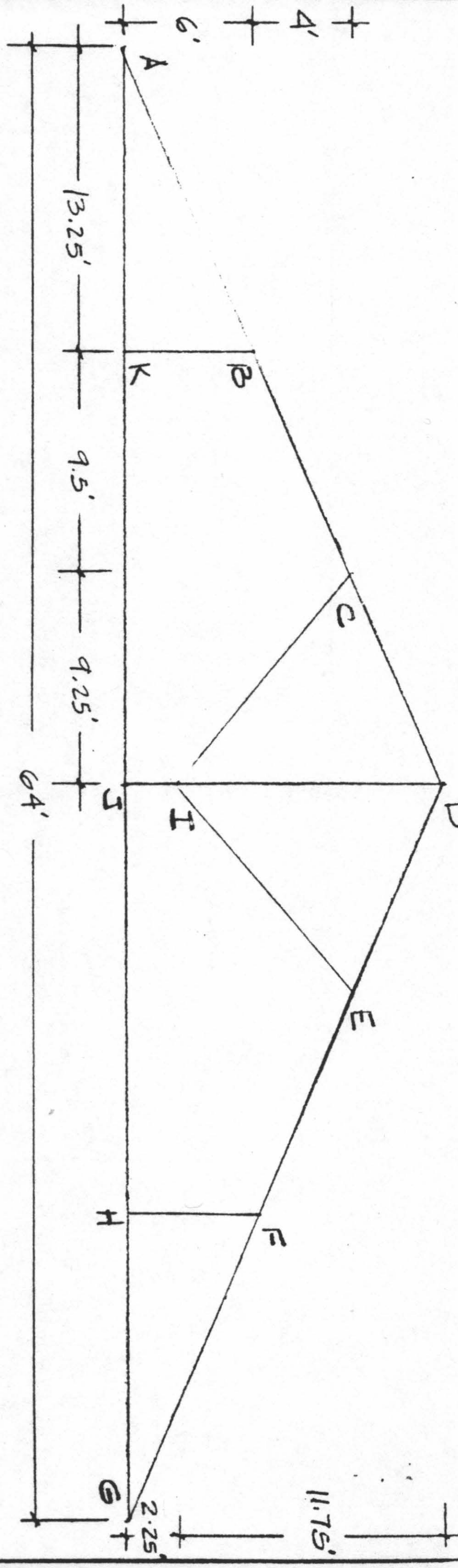
DEAD LOADS :

• SLATE SHINGLES	10 PSF
• ROOF SHEATHING (3/4" THICK)	2.5 "
• 3½" x 3½" RAFTERS @ 2'-0 O.C.	1.7 "
• 5½" x 6" PURLINS	9.1 PLF
• 6½" x 3" CEILING JOISTS @ 16" O.C.	4.1 PSF
• PLASTER CEILING	10 PSF
• 4⅞" x 9" UPPER CHORD TRUSS MEMBER	11.25 PLF
• 6" x 9" BOTTOM CHORD	15.0 "
• 6" x 9" KING POST	15.0 "
• 6" x 7" VERT. WEB MEM.	11.7 "
• 5½" x 5½" DIAGONAL WEB MEMBER	8.4 "

LIVE LOADS :

• SNOW	16 OR 20 PSF
• ATTIC FLOOR LOAD ON WEST SIDE	30 "

• WIND LOAD
FOR ROOF SLOPE = 23.63° AND RATIO OF SIDEWALL HEIGHT TO BLDG. WIDTH = 0.468, WIND PRESSURE
ON WINDWARD SIDE = 12 PSF (SUCTION)
ON LEEWARD " = 10 PSF (")



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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 2
Area ARLINGTON HOUSE			of
Project ENGINEERING STUDY	By MACDONALD	Checked	Pkg. 506
Feature ROOF TRUSS ANALYSIS	Date 11/30/81	Date	Account

ROOF LOADS

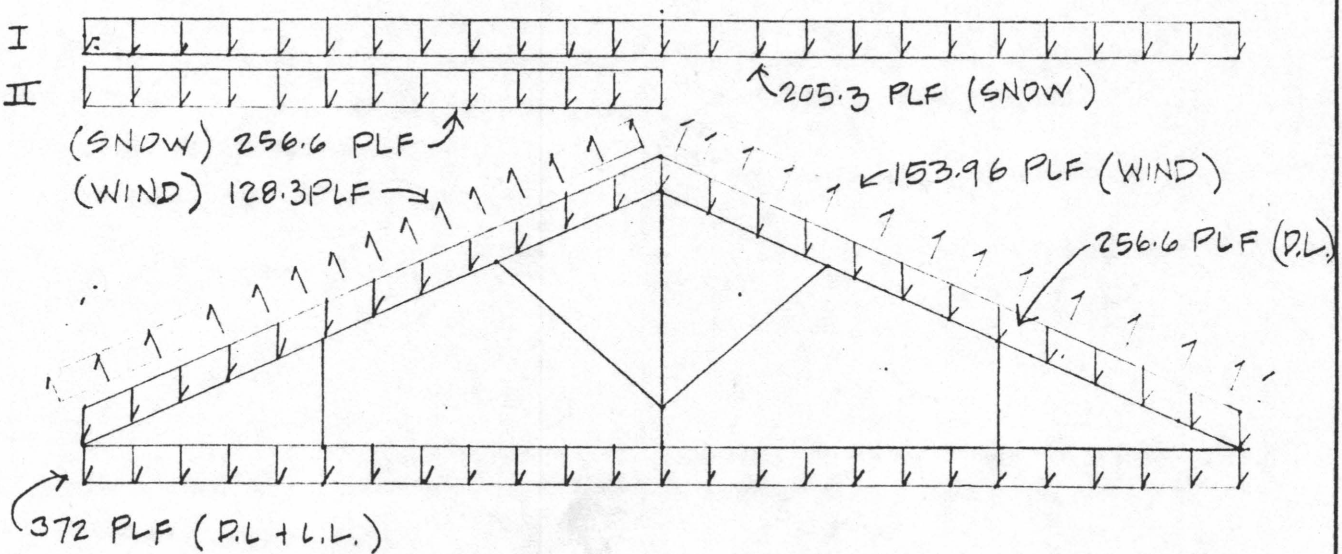
- TRUSS * 3.75 PSF
- SHINGLES 10 "
- SHEATHING 2.5 "
- RAFTERS 1.7 "
- PURLINS 1.8 "

19.75 PSF

- SNOW 16 OR 20 PSF
- WIND
  - WINDWARD SIDE -12 PSF
  - LEEWARD SIDE -10 "

FLOOR LOADS

- CEILING JOISTS 4 PSF
- PLASTER CEILING 10 "
- ATTIC LIVE FLOOR LOAD ON ONE SIDE OF ONE TRUSS plus floor boards 30 "
- BOARD FLOORING 3



LOADING COMBINATIONS TO ANALYZE FOR:

1. DEAD LOAD plus SNOW LOAD (I) plus ATTIC FLOOR LOAD
2. " " " " " (II) " " " " " plus WIND LOAD
3. DEAD LOAD plus SNOW LOAD (I)
4. " " " " " (II) plus WIND LOAD

FORM D S C - 44

Park GWMP	**NATIONAL PARK SERVICE** DENVER SERVICE CENTER	Sheet 3
Area ARLINGTON HOUSE		of
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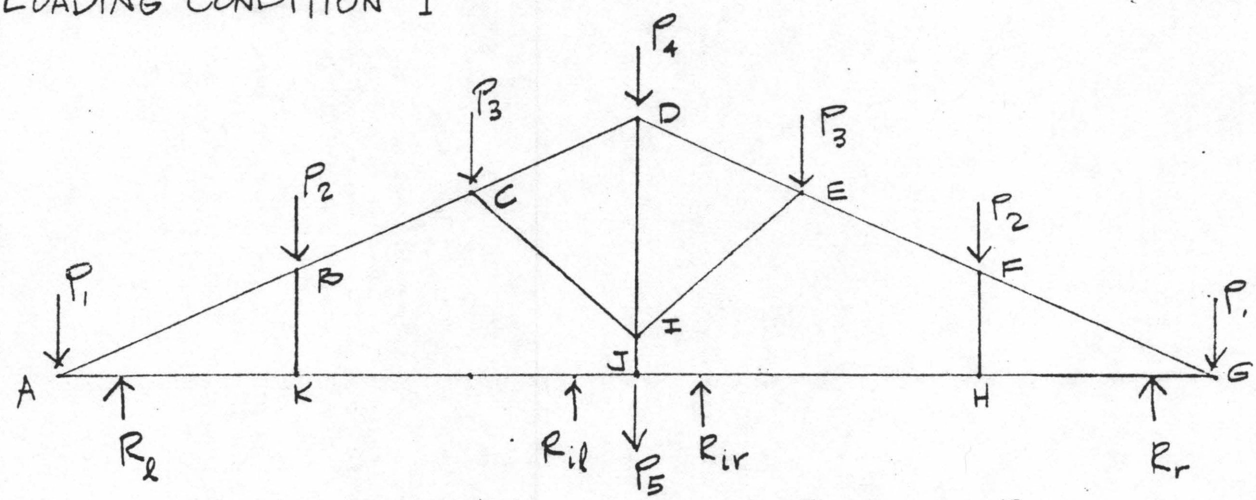
Feature ROOF TRUSS ANALYSIS	Date 11/30/81 / Date	Account

LOADING CONDITION 1



SNOW D.L FLOOR LOAD + D.L

$$P_1 = \frac{(13.25)}{2}(205.3) + \frac{(14.55)}{2}(256.6) + \frac{32}{2}(372) = 9.179^K$$

$$P_2 = (13.25 + 9.5)(205.3)/2 + \frac{24.85}{2}(256.6) = 5.524^K$$

$$P_3 = (9.5 + 9.25)(205.3)/2 + (20.39)(256.6)/2 = 4.541^K$$

$$P_4 = (9.25)(205.3) + 11.03(256.6) = 4.729^K$$

$$P_5 = (64)(372)/2 = 11.904^K$$

NOTE: ALTHOUGH BOTTOM CHORD OF TRUSS IS NOT CONNECTED AT "J", ASSUME $R_L = R_R$ & $R_{i\ell} = R_{ir}$. (ACTUAL LOCATIONS WHERE BOTTOM CHORD MEMBERS ARE CONNECTED VARIES. HOWEVER THEY ARE NEVER OVER A SUPPORT OR WHERE MET BY TRUSS WEB MEMBERS.)

$$\therefore R_L + R_{i\ell} = P_1 + P_2 + P_3 + \tfrac{1}{2}(P_4 + P_5)$$

$$= 27.560^K$$

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 4
Area ARLINGTON HOUSE			of
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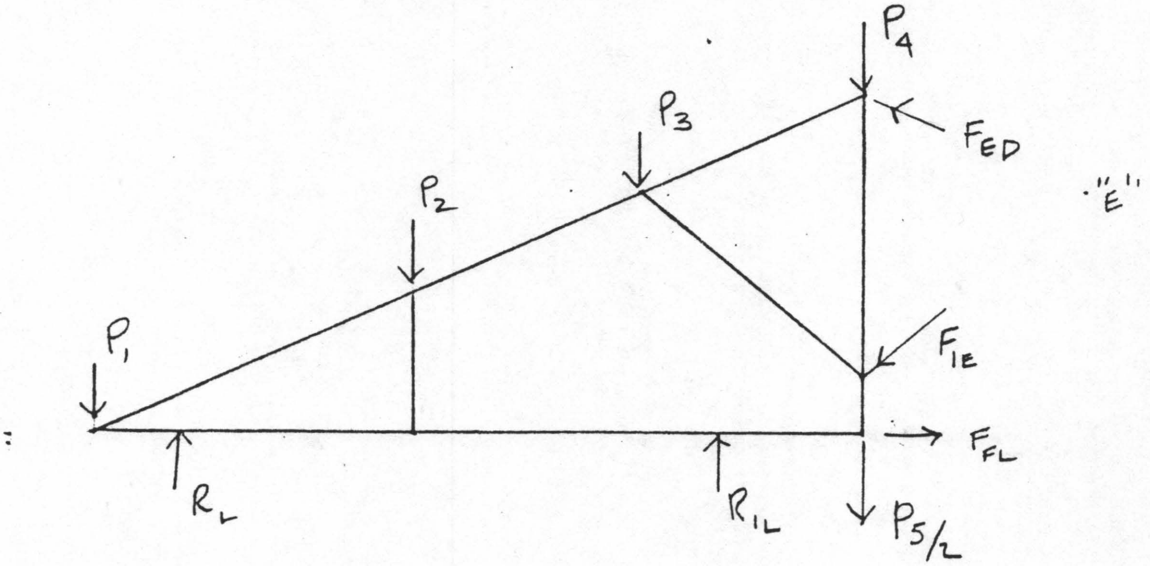
Feature ROOF TRUSS ANALYSIS	Date 11/30/81	Date	Account

TAKING THE MOMENT ABOUT "E" FOR THE LEFT HALF OF THE TRUSS.



$$\Sigma M_E = 9.25\,P_4 + \frac{9.25}{2}\,P_5 + 18.5\,P_3 + 28\,P_2 + 41.25\,P_1$$

$$- 37.75\,R_L - 15.25\,R_{1L} + 10\,F_{FL}$$

$$F_{FL} = \frac{F_{ED}}{1.092} + \frac{F_{IE}}{1.271}$$

ASSUME $F_{FL} = 1.0^K$

$$\Sigma M_E = 726.113 - 37.75\,(27.560 - R_{1L}) - 15.25\,R_{1L} = 0$$

$22.5\,R_{1L} = 314.277$

$R_{1L} = 13.968^K$

$R_L = 13.592^K$

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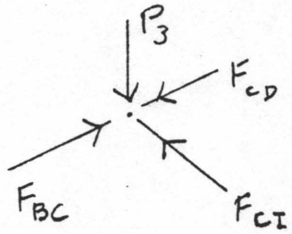
Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 5
Area ARLINGTON HOUSE			of
Project ENGINEERING STUDY	By MACDONALD	Checked	Pkg. 506
Feature ROOF TRUSS ANALYSIS	Date 11/30/81	Date	Account

JOINT "C"



$$\sum F_H = \frac{F_{CI}}{1.271} - \frac{F_{BC}}{1.092} + \frac{F_{CD}}{1.092} = 0$$

$$\sum F_V = 0.6168\,F_{CI} + 0.401\,F_{BC} - P_3 - 0.401\,F_{CD} = 0$$

$$F_{AB} = F_{BC}\ ;\ F_{CD} = F_{DE}\ ;\ F_{CI} = F_{EI}$$

$$F_{BC} = F_{CD} + \frac{1.092}{1.271}\,F_{CI}$$

$$\sum F_V = 0.6168\,F_{CI} + \cancel{0.401\,F_{CD}} + \frac{(0.401)(1.092)}{1.271}\,F_{CI}$$

$$- P_3 - \cancel{0.401\,F_{CD}} = 0$$

$$0.9613\,F_{CI} = P_3$$

$$F_{CI} = 4.724^{K}$$

$$F_{FL} = 3.717 + \frac{F_{ED}}{1.092}$$

$$= 3.717 + 0.916\,F_{ED}$$

$$\therefore \text{ IF } F_{FL} = 1.0^{K}$$

$$F_{ED} = -2.966 \quad = \quad F_{ED} = 2.966^{K}(T)$$

$$\sum M_A = 3.5\,R_L + 26\,R_{IL} - \frac{32}{2}\,P_5 - 32\,P_4 - 22.75\,P_3 - 13.25\,P_2$$

$$- 32(0.401)(F_{DE}) - \frac{14}{1.092}(F_{DE}) - 32(0.6168)(4.724)$$

$$+ \frac{2.25}{1.271}(4.724) = 410.74 - 190.464 - 151.328 - 103.308$$

$$- 73.193 - 32(0.401)F_{DE} - \frac{14}{1.092}(F_{DE}) - 93.24 + 9.734 =$$

$$- 267.144$$

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 6
Area ARLINGTON HOUSE			of
Project ENGINEERING STUDY	By MACDONALD	Checked	Pkg.
Feature ROOF TRUSS ANALYSIS	Date 11/30/81	Date	Account

ASSUMPTION THAT $F_{FL} = 1.0^K$ NO GOOD

TO ADJUST, ...

ASSUME $F_{FL} = 6.0^K$

$\therefore F_{ED} = 2.492^K$ (C)

$\Sigma M_E = 716.113 + 60 - 1043.788 + 22.5 R_{IL}$

$R_{IL} = 11.897^K$

$R_L = 15.753^K$

CHECK:

$\Sigma M_A = 364.458 - 190.464 - 151.328 - 103.308$

$-73.193 + 31.977 + 31.949 - 93.24 + 9.734$

$= -173.415$ ft-K NO GOOD

ASSUME $F_{FL} = 9.0^K$

$F_{ED} = 5.767^K$ (C)

$\Sigma M_E = 716.113 + 90 - 1043.788 + 22.5 R_{IL}$

$R_{IL} = 10.563^K$

$R_L = 16.997^K$

CHECK:

$\Sigma M_A = 334.126 - 518.292 + 74.002 + 73.936 - 93.24 + 9.734$

$= -119.733$ ft-K NO GOOD

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 7
Area ARLINGTON HOUSE			of
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Feature ROOF TRUSS ANAL.	Date 11/30/81	Date	Account

ASSUME $F_{FL} = 16.0^K$

$F_{ED} = 13.409^K$ (C)

$\Sigma M_E = 716.113 + 160 - 1043.788 + 22.5\, R_{IL}$

$R_{IL} = 167.675/22.5 = 7.452^K$

$R_L = 20.108^K$

CHECK:

$\Sigma M_A = 264.13 - 518.292 + 172.064 + 171.910 - 93.24$
$+ 9.734$

$= + 6.306$ ft-K

CLOSE

ASSUME $F_{FL} = 15.648^K$

$F_{ED} = 13.025^K$

$\Sigma M_E = 716.113 + 156.48 - 1043.788 + 22.5\, R_{IL}$

$R_{IL} = 7.609^K$

$R_L = 19.951^K$

CHECK:

$\Sigma M_A = 267.664 - 518.292 + 167.137 + 166.987$
$- 93.24 + 9.734$

$= - 0.010$ Ft-K

OK

Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 7A
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$$F_{DI} = 2(0.401)F_{ED} - P_1 = 5.717^K \ (T)$$

$$F_{IJ} = 2(0.6168)F_{CD} - F_{DI} = 0.111^K \ (C)$$

$$F_{BC} = F_{AB} = F_{CD} + \frac{1.092}{1.271} F_{CI} = 17.084^K \ (C)$$

$$F_{AJ} = 15.636^K$$

CHECK :

ΣM_A for bottom chord AJ = $3.5(19.951) + 26(7.609)$

$$- 13.25(5.524) - \frac{32}{2}(11.904) - \frac{32}{2}(1.111)$$

$$= 2.230 \text{ ft-K} \quad \text{OK}$$

$$\Sigma M_J = 6(7.609) + 28.5(19.951) - 18.75(5.524)$$

$$- \frac{32}{2}(11.904) - 32(0.401)(17.084) - 32(3.227)$$

$$= -2.267 \text{ ft-K} \quad \text{OK}$$

Park	GWMP
Area	ARLINGTON HOUSE
Project	ENGINEERING STUDY
Feature	ROOF TRUSS ANALYSIS

NATIONAL PARK SERVICE
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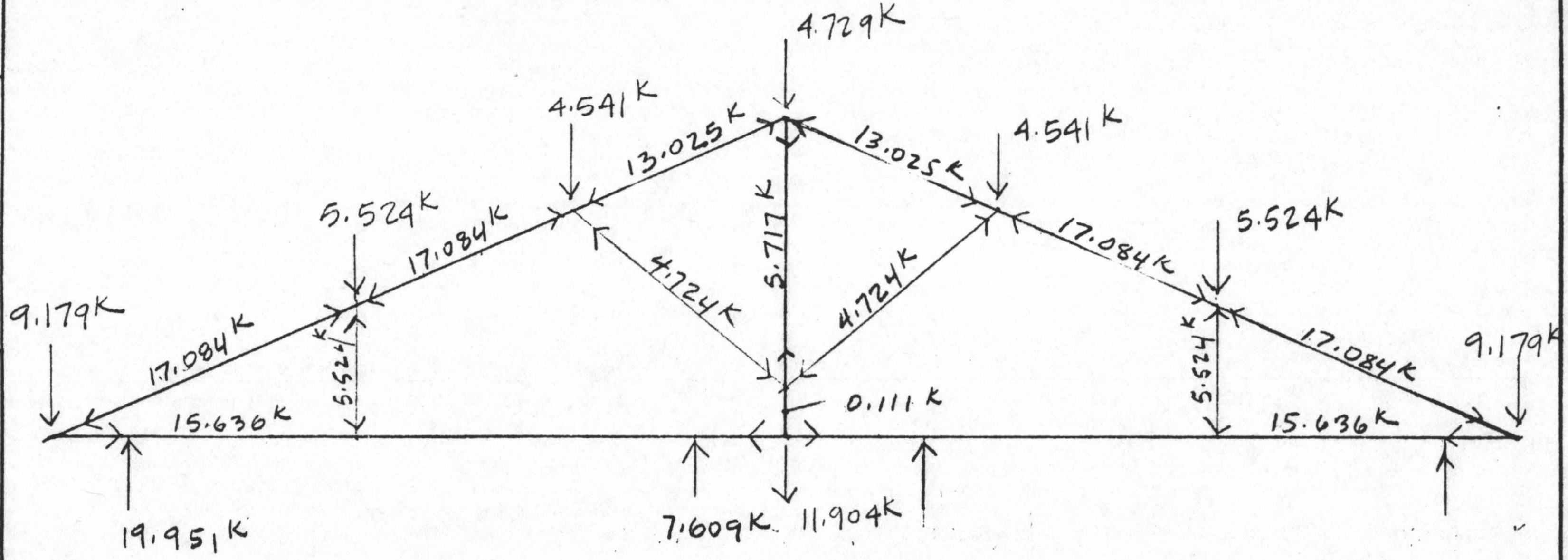

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 9
Area ARLINGTON HOUSE			of
Project ENGINEERING STUDY	By MACDONALD	Checked	Pkg. 506
Feature ROOF TRUSS ANAL.	Date 11/23/81	Date	Account

MEMBER STRESS ANALYSIS :-

ALLOWABLE STRESSES - EASTERN WHITE PINE

F_b = 1050 psi		F_b = 1310 psi
F_t = 700 "		F_t = 875 "
F_v = 65 "	INCREASE BY 1.25 FOR SNOW LOADS	F_v = 80 "
$F_{c\perp}$ = 220 "		$F_{c\perp}$ = 275 "
F_c = 675 "		F_c = 840 "
E = 1100 KSI		E = 1375 Ksi

MEMBER PROPERTIES :-

b × d		A (in²)	S(in³)	I(in⁴)
5½" × 6"	PURLINS	33.0	33.00	99.00
3 × 6½	CEILING JOISTS	19.5	21.12	65.66
3½ × 3½	RAFTERS	12.25	7.14	12.51
4½ × 9	TOP CHORD	40.5	60.75	273.38
6 × 9	BOTTOM CHORD	54.0	81.00	364.50
6 × 9	KING POST	54.0	81.00	364.50
6 × 7	VERT. WEB MEMB	42.0	49.00	171.50
5½ × 5½	DIAG. " "	30.25	27.73	76.26

FOR MEMBERS UNDER COMBINED LOADING :-

1. FLEXURE & AXIAL TENSION

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} \leq 1$$

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2. FLEXURE & AXIAL COMPRESSION :

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b} \leq 1 \quad (\text{SHORT COLUMN}, \ l/d \leq 11)$$

OR $$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} \quad \text{where } J = \frac{(l/d) - 11}{K - 11} \ \& \ K = 0.671\sqrt{\frac{E}{F_c}}$$

for $l/d > 11$

WEB MEMBERS - AXIAL LOADS ONLY

- MEMBERS BK & FH - 6"x7" ; L = 6 ; $P = 5.524^K$

$$f_c = \frac{5524}{(6)(7)} = 131.5 \text{ psi} < 833 \text{ psi}$$

OK

$$l/d = \frac{6 \times 12}{6} = 12$$

$$K = 0.671\sqrt{\frac{E}{f_c}}$$

$$= 27.1 > 12$$

$$F_c' = F_c\left[1 - \tfrac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$$

$$= 840(0.9877)$$

$$= 833 \text{ psi}$$

- MEMBERS CI & EI ; $L = \sqrt{(9.23)^2 + (7.75)^2} = 12.07'$

$P = 4.724^K$

$$f_c = \frac{4724}{(5.5)(5.5)} = 156.17 \text{ psi} < 590.5 \text{ psi}$$

$$l/d = \frac{12.07 \times 12}{5.5} = 26.33$$

$$K = 27.1 > 26.33$$

$$F_c' = F_c\left[1 - \tfrac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$$

$$= 590.5 \text{ psi}$$

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• MEMBER DI ; $L = 11.75'$; $6'' \times 9''$; $P = 5.717^K$

$$f_t = \frac{5717}{(6)(9)} = 105.87 \text{ psi}$$

875 $\gg$ 105.87 psi OK

$$l/d = \frac{11.75 \times 12}{6} = 23.5$$

$K = 27.1 > 23.5$

$F_t = 875$ psi

• MEMBER IJ ; $L = 2.25'$; $6'' \times 15''$; $P = 0.111^K$

$$f_c = \frac{111}{6 \times 15} = 1.23 \ll 840 \text{ psi}$$

OK

$$l/d = \frac{(2.25)(12)}{6} = 4.5$$

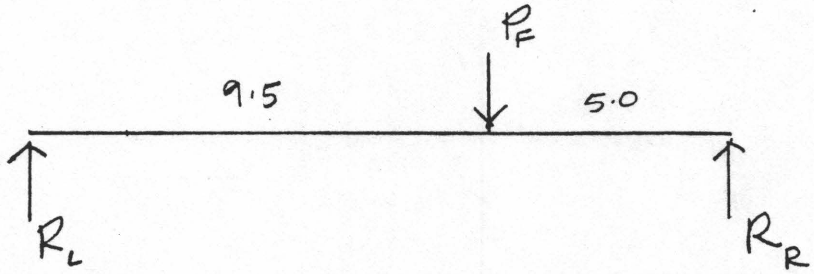
$l/d < 11$

$F'_c = F_c = 840$ psi

MEMBERS SUBJECTED TO AXIAL LOADS & FLEXURE

• TOP CHORD MEMBERS AB & FG ; $4\frac{1}{2}'' \times 9''$; $L = 14.5'$

(LATERAL SUPPORT PROVIDED BY PURLINS) $P_A = 17.084^K$



$$l/d = \frac{(14.5)(12)}{9} = 19.33$$

$$F'_c = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$$

$= 616.76$ psi

$$J = \frac{l/d - 11}{K - 11} = 0.52$$

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$$P = \left[(14.5)(12.83)(20) + (13.25)(12.83)(16)\right] \tfrac{1}{2} = 3220.33\ \#$$

$$\Sigma M_\ell = 9.5\,P - 14.5\,R_r = 0$$

$$R_r = 2109.9 \text{ lbs.}$$

$$R_\ell = 1110.4 \text{ lbs.}$$

$$M_{MAX.} = (5)(2109.9) = 10{,}549 \text{ Ft-lbs}$$

$$f_b = \frac{10{,}549\ (12)}{60.75} = 2083.75 \text{ psi}$$

$$F_b = 1310 \text{ psi}$$

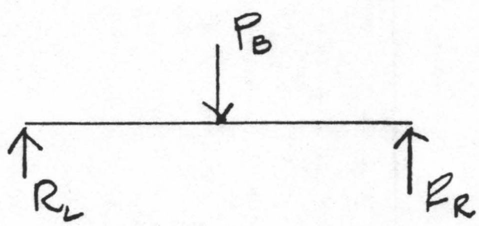
$$J = \frac{\ell/d - 11}{K - 11}$$

$$f_c = \frac{17084}{(4.5)(9)} = 421.83 \text{ psi}$$

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} = \frac{421.83}{616.76} + \frac{2083.75}{1092} = 2.59$$

NO GOOD

- TOP CHORD MEMBERS BC & EF; 4½" x 9"; L=10'
  LATERAL SUPPORT PROVIDED BY PURLINS; P = 17.084K



$$P_B = \left[(110)(12.83)(20) + (9.25)(12.83)(16)\right] \tfrac{1}{2}$$
$$= 2232.42 \text{ lbs}$$

$$\ell/d = \frac{(10)(12)}{9} = 13.33$$

$$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{\ell/d}{K}\right)^4\right]$$
$$= 661.83 \text{ psi}$$

$$J = \frac{\ell/d - 11}{K - 11} = 0.145$$

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$R_L = R_R = \frac{1}{2} P_B = 1116.21$ lbs

$M_{MAX} = 5(1116.21) = 5581.05$ Ft-lbs

$f_b = \dfrac{5581.05\ (12)}{60.75} = 1102.43$ psi

$f_c = \dfrac{17084}{(4.5)(9)} = 421.83$ psi

NO GOOD

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} = \frac{421.83}{661.83} + \frac{1102.43}{1310 - 61.17} = 1.52 > 1$$

- TOP CHORD MEMBERS CD & DE ; 4½" x 9" ; L = 10.08' LATERAL SUPPORT PROVIDED BY PURLINS ; P = 13.025 K

$P = 2232.42$ lbs $\qquad l/d = 13.33$

$R_L = R_R = 1116.21$ lbs. $\qquad F_c' = 661.83$ psi

$M_{MAX} = 5(1116.21) = 5,581.05$ ft-lbs. $\qquad J = 0.145$

$f_b = 1102.43$ lbs

$f_c = \dfrac{13025}{(4.5)(9)} = 321.605$ psi

NO GOOD

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} = \frac{321.6}{661} + \frac{1102.43}{1310 - 61.17} = 1.37 > 1$$

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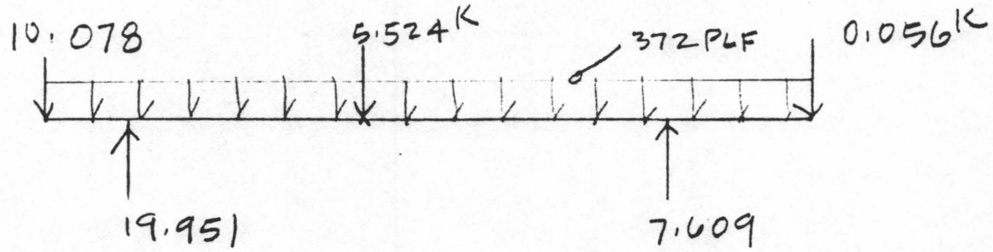
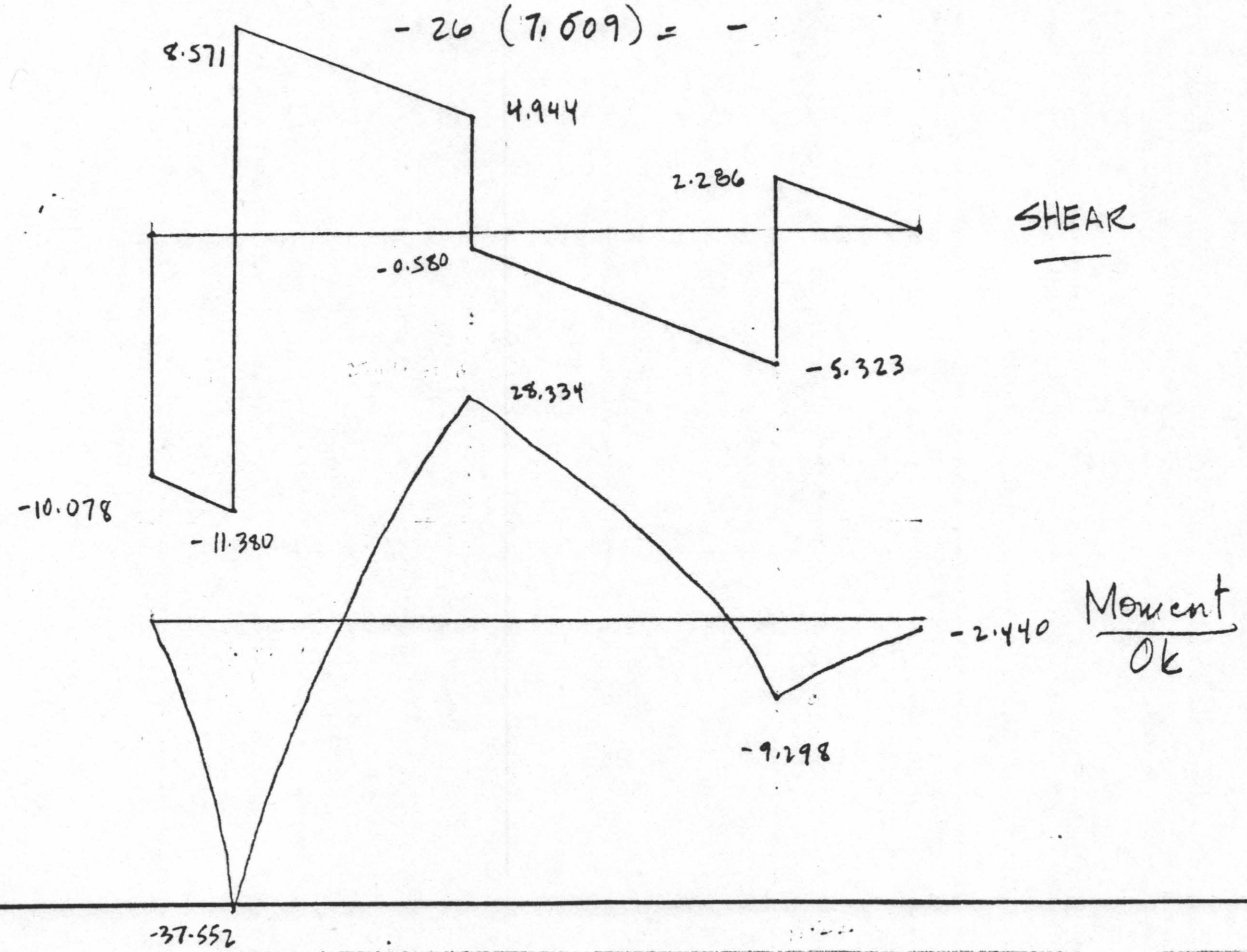
Feature ROOF TRUSS ANAL.	Date 11/30/82	Date	Account

• BOTTOM CHORD MEMBER AJ & JG; $P_t = 15.636^K$; 6" x 9"



$$\Sigma F_v = 10.078 + 5.524 + 0.056 + 0.372(32) - 19.951 - 7.609$$
$$= 0.002^K$$

$$\Sigma M_A = -3.5(19.951) + 13.25(5.524) + \frac{(0.372)}{2}(32)^2 + (0.056)(32)$$
$$-26(7.609) = -$$



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Feature ROOF TRUSS ANALYSIS	Date 11/30/81 / Date	Account

$F_b = 1310$ psi

$M_{MAX} = 37.552$ ft-K

$f_b = \frac{37552}{81}(12) = 5563.26$ psi

$f_t = \frac{15636}{54} = 289.6$ psi

$\frac{f_t}{F_t} + \frac{f_b}{F_b} = \frac{289.56}{875} + \frac{5563.3}{1310} = 4.58 > 1$

NO GOOD

MEMBER IS 358 % UNDERSIZED

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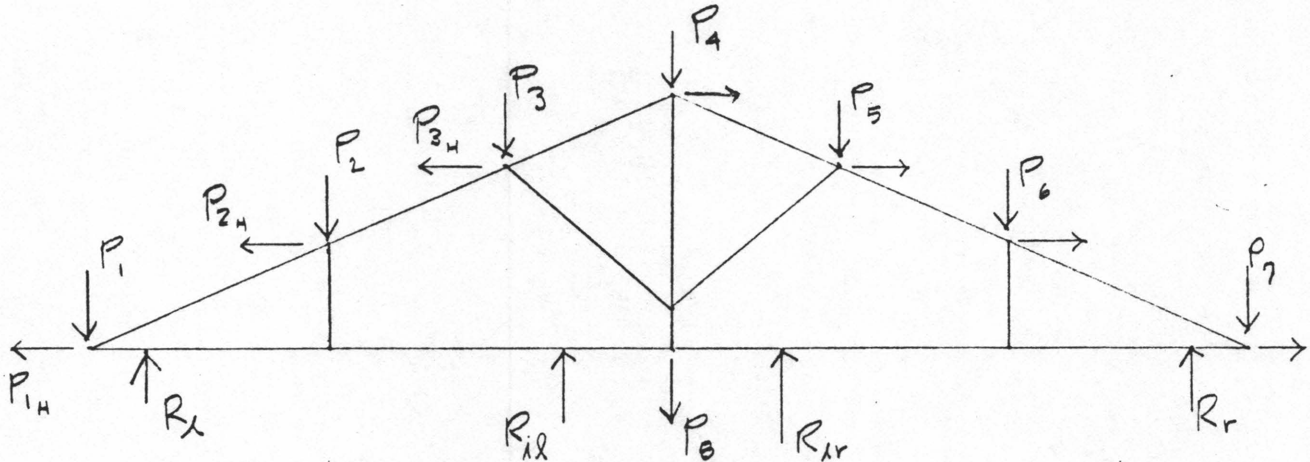
LOADING CONDITION 2 :-



D.L. (ROOF) SNOW FLOOR (D.L.&L.L) WIND

$$P_1 = \frac{14.55}{2}(256.6) + \frac{(13.25)}{2}(256.6) + \frac{(32)(372.0)}{2} - \frac{(14.55)(128.3)(1.0)}{(2)(1.092)}$$

$$= 8.664^K$$

$$P_{1_H} = \frac{(14.55)(128.3)(0.401)}{(2)(1.092)} = 0.343^K$$

$$P_2 = (24.85)(256.6)/2 + (13.25 + 9.5)(256.6)/2 - \frac{(24.85)(128.3)(1.0)}{(2)(1.092)}$$

$$= 4.647^K$$

$$P_{2_H} = \frac{(24.85)(128.3)(0.401)}{(2)(1.092)} = 0.585\,K$$

$$P_3 = (20.39)(256.6)/2 + (9.5 + 9.25)(256.6)/2 - \frac{(20.39)(128.3)(1.0)}{(2)(1.092)}$$

$$= 3.824\,K$$

$$P_{3_H} = (20.39)(128.3)(0.401)/(2)(1.092) = 0.480\,K$$

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$$P_4 = (11.03)(256.6) + \left(\frac{9.25}{2}\right)(256.6) - \frac{(11.03)(128.3 + 153.96)(1.0)}{(2)(1.092)}$$

$$= 2.592^K$$

$$P_{4_H} = \frac{(11.03)(128.3)(0.401)}{(2)(1.092)} - \frac{(11.03)(153.96)(0.401)}{(2)(1.092)} = 0.052^K$$

$\overset{+}{\longleftarrow}$

$$P_5 = (20.39)(256.6)/2 - \frac{(20.39)(153.96)(1.0)}{(2)(1.092)} = 1.179^K$$

$$P_{5_H} = (20.39)(153.96)(0.401)/(2)(1.092) = 0.576^K$$

$$P_6 = (24.85)(256.6)/2 - \frac{(24.85)(153.96)(1.0)}{(2)(1.092)} = 1.436^K$$

$$P_{6_H} = (24.85)(153.96)(0.401)/2(1.092) = 0.702^K$$

$$P_7 = (14.55)(256.6)/2 + (32)(372.0)/2 - \frac{(14.55)(153.96)(1.0)}{(2)(1.092)}$$

$$= 6.793^K$$

$$P_{7_H} = \frac{(14.55)(153.96)(0.401)}{(2)(1.092)} = 0.411^K$$

$$P_8 = (64)(372)/2 = 11.904^K$$

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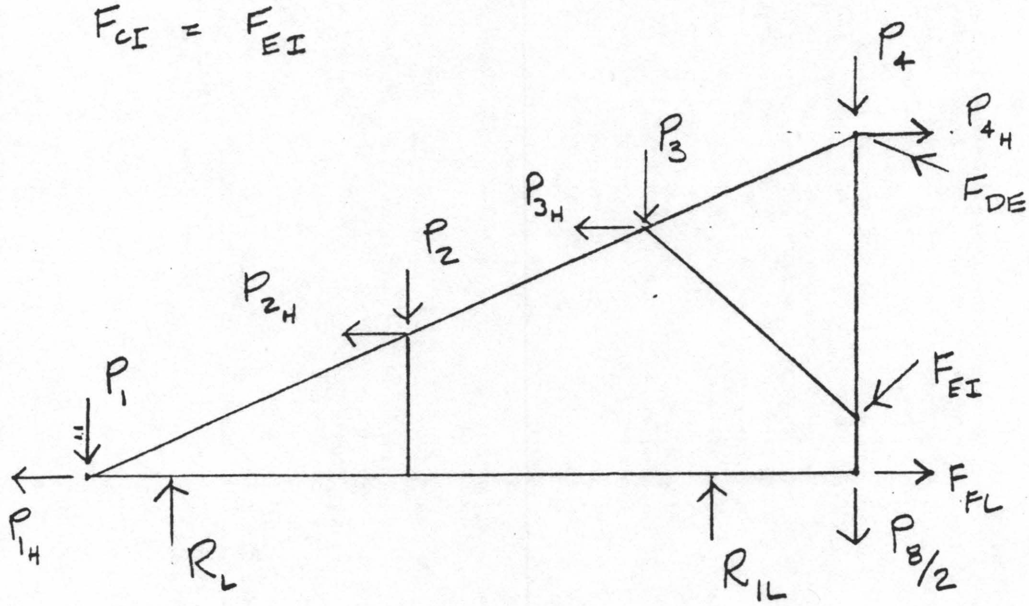
$$F_{CD_H} + P_{4_H} = F_{DE_H} \Rightarrow \frac{F_{CD}}{1.092} + 0.052 = F_{DE}/1.092$$

$$\therefore F_{CD} = F_{DE} - 0.057$$

$$F_{CI} = F_{EI}$$



ASSUME: $(R_L + R_{IL})$ support $1/2\ F_{IJ}$ AND F_{FL} includes all friction forces AND AXIAL REACTIONS on bottom chord. $\left(F_{FL} = \frac{F_{ED}}{1.092} + \frac{F_{IE}}{1.271} + P_{3_H} + P_{2_H} + P_{1_H} - P_{4_H}\right)$

$$\Sigma M_E = 1/2\,(9.25)P_8 + (9.25)P_4 + 18.5\,P_3 + 28\,P_2$$

$$+ 41.25\,P_1 - 15.25\,R_{IL} - 37.75\,R_L - 4\,P_{4_H}$$

$$- 4\,P_{2_H} - 10\,P_{1_H} + 10\,F_{FL}$$

$$= 55.056 + 23.976 + 70.744 + 130.116 + 357.39$$

$$- 15.25\,R_{IL} - 37.75\,R_L - 0.208 - 2.340 - 3.430$$

$$+ 10\,F_{FL}$$

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USING THE METHOD OF JOINTS, THE FOLLOWING RELATIONSHIPS WERE DERIVED :-

$$F_{BC} = F_{AB} - 1.092\,(P_{2H})$$

$$F_{CD} = F_{BC} - 0.70065\,P_{3_H} - 0.8937\,P_3$$

$$F_{DE} = F_{CD} + 1.092\,P_{4H} = F_{EF} - 0.70065\,P_{5_H} - 0.8937\,P_5$$

$$F_{EF} = F_{DE} + 0.70065\,P_{5_H} + 0.8937\,P_5 = F_{FG} - 1.092\,P_{6H}$$

$$F_{FG} = F_{EF} + 1.092\,P_{6H}$$

$$F_{FG} = 1.092\,P_{6H} + 0.70065\,P_{5_H} + 0.8937\,P_5 + 1.092\,P_{4H} - 0.70065\,P_{3_H} - 0.8937\,P_3 - 1.092\,P_{2H} + F_{AB}$$

$$= F_{AB} - 2.112^K$$

$$F_{GH} = \frac{1}{1.092} F_{FG} + P_{7_H} = \frac{1}{1.092}\left(F_{AB}\right) - 1.523^K$$

$$F_{AJ} = \frac{1}{1.092} F_{AB} + P_{1_H} = \frac{1}{1.092} F_{AB} + 0.343^K$$

$$\therefore\ F_{AJ} - F_{GH} = 1.866^K$$

ASSUME $F_{FL} = 5.000^K = \frac{F_{ED}}{1.092} + \frac{F_{IE}}{1.271} + P_{3_H} + P_{2_H} + P_{1_H} - P_{4_H}$

$$\therefore\ \Sigma M_E = 681.304 - 15.25\,R_{1L} - 37.75\,R_L$$

$$\Sigma F_V = P_1 + P_2 + P_3 + \tfrac{1}{2}P_4 + \tfrac{1}{2}P_8 - R_L - R_{1L} = 0$$

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$\Sigma F_V = 24.383 - R_L - R_{1L}$

$R_L = 24.383 - R_{1L}$

$\therefore \Sigma M_E = 681.304 - 15.25 R_{1L} - 37.75(24.383 - R_{1L})$

$22.5 R_{1L} = 239.154$

$R_{1L} = 10.629^K$

$R_L = 13.754^K$

~~For Right Side of Truss ...~~

$$\Sigma M_C = \tfrac{1}{2}\overset{55.056}{(9.25) P_8} + \overset{23.976}{(9.25) P_4} + \overset{21.812}{18.5 P_5} + \overset{40.208}{28 P_6}$$

$$+ \overset{280.211}{41.25 P_7} - 15.25 R_{1R} - 37.75 R_R + \overset{0.208}{4 P_{4H}}$$

$$+ \overset{2.808}{4 P_{6H}} - \overset{-4.11}{10 P_{7H}} + 10 F_{FR} = 0$$

Assume $F_{FR} = 5.000 - 1.866 = 3.134^K$

$= \frac{F_{CD}}{1.092} + \frac{F_{CI}}{1.271} + P_{5H} + P_{6H} + P_{7H} + P_{4H}$

$\therefore \Sigma M_C = 451.508 - 15.25 R_{1R} - 37.75 R_R$

$\Sigma F_V = \tfrac{1}{2} P_4 + \tfrac{1}{2} P_8 + P_5 + P_6 + P_7 - R_{1R} - R_R = 0$

$= 16.656 - R_{1R} - R_R$

$\Sigma M_C = 451.508 - 15.25 R_{1R} - 37.75 (16.656 - R_{1R})$

$22.5 R_{1R} = 177.256$

$R_{1R} = 7.878^K$

$R_R = 8.778^K$

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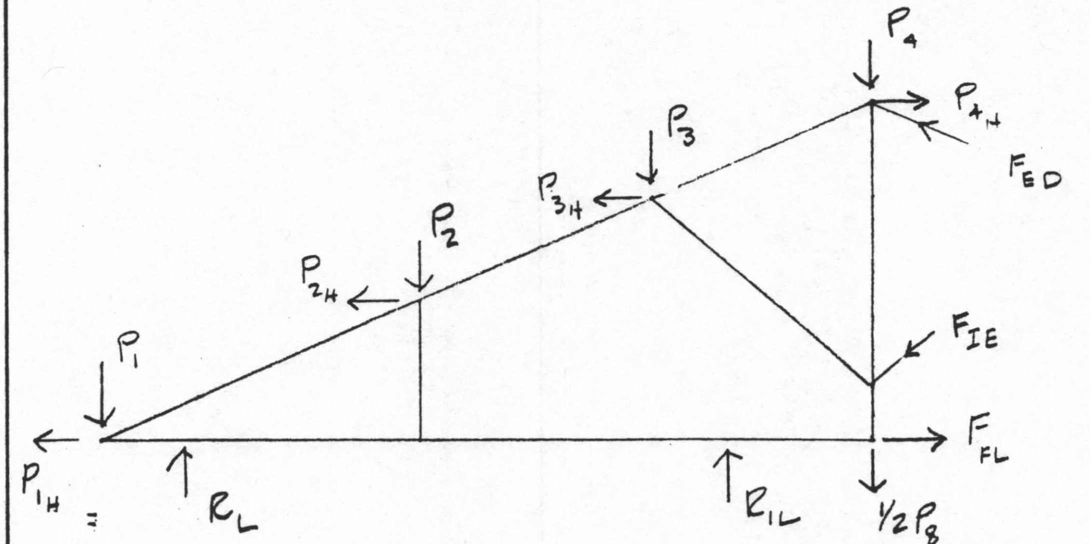
TO CHECK ASSUMPTIONS, TAKE MOMENT ABOUT "A" FOR PORTION OF TRUSS SHOWN BELOW.



$$\circlearrowleft^{+} \Sigma M_A = 3.5\ R_L + 26\ R_{1L} - \tfrac{1}{2}(32)\ P_8 - \underline{32}\ P_4 - 22.75\ P_3$$

$$-13.25\ P_2 + \frac{14\ (F_{ED})}{1.092} + 32(0.401)\ F_{ED} - 32(0.6168) F_{IE}$$

$$+ \frac{2.25}{1.271} F_{IE} + 6\ P_{2H} + 10\ P_{3_H} - 14\ P_{4_H}$$

JOINT "E"

$$\Sigma F_V = P_5 - 0.401\ F_{EF} + 0.401\ F_{DE} - 0.6168\ F_{IE}$$

$$\therefore F_{EF} = \frac{P_5}{0.401} + F_{DE} - \frac{0.6168}{0.401} F_{1E}$$

$$\Sigma F_H = \frac{F_{EF}}{1.092} - \frac{F_{DE}}{1.092} - \frac{F_{1E}}{1.271} - P_{5_H}$$

$$= \frac{1}{1.092}\left(\frac{P_5}{0.401} + F_{DE} - 1.5382\ F_{1E}\right) - \frac{F_{DE}}{1.092} - \frac{F_{1E}}{1.27}$$

$$- P_{5_H}$$

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 22
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$$\Sigma F_H = 2.2837\,P_5 - 1.4086\,F_{1E} - 0.7868\,F_{1E} - P_{5_H}$$

$$F_{1E} = 0.964^K$$

$$\therefore \Sigma M_A = 324.493 - 190.464 - 82.944 - 86.996 - 61.573$$
$$+ 25.6525\,F_{ED} - 19.027 + 1.707 + 3.51 + 4.8 - 0.728$$

$$25.6525\,F_{ED} = -107.222$$

$$F_{DE} = +4.180^K$$

$$F_{DE} = 1.092\,P_{4_H} - 0.70065\,P_{3_H} - 0.8937\,P_3 - 1.092\,P_{2_H} + F_{AB}$$

$$\therefore F_{FL} = \frac{F_{DE}}{1.092} + \frac{F_{1E}}{1.271} + P_{3_H} + P_{2_H} + P_{1H} - P_{4_H} = 5.942^K$$

ASSUMED VALUE OF $F_{AJ} = 5.000^K$

NO GOOD

ASSUME $F_{FL} = 7.000^K$

$$\therefore \Sigma M_E = 701.304 - 15.25\,R_{1L} - 37.75\,R_L$$

$$22.5\,R_{1L} = 219.154$$

$$R_{1L} = 9.740^K$$

$$R_L = 14.643^K$$

Check :

$$\Sigma M_A = 304.490 - 190.464 - 82.944 - 86.996 - 61.573$$
$$+ 25.6525\,F_{ED} - 19.027 + 1.707 + 3.51 + 4.8 - 0.728$$

$$F_{ED} = 4.960^K$$

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$F_{FL} = 6.708^K$ NO GOOD

ASSUME $F_{FL} = 6.5$

$\therefore \Sigma M_E = 696.304 - 15.25 R_{IL} - 37.75 R_L$

$22.5 R_{IL} = 224.154$

$R_{IL} = 9.962^K$

$R_L = 14.421^K$

CHECK:

$\Sigma M_A = 309.484 - 190.464 - 82.944 - 86.996 - 61.573 + 25.6525 F_{ED} - 19.027 + 1.707 + 3.51 + 4.8 - 0.728$

$=$

$F_{ED} = 4.765^K$

$F_{FL} = 6.477^K$ ASSUMED VALUE $= 6.50^K$

$\therefore$ ASSUME $F_{FL} = 6.450^K$

$\Sigma M_E = 695.804 - 15.25 R_{IL} - 37.75 R_L$

$R_{IL} = 9.985^K$

$R_L = 14.398^K$

CHECK:

$\Sigma M_A = 310.004 - 190.464 - 82.944 - 86.996 - 61.573 + 25.6525 F_{ED} - 19.027 + 1.707 + 3.51 + 4.8) - 0.728$

$F_{ED} = 4.744^K$

$\therefore F_{FL} = 6.459^K$ OK

ASSUMED VALUE $= 6.450^K$

$F_{DI} = 1.213^K (T)$

$F_{IJ} = 1.700^K (C)$

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Rt. Side of Truss

$$\Sigma M_C = \tfrac{1}{2}(9.75) P_8 + 9.25 P_4 + 18.5 P_5 + 28 P_6 + 14.25 P_7 - 15.25 R_{1R} - 37.75 R_R + 4 P_{4H} - 4 P_{6H} - 10 P_{7H} + 10 F_{FR} = 0$$

$$F_{FR} = \frac{F_{CD}}{1.092} + \frac{F_{CI}}{1.271} + P_{5H} + P_{6H} + P_{7H} + P_{4H}$$

$$F_{CI} = \frac{1}{2.1954}\left[2.23837 P_3 - P_{3H}\right]$$ SEE SHT 22

$$= 3.7592^k$$

$$F_{CD} = F_{DE} - 0.057$$ (SHT. 18)

$$= 4.687^k$$

$$\therefore F_{FR} = 8.991^k$$

$$\Sigma M_C = 420.169 - 15.25 R_{1R} - 37.75 R_R + 10 F_{FR}$$

$$22.5 R_{1R} = 118.686$$

$$R_{1R} = 5.275^k$$

$$R_R = 11.381^k$$

CHECK:

$$\Sigma M_G = 176.984 - 190.464 - 82.944 - 26.822 - 19.027 + 25.6525 F_{CD} - 74.198 + 6.655 + 4.212 - 5.76 + 0.72$$

$$25.6525 F_{CD} = 8.209^k$$

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Although this problem is statically indeterminate, it is apparent, based on the above calculations, that to compensate for the unsymetrical loading on the truss, members DI and IJ will support greater loads than calculated (and shown on SHT. 26) while the top and bottom chord members will be subject to smaller axial forces. Based on a comparison of SHTS. 8 and 26, it is clear that loading condition no. 1 is more severe for all members except possibly IJ and the bottom chords.

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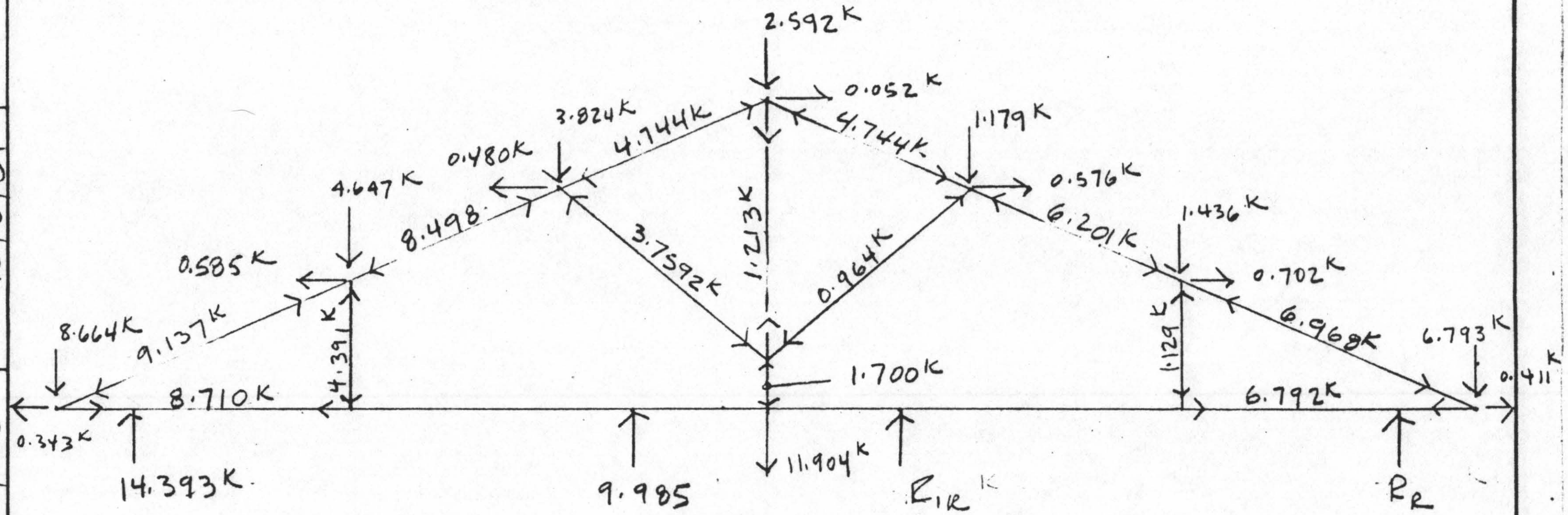

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MEMBER ANALYSIS :

- WEB (KINGPOST) MEMBER IJ ; L = 2.25' ; 6" x 15" ;

P_c (assumed worst cond.) = 2.592 + (0.6168)(0.964 + 3.759) = 5.50

$f_c = \frac{}{6 \times 15} =$ 61.2 psi << 840 psi

$l/d = 4.5$

$l/d < 11$

$\therefore F_c' = F_c = 840$

OK

- BOTTOM CHORD MEMBER AJ ; 6" x 9" ; $P_{T(MAX)} = 8.710^k$

WHEN F_{IJ} INCREASES TO COMPENSATE FOR THE UNSYMETRICAL TRUSS LOADING, R_{IL} WILL INCREASE AND R_L WILL DECREASE. COMPARING THE BOTTOM CHORD LOADING FOR CASE NO. 1 (- SHT. 14), AND THE PROJECTED LOADING FOR CASE NO. 2, IT IS APPARENT CASE NO. 1 IS MORE SEVERE AND SHOULD BE USED IN THE DESIGN OF A SOLUTION.

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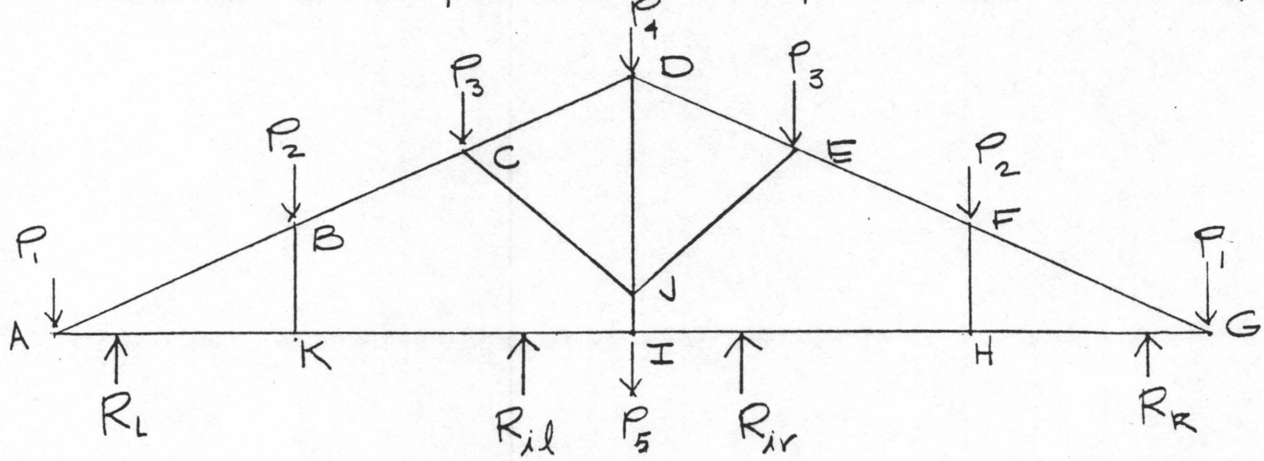
LOADING CONDITION 3: D.L. plus SNOW LOAD (I)



$$P_1 = \frac{(13.25)(205.3)}{2} + \frac{(14.55)(256.6)}{2} + \frac{32}{2}(14)(12.83)$$

$$= 6.101^K$$

$$P_2 = (13.25 + 9.5)(205.3)/2 + 24.85(256.6)/2 = 5.524^K$$

$$P_3 = (9.5 + 9.25)(205.3)/2 + 20.39(256.6)/2 = 4.541^K$$

$$P_4 = (9.25)(205.3) + 11.03(256.6) = 4.729^K$$

$$P_5 = (64)(14)(12.83)/2 = 5.748^K$$

$$R_L + R_{il} = P_1 + P_2 + P_3 + 1/2(P_4 + P_5)$$

$$= 21.404^K$$

Note: AXIAL LOADS FOR LOADING CONDITION 3 IDENTICAL TO THOSE FOR LOADING CONDITION No. 1. (SEE SHEET 8)

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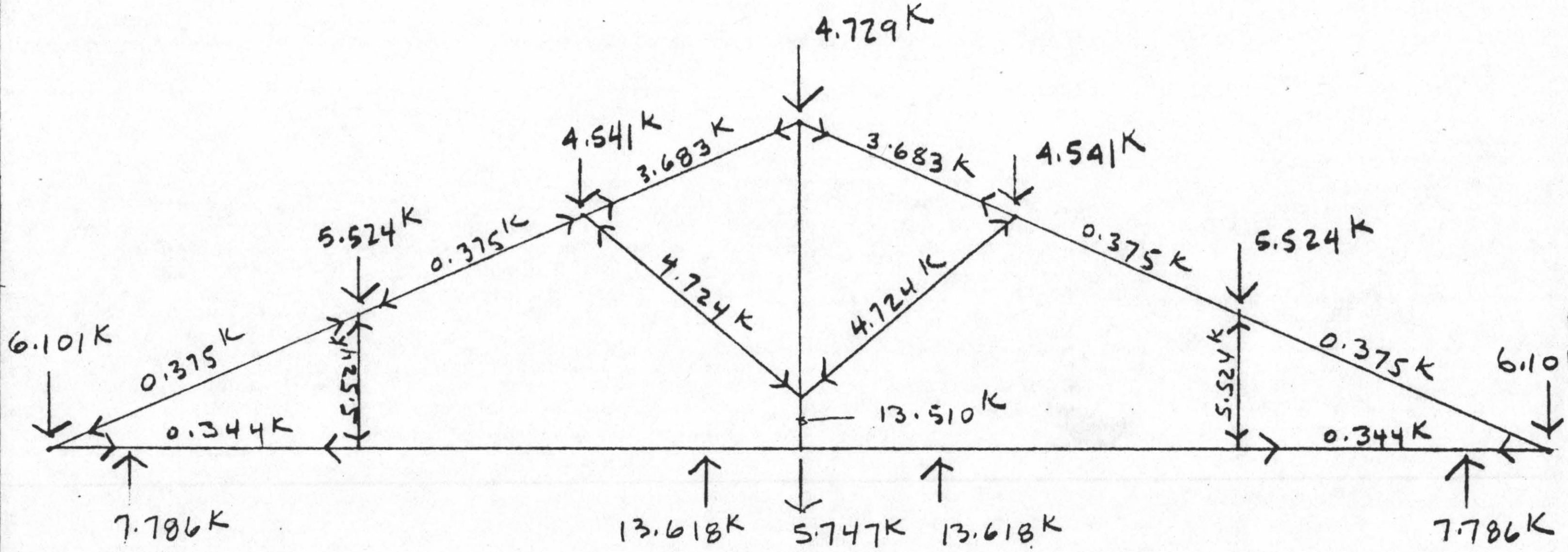

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ΣM_A (for floor live load) $= \dfrac{0.1924(32)^2}{2} - 3.5R_L - 26R_{1L}$

$R_L + R_{1L} = (0.1924)(32) = 6.157^K$

$\Sigma M_A = 98.509 - 3.5(6.157 - R_{1L}) - 26R_{1L}$

$22.5R_{1L} = 76.96$

$R_{1L} = 3.420^K$

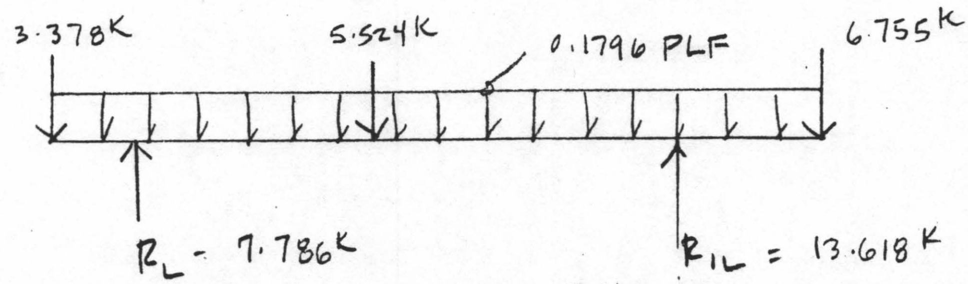
$R_L = 2.737^K$

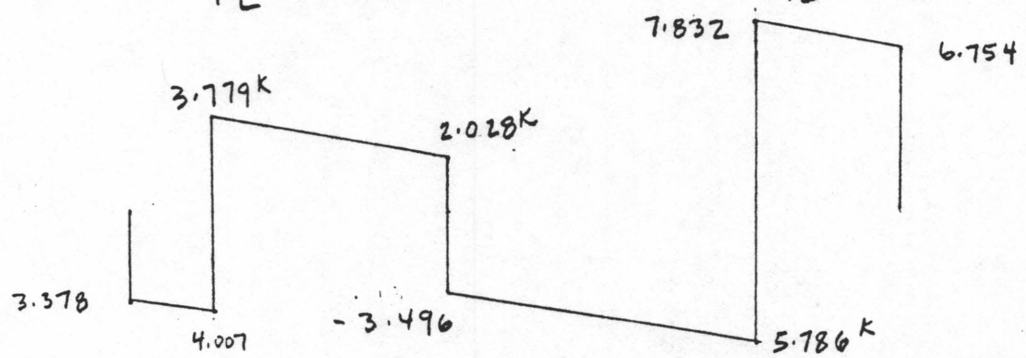
$\therefore$ R_L for loading condition 3 $= 7.786^K$

R_{1L} " " " " $= 13.618^K$

- BOTTOM CHORD MEMBERS AJ & JG ; $P = 0.344^K$

  $L = 32'$; $6'' \times 9''$





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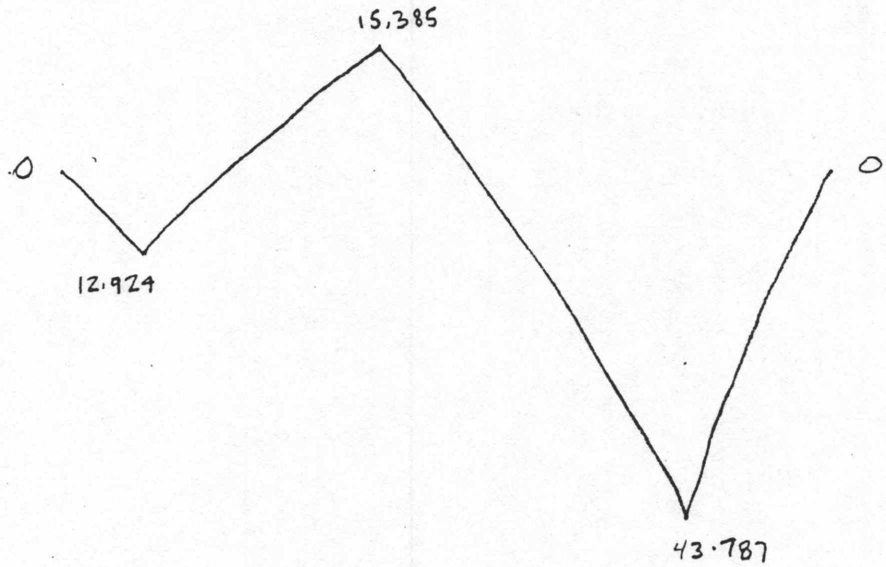

MOMENT

$M_{MAX} = 43.787$ ft-k

$f_b = \dfrac{43787\ (12)}{81} = 6,487$ psi

$f_t = \dfrac{0.344 \times 10^3}{54} =$

$\dfrac{f_t}{F_t} + \dfrac{f_b}{F_b} = 4.96 > 1$

NO GOOD

LOADING CONDITION 4. :- DEAD LOADS PLUS SNOW LOADS (II) PLUS WIND LOAD.

This loading condition results in identical member loading as exists for loading condition 2 with the exception that bottom chord members AJ & JG are subject to less flexure. They still, however, would be overloaded.

NATIONAL PARK SERVICE
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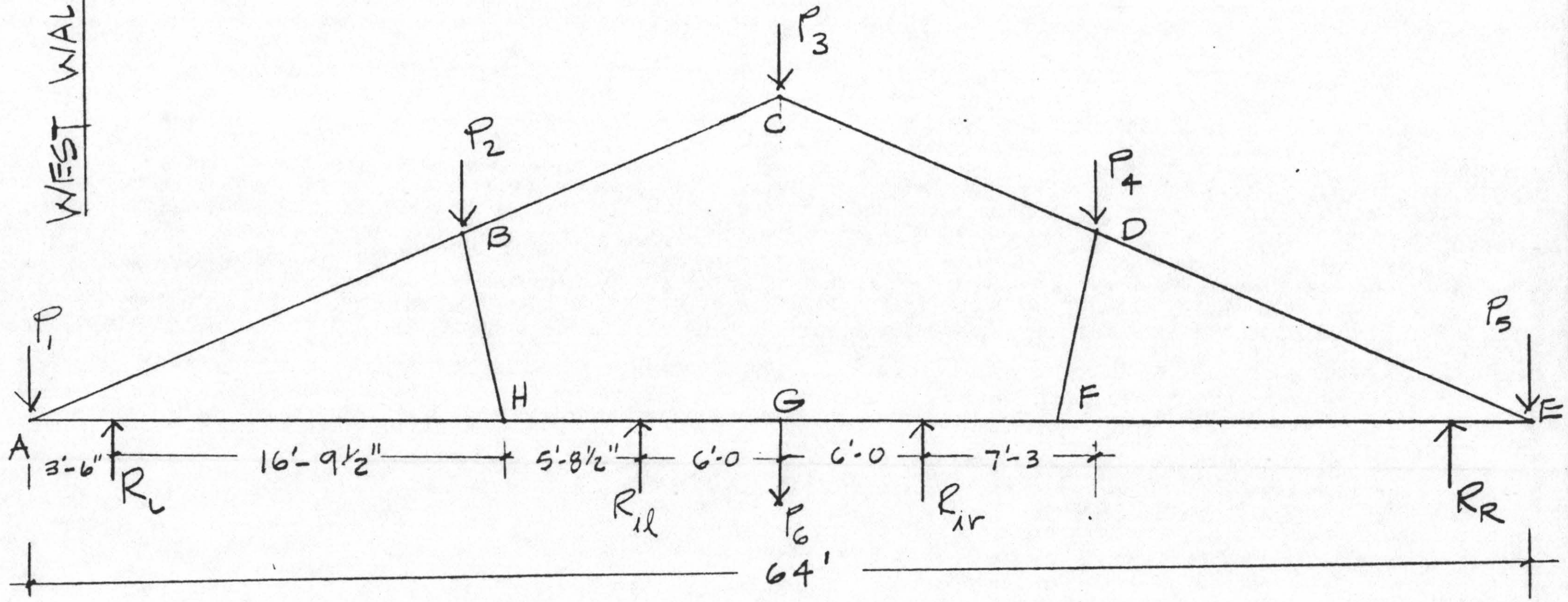
WEST WALL TRUSS



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FLOOR LOADS

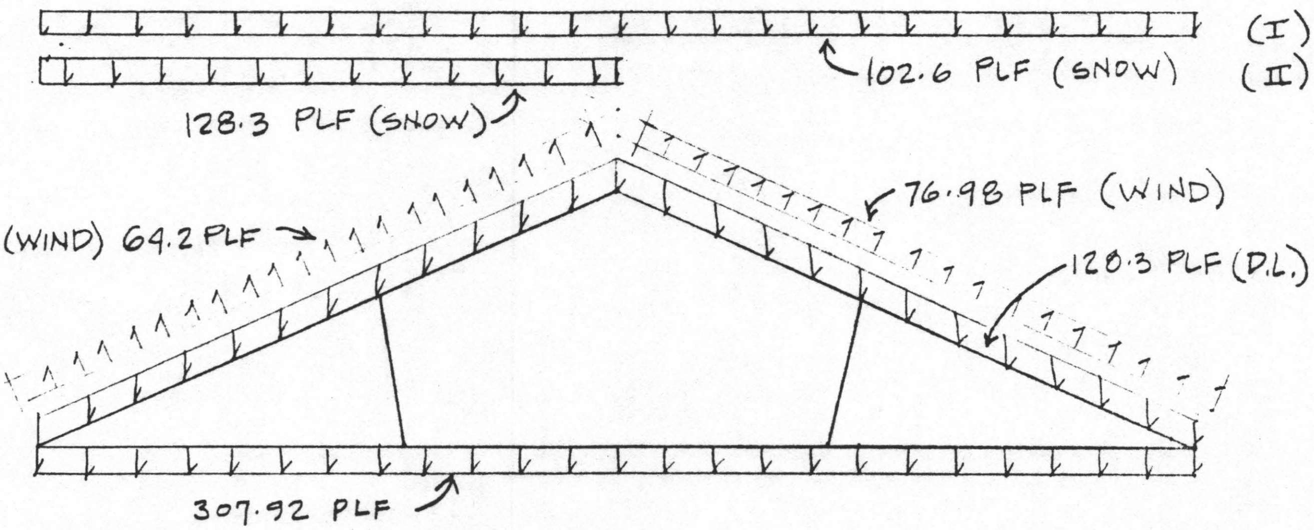
	PSF
• CEILING/FLOOR JOISTS (2x9s 16" o.c.)	5.0
• PLASTER CEILING	10.0
• FLOOR BOARDS	3.0
• ATTIC LIVE LOAD (THIS AREA ONLY)	30.0
	48.0

* SEE TABLE IV, PG 49, SIMPLIFIED DESIGN OF ROOF TRUSSES; PARKER

ROOF LOADS

	PSF
• TRUSS *	3.75
• SLATE SHINGLES	10.0
• SHEATHING	2.5
• RAFTERS	1.7
• PURLINS	1.8
	19.75
• SNOW	16 OR 20
• WIND	
WINDWARD	12
LEEWARD	10
• REACTION DUE TO OUTRIGGER	

LOADING ON WEST WALL TRUSS LESS EFFECTS DUE TO OUTRIGGERS



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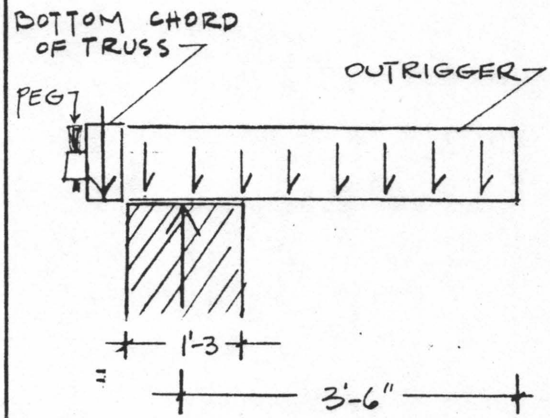
LOADS ON OUTRIGGERS AND THE SUBSEQUENT EFFECTS ON WEST WALL TRUSS.

ASSUME OUTRIGGERS SUPPORTING HORZ. CORNICE AND RAKING CORNICE OF PEDIMENT SUBJECT TO EQUAL DEAD & LIVE LOADS.



DEAD LOADS

2x9s 16" O.C. 4'-2" LONG		3.75 PSF
SHEATHING		2.5 "
SHINGLES		10.0 "
MISC.		3.75 "
	SUB - TOTAL	20 PSF

LIVE LOAD

SNOW LOAD	20 "
OR FOR OVERHANGING EAVES (710.3)	60 "

WIND LOAD (714.4 - OVERHANG UPLIFT SHALL BE DESIGNED TO WITHSTAND AN UPWARD PRESSURE OF TWO TIMES THE WIND PRES GIVEN IN TABLE 712.1 = 28 PSF

LOAD COMBINATIONS ON OUTRIGGERS :-

1. DEAD LOAD — 20 PSF ↓
2. DEAD LOAD plus LIVE LOAD*(NO SNOW) — 80 " ↓
3. DEAD LOAD plus WIND LOAD — ~~9.25~~ 8.0 " ↑
4. DEAD LOAD " LIVE LOAD plus WIND LOAD — 52 " ↓
5. DEAD LOAD " LIVE LOAD (SNOW) — 40 " ↓
6. DEAD LOAD " " " " + WIND — 12 " ↓

DETERMINE EFFECTS OF OUTRIGGER LOADING COMBINATIONS ON WEST END TRUSS.

* LIVE LOAD ON OUTRIGGER ONLY OF 60 PSF (710.3 BOCA)
– WOULDN'T BE CONCURRENT WITH MAJOR SNOW LOAD ON REST OF ROOF

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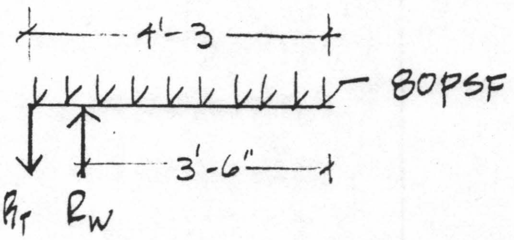
LOADING COMBINATION 2 (NO SNOW ON MAJOR PART OF ROOF)



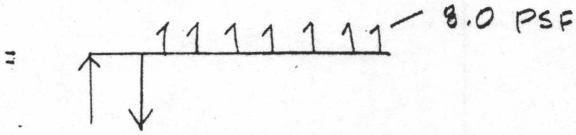
$\Sigma F_V = (80)(4.25) + R_T - R_W$

$\Sigma M_T = (80)(4.25)^2/2 - R_W(0'\text{-}9") = 0$

$R_W = 963.33$ lbs/Ft

$R_T = 623.3$ " / "

LOADING CONDITION 3



$\Sigma F_V = (8.0)(3) + R_T - R_W$

$\Sigma M_T = (8.0)(3)(4.25)/2 - R_W(0'\text{-}9)$

$R_W = 56.7$ PLF

$R_T = 32.7$ PLF

LOADING CONDITION 5 (SNOW)

(SAME DWG. AS # 2)

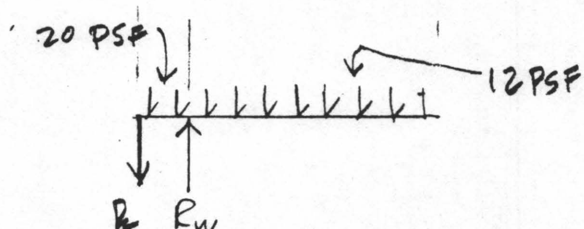
$\Sigma F_V = (40)(4.25) + R_T - R_W$

$\Sigma M_T = (40)(4.25)^2/2 - R_W(0.75)'$

$R_W = 481.67$ lbs/Ft

$R_T = 311.67$ " / "

LOADING CONDITION 6 – (SNOW + WIND)



$\Sigma F_V = (12)(3.5) + (20)(0.75) - R_W + R_T$

$\Sigma M_T = (12)(3.5)(4.25)/2 + \frac{(0.75)^2(20)}{2} - (0.75)R_W$

$R_W = 126.5$ lbs/Ft

$R_T = 69.5$ " / Ft

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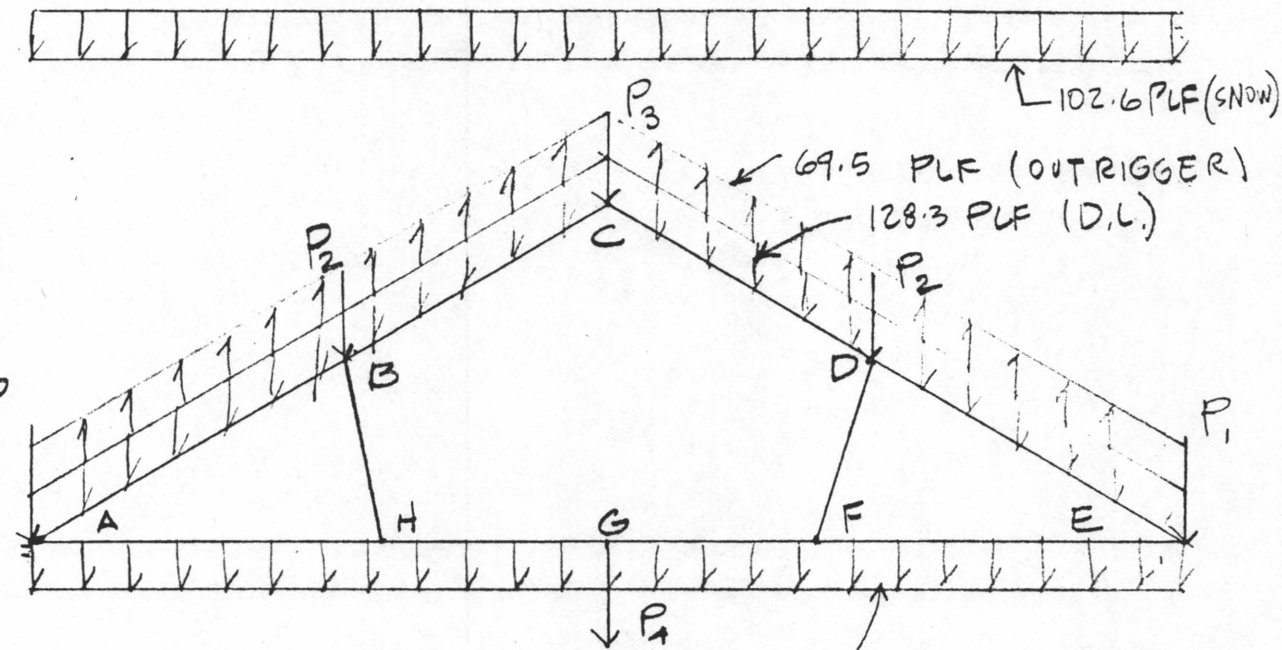
WEST END TRUSS ANALYSIS - D.L. plus SNOW LOAD plus WIND LOAD



(109.1 + 192.5 - 69.5) PLF = D.L. + L.L. + OUTRIGGER

$$P_1 = \underset{\text{SNOW}}{\frac{18.75}{2}(102.6)} + \underset{\text{FLOOR + OUTR}}{\frac{32}{2}(232.1)} + \underset{\text{OUTRIGGER + D.L.}}{\frac{20.475}{2}(58.8)} = 5277.44^{\#}$$

$$P_2 = \frac{32}{2}(102.6) + \frac{34.944}{2}(128.3 - 69.5) = 2668.95\ \#$$

$$P_3 = \frac{26.5}{2}(102.6) + \frac{28.938}{2}(128.3 - 69.5) = 2210.23\ \#$$

$$P_4 = \frac{64}{2}(232.1) = 7427.1 \text{ lbs}$$

$$R_L + R_{1L} = P_1 + P_2 + \tfrac{1}{2}(P_3 + P_4)$$

$$= 12.765^{K}$$

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$$F_{BC} = F_{CD} = \frac{P_3}{(2)(0.401)} = 2.756^K$$

TAKING THE MOMENT ABOUT "A" FOR HALF THE TRUSS.

$$\Sigma F_V = P_1 + P_2 + \tfrac{1}{2} P_3 + \tfrac{1}{2} P_4 - R_L - R_{IL}$$

$$= 12,765.06 - R_L - R_{IL}$$

$$\Sigma M_A = 18.75 P_2 + 32 P_3 + \frac{32}{2} P_4 - 32(0.401)(2.756)$$

$$= \frac{-14}{1.092}(2.756) - 3.5 R_L - 26 R_{IL} = 0$$

$$22.5 R_{IL} = 124.227$$

$$R_{IL} = 5.521^K = R_{IR}$$

$$R_L = 7.244^K = R_R$$

MEMBER BY MEMBER ANALYSIS BY METHOD OF JOINTS.

JOINT "A"

$$\Sigma F_V = 0.401 (F_{BC} + F_{CD}) - P_3 = 0$$

$$F_{BC} = \frac{P_3}{(2)(0.401)} = 2.756^K$$

$$F_{CD} = 2.756^K$$

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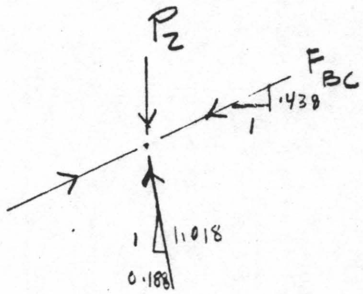
JOINT "B"



$$\Sigma F_V = P_2 + 0.401 F_{BC} - 0.401 F_{AB} - \frac{1}{1.018} F_{BH} = 0$$

$$\Sigma F_H = 0.9158 F_{BC} - 0.9158 F_{AB} + \frac{0.188}{1.018} F_{BH} = 0$$

$$\Sigma F_V = 3.774 - 0.401 F_{AB} - 0.9823 F_{BH}$$

$$\Sigma F_H = 2.524 - 0.9158 F_{AB} + 0.1847 F_{BH}$$

$$F_{AB} = 9.411 - 2.450 F_{BH}$$

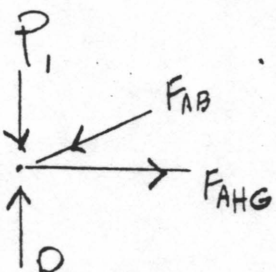
$$\Sigma F_H = 2.524 - 0.9158 (9.411 - 2.450 F_{BH}) + 0.1847 F_{BH}$$

$$2.428 F_{BH} = 6.095$$

$$F_{BH} = 2.510^K$$

$$F_{AB} = 3.261^K$$

JOINT "A"



R (reaction due to bottom chord)

$$F_{AHG} = F_{AB_H} = F_{AB}/1.092$$

$$= 2.986^K$$

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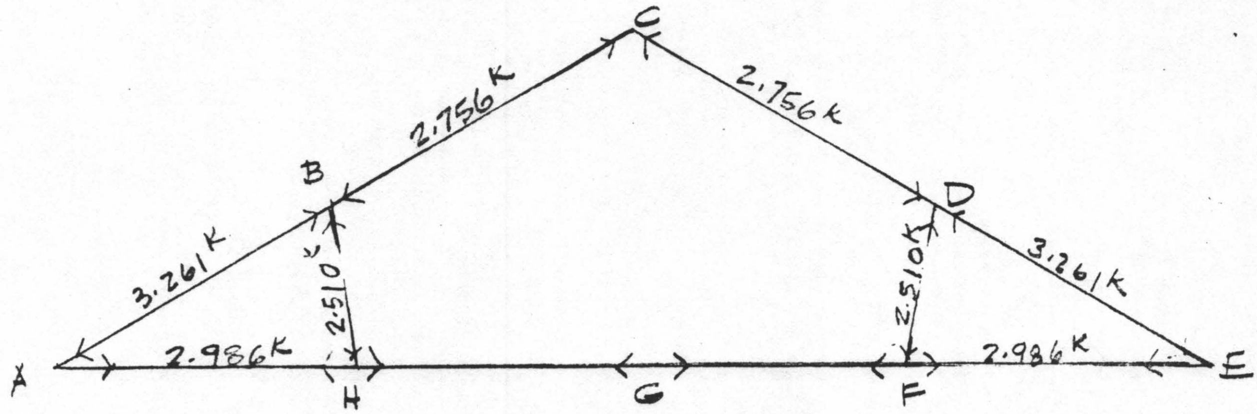
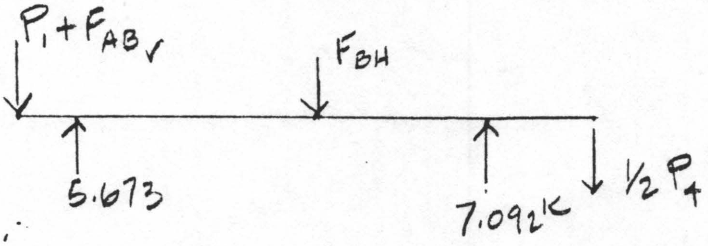


CHECK:-



$$\Sigma F_V:\ 5.277 + (0.401)(3.261) + \frac{2.510}{1.018} + 3.714 - 5.521 - 7.244 = 0$$

$$= 0.001 \qquad = 0 \qquad \text{OK}$$

MEMBER STRESS ANALYSIS:

MEMBER PROPERTIES:-

b	x	d		A (in^2)	S (in^3)	I (in^4)
4 3/4		8 5/8	BOTTOM CHORD	40.97	58.89	253.97
3 1/2		8	TOP CHORD	28	37.33	149.33
3		5 1/2	WEB MEMBER	16.5	15.12	41.59

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- WEB MEMBERS BH & DF (AXIAL LOAD ONLY)

$L = 8.36'$; $P = 4.329^K$; $3'' \times 5\frac{1}{2}''$

$$f_c = \frac{2510}{(3)(5\frac{1}{2})} = 152.12 \text{ psi} < 368.9 \text{ psi}$$

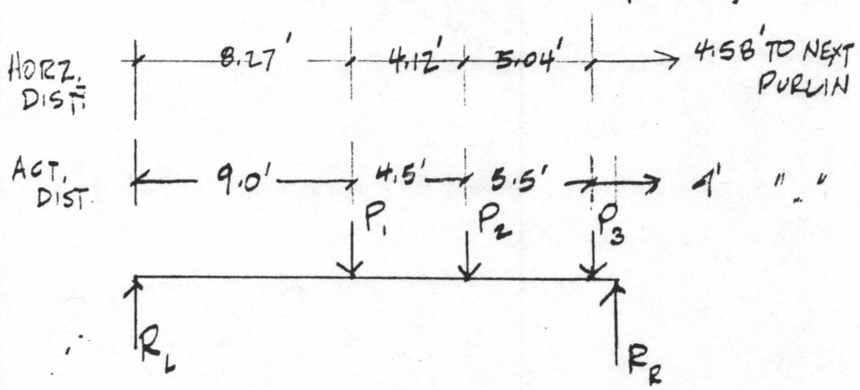
OK

$$l/d = \frac{(8.36)(12)}{3} = 33.44 \quad \checkmark$$

$$K = 0.671\sqrt{E/f_c} = 27.1$$

$$F'_c = \frac{0.3E}{(l/d)^2} = 368.9 \text{ psi}$$

- TOP CHORD MEMBERS AB & DE ; $P = 3.261^K$ (C) ; $L = 20.475'$



$$l/d = (20.5)(12)/8 = 30.75$$

$$F'_c = 0.3E/(l/d)^2$$

$$= 436.25$$

$$P_1 = \left[(8.27 + 4.12)(102.6)/2 + (9 + 4.5)(128.3 - 69.5)/2\right] = 1032.51 \text{ lbs}$$

$$P_2 = \left[(4.12 + 5.04)(102.6)/2 + (4.5 + 5.5)(128.3 - 69.5)/2\right] = 763.91 \text{ "}$$

$$P_3 = \left[(5.04 + 4.58)(102.6)/2 + (5.5 + 5)(128.3 - 69.5)/2\right] = 802.21 \text{ "}$$

$$\Sigma M_l = 8.27P_1 + 12.39P_2 + 17.43P_3 - 18.35R_R = 0$$

$$R_R = 31{,}986.15/18.35 = 1743.1.1 \text{ lbs}$$

$$R_l = P_1 + P_2 + P_3 - R_R = 855.52 \text{ lbs.}$$

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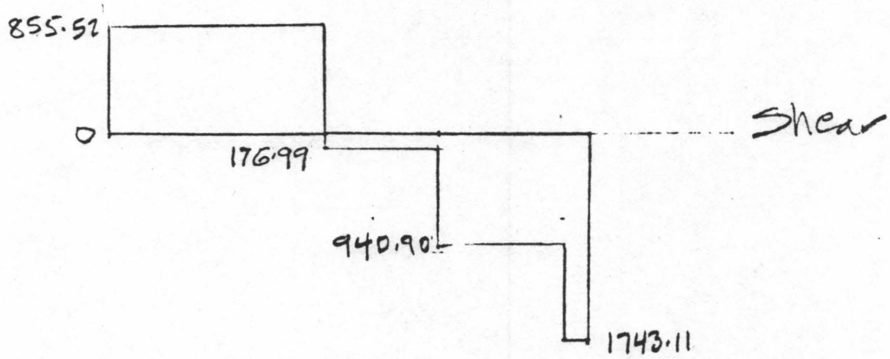

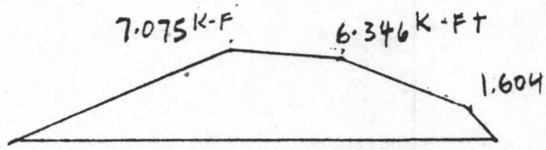

$M_{MAX} = 7.075^{K}\text{-FT}$

$F_b = 1310$ psi

$J = 1$

$$f_b = \frac{(7075)(12)}{37.33} = 2274.31 \text{ psi}$$

No Good

$$f_c = \frac{3261}{(3\tfrac{1}{2})(8)} = 116.46 \text{ psi}$$

$$\frac{f_c}{F'_c} + \frac{f_b}{F_b - Jf_c} = \frac{116.5}{436.25} + \frac{2274.3}{1310 - 116.5} = 2.17 > 1$$

MEMBER IS UNDERDESIGNED

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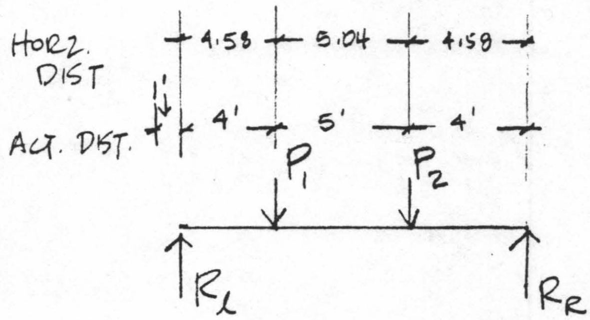
• TOP CHORD MEMBERS BC & CD ; $L = 14.47'$; $P = 2.756^K$



$$l/d = (14.5)(12)/8 = 21.75$$

$$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$$

$$= 723.8$$

$$J = \frac{21.75 - 11}{27.1 - 11} = 0.667$$

$$P_1 = \left[(4.58 + 5.04)(102.6)/2 + (5.0 + 5.5)(58.8)/2\right] = 802.21 \text{ lbs}$$

$$P_2 = \quad '' \quad = 802.21 \text{ ''}$$

$$R_L = R_R = 802.21 \text{ lbs}$$

$$M_{MAX} = 5(802.21) = 4011.03 \text{ Ft-lbs}$$

$$f_b = \frac{(4011.03)(12)}{37.33} = 1289.4 \text{ psi} \qquad F_b = 1310 \text{ psi}$$

$$f_c = \frac{2756}{(3\tfrac{1}{2})(8)} = 98.43 \text{ psi}$$

$$\frac{f_b}{F_b - Jf_c} + \frac{f_c}{F_c'} = \frac{1289.4}{1244.35} + \frac{98.43}{723.8} = 1.17 < 1$$

NO GOOD

• BOTTOM CHORD MEMBERS AG & EG ; $P = 2.986^K$ (T)

$4\tfrac{3}{4}'' \times 8\tfrac{5}{8}''$; $L = 32$

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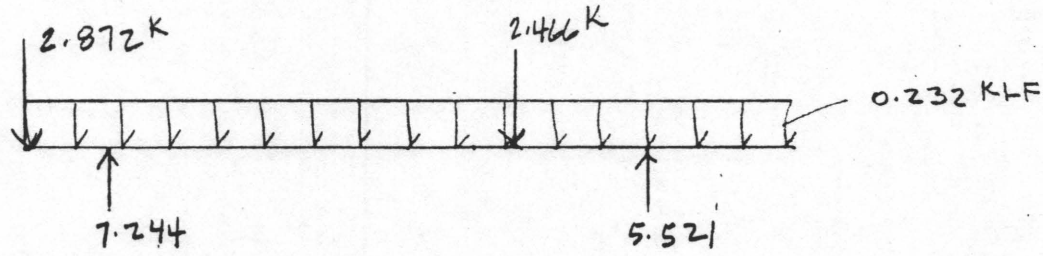
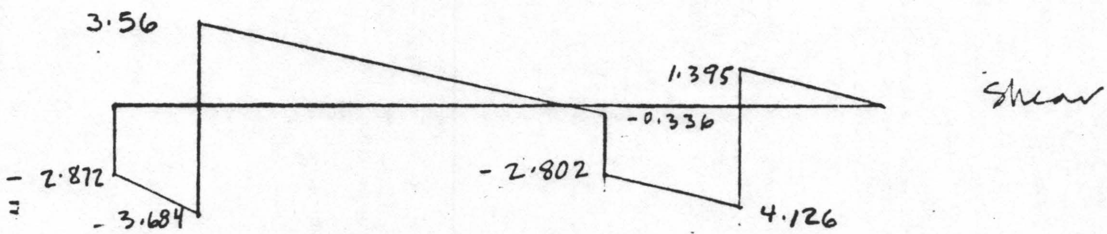
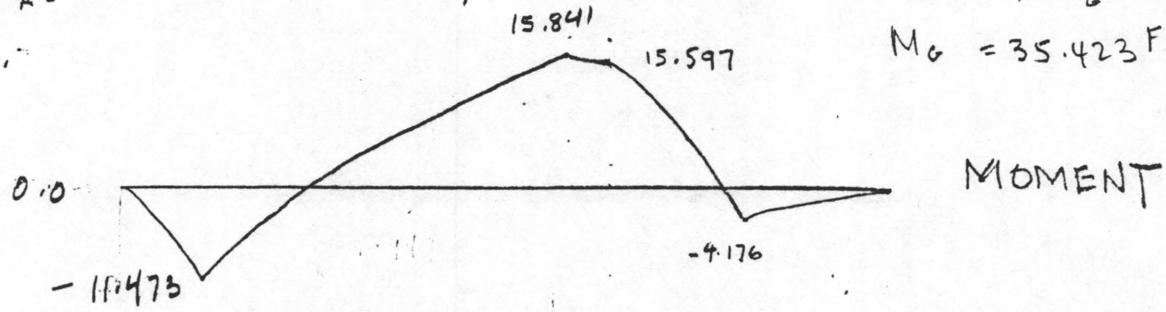




$M_A = 3.5(5.673) - (0.232)(32)^2/2 - 2.466\,(20.292) + 26(7.092) - M_G = 0$

$M_G = 35.423$ FT-K



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$M_{MAX} = 15.841$ K·Ft

$F_b = 1310$ psi

$$f_b = \frac{15841(12)}{58.89} = 3228 \text{ psi}$$

$$f_t = \frac{2986}{40.97} = 72.88$$

$$\frac{f_b}{F_b} + \frac{f_t}{F_t} = \frac{3228}{1310} + \frac{72.88}{875} = 2.55 > 1$$

MEMBER IS UNDERDESIGNED

=

WEST END TRUSS - LOADING CONDITION I.

D.L. + W.L. (ON ROOF and outriggers)

+ SNOW ON ONE SIDE OF BLDG.

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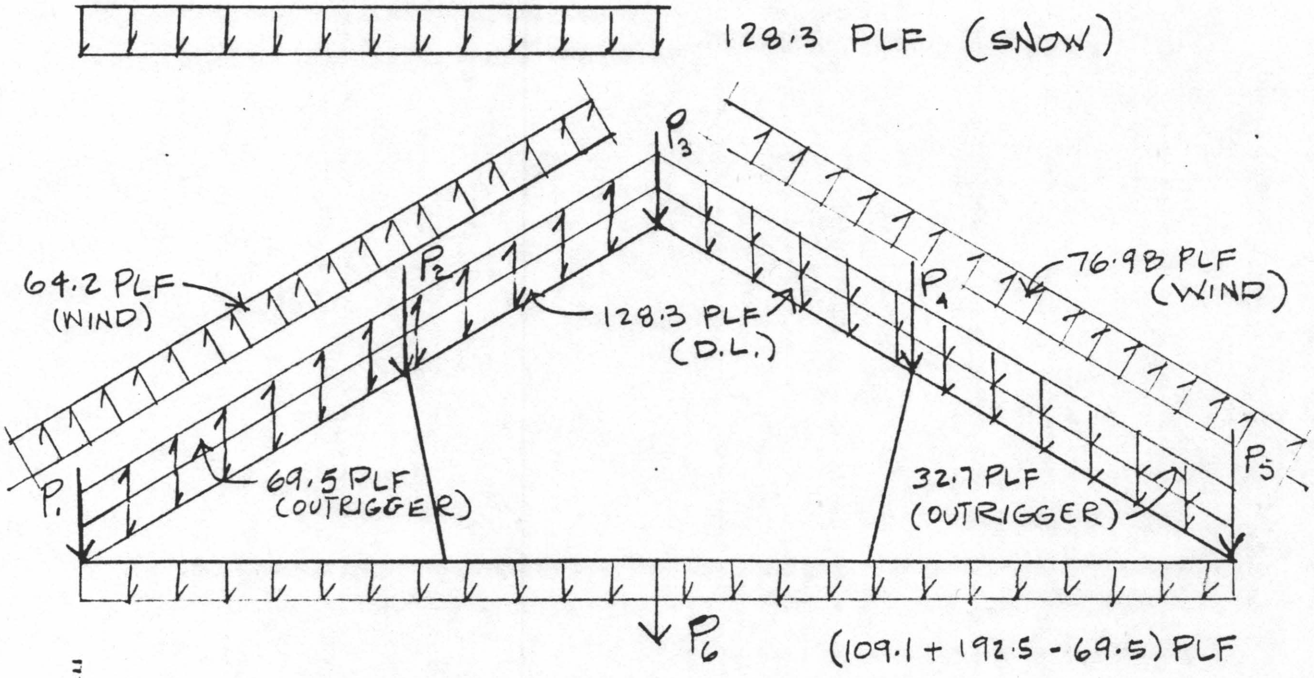

SNOW FLOOR D.L. OUTRIG WIND

$$P_1 = \left(\frac{18.75}{2}\right)(128.3) + \frac{32}{2}(232.1) + \frac{20.475}{2}\left(128.3 + 69.5 - \frac{64.2}{1.092}\right)$$

$$\therefore = 4916.4 \text{ lbs}$$

$$P_2 = \left(\frac{32}{2}\right)(128.3) + \frac{34.944}{2}\left(128.3 - 69.5 - \frac{64.2}{1.092}\right) = 2052.8 \text{ lbs}$$

$$P_3 = \left(\frac{13.25}{2}\right)(128.3) + \frac{28.938}{2}\left[128.3 - (69.5 - 32.7) - \frac{(64.2 + 76.98)}{(2)(1.092)}\right] = 1238.58$$

$$P_4 = \left(\frac{34.944}{2}\right)\left(128.3 + 32.7 - \frac{76.98}{1.092}\right) = 1581.31 \text{ lbs}$$

$$P_5 = \frac{32}{2}(232.1) + \frac{20.475}{2}\left(32.7 + 128.3 - \frac{76.98}{1.092}\right) = 4640.17 \text{ lbs}$$

$$P_6 = \frac{64}{2}(232.1) = 7427.2 \text{ lbs.}$$

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$$P_{1_H} = \frac{20.475}{2}(64.2)(0.401) = 0.264^K$$

$$P_{2_H} = \frac{34.944}{2}(64.2)(0.401) = 0.450^K$$

$$P_{3_H} = \frac{(26.5)}{2}(76.98-64.2)(0.401) = 0.068^K$$

$$P_{4_H} = \left(\frac{34.944}{2}\right)(76.98)(0.401) = 0.539^K$$

$$P_{5_H} = \frac{20.475}{2}(76.98)(0.401) = 0.316^K$$

$$R_L + R_{1L} = P_1 + P_2 + \frac{(P_3 + P_6)}{2}$$

$$= 11.302^K$$

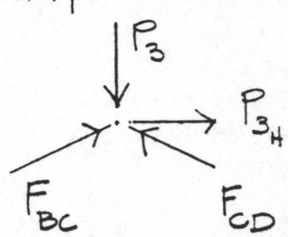
$$\Sigma M_A = 18.25\, P_2 + 32\, P_3$$
$$-26 R_{1L} - 3.5 R_L + \frac{32}{2} P_6$$
$$-32\,(0.401)(1.5814)$$
$$+14\left(0.068 - \frac{1.5814}{1.092}\right)$$
$$-8.21\,(0.450)$$

$$22.5\, R_{1L} = 113.067$$

$$R_{1L} = 5.025\ K$$

$$R_L = 6.277^K$$

JOINT "C"



$$\Sigma F_V = P_3 - F_{BC}(0.401) - F_{CD}(0.401)$$

$$\Sigma F_H = F_{CD_H} - F_{BC_H} - P_{3_H}$$

$$F_{BC} = 3088.73 - F_{CD}$$

$$\Sigma F_H = \frac{F_{CD}}{1.092} - \frac{F_{BC}}{1.092} - 67.9 = 0$$

$$F_{CD} - 3088.73 + F_{CD} - 74.15 = 0$$

$$2F_{CD} = 3162.88$$

$$F_{CD} = 1581.44 \text{ lbs}$$

$$F_{BC} = 1507.29 \text{ ''}$$

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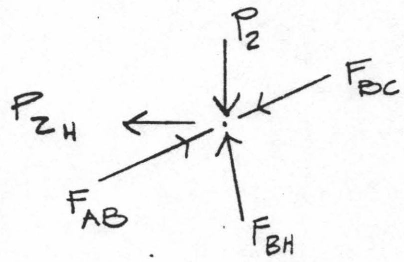
JOINT "B"



$$\Sigma F_V = 0.401\,F_{BC} + P_2 - \frac{F_{BH}}{1.018} - 0.401\,F_{AB}$$

$$= 2.657 - 0.9823\,F_{BH} - 0.401\,F_{AB}$$

$$\Sigma F_H = \frac{F_{BC}}{1.092} - \frac{F_{AB}}{1.092} + \frac{0.188}{1.018}\,F_{BH} + P_{2_H}$$

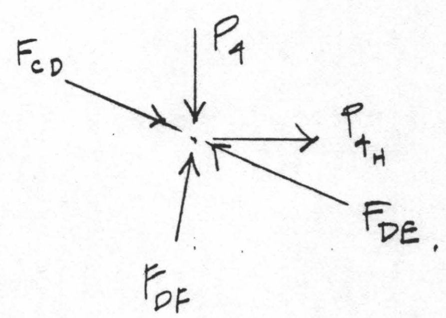
$$F_{AB} = 1.998 + 0.2017\,F_{BH}$$

$$\Sigma F_V = 2.657 - 0.9823\,F_{BH}$$
$$-0.401\,(1.998 + 0.2017\,F_{BH})$$

$$1.0632\,F_{BH} = 1.856^K$$

- $F_{BH} = 1.746^K$
- $F_{AB} = 2.350^K$
- $F_{AH} = F_{AB_H} + P_{1_H}$
  $= 2.416^K$

JOINT "D"



$$\Sigma F_V = 0.401\,F_{CD} + P_4 - 0.401\,F_{DE}$$
$$- \frac{F_{DF}}{1.018}$$

$$= 2.215 - 0.401\,F_{DE} - 0.9823\,F_{DF}$$

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$$\Sigma F_H = P_{4_H} + \frac{F_{CD}}{1.092} - \frac{F_{DE}}{1.092} + \frac{0.188}{1.018} F_{DF}$$

$$F_{DE} = 1.987 + 0.2017 F_{DF}$$

$$\Sigma F_V = 2.215 - 0.401(1.987 + .2017 F_{DF}) - 0.9823 F_{DF}$$

$$1.063 F_{DF} = 1.418$$

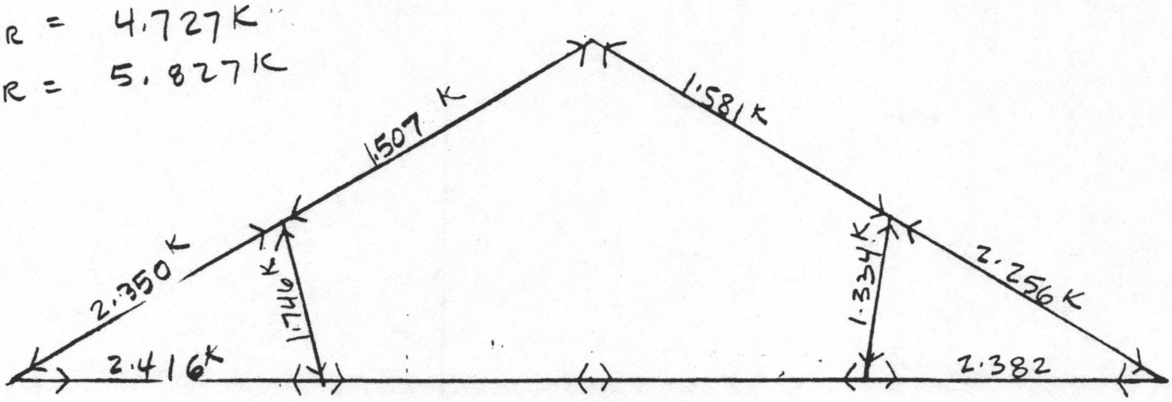
- $F_{DF} = 1.334^K$
- $F_{DE} = 2.256^K$
- $F_{EF} = F_{DE_H} + P_{5_H} = 2.382^K$

$R_R = 10.554^K - R_{1L}$

$$\Sigma M_E = 18.25 P_4 + 32 P_3 - 26 R_{1R} - 3.5 R_R + 16 P_6 - 32(0.401)(1.5073) - 14\left(0.068 + \frac{1.5073}{1.092}\right) - 8.21(0.539)$$

$R_{1R} = 4.727^K$

$R_R = 5.827^K$



Difference between F_{GH} & F_{FG} to be made up in friction forces or restraints at the various supports.

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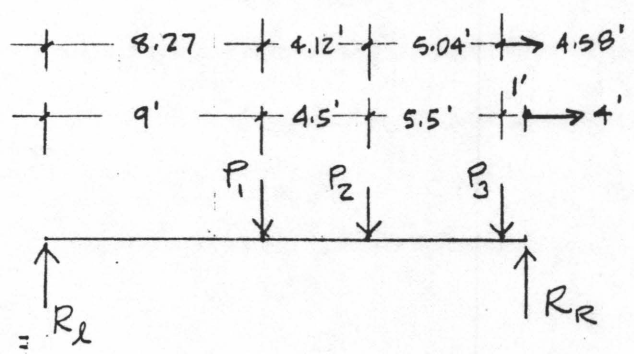
- WEB MEMBERS O.K AS AXIAL LOADS LESS THAN SAFE LOADING OF CASE I

- TOP CHORD MEMBER AB; 3½ x 8 ; Γ = 2.350K (C)

$L = 20.475'$



$\ell/d = 30.7125$

$F_c' = 436.25$

$$P_1 = \left[(8.27+4.12)(128.3)/2 + (9+4.5)(128.3 - \overset{0}{69.5} - \frac{64.2}{1.092})/2\right] = 794.82^{\#}$$

$$P_2 = \left[(4.12+5.04)(128.3)/2 + (4.5+5.5)(128.3 - \overset{0}{69.5} - \frac{64.2}{1.092})/2\right] = 587.61^{\#}$$

$$P_3 = \left[(5.04+4.58)(128.3)/2 + (5.5+5)(128.3 - \overset{0}{69.5} - \frac{64.2}{1.092})/2\right] = 617.12^{\#}$$

$$\Sigma M_\ell = 8.27 P_1 + 12.39 P_2 + 17.43 P_3 - 18.35 R_R = 0$$

$R_R = 1341.15^{\#}$

$R_\ell = P_1 + P_2 + P_3 - R_R = 658.40^{\#}$

$M_{MAX} = 658.4(9) = 5.926$ K-FT

$$f_b = \frac{(5926)(12)}{37.33} = 1904.83 \text{ psi}$$

$F_b = 1310$

$J = 1.0$

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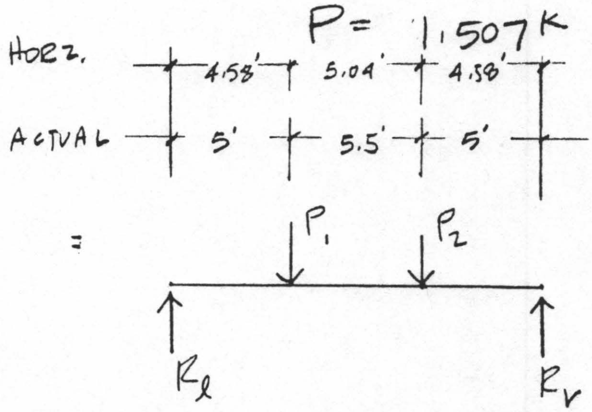
$$f_c = \frac{2350}{(3.5)(8)} = 83.93\,psi$$

MEMBER IS UNDERDESIGNED!

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} = \frac{89.93}{436.25} + \frac{1905}{1220.1} = 1.767 > 1$$

• TOP CHORD MEMBER BC ; $L = 14.47'$; $3\frac{1}{2} \times 8$



$l/d = 21.75$

$F_c' = 723.8\ psi$

$$J = \frac{21.75 - 11}{27.1 - 11} = 0.668$$

$$P_1 = \left[(4.58 + 5.04)(128.3)/2 + (5.0 + 5.5)\left(128.3 - \cancel{69.5} - \frac{64.2}{1.092}\right)/2\right] \quad 69.5 \to 0$$

$= 617.12\ lbs. = P_2$

$$M_{MAX} = 5(617.12) = 3085.62\ Ft\cdot lbs$$

$$f_b = \frac{3085.6(12)}{37.33} = 991.89\ psi \qquad F_b = 1310\ psi$$

$$f_c = \frac{1507}{(3\frac{1}{2})(8)} = 53.82$$

$$\frac{f_b}{F_b - Jf_c} + \frac{f_c}{F_c'} = 0.853 < 1$$

OK

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Feature ROOF TRUSS ANAL	Date 12/8/81	Date	Account

- TOP CHORD MEMBER CD ; $P = 1.581^k$; $L = 14.47'$

SAME DWG. AS MEMBER BC

$$P_1 = \left[\frac{(5+5.5)}{2}\left(128.3+32.7-\frac{76.98}{1.092}\right)\right] = 475.15 \text{ lbs} = P_2$$

$$M_{MAX} = 5(475.15) = 2375.77 \text{ Ft-lbs}$$

$$f_b = \frac{2375.77\,(12)}{37.33} = 763.71 \text{ psi} \qquad F_b = 1310 \text{ psi}$$

$$f_c = \frac{1581}{(3\tfrac{1}{2})(8)} = 56.46 \text{ psi} \qquad F_c' = 723.8 \text{ psi}$$

$$J = 0.668$$

$$\frac{f_b}{F_b' - Jf_c} + \frac{f_c}{F_c} = \frac{763.71}{1272.3} + \frac{56.46}{723.8} = 0.678 < 1 \quad \text{OK}$$

- TOP CHORD MEMBER DE ; $P = 2.256^k$

SAME DWG AS FOR MEMBER AB

$$P_1 = \left[(9+4.5)\left(128.3+32.7-\frac{76.98}{1.092}\right)\right] = 1221.82 \text{ lbs}$$

$$P_2 = \left[(4.5+5.5)(\quad '' \quad)\right] = 905.05 \text{ lbs}$$

$$P_3 = \left[(5.5+5.0)(\quad '' \quad)\right] = 859.80\ ''$$

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$$\Sigma M_{\ell} = 8.27\,P_1 + 12.39\,P_2 + 17.43\,P_3 - 18.35\,R_R = 0$$

$$R_R = 1.978^k$$

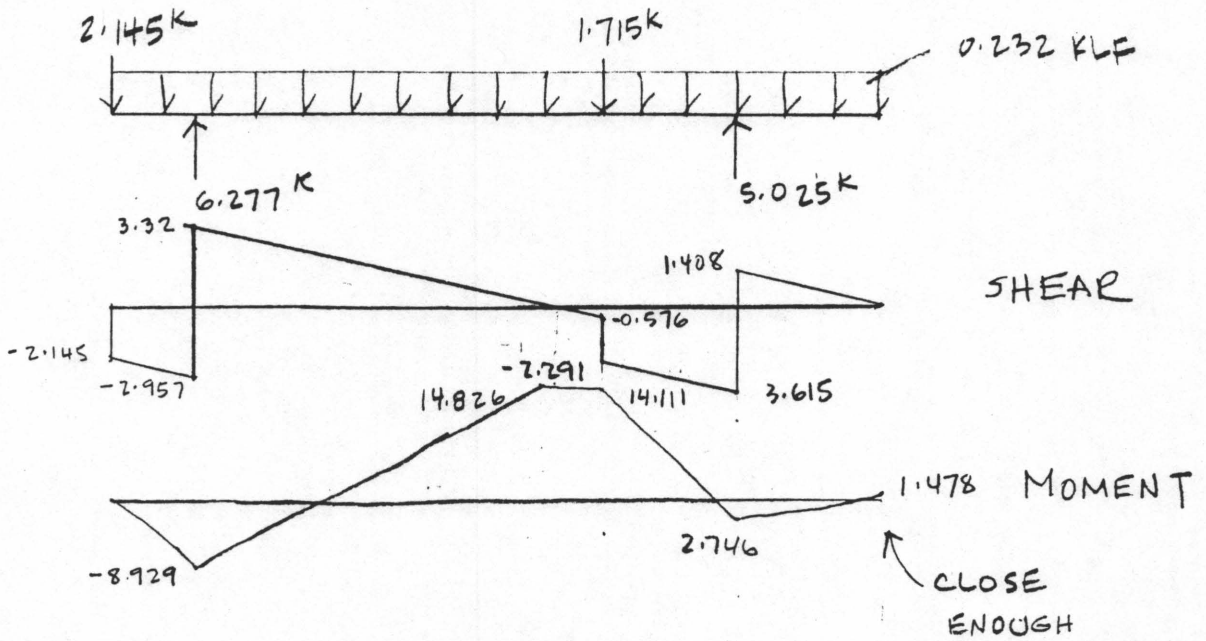
$$R_{\ell} = P_1 + P_2 + P_3 - R_R = 1.008^k$$

$$M_{MAX} = 9(1.008) = 9.072\ \text{Ft-k}$$

$$f_b = \frac{(9072)(12)}{37.33} = 2916.3\ \text{psi} \qquad F_b = 1310\ \text{psi}$$

NO GOOD - MEMBER UNDERDESIGNED

- BOTTOM CHORD MEMBER AG ; $P = 2.416^k$;
  $4\frac{3}{4}'' \times 8\frac{5}{8}''$; $L = 32'$



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Feature ROOF TRUSS ANAL.	Date 12/8/81 / Date	Account

$M_{MAX} = 14.826$ K-Ft

$$f_b = \frac{14826\,(12)}{58.89} = 3021.1 \text{ psi}$$

$$f_t = \frac{2416}{40.97} = 58.97 \text{ psi}$$

$$\frac{f_b}{F_b} + \frac{f_t}{F_t} = \frac{3021.1}{1310} + \frac{58.97}{875} = 2.37 > 1$$

No Good

EAST-END TRUSS - SIMILAR TO MAJORITY OF TRUSSES EXCEPT :-

1. SUPPORTED AT BRICK COLUMNS
2. HAS OUTRIGGER LOADING ON ONE SIDE.

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 54
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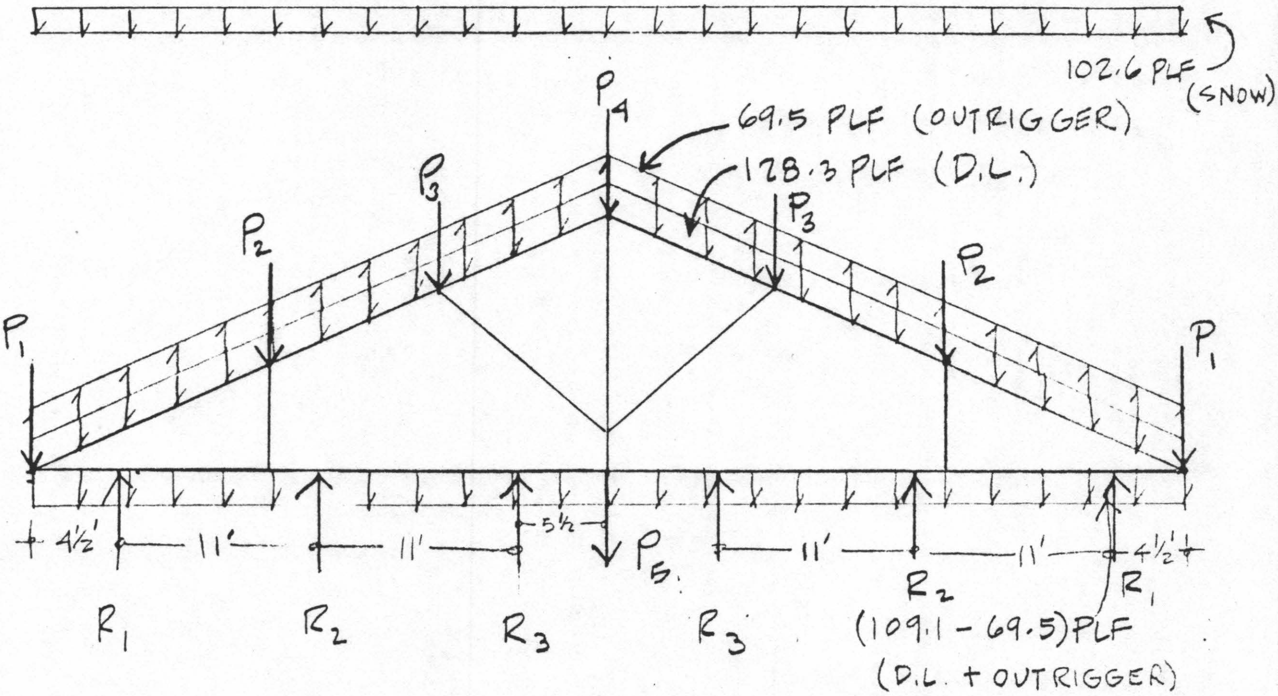
Feature ROOF TRUSS ANALYSIS	Date 12/18/81	Date	Account

EAST END TRUSS - LOADING CONDITION I

LOADING CONDITIONS - SAME AS FOR WEST END TRUSS

SEE SHEETS 31 - 34



$P_1 = (13.25)(102.6)/2 + (14.55)(128.3 - 69.5)/2 + \frac{32}{2}(109.1 - 69.5) = 1.741^K$

$P_2 = (13.25 + 9.5)(102.6)/2 + \frac{24.85}{2}(128.3 - 69.5) = 1.898^K$

$P_3 = (9.5 + 9.25)(102.6)/2 + \frac{20.39}{2}(128.3 - 69.5) = 1.561^K$

$P_4 = (9.25)(102.6) + 11.03\,(128.3 - 69.5) = 1.598^K$

$P_5 = (64)(109.1 - 69.5)/2 = 1.267^K$

$R_1 + R_2 + R_3 = P_1 + P_2 + P_3 + \frac{1}{2}(P_4 + P_5) = 6.632^K$

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Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER	Sheet 55
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Feature ROOF TRUSS ANALYSIS	Date 12/18/81 / Date	Account

ASSUME $R_1 = R_2 = R_3 = 2.211^K$

USING THE METHOD OF SECTIONS, TAKE MOMENT ABOUT "A" FOR AEIJA.

$$\Sigma M_A = 4.5 R_1 + 15.5 R_2 + 26.5 R_3 - \tfrac{1}{2}(32)P_5 - 32 P_4$$
$$- \frac{41.25}{2} P_3 - 22.75 P_3 - 13.25 P_2 + 41.25 F_{EF_V}$$
$$+ 10 F_{EF_H} = 0$$

$25.703 F_{EF} = 61.453$

- $\therefore F_{EF} = 2.391^K$
- $F_{FH} = P_2 = 1.898^K$
- $F_{FG} = F_{EF} = 2.391^K$
- $\therefore F_{HG} = \frac{F_{FG}}{1.092} = 2.189^K$
- JOINT "E"

$\Sigma F_V = 1.561 + 0.401 F_{DE} - 0.9588 - 0.6168 F_{IE} = 0$

$\Sigma F_H = 2.1896 - 0.9158 F_{DE} - 0.7868 F_{IE} = 0$

$F_{DE} = 1.538 F_{IE} - 1.502$

$\Sigma F_H = 2.1896 - 0.9158(1.538 F_{IE} - 1.502) - 0.7868 F_{IE} = 0$

$2.1953 F_{IE} = 3.565$

- $F_{IE} = 1.624^K$
- $F_{DE} = 0.996^K$

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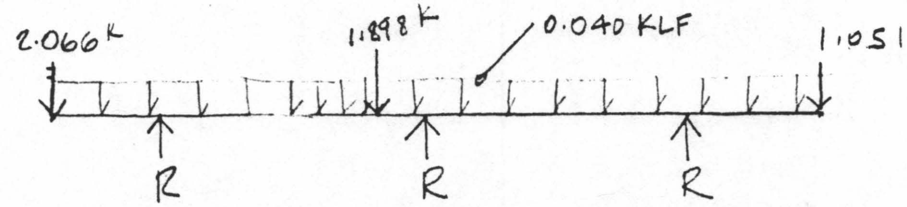
Feature ROOF TRUSS ANALYSIS	Date 12/18/81	Date	Account

- $F_{CD} = F_{DE} = 0.996$
- $F_{DI} = P_4 - 0.401(0.996)2 = 0.800^K$
- $F_{IJ} = 2(0.401)(1.624) + F_{DI} = 2.102$

MEMBER BY MEMBER ANALYSIS :-

BOTTOM CHORD MEMBERS AK & HG ; $P = 2.189^K$;

$L = 32'$, $6'' \times 9''$



$\Sigma F_V = 2.066 + 1.898 + 1.051 + 0.04(32) - 6.632 = -0.338^K$

Although this means that the assumption that $R_1 = R_2 = R_3$ is incorrect, for the purposes of this analysis and because it wasn't too far off, the bottom chord will be evaluated for the loading conditions just derived.

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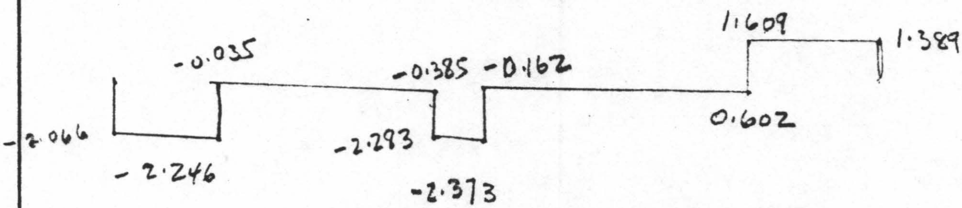
Feature ROOF TRUSS ANALYSIS	Date 12/18/81	Date	Account



$M_{MAX} = 20.98$ Ft-K.

$$f_b = \frac{20{,}980\ (12)}{81.00} = 3108 \text{ psi} \qquad F_b = 1310 \text{ psi}$$

$$f_t = \frac{2.189 \times 10^3}{54} = 40.54 \text{ psi}$$

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} = 2.42 > 1$$

NO GOOD

Note: Upon closer inspection, it is apparent that the bottom chord is supported by other more closely spaced posts which rest on a second (lower) beam. These posts lie in direct line with the vertical braces within the east-end truss. Because of this almost continuous-type support, flexure is reduced to an appreciably lower amount and, as the axial loads are small, the bottom chord is O.K.

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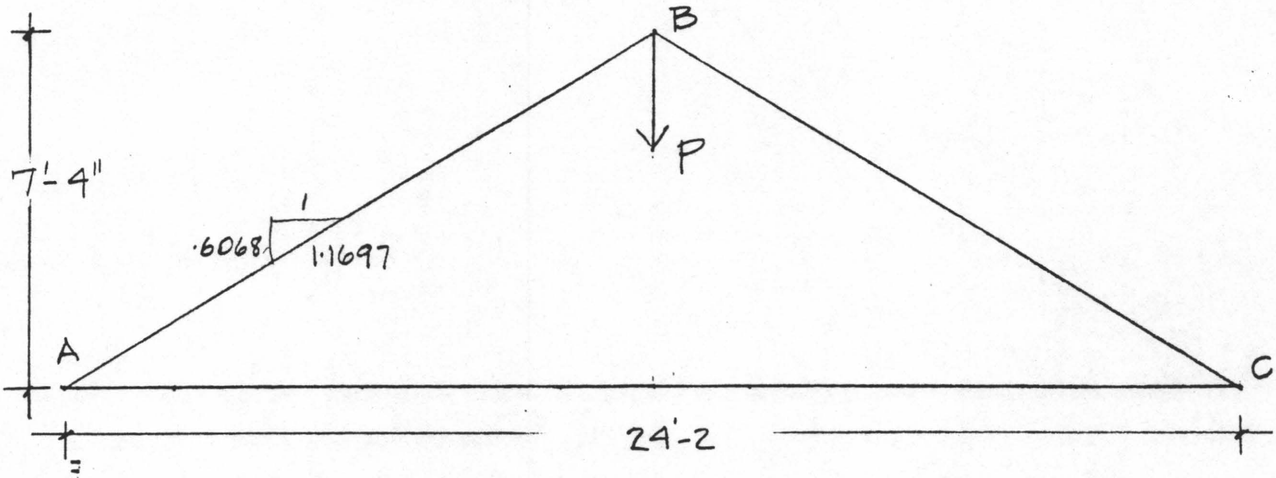
Feature ARMY TRUSS	Date 12/10/81	Date	Account

ARMY TRUSS - CIRCA 1930



MEMBERS		DIMENS.	LENGTH	WEIGHT
TOP CHORD	AB & BC	6" x 6"	14.134'	10 PLF
BOT. "	AC	8" x 6"	24.167'	13.33 "

P (from calcs for main roof truss) = $R_{i\ell_{Max}}$ + $(10)(14.134)$ + 50
(wgt. of AB&BC) wgt of rod

= 15.575^K

MEMBER BY MEMBER ANALYSIS BY METHOD OF JOINTS :-

JOINT "B"

$$\Sigma F_V = P = (F_{AB} + F_{BC}) \frac{0.6068}{1.1697}$$

$$F_{AB} = F_{BC}$$

$$F_{AB} = \frac{(1.1697)P}{(0.6068)(2)} = 15.012^K = F_{BC} \ (C)$$

$$F_{BC} = 2 F_{AB_H} = 25.668^K \ (T)$$

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Feature ARMY TRUSS	Date 12/10/81	Date	Account

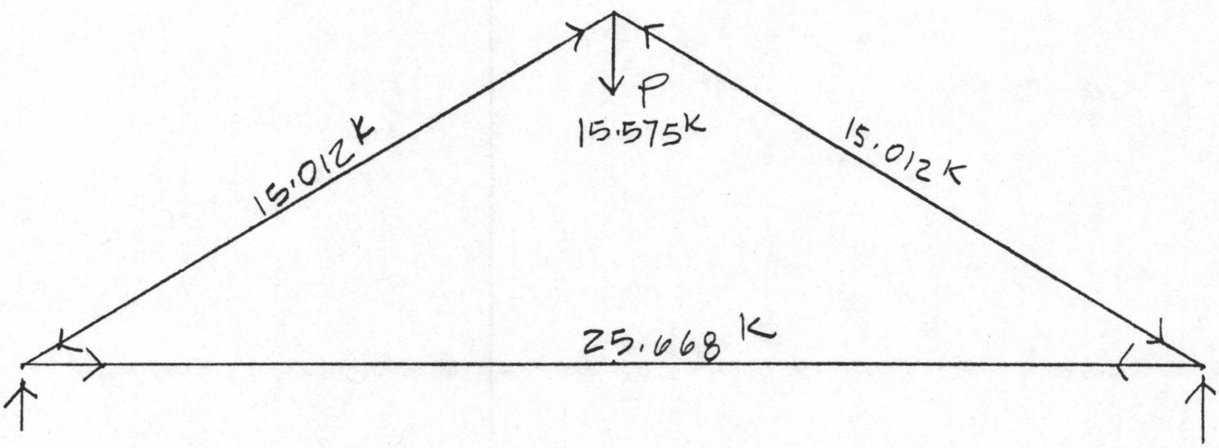
MEMBER STESS ANALYSIS :-

ALLOWABLE STRESSES - ASSUME SOUTHERN PINE (No. 2SR)

F_b = 1200 psi		INCREASE 1.25 FOR SNOW LOADS	F_b = 1500 psi
F_t = 650 "			F_t = 810 "
F_v = 85 "			F_v = 105 "
$F_{c\perp}$ = 390 "			$F_{c\perp}$ = 485 "
F_c = 900 "			F_c = 1125 "
E = 1500 Ksi			E = 1875 Ksi

MEMBER PROPERTIES

b x d		A (in²)	S (in³)	I (in⁴)
6" x 6"	top chord	36.0	36.0	108.0
6" x 8"	bottom "	48.0	64.0	256.0



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Feature ARMY TRUSS	Date 12/10/81	Date	Account

- TOP CHORD MEMBERS AB & BC – $P = 15.012^K$, $L = 14.134'$, $6'' \times 6''$

$f_c = \frac{15012}{(6)(6)} = 417 \text{ psi} < 637.5 \times 1.25$

OK

$l = 13'.3$

$l/d = \frac{13.25 \times 12}{6} = 26.5$

$K = 0.671\sqrt{E/f_c}$

$= 27.4 > 26.5$

$F'_c = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$

$= 900\,(0.708)$

$= 637 \text{ psi}$

- BOTTOM CHORD MEMBER AC ; $P = 25.668^K$; $L = 24'\text{-}2''$;
  $6'' \times 8''$ minus notch of $2''$ I

$f_t = \frac{25668}{(6)(6)} = 713 \text{ psi} < 845 \text{ psi}$

OK

$F_t = 845 \text{ psi}$

ANALYSIS OF $1''\phi$ STL. ROD SUPPORTING $15.384^K + \text{WGT} \approx 15.5^K$

Area $= 0.7854 \text{ in}^2$

For $F_t = 20$ ksi, allowable load in kips $= 15.71^K$ OK

$F_t = 27$ " , " " " " $= 21.21^K$ OK

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Feature ARMY TRUSS	Date 12/10/81	Date	Account

For $F_t = 40$ ksi (A325), allow. load = 31.42 kips.

Note: Tension values for unthreaded shanks of bolts and other unthreaded parts.

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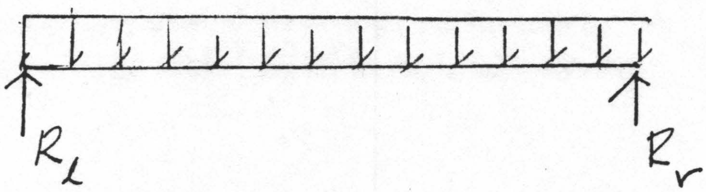
Feature ANALYSIS OF PURLINS	Date 12/9/81 · Date	Account

PURLINS

AVG. DIMENSIONS - 5½" x 6"
AVG LENGTH - 12.83'
SPACING (AVG) - 5' O.C.
(MAX.) - 9'-6" (first purlin from bottom)
S = 33 in^3

LOADS:

SLATE SHINGLES	10 PSF	
ROOF SHEATHING	2.5 "	
RAFTERS	1.7 "	
	14.2 PSF x 14.25/2 =	101.2 PLF
PURLINS		9.1 "
LIVE (SNOW)	20 PSF x 14.25/2 =	142.5 PLF
		252.8 PLF



$R_l = R_r = 12.83\,(252.8)/2 = 1621.7$ lbs.

$M_{max} = (252.8)(12.83)^2/8 = 5201.6$ FT-lbs

$f_b = (5201.6)(12)/33.0 = 1891.5$ psi $\qquad F_b = 1310$ psi

$f_b/F_b = 1891.5/1310 = 1.44 > 1$ MEMBER UNDERDESIGNED

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Feature PURLINS	Date 12/9/81	Date	Account

FOR PURLINS SPACED 5' O.C.

LOADS:

Slate shingles	10 PSF	
SHEATHING	2.5 "	
RAFTERS	1.7 "	
	14.2 x 5' =	71 P.L.F.
PURLINS		9.1 "
LIVE (SNOW)	20 x 5	100.0 "
	TOTAL	180.1 "

$R_l = R_r = 12.83(180.1)/2 = 1155.3$ lbs

$M_{max} = (180.1)(12.83)^2/8 = 3705.8$ Ft-lbs

$f_b = (3705.8)(12)/33 = 1347.5$ psi

$\frac{f_b}{F_b} = \frac{1347.5}{1310} = 1.028 < 1$ NO GOOD

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Feature RAFTERS	Date 12/9/81 / Date	Account

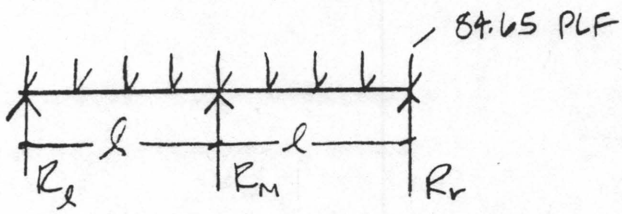
ANALYSIS OF RAFTERS

AVG DIMENSIONS	3½"x 3½ to 2¼ x 3¼
SPACING	2'-6"
AVG UNSUPPORTED LENGTH	5'-0
MAX. " "	9'-6

$S_{(MIN)}$ = 3.96 in^3

LOADS : FOR AVG. CONDITIONS – 5' UNSUPPORTED LENGTHS.

SLATE SHINGLES	10 PSF	
ROOF SHEATHING	2.5 "	
	12.5 " x 2.5'	= 31.25 PLF
RAFTERS		3.4 "
LIVE (SNOW)	20 x 2.5	50.0 "
		84.65 "



$l = 5'$

$R_r = R_l = \frac{3wl}{8} = 158.7$#

$R_M = \frac{5wl}{8} = 264.53$ #

$M_{max} = \frac{wl^2}{8} = 264.53$ Ft-lbs

(FOR SIMPLY SUPP.)

$f_b = (264.53)(12)/3.96 = 801.61$ psi

$F_b = 1310$ psi

OK

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Feature RAFTERS	Date 12/9/81	Date	Account

FOR MOST CRITICAL LOADING CONDITION :-

9'-6 LENGTH SIMPLY SUPPORTED. W/ UNIFORM LOAD OF 84.65 PLF

$$M_{MAX} = wl^2/8 = (84.65)(9.5)^2/8 = 954.96 \text{ Ft-lbs}$$

$$f_b = \frac{954.96\,(12)}{3.96} = 2894 \text{ psi} \qquad F_b = 1310 \text{ psi}$$

NO GOOD

Check in field to see if rafters for 9'-5" span are greater in dimension than 2¼" x 3¼".

$$f_b \left(\text{for the above loading condition for } 3\tfrac{1}{2} \times 3\tfrac{1}{2} \text{ rafters}\right) = \frac{954.96\,(12)}{7.15}$$

$$= 1603.67 \text{ psi}$$

STILL NO GOOD BUT IN THE BALL PARK

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Feature CEILING JOISTS	Date 12/9/81	Date	Account

ANALYSIS OF CEILING JOISTS

AVG DIMENSIONS - VARIES FROM 3 x 3½ TO 3 x 6½
SPACING - 16" O.C.
AVG SPAN - 12.83'

$S_{(MIN)}$ - 6.125 in^3 $\qquad I = 10.72\ in^4$

$S_{(MAX)}$ - 21.125 " $\qquad E = 1.1 \times 10^6\ psi \times 1.25$

LOADS:

PLASTER CEILING	10 PSF × 1.33 =	13.33 PLF
JOISTS		5.42 "
ASSUME 10 PSF LIVE LOAD		13.33 "
		32.1 "

FOR BENDING :-

$\therefore M_{MAX} = w l^2/8 = (32.1)(12.83)^2/8 = 660.22$ Ft-lbs

$f_b = (660.22)(12)/6.125 = 1293.5$ psi

$F_b = 1310$ psi

OK

FOR DEFLECTION :-

$D_{all} = \frac{1}{240} l = \frac{1}{240} \times (12.83)(12) = 0.6415"$

$D_{MAX} = \frac{5}{384} \frac{w l^4}{EI} = \frac{(5)(32.1)(12.83)^4 (10)^3}{(384)(1.375 \times 10^6)(10.72)} = 0.77"$

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Feature CEILING JOISTS.	Date 12/9/81	Date	Account

NOTE: ALTHOUGH D_{MAX} IS SLIGHTLY GREATER THAN D_{ALL} FOR THE CEILING JOISTS, THESE CALCS ARE BASED ON THE ASSUMPTIONS THAT ALL JOISTS ARE 3 x 3½, THAT A LIVE LOAD OF 10 PSF WOULD BE CARRIED BY THESE MEMBERS, AND THAT NO COOPERATIVE ACTION TAKES PLACE FOR THESE MEMBERS IN SERIES.

FORM D S C - 44

Park GWMP	NATIONAL PARK SERVICE DENVER SERVICE CENTER		Sheet 68
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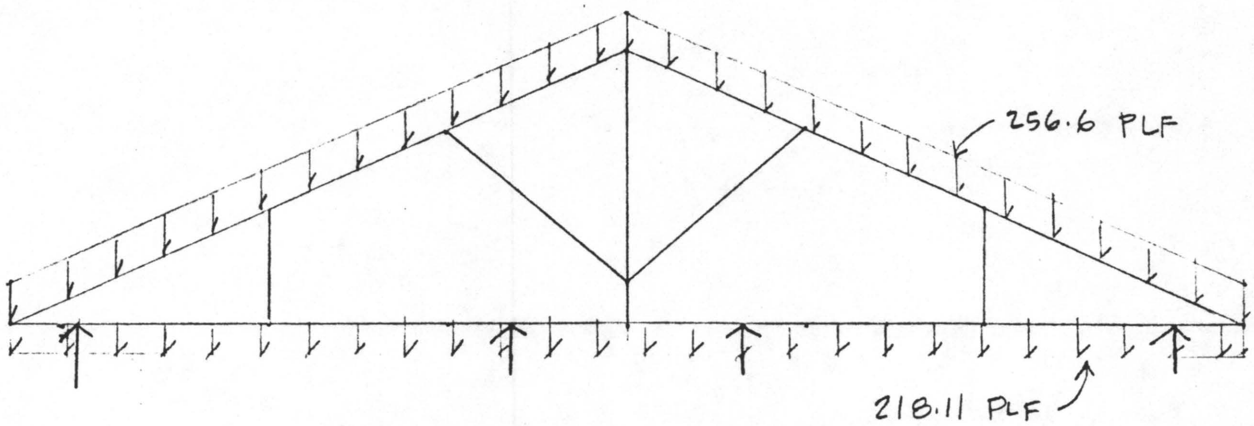
Feature ROOF TRUSS ANALYSIS	Date 1/1/82	Date	Account

TRUSS # 1 – ANALYSIS OF ABILITY TO SUPPORT DEAD LOADS ONLY.



ROOF LOADS

· TRUSS *	3.75	PSF
· SHINGLES	10	"
· SHEATHING	2.5	"
· RAFTERS	1.7	"
· PURLINS	1.8	"
	19.75	"

SAY 20 PSF

FLOOR LOADS

· CEILING JOISTS	4 PSF
· PLASTER CEILING	10 "
· BOARD FLOORING	3 "
	17 PSF

* Pg. 49, PARKER'S Simplified Design of Roof Trusses for Architects & Builders.

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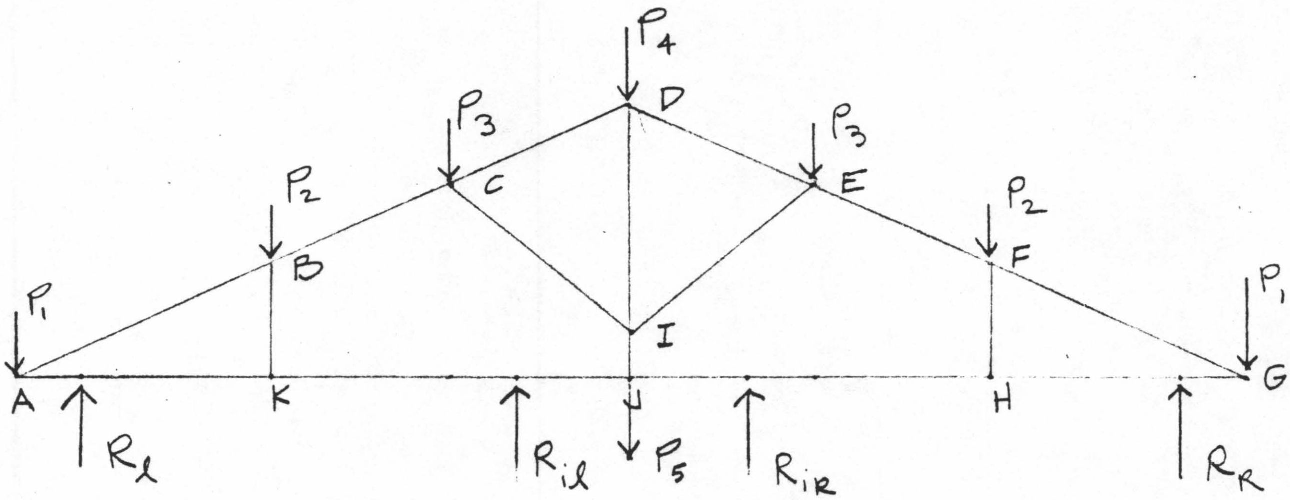

$P_1 = (14.55)(256.6)/2 + \frac{32}{2}(218.11) = 5356.525\#$

$P_2 = (256.6)(24.85)/2 = 3188.26\#$

$P_3 = (20.39)(256.6)/2 = 2616.04\#$

$P_4 = (11.03)(256.6) = 2830.30\#$

$P_5 = (64)(218.11)/2 = 6979.52\#$

ASSUME $R_L = R_R$ & $R_{iL} = R_{iR}$ (SEE SHT. 3 OF PREVIOUS TRUSS CALCS.)

$R_L + R_{iL} = P_1 + P_2 + P_3 + 1/2\,(P_4 + P_5) = 16.066^K$

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$$\Sigma M_E = 9.25 P_4 + \frac{9.25}{2} P_5 + 18.5 P_3 + 28 P_2 + 41.25 P_1$$
$$- 37.75 R_L - 15.25 R_{1L} + 10 F_{FL}$$

$$F_{FL} = \frac{F_{ED}}{1.092} + \frac{F_{IE}}{1.271}$$ ASSUME $= 1.0^K$

$$\therefore \Sigma M_E = 26.180 + 32.280 + 48.397 + 89.271 + 220.957$$
$$- 37.75 R_L - 15.25 R_{1L} + 10$$
$$= 427.085 - 37.75 R_L - 15.25 R_{1L}$$

$$\Sigma F_V = P_1 + P_2 + P_3 + \tfrac{1}{2}(P_4 + P_5) = 16.066^K$$

$$\therefore \Sigma M_E = 427.085 - 37.75(16.066 - R_{1L}) - 15.25 R_{1L} = 0$$

$$22.5 R_{1L} = 179.396$$
$$R_{1L} = 7.973$$
$$R_L = 8.093$$

JOINT "C"

$$\Sigma F_H = \frac{F_{CI}}{1.271} - \frac{F_{BC}}{1.092} + \frac{F_{CD}}{1.092} = 0$$

$$\Sigma F_V = 0.6168 F_{CI} + 0.401 F_{BC} - P_3$$
$$- 0.401 F_{CD} = 0$$

$$F_{BC} = F_{CD} + \frac{1.092}{1.271} F_{CI}$$

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Feature ROOF TRUSS ANALYSIS	Date 1/13/82	Date	Account

$$\therefore \Sigma F_V = 0.6168\,F_{CI} + \cancel{0.401\,F_{CD}} + \frac{(0.401)(1.092)}{1.271} F_{CI} - P_3 - \cancel{0.401\,F_{CD}}$$

$$0.9613\,F_{CI} = P_3$$

$$F_{CI} = 2.721^K = F_{EI}$$

$$\therefore F_{FL} = 2.141 + 0.916\,F_{ED}$$

IF $F_{FL} = 1.0^K$

$$F_{ED} = -1.246^K = 1.246^K\ (T)$$

$$\therefore \Sigma M_A = 3.5\,R_L + 26\,R_{IL} - \frac{32}{2} P_5 - 32\,P_4 - 22.75\,P_3 - 13.25\,P_2 - 32(0.401)\,F_{DE} - \frac{14}{1.092} F_{DE} - 32(0.6168)(2.721) + \frac{2.25}{1.271}(2.721)$$

$$= 28.326 + 207.298 - 111.672 - 90.570 - 59.515 - 42.244 - 15.989 - 15.974 - 53.706 + 4.817$$

$$= -149.230 \text{ Ft-K}$$

$\therefore$ ASSUMPTION THAT $F_{FL} = 1.0^K$ NO GOOD

TO ADJUST, ...

ASSUME $F_{FL} = 16.0^K$

$$\therefore F_{ED} = 4.213^K\ (C)$$

$$\therefore \Sigma M_E = 417.085 + 60 - 606.492 + 22.5\,R_{IL}$$

$$R_{IL} = 5.751^K$$

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Feature ROOF TRUSS ANALYSIS	Date 1/13/82	Date	Account

$R_L = 10.315^K$

CHECK:

$\Sigma M_A = 36.102 + 149.526 - 111.672 - 90.570 - 59.515 - 42.244 + 54.061 + 54.013 - 53.706 + 4.817$

$= -59.188$ Ft-K

No Good

ASSUME $F_{FL} = 9.0K$

$\therefore F_{ED} = 7.488^K$

$\therefore \Sigma M_E = 417.085 + 90 - 606.492 + 22.5 R_{IL}$

$22.5 R_{IL} = 99.407$

$R_{IL} = 4.418^K$

$R_L = 11.648^K$

CHECK:

$\Sigma M_A = 40.768 + 114.868 - 111.672 - 90.570 - 59.515 - 42.244 + 96.086 + 96 - 53.706 + 4.817$

$= -5.168$ Ft-K

No. Good

F_{FL}	F_{ED}	ΣM_A
1	1.246^K (T)	-149.230
6	4.213^K (C)	-59.188
9	7.488^K (C)	-5.168

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$$\frac{\Delta F_{FL}}{\Delta \Sigma M_A} = \frac{3}{54.024} = 0.0555$$

For $\Delta \Sigma M_A = 5.168$, $\Delta F_{FL} = 0.287^K$

ASSUME $F_{FL} = 9.287^K$

$F_{ED} = 7.801^K$ (c)

$\therefore \Sigma M_E = 417.085 + 92.87 - 606.492 + 22.5 R_{IL}$

$22.5 R_{IL} = 96.537$

$R_{IL} = 4.291^K$

$R_L = 11.775^K$

CHECK:

$\Sigma M_A = 41.214 + 111.565 - 111.672 - 90.570 - 59.515$

$- 42.244 + 100.102 + 100.013 - 53.706 + 4.817$

$= 0.004$ Ft.K OK

MEMBER BY MEMBER ANALYSIS

$F_{BK} = F_{FH} = P_2 = 3.188^K$

$F_{ED} = F_{CD} = 7.801^K$

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$$F_{BC} = F_{EF} = F_{CD} + \frac{1.092}{1.271} F_{CI} = 10.139^{K}$$

$$F_{AB} = F_{FG} = 10.139^{K} \ (C)$$

$$F_{AJ} = F_{GJ} = 9.285^{K} \ (T)$$

$$F_{DI} = 2(0.401)(7.801) - 2.830 = 3.426^{K} \ (T)$$

$$F_{JI} = 2(.6168)(2.721) - 3.426 = -0.070^{K} = 0.070^{K} (T)$$

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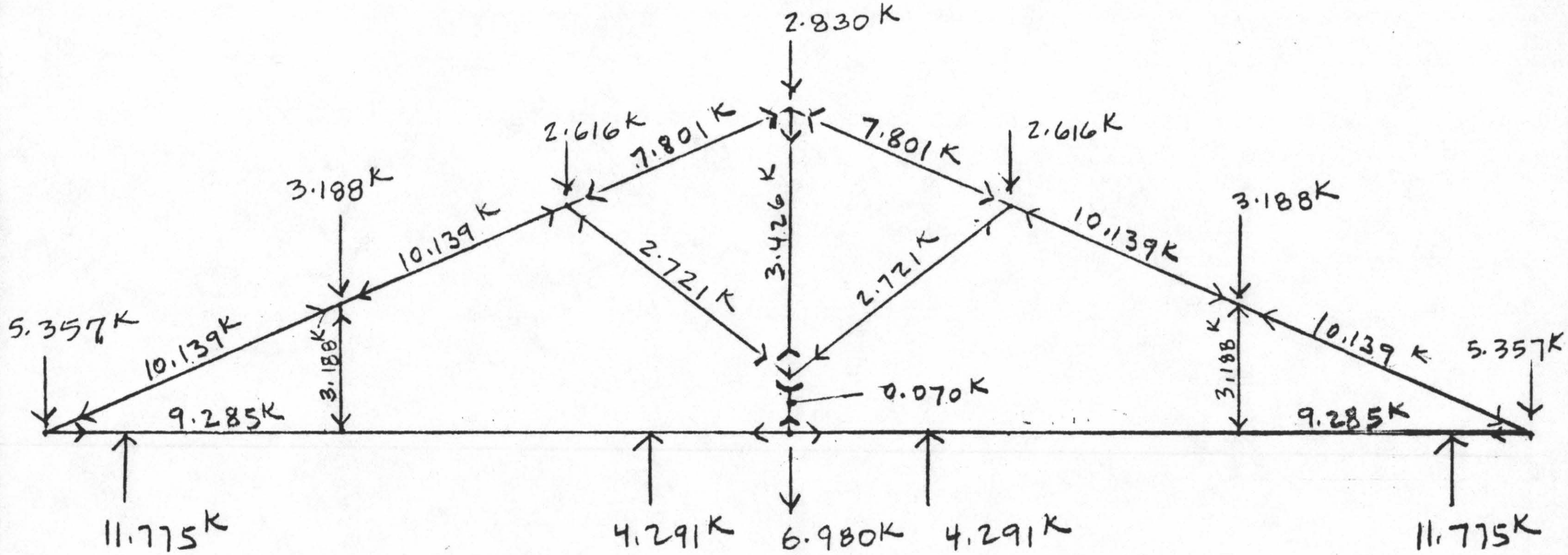
Feature ROOF TRUSS ANAL



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MEMBER STRESS ANALYSIS :- ALLOWABLE STRESSES FOR EASTERN WHITE PINE SELECT STRUCT. No. 1 (BEAMS)

INCREASE 1.25 FOR SNOW LOADS

F_b = 1050 psi		F_b = 1312 psi
F_t = 700 "		F_t = 875 "
F_v = 65 "	INCREASE 1.25 FOR SNOW LOADS	F_v = 81.25 "
$F_{c\perp}$ = 220 "		$F_{c\perp}$ = 275 "
F_c = 675		F_c = 844 "
E = 1100 KSI		E = 1375 KSI

MEMBER PROPERTIES :-

b	x	d		A(in^2)	S(in^3)	I(in^4)
5½		6	PURLIN	33.0	33.0	99.0
3		6½	CEILING JOIST	19.5	21.12	65.66
3½		3½	RAFTERS	12.25	7.14	12.51
4½		9	TOP CHORD	40.5	60.75	273.38
6		9	BOTTOM CHORD	54.0	81.00	364.50
6		9	KING POST	54.0	81.00	364.50
6		7	VERT. WEB MEMB	42.0	49.00	171.50
5½		5½	DIAG " "	30.25	27.73	76.26

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WEB MEMBERS – AXIAL LOADS ONLY

· MEMBERS BK & FH – 6"×7" ; L = 6' ; P = 3.188

$f_c = \frac{3188}{(6)(7)} = 75.9 << 666.3$ psi

OK

$l/d = \frac{6 \times 12}{6} = 12$

$K = 0.671\sqrt{\frac{E}{F_c}} = 27.09$

$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$

$= 666.3$ psi

· MEMBERS CI & EI ; L = 12.07' ; P = 2.721K

$f_c = \frac{2721}{(5.5)^2} = 89.95$ psi

89.95 < 474.21 psi

OK

$l/d = \frac{(12.07)(12)}{5.5} = 26.33$

$K = 27.09 > 26.33$

$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$

$= 474.21$ psi

· MEMBER DI ; L = 11.75' ; 6"×9" ; P = 3.246K

$f_t = \frac{3246}{(6)(9)} = 60.1$ psi.

60.1 < 875 psi OK

$l/d = \frac{11.75 \times 12}{6} = 23.5$

$F_t = 875$ psi

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- MEMBER IJ ; $L = 2.25'$; $L = 6'' \times 15''$; $P = 0.070^K$

$f_c = \frac{70}{(6)(15)} = 1.3$ psi < 875 psi

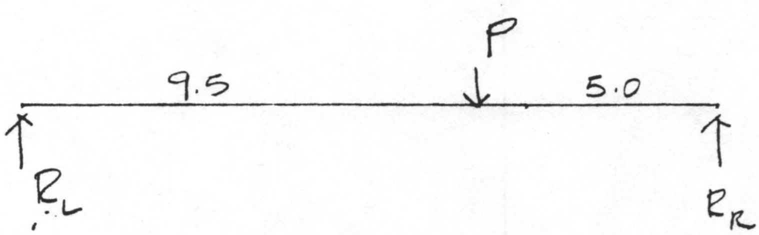
OK

$l/d = \frac{(2.25)(12)}{6} = 4.5$

$F_t = 875$ psi

MEMBERS SUBJECTED TO AXIAL LOADS & FLEXURE

- TOP CHORD MEMBERS AB & FG ; $4\frac{1}{2}'' \times 9''$; $L = 9.5$
  (LATERAL SUPPORT PROVIDED BY PURLINS) $P_A = 10.139^K$



$l/d = \frac{(14.5)(12)}{9} = 19.33$

$F_c' = F_c\left[1 \cdot \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$

$= 616.76$ psi

$J = \frac{l/d - 11}{K - 11} = 0.52$

$P = \left[(14.5)(12.83)(20)\right] \frac{1}{2} = 1860.4$ lbs

$\Sigma M_L = 9.5P - 14.5 R_R = 0$

$R_R = 1218.5$ lbs.

$R_L = 641.55$ "

$M_{MAX.} = (5.0)(1218.85) = 6094.25$ Ft-lbs

$f_b = \frac{6094.25}{60.75}(12) = 1203.8$ psi > 1050 psi

No GOOD

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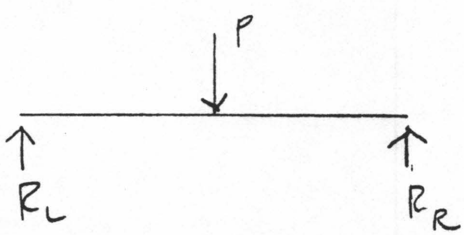
• TOP CHORD MEMBERS BC & EF ; 4½" × 9" ; L = 10'

$P_{AX.} = 10.139^{K}$



$l/d = \frac{(10)(12)}{9} = 13.33$

$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$

$= 661.83$ psi

$J = \frac{l/d - 11}{K - 11} = 0.145$

$P = (10)(12.83)(20)/2 = 1283$ lbs.

$R_L = R_R = 641.5$ lbs

$M_{MAX} = (5)(641.5) = 3207.5$ Ft-lbs.

$f_b = \frac{3207.5 \quad (12)}{60.75} = 633.6$ psi

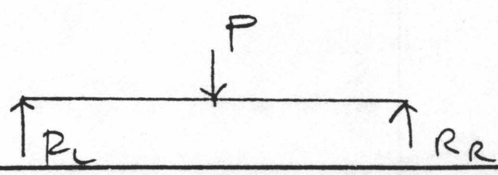
$f_c = \frac{10139}{(4.5)(9)} = 250.3$ psi

OK

$\frac{f_c}{F_c'} + \frac{f_b}{F_b - J f_c} = \frac{250.3}{661.83} + \frac{633.6}{1050 - 36.29} = 1.00$

• TOP CHORD MEMBERS CD & DE ; 4½" × 9 ; L = 10'

$P_A = 7.801^{K}$



$l/d = 13.33$

$F_c' = 661.83$ psi

$J = 0.145$

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$P = (10)(12.83)(20)/2 = 1283$ lbs.

$R_L = R_R = 641.5$ lbs.

$M_{MAX} = (5)(641.5) = 3207.5$ Ft-lbs

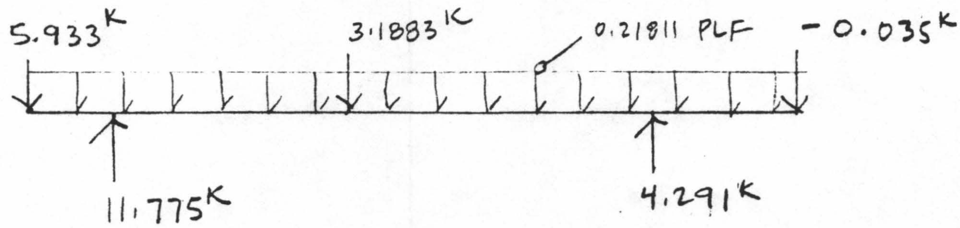
OK

$f_b = \frac{3207.5}{60.75}(12) = 633.6$ psi ; $f_c = 192.96$ psi

$\frac{f_c}{F_c'} + \frac{f_b}{F_b - Jf_c} = \frac{192.96}{661.83} + \frac{633.6}{1013.71} = 0.91 < 1$

- BOTTOM CHORD MEMBERS AJ & JG ; P = 9.285 K (T)

6" x 9"



$\Sigma M_A = (0.21811)(32)^2/2 + 13.25(3.1883) - 32(0.035)$

$- 3.5(11.775) - 26(4.291) = 0.048$ Ft-lbs OK

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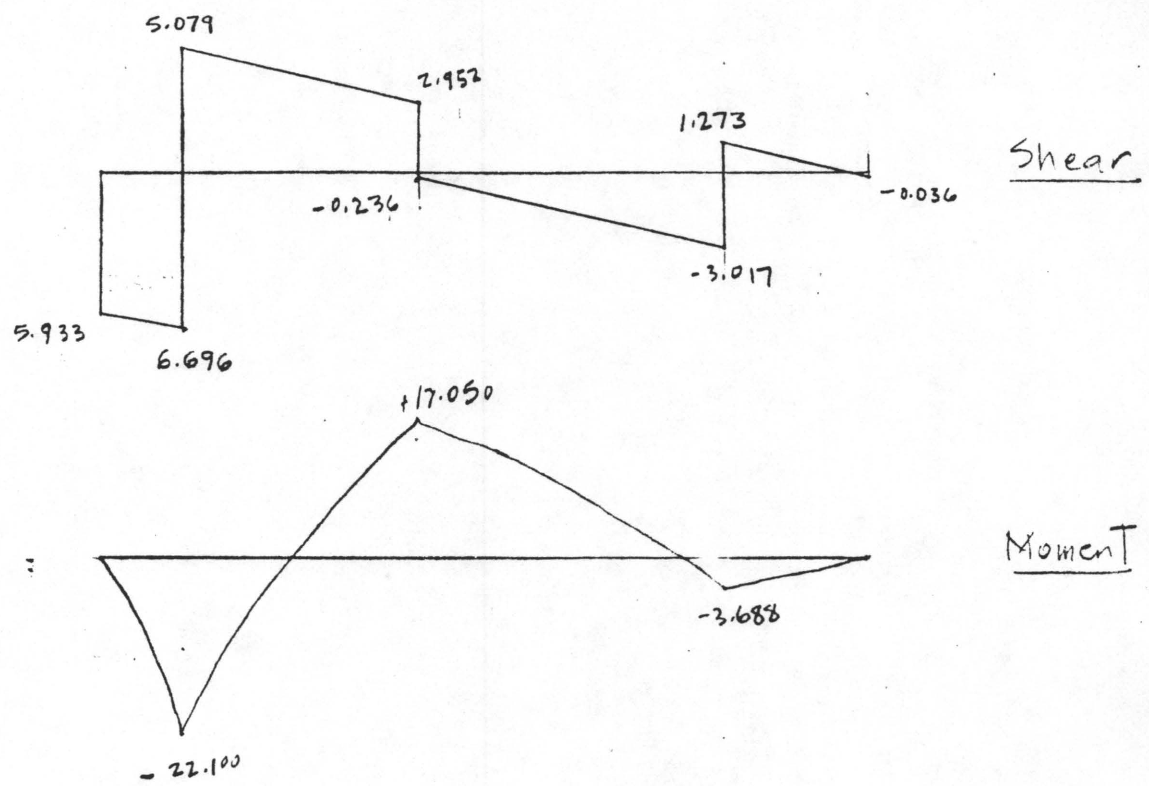

M_{MAX} = 22 100 ft-lbs.

f_b = 3274.1 psi

$f_t = \frac{9285}{54} = 171.94$ psi

$\frac{f_t}{F_t} + \frac{f_b}{F_b} = 2.70$ No GOOD - MEMBER UNDERSIZED

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DETERMINE ADEQUACY OF PURLINS (NEW & EXIST.) SPACED 2.5 ft O.C.

LOADS

SHINGLES	10 PSF	
SHEATHING	2.5 "	
RAFTERS	1.7 "	
	14.2 PSF x 2.5	= 35.5 PLF
PURLINS		9.1 "
SUB-TOTAL		44.6 "
LIVE (SNOW)	20 x 2.5	= 50
		94.6 "

$R_l = R_r = 12.83\,(94.6)/2 = 606.86$ lbs.

$M_{MAX} = (94.6)(12.83)^2/8 = 1946.50$ ft-lbs

$f_b = (1946.50)(12)/33 = 707.82$ psi

$f_b/F_b = \dfrac{707.82}{1050} = 0.67$

OK

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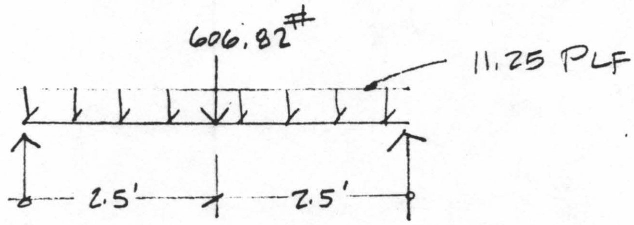
Feature ROOF TRUSS ANALYSIS	Date 1/13/82	Date	Account

DETERMINE MAX. SPAN OF TOP CHORD TO SUPPORT DESIGN LIVE AND DEAD LOADS.

WGT. OF TOP CHORD = 11.25 PLF

WGT. OF PURLINS, RAFTERS, SHEATHING, SHINGLES, AND 20 PSF LIVE LOAD = 606.82# 2.5' O.C.



$$M_{MAX.} = 2.5\,(606.82)/2 + (11.25)(5)^2/2 = 899.15 \text{ Ft-lbs}$$

$$f_b = \frac{899.15\,(12)}{60.75} = 177.61 \text{ psi}$$

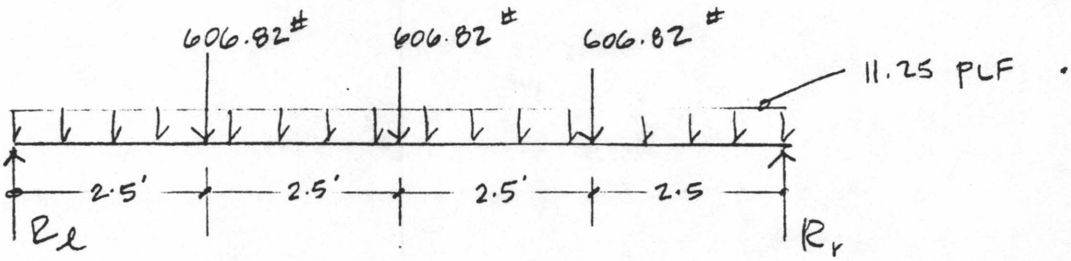
$$f_{c\,(max.)} = \frac{9328^{*}}{(4.5)(9)} = 230.32 \text{ psi}$$

* SHT 23 - LOADING COND. 2

$$\frac{f_c}{F_c} + \frac{f_b}{F_b} = \frac{230}{840} + \frac{177.61}{1310} = 0.409$$

OK

TRY



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Feature ROOF TRUSS ANAL.	Date 1/13/82	Date	Account

$$M_{MAX} = 2.5\,(606.82) + 5.0\,(303.41) + \frac{(11.25)(10)^2}{2}$$

$$= 3596.6 \text{ Ft-lbs}$$

$$f_b = \frac{3596.6\,(12)}{60.75} = 710.44 \text{ psi}$$

$$f_{c_{max}} = 230.32 \text{ psi}$$

$$\frac{f_c}{F'_c} + \frac{f_b}{F_b - Jf_c} = \frac{230.3}{823.6} + \frac{710.44}{1310 - 33.33}$$

$$= 0.836 \leq 1 \quad \text{Ok}$$

$$l/d = \frac{(10)(12)}{9} = 13.33$$

$$J = \frac{(l/d) - 11}{K - 11} = 0.145$$

$$F'_c = 840\left[1 - \tfrac{1}{3}\left(\frac{l/d}{K}\right)^4\right]$$

$$= 823.61 \text{ psi}$$

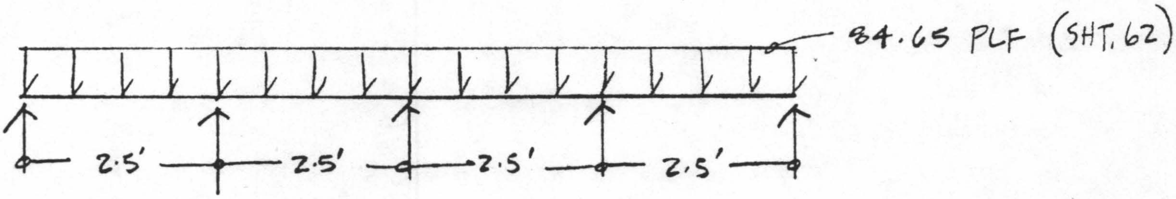
∴ TOP CHORD OF TRUSS TO BE SUPPORTED BY VERT. BRACES A MIN OF TEN FEET APART. PURLINS (5½" × 6" OR GREATER) TO BE SPACED 2.5' ON CENTER.

DETERMINE ADEQUACY OF RAFTERS.

ASSUME WORST CONDITIONS. – $A = 7.3125 \text{ in}^2$

$S = 3.96 \text{ in}^3$

$I = 6.44 \text{ in}^4$



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Feature ROOF TRUSS ANAL.	Date 1/13/82	Date	Account

$$M_{MAX} = 0.1071\, w l^2 = 56.66 \text{ Ft-lbs.}$$

$$f_b = \frac{56.66\,(12)}{3.96} = 171.7 \text{ psi}$$

$$F_b = 1310 \text{ psi}$$

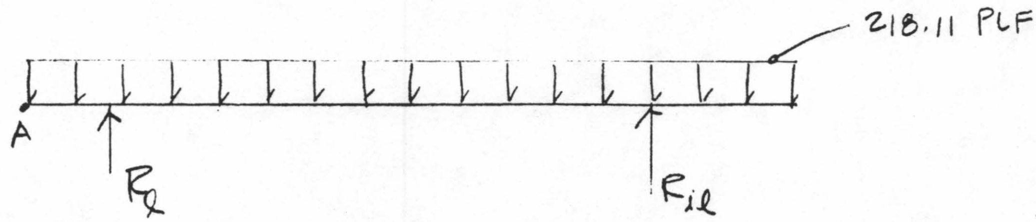
OK

RAFTERS O.K. for a span of 5 ft between supports.

DETERMINE ADEQUACY OF BOTTOM CHORD TO SUPPORT:

A.) JUST FLOOR DEAD LOAD

B) JUST ROOF DEAD LOAD



$$R_l + R_{il} = wl = (0.218)(32) = 6.98 \text{ K}$$

$$\Sigma M_A = 3.5\, R_l + 26\, R_{il} - wl^2/2$$

$$= 3.5\,(6.98 - R_{il}) + 26\, R_{il} - 111.67$$

$$22.5\, R_{il} = 87.24$$

$$R_{il} = 3.88^K$$

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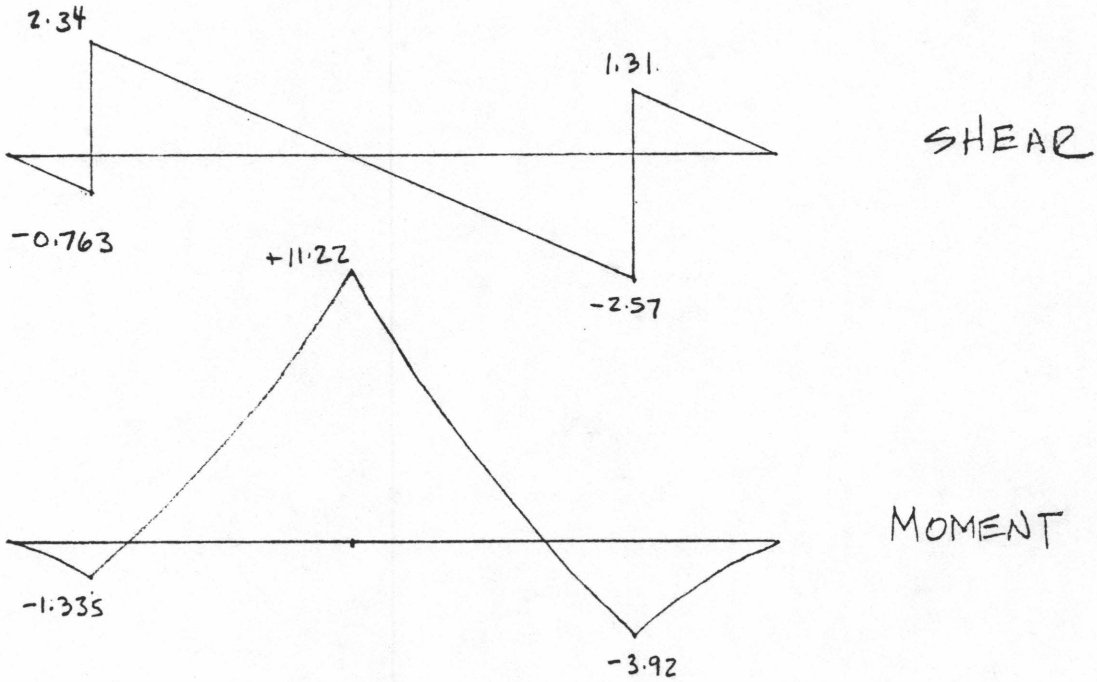
$R_l = 6.98 - R_{il} = 3.10^K$



$M_{MAX} = 11220$ Ft-lbs

$f_b = 11,220\ (12)/81 = 1662$ psi $>$ 1050 psi

No. GOOD

DOES NOT INCLUDE AXIAL LOADS.

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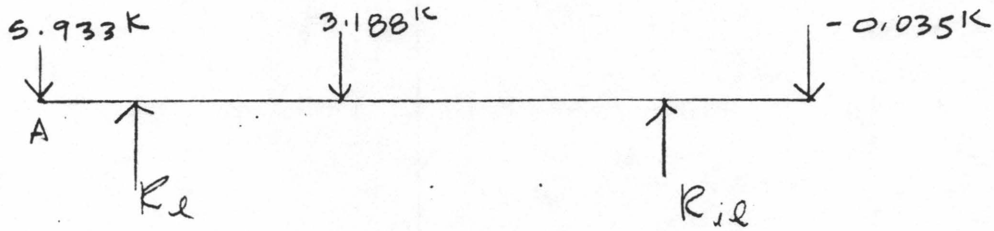
B.) ROOF DEAD LOADS ONLY



$$R_l + R_{il} = 5.933 + 3.188 - 0.035 = 9.086^K$$

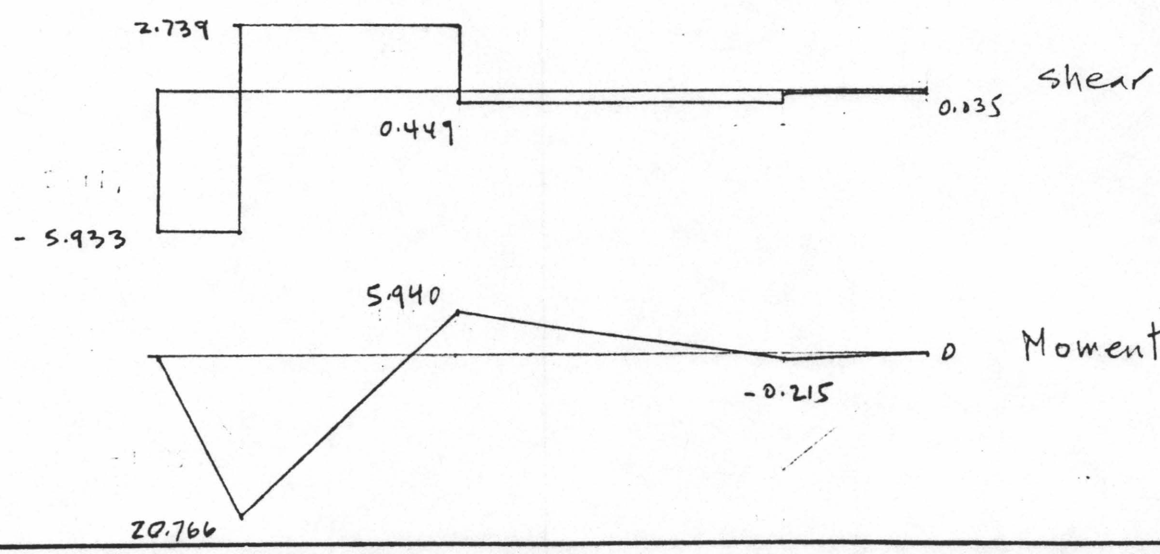
$$\Sigma M_A = 3.5 R_l + 26 R_{il} - 13.25(3.188) + 32(0.035)$$

$$= 3.5(9.086 - R_{il}) + 26 R_{il} - 41.121$$

$$22.5 R_{il} = 9.32$$

$$R_{il} = 0.414^K$$

$$R_l = 8.672^K$$



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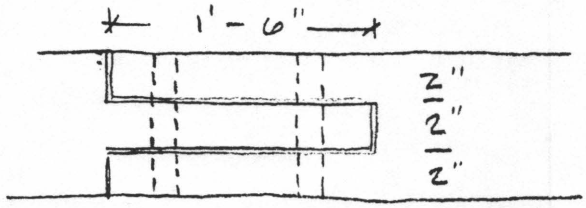
Feature ROOF TRUSS ANALYSIS	Date 1/13/82	Date	Account

$M_{MAX} = 20766$ ft-lbs

$f_b = (20766)(12)/81 = 3076.44 > 1050$ psi

NO GOOD

ADEQUACY OF BOTTOM CHORD SPLICES



NET SECTION $= 2 \times 9 - (2)(1)$

$= 16$ in^2

MAX. TENSILE $= 15.636$ K

actual tensile stress = 977 psi

$F_t = 875$ psi

NO GOOD

BENDING

FORCE ON PINS PRODUCING BENDING $= 15{,}636/2$

$= 7{,}818$ lbs

BENDING MOMENT $= 7{,}818 \times 2 = 15{,}635$ in-lbs

$F_b = 1310$ psi

$\therefore \frac{15{,}635}{1310} = 11.936$ in^3 $= S_{req'd}$

NO GOOD

$S_{act} = 2(\pi)(1)^3/32 = 0.196$ in^3

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Feature HEEL JOINT CONNECTION	Date 1/25/82	Date	Account

THESE FIGURES ARE BASED ON THE ASSUMPTION THAT THE ENTIRE TENSILE FORCE AT "A" EXISTS AT THE SPLICE. IN REALITY, FRICTION AT THE SUPPORTS WILL REDUCE THE TENSILE STRESS SIGNIFICANTLY. HOWEVER, THE TWO WOOD DOWELS SEEM TO BE TOO SMALL.

FOR SHEAR

Shear area $= \pi \frac{(1)^2}{4} = 0.785 \text{ in}^2$

$F_v = 80\text{psi}$

allowable shear stress for one pin, single shear

$= (80)(0.785) = 62.83 \text{ lbs}$

in double shear

$= (2)(62.83) = 125.66 \text{ lbs.}$

for two dowels,

$= (2)(125.66) = 251.32 \text{ lbs.}$

Max. force $= 15.636^k$

No Good

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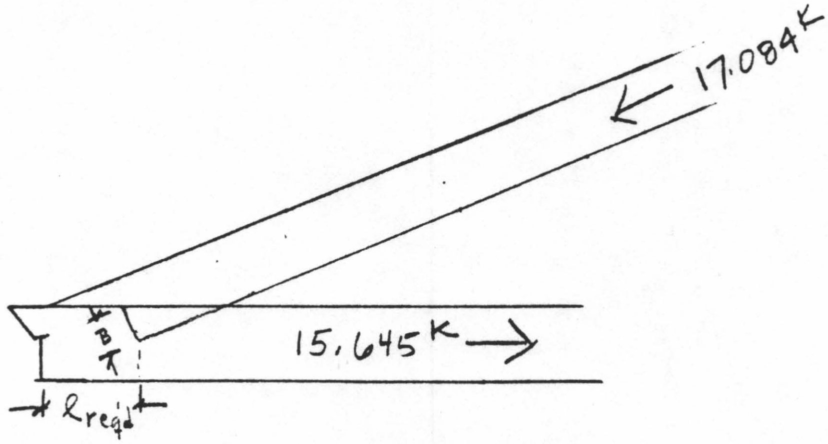
Feature ROOF TRUSS ANALYSIS	Date 2/3/82	Date	Account

ADEQUACY OF HEEL JOINT



Force to be resisted by horz. shear = 15.645^K

allowable horz. shear = 85 psi

∴ REQD. SURFACE AREA = $15,645/85$ = 184.055 in^2

ASSUMING THE WHOLE BOTTOM CHORD ACTS IN SHEAR,

THE REQD. LENGTH IS $184.055/6$ = 30.676 in

SAY 31" = l_{reqd}

No way this occurs

∴ NO NEED TO CALCULATE "B" AS EVEN IF THE LOWER CHORD COULD PROVIDE ADEQUATE COMPRESSION, IT WOULD FAIL IN SHEAR AND POSSIBLY IN TENSION.

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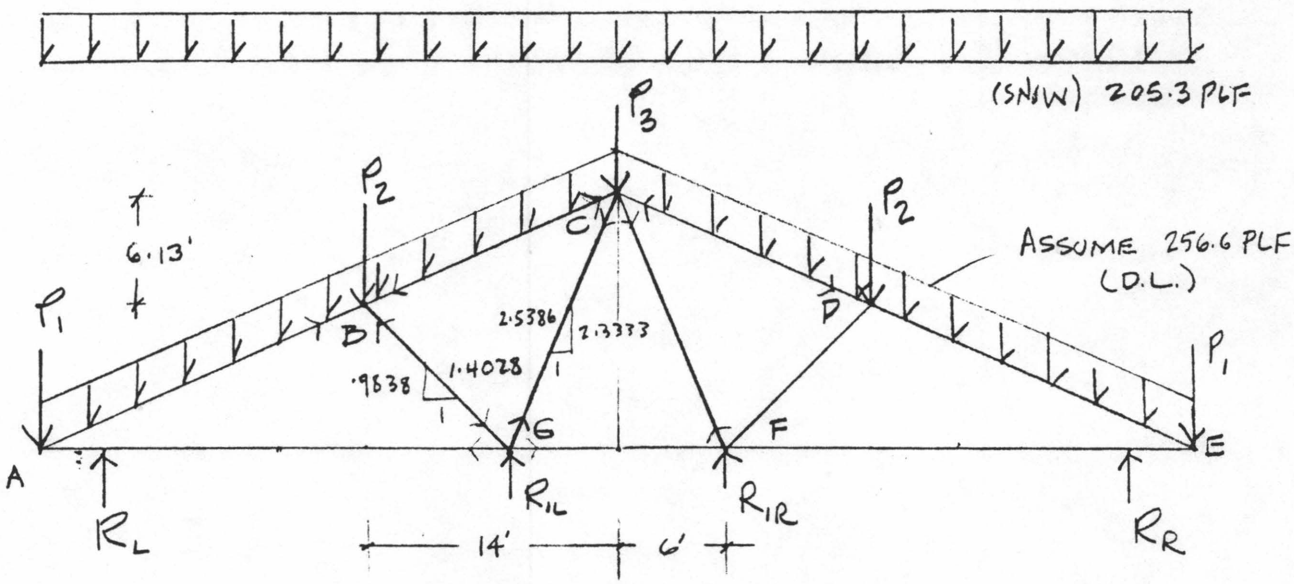
Feature ROOF TRUSS ANALYSIS	Date 1/29/82	Date	Account

DESIGN FINK TRUSS TO SUPPORT ROOF LOADS FOR LOADING CONDITION No 1



$P_1 = (18)(205.3)/2 + (18)(1.092)(256.6)/2 = 4.3696^K$

$P_2 = (32)(205.3)/2 + (32)(1.092)(256.6)/2 = 7.7681^K$

$P_3 = (14)(205.3) + (14)(1.092)(256.6) = 6.7971^K$

$R_L + R_{1L} = P_1 + P_2 + 1/2\, P_3 = 15.536^K$

$\Sigma M_C = 6 R_{1L} - 14 P_2 - 32 P_1 + 28.5 R_L - 14 F_{FG}$

FOR ABCGA

$\Sigma F_H = F_{CD}/1.092 + F_{CF}/2.5386 - F_{GF} = 0$

FOR ABCGA

$F_{FG} = F_{AB}/1.092 - F_{BG}/1.4028 - F_{CG}/2.5386$ (FROM SHT. 2)

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JOINT "B"

$$\Sigma F_H = F_{BG}/1.4028 + F_{BC}/1.092 - F_{AB}/1.092$$

$$\Sigma F_V = \frac{.9838}{1.4028} F_{BG} + 0.401 F_{AB} - 0.401 F_{BC} - P_2$$

$$F_{AB} = F_{BC} + \frac{1.092}{1.4028} F_{BG} = F_{BC} + 0.7784 F_{BG}$$

$$\therefore \Sigma F_V = \frac{.9838}{1.4028} F_{BG} + \frac{0.438}{1.4028} F_{BG} + 0.401 F_{BC} - 0.401 F_{BC} - P_2$$

$$= 1.0135 F_{BG} - P_2 = 0$$

$$F_{BG} = 7.6649^K$$

$$F_{AB} = F_{BC} + 5.9667^K$$

JOINT "G"

$$\Sigma F_V = \frac{0.9838}{1.4028} F_{BG} - R_{IL} - \frac{2.3333}{2.5366} F_{CG}$$

$$= 5.3755 - R_{IL} - 0.9199 F_{CG}$$

$$F_{CG} = +5.8439 - 1.0871 R_{IL}$$

$$\Sigma M_A = 18 P_2 + 32 P_3 - 3.5 R_L - 26 R_{IL} + \frac{32\,(2.3333)}{2.5386} F_{CF}$$

$$+ \frac{14\,(1)}{2.5386} F_{CF} - 32(.401) F_{CD} - \frac{14\,(1.0)}{1.092} F_{CD} = 0$$

$$= 139.826 + 217.507 + 34.927 F_{CF} - 25.6525 F_{CD}$$

$$- 3.5 R_L - 26 R_{IL} = 0$$

$$= 139.826 + 217.507 + 34.927 (5.8439 - 1.0871 R_{IL})$$

$$- 25.6525 F_{CD} - 3.5 (15.536 - R_{IL}) - 26 R_{IL} = 0$$

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$$\Sigma M_A = 507.067 - 60.469\ R_{1L} - 25.6525\ F_{CD} = 0$$

$$\Sigma M_C = 6(15.536 - R_L) - 108.7534 - 139.8272 + 28.5 R_L - 14 F_{FG}$$

$$= 22.5 R_L - 155.365 - 14\left[F_{AB}/1.092 - F_{BG}/1.4028 - F_{CG}/2.5386\right]$$

JOINT "C"

$$\Sigma F_V = \tfrac{1}{2} P_3 - 0.401 F_{BC} + \frac{2.3333}{2.5386} F_{CG} = 0$$

$$= 3.39855 - 0.401\ F_{AB} + (0.401)(5.9667) + 0.9191\ F_{CG}$$

$$\therefore F_{AB} = 5.7912/0.401 + 2.2921 F_{CG} = 14.442 + 2.2921\ F_{CG}$$

$$\Sigma M_C = 22.5 R_L - 155.365 - \frac{14(14.442)}{1.092} - \frac{14(2.2921) F_{CG}}{1.092} + 76.496$$

$$+ 5.5149\ F_{CG} = 0$$

$$\therefore = 22.5 R_L - 23.871\ F_{CG} - 264.023$$

$$\Sigma M_A = 507.067 - 60.469(15.536 - R_L) - 25.6525\left[F_{AB} - 5.9667\right]$$

$$= -279.318 + 60.469 R_L - 25.6525(14.442 + 2.2921\ F_{GC})$$

$$= -649.792 + 60.469 R_L - 58.7981 F_{CG}$$

$$\therefore R_L = +0.9724\ F_{CG} + 10.746$$

$$\therefore \Sigma M_C = 22.5(+0.9724\ F_{CG} + 10.746) - 23.871\ F_{CG} - 264.023$$

$$1.992 F_{CG} = -22.238$$

$$F_{CG} = -11.164^K = 11.164^K\ (C)$$

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Feature ROOF TRUSS ANAL.	Date 1/29/82	Date	Account

$R_L = -0.110^K$

$R_{IL} = 15.646^K$

JOINT "C" $\quad \Sigma F_V = \frac{1}{2}P_3 + \frac{2.3333}{2.5386} F_{CG} - 0.401 F_{BC} = 0$

$F_{BC} = 17.114^K$ (T)

$F_{AB} = 11.147^K$ (T)

$F_{AG} = +10.208^K$ (C)

$F_{FG} = 11.274^K$ (C)

$F_{BG} = 7.6649^K$

CHECK:

ΣF_H (ABCGA) $= -15.672 + 11.274 + 4.3977 = 0$ OK

CHECK:

$\Sigma M_A = 139.826 + 217.507 + 34.927 F_{CF} - 25.6525 F_{CD} - 3.5 R_L - 26 R_{IL}$

$= 357.333 - 389.925 + 439.016 - 0.385 - 406.796$ OK

$= 0.76$ K-FT

ΣM_A (ACEA) $= 18 P_2 + 32 P_3 + 46 P_2 + 64 P_1 - 3.5 R_L - 26 R_{IL} - 38 R_{IR} - 61.5 R_R$

$= 64(7.7681) + 32(6.7971) + 64(4.3696) - 64(-0.110) - 64(15.646)$

$= 0.016$ K-FT OK

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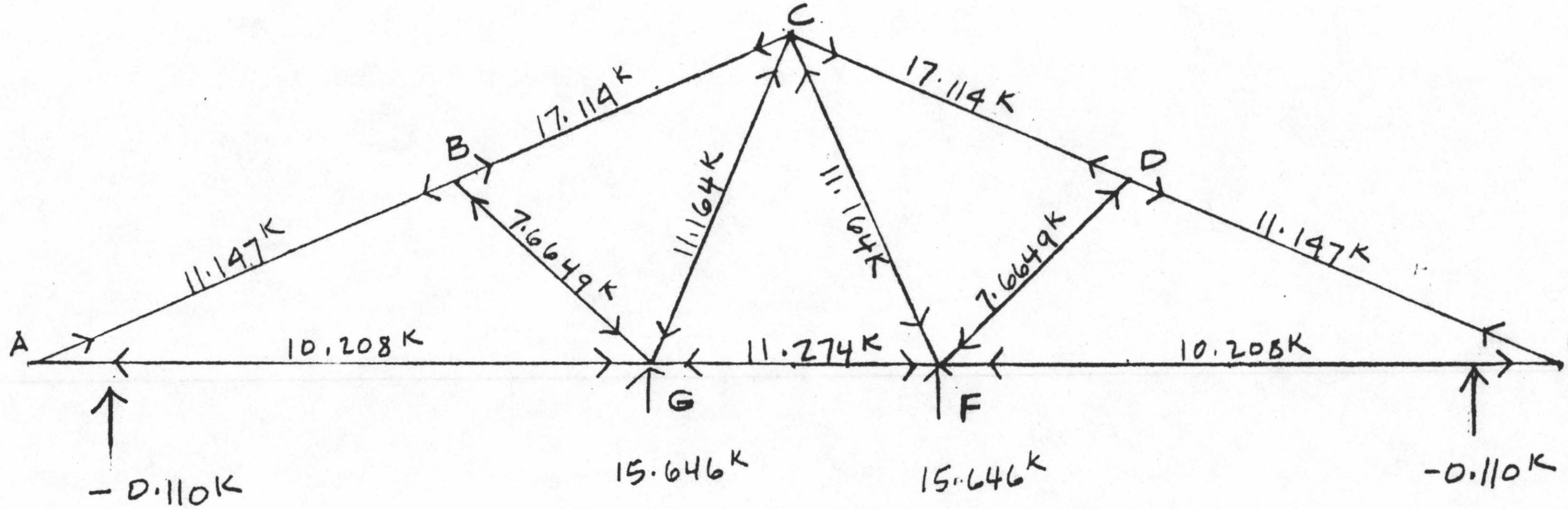

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Feature ROOF TRUSS ANAL.	Date 2/1/82	Date	Account

DESIGN FINK TRUSS FOR LOADS SHOWN ON SHT 95

- WEB MEMBERS BG & DF ; $P = 7.6649^k$ (C)

TRY DOUBLE ANGLES w/ 3/8" PLs & 4½" wood block for spacers ; $l = 8.75'$; $F_y = 36$ ksi

$KL = (1.0)(8.75) = 8.75'$

TRY 2 ∠s 3×3×3/8

(STRUCT. ENGR. HANDBOOK p. 6-13)

$r_x = 0.31\,h = 0.93$ in

$r_y = (4.5 + 0.75 + 6)(0.21 + 0.02(4.5 + 0.75)) = 3.544$ in

GOOD FOR 44^k w/ an effective length of 9 (X-X axis)

$r_x / r_y = 0.93 / 3.544 = 0.262$

EQUIV. EFFECTIVE LENGTH FOR Y-Y axis $= 9' / (0.262)$

OK $= 34.35' > 8.75'$

X-X AXIS CONTROLS.

∠3×3×3/8 OK for 44+k

Check. $b/t = 3 / 3/8 = 8 < 76/\sqrt{F_y} = 12.67$

- WEB MEMBERS CG & CF ; $P_A = 11.164^k$ (C)

$l = 15.25'$

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$KL = 15.25$

TRY $\angle s$ $6 \times 4 \times 1/2$

GOOD FOR 99^k w/ an effective length of 19' (X-X axis)

$r_x = 0.32h = 1.92$

$r_y = b(0.19 + 0.02s) = (4.5 + 0.75 + 8)(0.19 + 0.02(0.75 + 4.5))$

$= 3.909$

OK

EQUIV. EFFECTIVE LENGTH FOR Y-Y AXIS = $19/1.92/3.909 = 38.7'$

CHECK $b/t = 4/1/2 = 8.0 < 12.67$

- BOTTOM CHORDS AG & EF ; $P = 10.208^k$ (c)

$\therefore l = 26'$

BRACING AT INTERVALS OF < 9' (esp. in X-X axis), $\angle s$ $3 \times 3 \times 3/8$ WOULD BE O.K.

- BOTTOM CHORD FG — SAME AS ABOVE

- TOP CHORDS AB & DE ; $l = 19'$; $P = 11.147^k$

ASSUME 3/4"ϕ BOLTS

AREA TO BE DEDUCTED FOR BOLT HOLE = $3/8 \times 7/8 = 0.3281 \text{ in}^2$

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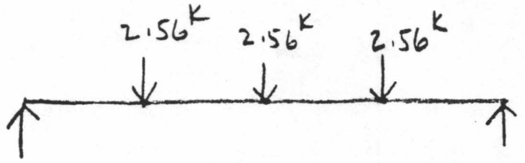
Feature ROOF TRUSS (FINK)	Date 2/1/82	Date	Account



$W = 205.3 + (1.092)(256.6) + \text{WGT OF} \angle\text{S}$

$\cong 510$ PLF

$M = (3.84)(5) + (1.28)(5) = 25.624$ K-FT

TRY 2 $\angle$S $5 \times 3 \times 1/2$

$I = 18.9 \text{ in}^4$

$S = 5.82 \text{ in}^3$

$A = 7.50 \text{ in}^2$

$$f = \frac{P}{A} + (M + Pd)\frac{c}{I}$$

$$d = \frac{d_o}{1 - (P/P_c)} = \frac{d_o}{0.893}$$

$P_c = \pi^2 EI/L^2$

$= 1.041 \times 10^5$ lbs

$P = 11.147^K$

$d = 3.06''$

$$d_o = \frac{5wL^4}{384EI} = 2.73''$$

$c = 3.25''$

tension

$$f = \frac{11.147}{7.50 - 0.328} \times 10^3 + \left[(25.624)(12) - 11.147(3.06)\right]\frac{3.25}{18.9} \times 10^3$$

$= 48659.55$ psi

$f_{all} = 20$ ksi

NO GOOD

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Feature ROOF TRUSS (FINK)	Date 2/1/82	Date		Account

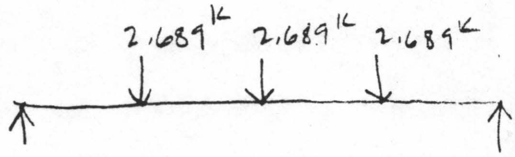
TRY 2 $\angle$S $7 \times 4 \times 3/4''$

$S = 16.8 \text{ in}^3$

$A = 15.4 \text{ in}^2$

WGT = 52.4 PLF

$I = 75.6 \text{ in}^4$



$M_{MAX} = 26.897$ ft-k

$$d = \frac{d_o}{1-(P/P_c)} = 0.989''$$

$P_c = \pi^2 EI/L^2$

$= 3.757 \times 10^5$ lbs

$P = 11.147$

$d_o = \dfrac{5(537.91)(20 \times 12)^4}{384\, EI}$

$= 0.883''$

$c = 4.49''$

$$f = \frac{11.147}{15.4 - 0.328} + \left[(26.897)(12) - 11.147(0.989)\right] \times \frac{4.49}{75.6}$$

$= 19.25$ ksi < 20 ksi

OK

$$\frac{P/A}{F_{ALL}} = \frac{0.74 \text{ ksi}}{20} = 0.04 \leq 0.15$$ OK

- TOP CHORD MEMBERS BC & CD ; $l = 16'$

$P = 17.114^K$

USE 2 $\angle$S $7 \times 4 \times 3/4''$ (LARGER AXIAL FORCE SMALLER MOMENT - OK)

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Feature ROOF TRUSS	Date 2/1/82	Date	Account

Threaded fasteners

$$A_s = \frac{\pi}{4}\left(D - \frac{0.9743}{n}\right)^2$$

Gaylord & GAYLORD, 6-5

FOR WEB MEMBERS BG & DF - AXIAL LOAD = 7.665^K

USING FRICTION TYPE BOLTS (-A325) W/ SHORT SLOTTED HOLES - F_V = 15.0 Ksi

For 3/4"ϕ bolts - All. load in kips for single shear = 6.6^K

Use two 3/4"ϕ bolts - total allowable = 13.2^K

FOR WEB MEMBERS CG & CF - AXIAL LOADS = 11.164^K

USE three 3/4"ϕ BOLTS - ALL LOAD = 3x6.6 = 19.8^K

FOR BOTTOM CHORD MEMBERS AG & FE, AXIAL LOAD = 10.208^K

USE three 3/4"ϕ BOLTS - ALL. LOAD (SHEAR) = 19.8^K

BOTTOM CHORD FG, AXIAL LOAD = 11.274^K

USE three 3/4"ϕ bolts - ALL. LOAD (SHEAR) = 19.8^K

FOR TOP CHORDS AB & DE, AXIAL LOAD = 11.147^K plus

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Feature TRUSS FASTENERS	Date 4/9/82	Date	Account

FOR TRUSS SYSTEM OF DOUBLE ANGLES W/ DOUBLE 3/8" PLs AND WOOD BLOCK SPACER, WE HAVE THE FOLLOWING CONDITIONS;

a) BOLTS IN SINGLE SHEAR (CONSERVATIVE ASSUMP.)

B) GUSSET PLs IN SINGLE BEARING

c) LEGS OF ∠s IN SINGLE BEARING

B.) FOR 3/4" ϕ BOLTS, 3/8" PLs, THE BEARING AREA $= (0.75)(0.375) = 0.28125 \text{ in}^2$

FOR ALL. UNIT BEARING STRESS = 22 ksi,

WORKING VALUE = 6187.5^K

FOR TWO PLs, WORKING VALUE = 12,375 K

∴ TWO BOLTS WOULD BE O.K. FOR ANY JOINT IN OUR TRUSS USING THE GUSSET PLs IN SINGLE SHEAR AS OUR PARAMETER

C.) FOR LEGS OF ∠s IN SINGLE BEARING W/ MIN. THICKNESS OF 3/8", CONDITIONS SAME AS FOR (B.)

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Feature TOP CHORD CONNECTIONS	Date 4/9/82	Date	Account

- MEMBERS AB & DE, AXIAL LOAD = 11.147^K

  USE THREE 3/4" ϕ BOLTS A325 F

  GOOD FOR 19.8^K

- MEMBERS BC & DE, AXIAL LOAD = 17.114^K

  USE THREE 3/4" ϕ BOLTS A325 F

  GOOD FOR AT LEAST 19.8^K

NOTE: THIS ASSUMES NO ECCENTRICITY OR MOMENT AT CONNECTION.

SHOULD VERIFY W/ MAURY

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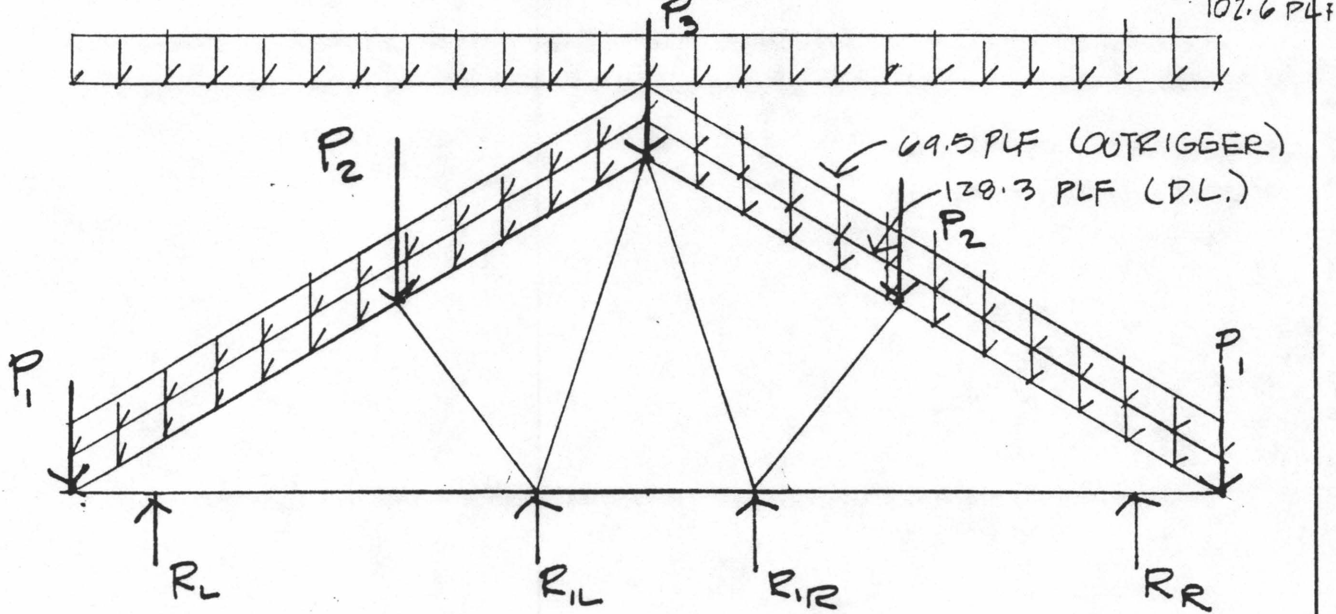
Feature FINK TRUSS FOR "F"	Date 4/20/82	Date	Account

DESIGN FINK TRUSS TO SUPPORT ROOF LOADS ON TRUSS "F" FOR LOADING CONDITION NO. 1



$$P_1 = (18)(102.6)/2 + (18)(1.092)(128.3 + 69.5)/2 = 2867.4\#$$

$$P_2 = (32)(102.6)/2 + (18)(1.092)(128.3 + 69.5)/2 = 3585.6\#$$

$$P_3 = (14)(102.6) + (14)(1.092)(128.3 + 69.5) = 4460.4\#$$

$$R_L + R_{1L} = P_1 + P_2 + \tfrac{1}{2} P_3 = 8683.2 \text{ lbs.}$$

$$\Sigma M_c = 6R_{1L} - 14P_2 - 32P_1 + 28.5R_L - 14F_{FG}$$

FOR ABCGA

$$\Sigma F_H = F_{CD}/1.092 - F_{CF}/2.5386 - F_{GF} = 0$$

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FOR ABCGA

$$F_{FG} = F_{AB}/1.092 - F_{BG}/1.4028 - F_{CG}/2.5386$$ FROM SHT. 91

(SHT. 92)

JOINT "B" $$F_{AB} = F_{BC} + \frac{1.092}{1.4028} F_{BG} = F_{BC} + 0.7784 F_{BG}$$

$$\therefore \Sigma F_V = 1.0135 F_{BG} - P_2 = 0$$

$$F_{BG} = 3585.6/1.0135 = 3537.84 \text{ lbs.}$$

$$F_{AB} = (F_{BC} + 2753.85) \text{lbs}$$

JOINT "G" $$\Sigma F_V = 2481.13 - R_{1L} - 0.9199 F_{CG}$$

$$F_{CG} = 2697.2 - 1.0871 R_{1L}$$

$$\Sigma M_A = 18 P_2 + 32 P_3 - 3.5 R_L - 26 R_{1L} + \frac{32(2.3333)}{2.5386} F_{CF}$$
$$+ \frac{14(1)}{2.5386} F_{CF} - 32(0.401) F_{CD} - \frac{14(1.0)}{1.092} F_{CD} = 0$$

$$= 64,540.8 + 142,732.8 + 34.927 F_{CF} - 25.6525 F_{CD}$$
$$- 3.5 R_L - 26 R_{1L} = 0$$

$$= 207,273.6 + 34.927 F_{CF} - 25.6525 F_{CD} - 3.5 R_L$$
$$- 26 R_{1L} = 0$$

$$= 207,273.6 + 34.927(2697.2 - 1.0871 R_{1L})$$
$$- 25.6525 F_{CD} - 3.5(8683.2 - R_{1L}) - 26 R_{1L} = 0$$

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$$\Sigma M_A = 271,087.5 - 60.469 R_{1L} - 25.6525 F_{CD} = 0$$

$$\Sigma M_C = 6(8683.2 - R_L) - 50198.4 - 91756.8 + 28.5 R_L - 14 F_{FG}$$

$$= -89,856 + 22.5 R_L - 14\left[F_{AB}/1.092 - F_{BG}/1.4028 - F_{CG}/2.5386\right]$$

JOINT "C" $\quad \Sigma F_V = \frac{1}{2} P_3 - 0.401 F_{BC} + \frac{2.3333}{2.5386} F_{CG} = 0$

$$= 2230.2 - 0.401 F_{AB} + 0.401(2697.2) + 0.9191 F_{CG}$$

$$\therefore F_{AB} = 2.2921 F_{CG} + 3311.78/0.401$$

$$= 2.2921 F_{CG} + 8258.80$$

$$\Sigma M_C = 22.5 R_L - 89,856 - 105,882 - 29.3859 F_{CG} + 35307.8 + 5.5199 F_{CG} = 0$$

$$= 22.5 R_L - 23.871 F_{CG} - 160,430.2$$

$$\Sigma M_A = 271,087.5 - 60.469(8683.2 - R_L) - 25.6525\left[F_{AB} - 2753.85\right]$$

$$= -183,333.784 + 60.469 R_L - 25.6525\left[8258.80 \cdot 2.2921 F_{CG}\right]$$

$$= -395,192.65 + 60.469 R_L - 58.7981 F_{CG}$$

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$\therefore R_L = 0.9724\, F_{CG} + 6535.46$

$\therefore \Sigma M_C = 22.5\,(0.9724\, F_{CG} + 6535.46) - 23.871\, F_{CG}$
$- 160{,}430.2$

$1.992\, F_{CG} = -13{,}382.4$

$F_{CG} = -6718.1$ lbs

$= 6718.1$ lbs (C)

$R_L = 2.82$ lbs

$R_{1L} = 8683.2 - 2.82$

$= 8680.4$ lbs.

JOINT "C"

$$\Sigma F_Y = 1/2\, P_3 + \frac{2.3333}{2.5386} F_{CG} - 0.401\, F_{BC} = 0$$

$$F_{BC} = \left[2230.2 + \frac{2.3333}{2.5386}(-6718.1)\right] / 0.401$$

$= 9836.9$ lbs (T)

$F_{AB} = -9836.9 + 2753.85$

$= -7083.05$ lbs $= 7083.05$ lbs (T)

$F_{AG} = 6486.3$ lbs (C)

$F_{FG} = 2646.4 - 2522.0 - 6486.3 = 6361.9$ lbs (T)

$F_{BG} = 3537.84$ lbs (C)

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"CHECK"

$\Sigma F_H = -9008.2 + 6361.9 + 2646.38 = 0$ OK

ABCGA

"CHECK"

$\Sigma M_A = 207,273.6 - 234,643.1 + 252,341.1 - 9.87$
$- 225,690.40 = 0.729$ ft.K OK

"CHECK"

$\Sigma M_A = 18 P_2 + 32 P_3 + 46 P_2 + 64 P_1 - 3.5 R_L - 26 R_{1L}$
ACEA
$-38 R_{1R} - 61.5 R_R = 0$

$= 229,478.4 + 142,732.8 + 183,513.6 - 180.5$
$- 555545.6$

$= -1.3$ ft-lbs OK

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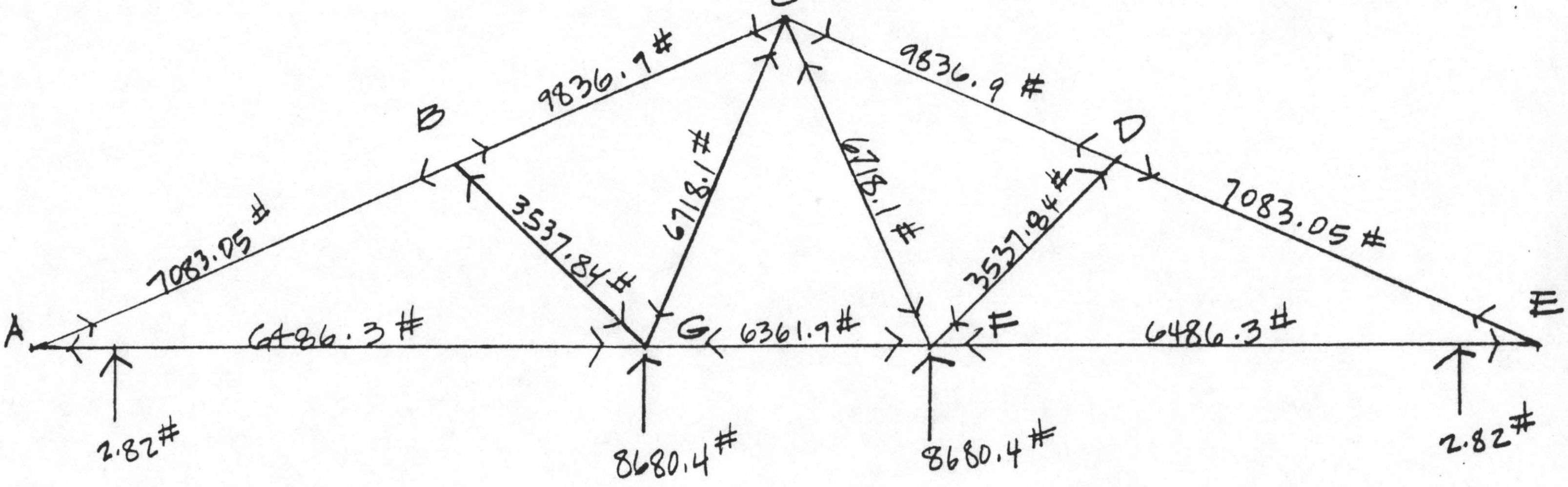

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WEB MEMBERS BG & DF ; P = 3537.8 lbs (C)

$l = 8.75'$; $F_y = 36$ KSI

$KL = (1.0)(8.75) = 8.75'$

USE 2 $\angle$s 3 x 3 x 1/4

GOOD FOR 11^K W/ AN EFFECTIVE LENGTH OF 15 Ft. (X-X AXIS).

CHECK: $b/t = 3/1/4 = 12 < 76/\sqrt{F_y} = 12.67$ OK

WEB MEMBERS CG & CF ; $P_A = 6718.1$ lbs (C)

$l = 15.25'$, $KL = 15.25'$

USE 2 $\angle$s 3½ x 3½ x 1/4

GOOD FOR 13^K W/ AN EFFECTIVE LENGTH OF 18 Ft (X-X AXIS)

CHECK: $b/t = 3.5/1/4 = 14 \not< 12.67$ NO GOOD

TRY 2 $\angle$s 3½ x 3½ x 5/16

$b/t = 3.5/5/16 = 11.2 < 12.67$ OK

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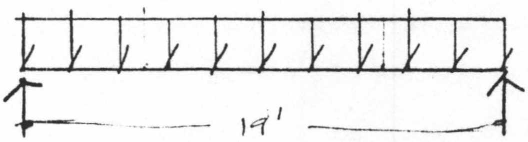
- BOTTOM CHORD AG & EF ; P = 6486.3 lbs. (C) ; $l = 26'$

  W/ BRACING AT INTERVALS OF 15 FT. (MAX), GOOD FOR 11^K

- BOTTOM CHORD FG - SAME AS FOR AG & EF

- TOP CHORDS AB & DE ; $l = 19'$ − KL = 19 ; P = 7083.1#

  ASSUME USE OF 3/4" ϕ BOLT HOLES. ∴ AREA TO BE DEDUCTED FOR BOLT HOLES = $1/4 \times 7/8 = 0.3281 \text{ in}^2$



W = 102.6 + (1.092)(128.3 + 69.5)

+WGT OF Ls ≈ 40 lbs/ft

= 358.6 PLF

$$M_{MAX} = \frac{wl^2}{8} = (358.6)(19)^2/8 = 16{,}182 \text{ ft-lbs}$$

TRY 2Ls 5 x 3 x 1/2 =

$I = 18.9 \text{ in}^4$

$S = 5.82 \text{ in}^3$

$A = 7.50 \text{ in}^2$

$$f = \frac{P}{A} + (M + Pd)\frac{c}{I}$$

$$P_c = \pi^2 EI/L^2$$

$$= 1.041 \times 10^5 \text{ lbs}$$

$$d = \frac{d_o}{1 - (P/P_c)} = \frac{d_o}{0.893}$$

$P = 7.083^K$

$$d_o = \frac{5WL^4}{384EI}$$

$$= 1.92''$$

$$d = 1.92/0.893 = 2.15''$$

$c = 3.25''$

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$$f = \frac{7.083 \times 10^3}{7.5 - 0.328} + \left[(16.182)(12) - (7.083)(2.15)\right] \frac{3.25}{18.9} \times 10^3$$

$$= 9.88 \times 10^2 + 3.077 \times 10^4 = 3.176 \times 10^4$$

$f_{all} = 20$ ksi   NO GOOD

$M = (358.6 + 10)(19)^2/8$
$= 16.633$ ft-lbs.

TRY 2 ∠s 6 x 4 x 3/4

$c = 6 - 2.08$
$= 3.92''$

$$f = \frac{P}{A} + (M + Pd)\frac{c}{I}$$

$I = 49.0 \text{ in}^4$

$S = 12.5 \text{ in}^3$

$A = 13.9 \text{ in}^2$

$P_c = 2.70 \times 10^5$

$$d = \frac{d_0}{1 - (P/P_c)} = \frac{0.74}{1 - \left(\frac{7.083 \times 10^3}{2.70 \times 10^5}\right)}$$

$$d_0 = \frac{5wL^4}{384EI}$$

$\therefore = 0.76''$

$= 0.74''$

$$f = \frac{7.083 \times 10^3}{13.9 - 0.328} + \left[(16.633)(12) - (7.083)(0.76)\right] \frac{3.92}{49.0} \times 10^3$$

$$= 5.219 \times 10^2 + 1.554 \times 10^4 = 1.6062 \times 10^4$$

$= 16.062$ ksi < 20 ksi

OK

• TOP CHORD MEMBERS BC & CD ; $l = 16'$

$P = 9836.7$ lbs.

USE 2 ∠s 6 x 4 x 3/4   (Larger axial force, smaller moment)   OK

CORRESPONDENCE

United States Department of the Interior

NATIONAL PARK SERVICE

DENVER SERVICE CENTER - NATIONAL CAPITAL TEAM

5600 Columbia Pike / Room 312, Falls Church, Va. 22041

H3015 (DSC-TNC)
GWMP-506-35

FEB 18 1982

Memorandum

To: Superintendent, George Washington Memorial Parkway

From: Chief, Branch of Cultural Resources, National Capital Team, DSC

Reference: George Washington Memorial Parkway, Package No. 506, Arlington House, Historic Structure Report

Subject: Engineering Study of Central Section Roof

As part of the engineering study of the roof support structure in the main block of the Arlington House, an investigation was conducted on February 11 to determine tbe condition of the truss members which pass through two masonry chimneys. Participating in this investigation were Jim Askins, Bill Hose, and Jim Gilbert of the Williamsport Training Center and Pat Macdonald of this office. Working in the attic, they removed the brick on the interior side of the chimneys in the area surrounding the truss members. Exposed was one truss end in satisfactory condition (south side) and the other (north side) seriously damaged by no longer active powder-post beetle and brown rot. The wood deterioration problems stem from moisture intrusion from around the flashing which has since been brought under control.

Because of the extent of the damage already incurred, it was necessary that they install a temporary heel brace to distribute the loads beyond the points of damage and provide additional strength to compensate for the section loss. This measure, however, is only a temporary solution. It is therefore recommended that structural repairs be undertaken as soon as possible to reinforce the remaining sound wood to enable a load-supporting capability equal to the original full sections.

One technique which could be used to accomplish this would be by application of epoxies such as the Beta materials manufactured by the Dell Corporation. This method would return the truss members to their original dimensions and to at least their original strength. This method is suggested as it would be compatible with the general design to be proposed for a system to reinforce all trusses. Also, the expertise to accomplish this undertaking is readily available at the Williamsport Training Center. If you are interested in pursuing this type of solution to the problem described, please contact Jim Askins at 301-223-7872 for a cost estimate and time schedule.

/s/

Charles F. Bohannon

TNC:PSMacdonald:hds:2/18/82:2224

cc: NCR, Mr. Goeldner

cc: NCR-PHR, Mr. Scott
GWMP-Arl. House, Ms. Fuqua
DSC-WTC, Mr. Askins

Year of the Visitor

United States Department of the Interior

DENVER SERVICE CENTER - NATIONAL CAPITAL TEAM
NATIONAL PARK SERVICE
WASHINGTON, D.C. 20240

IN REPLY REFER TO:

H30 (DSC-TNC)

16 JUN 1982

Memorandum

To: Assistant Manager, National Capital Team, Denver Service Center

From: Branch Chief, Williamsport Preservation Training Center, National Capital Team, Denver Service Center

Reference: George Washington Memorial Parkway, Package Number 506, Arlington House, Historic Structures Report

Subject: Field Investigation: Central Roof Section

In response to a request from Structural Engineer Pat Macdonald, Denver Service Center, National Capital Team (DSC-TNC), to assist with an ongoing structural investigation of the central roof section, a site visit was conducted on February 10 and 11, 1982. Present were Mr. Macdonald, W. Hose, J. Gilbert and, J. Askins from the Williamsport Preservation Training Center (WPTC).

The purpose of this request was to assist Pat with a limited destructive investigation of the historic fabric to determine, if possible, their current condition. Various methods and techniques to correct obvious deficiencies in the original design and, to identify the extent of damage caused by fungal attack or insect damage to structural members were also discussed.

The findings of our investigation are recorded in a memorandum dated February 18, 1982 to the Superintendent, George Washington Memorial Parkway from the Branch Chief, Cultural Resources, DSC-TNC.

While carrying out this assignment and in indepth discussions, Pat revealed a preliminary design concept for strengthening the existing trussess. This concept, the use of a "fink truss" to provide supplemental support for the live load is an excellent idea.

This concept, I believe is a positive solution to a most difficult problem. After reviewing the preliminary design and accompanying recommendations, I support Pat's recommendations completely. It is a very well thought out design and demonstrates a keen awareness and appreciation for the value of the existing historic fabric and the need for its preservation.

The sensitivity of the Arlington House site and the complexity of the proposed design supports Pat's recommendation of the planned work to be accomplished by day labor. I support this recommendation completely.

I feel Pat should be recognized for the superior engineering work he produced as part of this project. His design concept for the stabilization of the central roof section clearly demonstrates an outstanding structural engineering ability and great sensitivity toward the preservation of historic fabric.

Jim Askins

James S. Askins

cc:
DSC-TNC-F. Bohannon
DSC-TNC-P. Macdonald
File

ACKNOWLEDGEMENTS

Team Members

1. Patrick Macdonald, Civil Engineer, DSC-TNC
2. Maury Paul, Structural Engineer, Professional Support, Denver Service Center
3. John Sligh, Architect, DSC-TNC
4. Karen Sporn, Editor
5. Hilde DeSouza, Secretary

Acknowledgements

1. For cooperation and assistance provided, special thanks to Ann Fuqua, Agnes Mullins, and the entire Arlington House staff.
2. For technical consultation, thanks to Jim Askins, Bill Hose, and Jim Gilbert of the Williamsport Training Center, DSC-TNC.