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A Study Rigid-Frame Design versus Truss-and-Column Design

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A STUDY

RIGID-FRAME DESIGN versus TRUSS-and-COLUMN DESIGN

THESIS

PRESENTED IN PARTIAL FULFILMENT OF THE REQUIRMENTS FOR THE DEGREE OF BACHOLEOR OF SCIENCE IN CIVIL ENGINEERING

AT

THE COLLEGE OF THE PACIFIC

BY

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_______________ Head of Department.

CONTENTS

SYNOPSIS

This paper describes the investigation of a mill building bent of truss and column design, a redesign using a rigid-frame, and an estimated cost of each design.

For sake of brevity the overall design of the building (sway bracing, lateral bracing, purlins, girts, roofing, siding, and bay spacing) was assumed to be the same for both designs. It is possible that this assumption cannot be made and the estimates of cost, therefore, are in error. In actual practice the complete redesign of the building should be considered.

INTRODUCTION

During the past two decades there has been a growing appreciation of the many structural and esthetic advantages of the rigid-frame type of construction particularly as applied to short span bridges. It has only been in the last few years that rigid-frame construction has become popular in building construction. The chief advantages of this type of construction being its ease of fabrication and erection, and in some cases overall economy of construction.

In comparing the two types of buildings a more or

less typical mill building was chosen for study. The building differs from the usual proportions in that it has excessive height to the eaves to allow clearance for standard gage track and box cars.

The focal point of interest in the comparison of the two types of construction was the design of the rigid-frame. This design made use of the theory of virtual work and uses experimental data to calculate actual stresses in certain sections of the frame.

CHAPTER I

INVESTIGATION OF BUILDING DESIGN

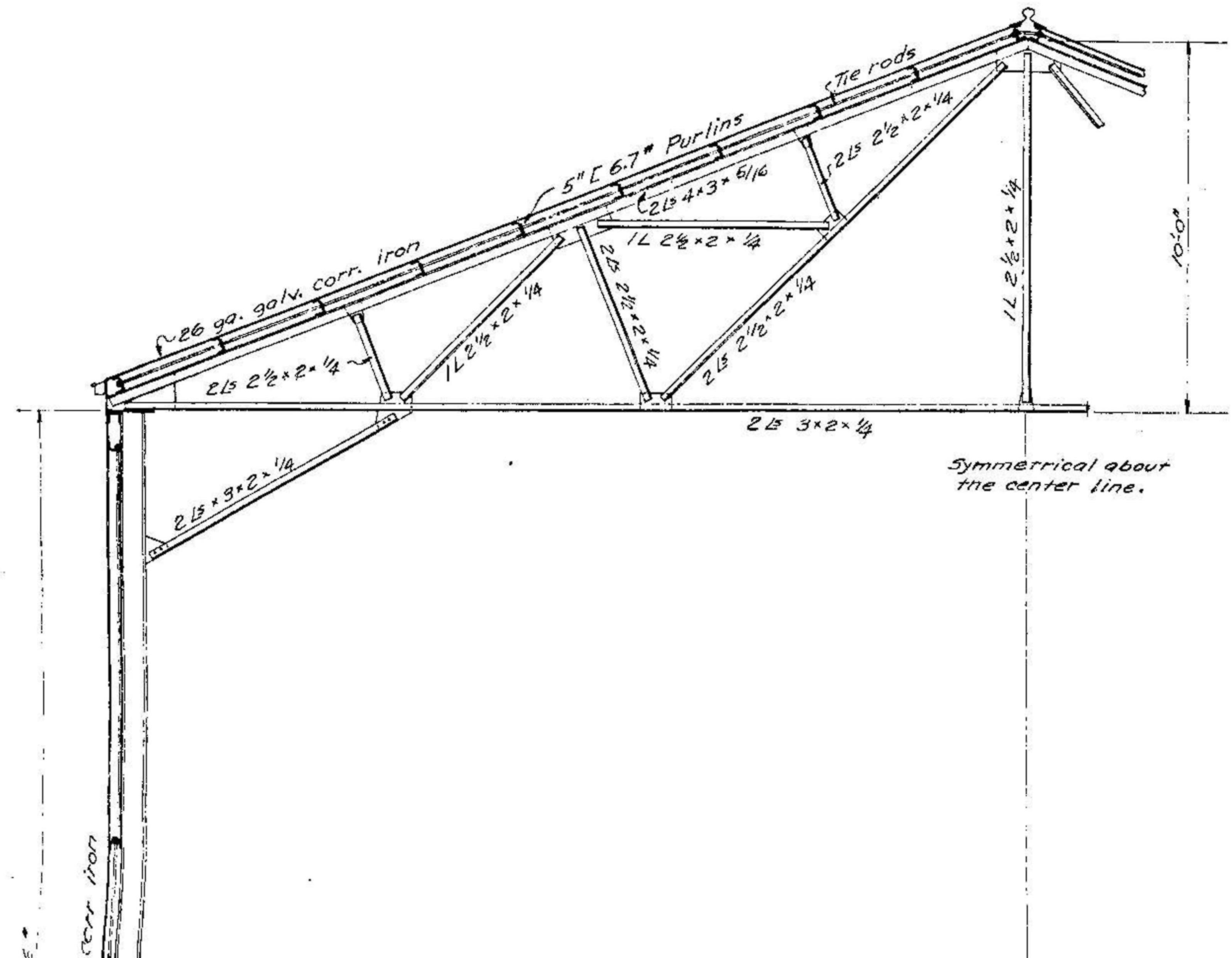
1. Description of Building. The building studied was designed and built for use as a warehouse for a steel wholesaler in Stockton, California. It was designed under the provisions of the Pacific Coast Uniform Building Code which apply to all building construction in that city.

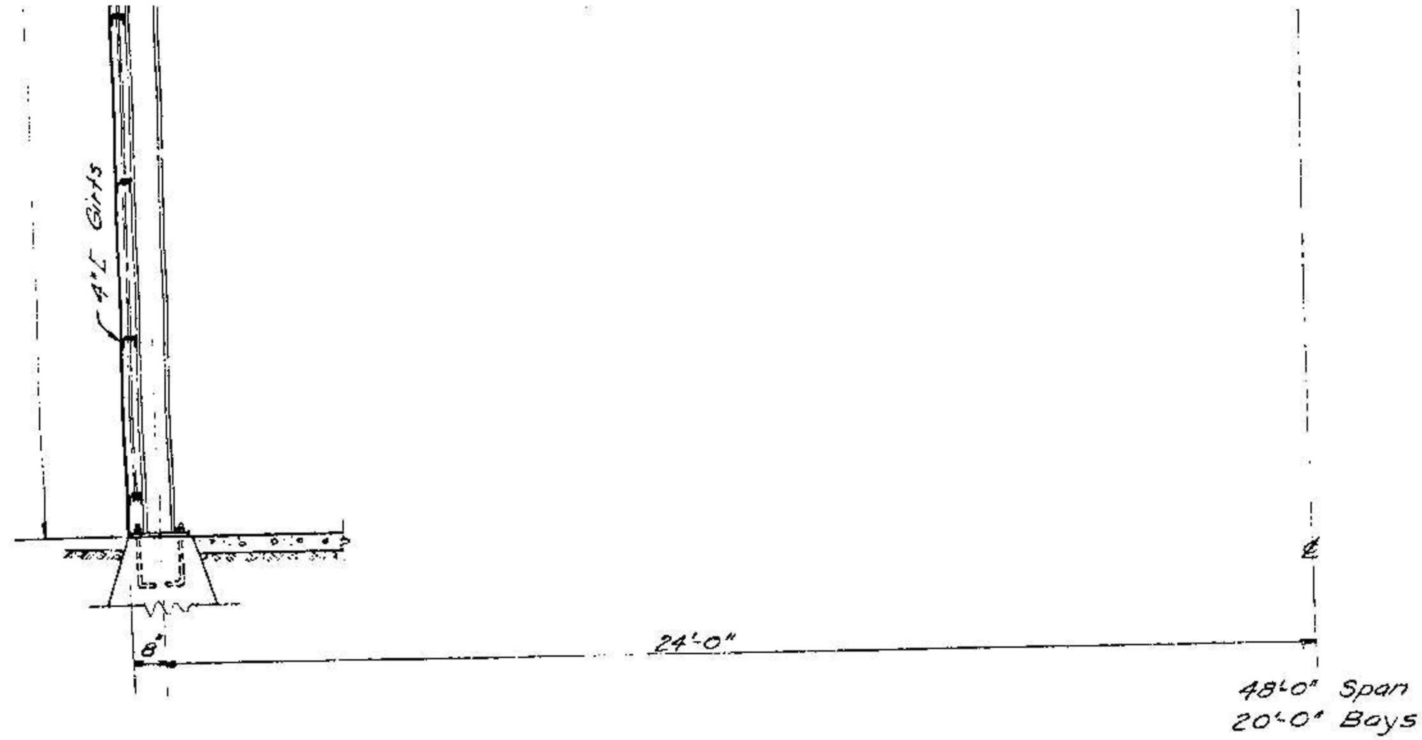
The structural framework is of typical steel mill building design with trussed and column bents. Unusual, however, was the excessive height of the columns. The height was necessary in order to provide clearance between an overhead crane and a standard gage box car.

Arc welding was used to connect the various members that were prefabricated. The field joints of the column, truss, and lateral bracing were made with field bolts. For further details of a typical bent refer to Fig. 1.

2. Theoretical Considerations. The trussed part of the bent, which consists of a Fink roof truss, was investigated for combined live and wind loads. Section 2305 of the Uniform Building Code states the following:

"Roofs shall be designed for a vertical


26 ga. galv. corr. iron
5" [6.7# Purlins
Tie rods
2 Ls 4×3×5/16
2 Ls 2½×2×¼
1 L 2½×2×¼
1 L 2½×2×¼
2 Ls 2½×2×¼
2 Ls 2½×2×¼
1 L 2½×2×¼
2 Ls 2½×2×¼
2 Ls 3×2×¼
2 Ls ×3×2×¼
10'-0"
corr iron
Symmetrical about
the center line.



Quan.	Item	Wt.	Quan.	Item	Wt.
	Truss & Column Members		2	PL - 6 × 1/4 × 0'-10"	8#
4	Ls - 4 × 3 × 5/16 × 28'	862#	2	PL - 6 × 1/4 × 0'-9"	8
2	Ls - 3 × 2 × 1/4 × 50'	410	4	PL - 6 × 1/4 × 0'-7"	12
4	Ls - 2 1/2 × 2 × 1/4 × 15'	217	1	PL - 6 × 1/4 × 0'-6"	3
1	L - 2 1/2 × 2 × 1/4 × 10'	36	4	Ls - 3 × 2 × 1/4 × 8'	131
4	Ls - 2 1/2 × 2 × 1/4 × 8'	116	2	PL - 8 × 5/16 × 0'-9"	13
4	Ls - 2 1/2 × 2 × 1/4 × 6'	87	2	WF - 8" - 31# × 29'	1800
8	Ls - 2 1/2 × 2 × 1/4 × 3'	87	4	PL - 14 1/2" × 1/2 × 2'-0"	93
1	PL - 11 × 5/16 × 1'-8"	20	2	PL - 16 1/2 × 1/2 × 2'-10"	112
2	PL - 12 × 5/16 × 1'-4"	34	28	Bolts - 3/4 × 1 1/2"	13
2	PL - 8 × 1/4 × 1'-8"	23	4	Bolts - 3/4 × 1'-6"	9
2	PL - 12 × 1/4 × 1'-0"	20		Total Weight	4114

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TYPICAL BENT
TRUSS CONSTRUCTION

Scale $\frac{1}{4}$" = 1'-0" 5-8-49 J.H. Goodman
Fig. 1 Plate No. 1

live load of 20 pounds per square foot of horizontal projection applied to any and all slopes, except herinafter provided.
Where the rise exceeds twelve inches (12") per foot no vertical live load shall need be assumed, but the roof shall be designed for the dead load of 15 pounds per square foot of vertical projection."

Concerning wind loading Section 2307 of the Code specifies:

"For purpose of design the wind pressure upon the gross area of the vertical projection of buildings and structures shall be taken at not less than 15 pounds per square foot for those portions of the building less than sixty feet (60') above the ground and not less than 20 pounds per square foot for those portions more than sixty feet above the ground."

The roof of the building has a pitch of about 1 to 2½. Therefore, 20 pounds per square foot of horizontal projection of the roof was used for the live loading. The horizontal force of the wind was resolved into a force normal to the roof surface by the relationship,

$$P_n = \frac{P\,2\sin\alpha}{1+\sin\alpha},$$

where α is the angle between the upper and lower chords. For the truss being considered,

$$P_n = 15(.640) = 9.6\ \#/\square'.$$

The dead load of the roofing material plus the live load on the roof were resolved into a uniform

load acting along the length of the purlins which span between the trusses and were assumed to act as simple supported beams. These purlins, supported at intervals along the top chord of the truss, bring the roof loading into the truss. The top chord, in turn, distributes the roof load into the truss as panel point loadings. The dead load of the truss members are also assumed to act at the panel points.

In order to find the maximum possible stress in the truss members, two separate graphical analyses were made, one with dead and live load acting and the other with wind load alone acting.

The top chord of the truss is assumed to distribute the purlin loads into the truss at the panel points. Therefore, the top chord must function as a continious beam spanning the panel points and, as well, function as the top chord of the truss. The method of moment distri- bution was used to calculate the reactions of this contin- uous beam. The unit stresses due to the beam action of the top chord was combined with the unit stresses due to truss action to give maximum possible stress.

The reactions acting on the columns supporting the truss were found by considering the wind acting on one side of the building and assuming the semi-fixed ended columns to have points of inflection (point of zero

moment) at one-third of the way up from their bases. Moments were evaluated about these points to obtain the various reactions acting on the columns.

Simple structural theory was used for the investigation of the building bent except for the analysis of the top chord of the truss which acts as a continuous beam and, therefore, is indeterminate.[1] An analysis by moment distribution was made in order to determine the bending stresses in this indeterminate top chord.

3. Computations. The computations and results have been summarized in graphical and tabular form and placed in the appendix for the reader's convenience and needs no further explanation.

4. Results of Investigation. Section 2307 of the Uniform Building Code states:

> "For combined stresses due to wind and other loads the allowable unit stresses may be increased 33 1/3 per cent in excess of the values specified in Chapters 24, 25, 26, and 27. For member carrying wind stresses only the allowable stress may be increased 33 1/3 per cent. In no case shall the section be less than required if the wind stress is neglected."

All allowable unit stresses were increased by the 33 1/3 per cent since wind loading in combination with dead and live loads were used in the investigation.

1. L. E. Grinter, Design of Modern Steel Structures pp. 279-307

The truss members were found to be stressed within the allowable limits as specified by the Uniform Building Code. The columns were found to be overstressed by 52% when combined dead, live and wind loads were assumed to be acting. This calculation was made with the points of inflection of the columns assumed to be at one-third of the unsupported length above the base of the same. If the points of inflection are assumed to be at the mid-point of the unsupported length of the column, which throws a greater moment into the footing, the column is overstressed by approximately 10%.

A 10 x 8 WF-38# column section or comparable will be stressed within allowable limits if the points of inflections are located approximately mid-way between the one-third point and one-half point of the unsupported length of the column.

CHAPTER II

RIGID-FRAME DESIGN

5. Theoretical Considerations. The three fundamental equations of statics, ($\Sigma H=0, \Sigma V=0$, and $\Sigma M=0$) provide sufficient conditions for determining the reactions and stresses of a structure that is determinate both internally and externally. A structure which contains a greater number of reactions than can be determined by the three fundamental equations of statics, on the other hand, is indeterminate. The degree to which a structure is indeterminate is equal to the number of reactions in excess of the three fundamental equations of statics.

The single-span rigid-frame with pinned ends is indeterminate to the first degree since there are four unknown reactions as shown by Fig. 2.

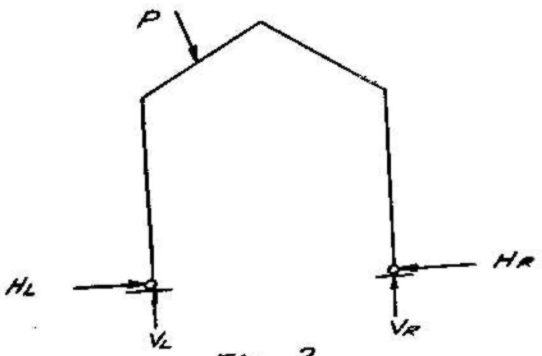


Fig. 2

In order to complete the analysis of the structure, one of the unknown reactions must be determined on the basis of the elastic distortion of the frame under the

imposed loads. Note that if the ends were not pinned (able to transmit moment) the structure would have two additional reactions and would be three degrees indeterminate.

The MAXWELL-MOHR METHOD OF WORK (Unit Dummy Load)[2] provides a method of approach to the analysis of any indeterminate structure and was used for the determination of the redundant reaction of the rigid-frame herein considered. The Maxwell-Mohr method is based on the fact that if a redundant reaction is removed, the structure will deflect some amount, δ_m, which can be determined on the basis of a determinate frame. If the dummy load (unit load), assumed to be one pound, pushes the structure back a given amount, $\delta_{m'}$, the total reaction will be deflection δ_m, divided by the amount that one pound will push the structure back into its normal position.[3,4] The sequence of calculation are graphically illustrated in Fig. 3.

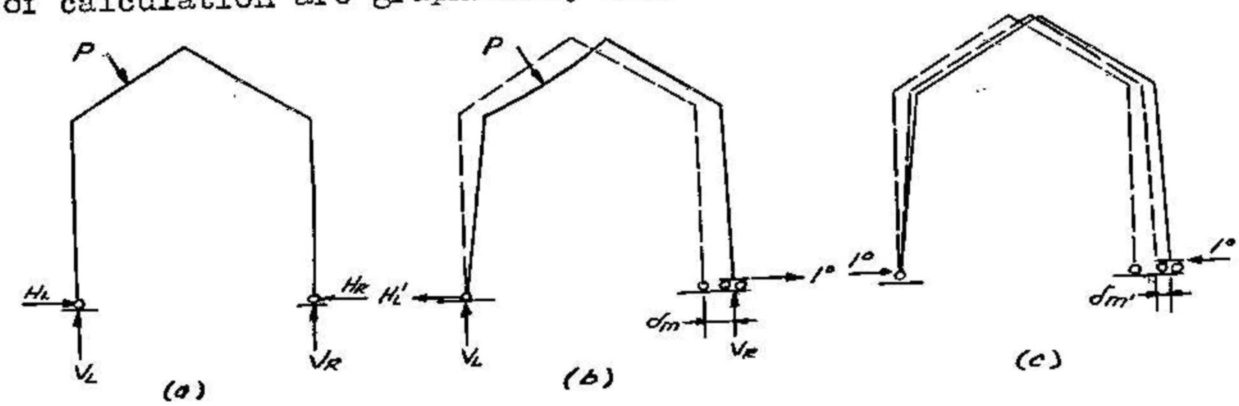


Fig. 3

2. Parcel and Maney, Statically Indeterminate Stresses pp. 11-20
3. ibid. pp. 90-101
4. A.I.S.C., Single Span Rigid-Frames in Steel pp. 11-12

The Maxwell-Mohr Method of Work formulas for deflection and elongation of members will not be derived, but will be stated and elaborated upon.

The elongation of a member under the action of an axial load is,

$$\delta_e = \sum \frac{SL}{AE} u \ ;$$

where, δ_e = elongation
S = stress in the member due to the load
L = length of the member
U = stress in the member due to the dummy load
A = cross-section area of member
E = modulus of elasticity of the member.

The deflection of a member under the action of transverse load is,

$$\delta_m = \sum \int \frac{M'm \, dx}{EI} \ ;$$

where, δ_m = deflection
M^1 = the determinate moment at any point in the member
m = the dummy load moment at any point in the member
E = the modulus of elasticity of the member
I = the moment of inertia of the cross-section at which the moment M^1 is calculated.

The deflection of a member due to the dummy load acting alone and axially is,

$$\delta_{e_1} = \sum \frac{u^2 L}{AE} \ ;$$

and the deflection of a member due to the dummy load acting alone and other than axially is,

$$\delta_{m_1} = \sum \int \frac{m^2 dx}{EI} \ .$$

Therefore, since the redundant reaction can be calculated by dividing the full load deflection by the dummy load deflection, the redundant reaction equals,

$$H_R = \frac{\delta_m}{\delta_{m_1}} = \frac{\Sigma \int \frac{M'm\,dx}{EI}}{\Sigma \int \frac{m^2\,dx}{EI}}$$

The effect of shortening of the members has been neglected since they are generally small in single-span rigid-frames.

These basic principles of the Maxwell-Mohr Method of Work will be applied in the design of a single-span rigid-frame in the following pages of this discussion. In order to apply the Method of Work, a rigid-frame of a given section must be chosen and then investigated since the deflections are dependent upon the relative stiffness of the frame. Once the reactions of the given rigid-frame are determined, the unit stresses in the various sections of the rigid-frame may be computed and checked against the allowable unit stresses. If this stress is exceeded at any point, another rigid-frame section must be chosen and the calculations for reactions and stresses repeated.

The loadings acting upon the rigid-frame are essentially the same as those assumed to be acting upon the truss and column bent. Since, the outward dimensions

of the building are the same in both designs, the same provisions of the Uniform Building Code will apply in regards to loadings (see pages 3 and 4). In addition, the affect of earthquake action and thermal expansion was considered. Section 2313 of the Uniform Building Code states:

> "Horizontal Force Formula. In determing the horizontal force to be resisted, the following formula shall be used:
>
> $$F = CW$$
>
> Where F = the horizontal force in pounds
>
> W = the total dead load plus one-half the total vertical designed live load, at and above the point of elevation under consideration, except for warehouses and tanks, in which case W shall equal the total dead load plus the total vertical designed live load at and above the point or elevation under consideration. Machinery or other fixed concentrated loads shall be considered as part of the dead load.
>
> C equals a numerical constant as shown in Tabel No. 23-A
>
> Where wind load as set forth in Section 2307 would produce higher stresses, this load should be used in lieu of the factor shown."

A check computation of the maximum possible stress caused by earthquake loading showed that wind load stress was greater. Therefore, wind loading was used in lieu of earthquake loading.

The reactions caused by thermal expansion were

readily found since the free end deflection of the rigid-frame is the amount that a steel member equal to the span of the frame will expand under a given temperature differential.

The rigid-frame finally chosen is of a constant cross-section except at the knee where it is reinforced to form a circular knee, (see Fig. 5).

The square knee and circular knee of rigid-frames have been the subject of numerous studies and investigations. A report written by Mr. Inge Lyse and Mr. W.E. Black[5] does an excellent job of correlating theoretical assumptions with experimental data to arrive at conclusions to be followed in practical design.

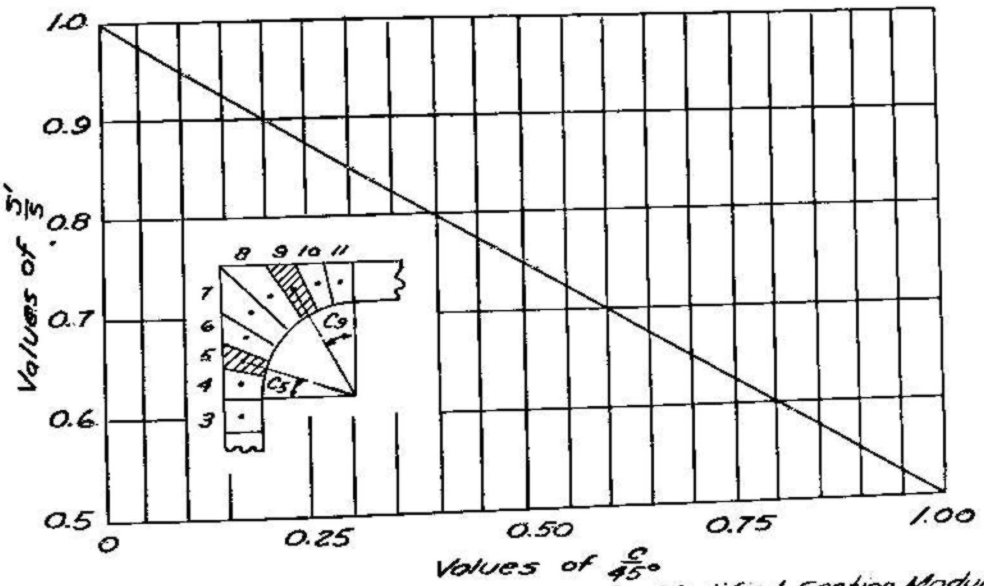
In the report, they note that the observed unit stresses in the compression flange of the knee is sometimes 50% greater than that calculated. Therefore, the report recommended that the actual section modulus of the various points around the curved knee be reduced by an amount which varies with the angular distance of the section from the point of tangency. More specifically, moment of inertia of this section should be reduced in accordance with the relationship,

$$I = S'C$$

where S' = reduction factor
C = one-half the actual depth of the section

5. Lyse and Black, A.S.C.E., <u>An Investigation of Steel Rigid Frames</u> Reprinted from the A.S.C.E. Proceedings, Nov. 1940

The values of the reduction factor were given in a graph which has been reproduced.



Graph for the Determination of a Modified Section Modulus
From, "An Investigation of Steel Rigid Frames", by
Lyse and Black

Fig. 4

The moment of inertia of the various sections of the circular knee were reduced by the procedure recommended. The maximum reduction being about 71% of the actual section modulus.

Once the reactions were determined and the section checked for the maximum allowable unit stress, the design of the incidental details were considered. The detail that provided the most complexities of design was the circular knee. Here the stress introduced by the bent outer flange had to be taken by the wide, comparatively thin, web.

Web stiffners were added at five points to insure against buckling and to promote even distribution of the web stress. The various welds in the curved knee area were designed to transmit the stresses carried by the members which they joined together.

The base plate was much simpler in design. The point to note is that for practical purposes the lead sheet beneath the base plate and the two anchor bolts placed along the possible line of rotation can be assumed to act as a pin connection which is essential for the conditions of design.

The field splice does not differ from the design of any other such splice. The stress due to flexure is transmitted by the flange splice plates, while the web splice plates transmits the shearing stress from one member to the other. The stress in the welds are that allowed by the A.I.S.C. and the Uniform Building Code specifications.

The peak or rigid connection design involved calculating the necessary weld to transmit the stresses in the joining members. Since the shear at this point is not critical, there is no danger of the web buckling.

These are fundamentaly the theories used to design

the single-span rigid-frame. It does not pretend to cover all the rammifications of design, since it would take volumes to completly exhaust the subject.

6. <u>Computations</u>. The computations are compiled in a graphical and tabular form and placed in the Appendix.

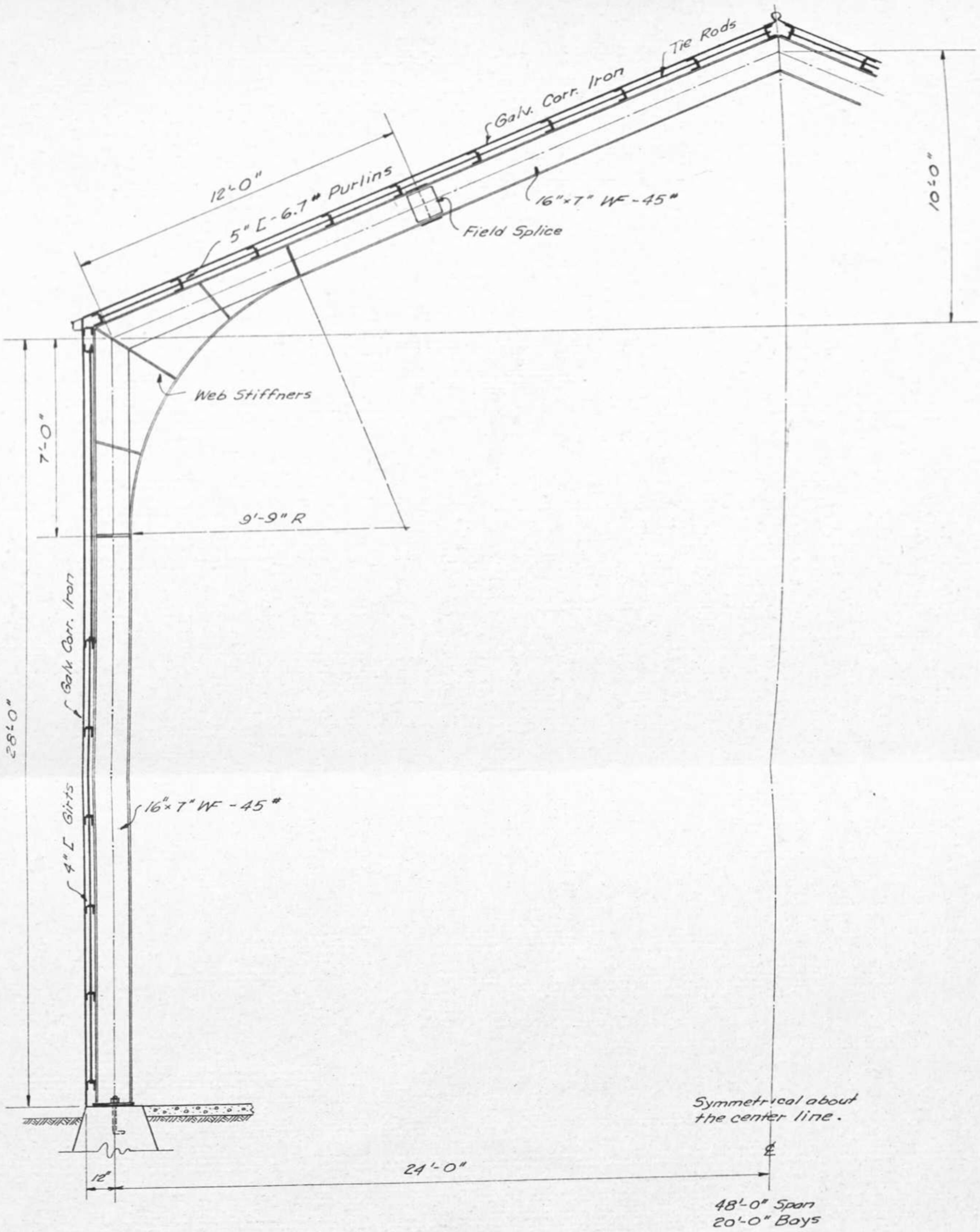
The rigid-frame was divided into short sections to facilitate the calculations of the indeterminate reaction and also the unit stresses at various sections. Unit shears were not calculated since a few check computations showed that the unit shearing stress was far below critical for all points. For further information on computation procedures, refer to pages of the Appendix.

7. <u>Description of the Single-Span Rigid-Frame</u>. The rigid-frame section found to be adequate in regards to allowable unit stresses is a 16" x 7" WF-45# with reinforced circular knees. As previously mentioned, the redesign of the whole building is not within the scope of this paper. However, inspection of the two types of framework will show that a minimum of redesign of the roofing, siding and sway bracings would be required. The economical bay spacing will be approximately the same for the two types[6],

6. A.I.S.C. <u>Single Span Rigid Frames in Steel</u> pp. 3-4

also the purlins, girts and siding will be the same in both cases. The major redesign would be that of the longitudinal bracing.

For the typical rigid-frame bent, bill of material, and details refer to Figs. 5 and 6.



Quan.	Item	Wt.
2	WF-16"×7"-45# 41'	3690#
1	WF-16"×7"-45# 29'	1305
2	PL-9"×½"×1'-6"	46
4	PL-5"×½×0'-10"	28
1	PL-19"×3/8×7'-2"	173
4	PL-3"×3/8"×3'-6"	54
8	PL-3"×3/8"×1'-8"	51
2	PL-12"×3/8"×1'-1"	17
4	Bolts-1"×1'-6"	19
2	PL-9"×1/8"×1'-6" (lead)	
	Total Weight	5383

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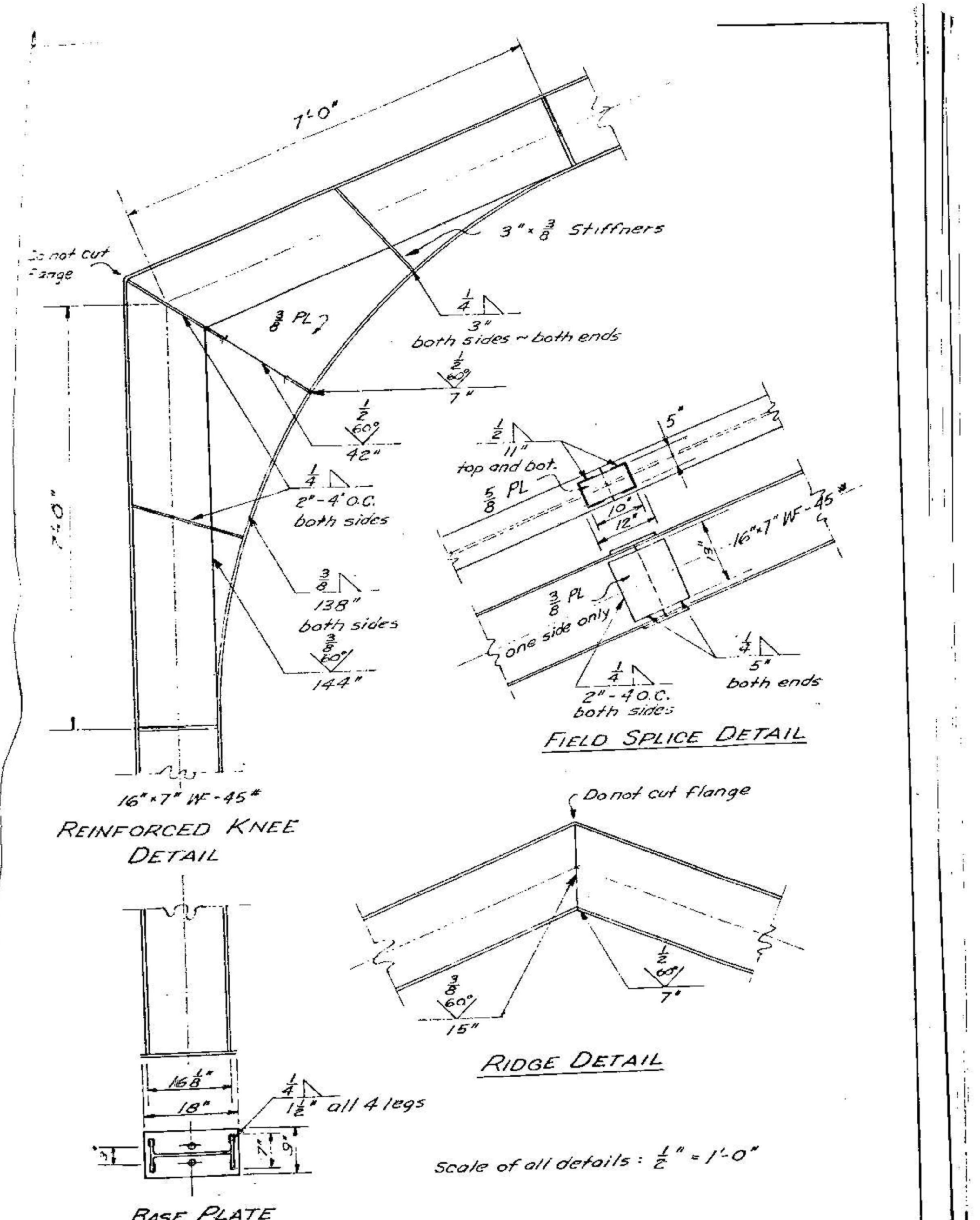
TYPICAL BENT
RIGID-FRAME CONSTRUCTION

Scale: $\frac{1}{4}$" = 1'-0" — 5-21-49 S.H. Goodman
Fig. 5 — Plate No. 2



7'-0"
Do not cut flange
3" × 3/8 Stiffners
3/8 PL
1/4
3"
both sides ~ both ends
1/2
60°
7"
1/2
60°
42"
1/4
2"-4" O.C.
both sides
3/8
138"
both sides
3/8
60°
144"
7'-0"
16" × 7" WF-45#
REINFORCED KNEE DETAIL
1/2
11"
top and bot.
5"
5/8 PL
10"
12"
16" × 7" WF-45#
13"
3/8 PL
one side only
1/4
5"
both ends
1/4
2"-4 O.C.
both sides
FIELD SPLICE DETAIL
Do not cut flange
3/8
60°
15"
1/2
60°
7"
RIDGE DETAIL
16 1/8"
18"
1/4
1 1/2" all 4 legs
7"
9"
BASE PLATE DETAIL
Scale of all details : 1/2" = 1'-0"

Fig. 6

CHAPTER III

A COST STUDY

8. <u>General Assumptions Used in Cost Estimates</u>. The primary assumption throughout the whole study has been that the outer shell of the building and longitudinal bracing will remain the same in both types of structures. Therefore, the cost of material, fabrication, and erection of the outer shell and longitudinal bracing will be neglected in the cost comparison since they are equal in both cases.

Fabrication and erection were assumed to take place in the vicinity of Stockton, California where the following hourly wage rates apply:

For a 8 Hour Day	
Carpenters	$2.16
Structural Iron Workers	2.40
Painters	2.46
Welders	2.15
Hoist Eng. (1-drum)	2.13
Truck Driver (over $1\frac{1}{2}$ ton)	1.78
Common Labor	1.53

The price for steel shapes and plates required were quoted by a local steel wholesaler and included the cost of delivery to the site of fabrication. The average steel price is $6.50 cwt.

In estimating the cost of fabrication, time-motion operations were visualized in order to obtain the cost of

various fabrication procedures such as shearing, cutting, assembly, and welding. The actual cost of welding materials were calculated by the use of cost data obtained from a textbook on arc welding procedure.[7] From this data the following table of welding cost was compiled.

Type of Weld	Position	Ft. of Joint Welded per Hour*	Lbs. of Electrode per Ft. of Weld	Cost of Power per Ft. of Weld	Total Cost per Inch	
					Matl.	Labor
1/4" U-Butt	Horz.	11	0.24	2 ¢	0.4¢	1.6¢
3/8" 60°	Flat	12	0.55	3	0.7	1.5
1/2" 60°	Vert.	4.5	0.85	4	1.1	4.0
3/16" fillet	Flat.	33	0.14	1	0.2	0.5
1/4" fillet	Flat	30	0.22	2	0.4	0.6
1/4" fillet	Vert.	26	0.14	1	0.2	0.7
3/8" fillet	Flat	20	0.36	3	0.6	0.9
1/2" fillet	Vert.	7.5	0.52	3	0.7	2.4

*Assuming the welder will be welding 75% of the time.
**Cost of electrodes assumed to be $0.10 per pound.

Table I

The erection crew was assumed to consist of one foreman, two steel iron workers, one hoist operator, and two laborers. This crew is ample to handle any small size erection job.

These various assumptions and cost data were used to arrive at a cost estimate of both types of structures. The actual cost of the structures may be inaccurate, but the comparative cost should be approximately correct.

7. The Lincoln Electric Company, <u>Procedure Handbook of Arc Welding Design and Practice</u>. pp. 188-262

since the same assumptions were made in both cases.

9. <u>Cost Estimate of Truss and Column Structure</u>. The cost estimate was made for one truss and two columns only for reasons stated in Sec. 8. The cost estimate follows closely the standard procedure as illustrated in texts on cost estimating.[8]

COST ESTIMATE

TRUSSED AND COLUMN BENT

(Total Weight 4114 lbs.)

ITEM	LABOR	MATERIAL	TOTAL
Structural Steel.			
Columns - 2 WF-8" 31#, 29' long		$126.00	
Details		7.23	
Bolts		2.92	
Truss - Assorted angles		108.69	
Details		8.40	
Knee braces - 2 ∠s -3x2x$\frac{1}{4}$ 8' long		8.09	
Details		.84	
Bolts		.72	
Totals		262.89	$262.89
Fabrication.			
Shearing, punching and cutting:			
Columns (4 cuts)	$1.00		
Angles (30 cuts)	4.00		
Sketch plates, including layout	12.80		
Punching 68 bolt holes	3.50		
Assembly (fitting up):			
Columns with base, truss, and knee plates	1.00		
Truss, angles and gusset plates are set in a specially prepared jig	4.10		
After one side is welded the truss must be flopped over.	1.50		

8. H.E. Pulver, <u>Construction Estimates and Costs</u> pp. 260-282

COST ESTIMATE (Cont.)

ITEM	LABOR	MATERIAL	TOTAL
Welding:			
Column and Truss			
154" of $\frac{1}{4}$ (horz.) butt weld	$ 2.50	$ .65	
142" of $\frac{1}{4}$ (flat) fillet weld	.90	.60	
64" of 3/8 fillet weld	.60	.40	
510" of 3/16 (flat) fillet weld	2.60	1.05	
Paint 591 □' (small members-1$\frac{1}{4}$hrs.)	9.50	3.60	
Handling of members	1.00		
Totals	45.00	6.30	$ 51.30
Transportation to site of erection			
2-columns (10 per load) includes loading and unloading	.70		
1-truss (4 per load) includes loading and unloading	1.50		
Totals	2.20		2.20
Erection			
2-columns	7.00		
1-truss	10.00		
Totals	17.00		17.00
Painting - 591 □' (small members - 2 hrs.)	12.80	3.60	16.40
Total Labor Cost	77.00		
Total Cost of Labor and Material			349.79
Equipment Ownership expense			10.00
Overhead, including insurance, power etc. @ 40% of 349.79			140.00
Total Cost			$499.79

10. Cost Estimate of Rigid-Frame Structure. Here, again, only the cost of the rigid-frame was considered since the other factors were assumed to be equal in both types of structures.

COST ESTIMATE

RIGID-FRAME BENT

(Total Wt. 5383 lbs.)

ITEM	LABOR	MATERIAL	TOTAL
Structural Steel			
Frame - WF 16x7 - 45#, 111' long		$344.60	
Details		23.76	
Lead plates		2.00	
Bolts		2.15	
Totals		372.51	$372.51
Fabrication			
Shearing, punching and cutting:			
Shearing plates, includes lay-out	$ 2.00		
Punching 4 bolt holes	.50		
4 cuts through frame	1.00		
3 notches of frame, includes layout	2.00		
Grinding edges	2.00		
Bending flanges and cutting plate for reinforced hip	8.00		
Assembly (fitting up)			
Base plates, reinforced section and splice plates	2.50		
Center section, bent to shape	1.00		
Welding:			
21" of $\frac{1}{2}$" (vertical) butt weld	.85	.25	
387" of 3/8" (flat) butt weld	5.80	4.10	
252" of 3/8" (flat) fillet weld	2.30	1.50	
88" of $\frac{1}{2}$" (vert.) fillet weld	2.10	1.70	
328" of $\frac{1}{4}$" (flat) fillet weld	2.00	1.30	
78" of $\frac{1}{4}$" (vert.) fillet weld	.55	.15	
turning frame over	.75		
Shop Painting: Area 580 □'			
Large member - 1hr. time	6.50	3.50	
handling members	1.00		
Totals	21.85	12.50	34.35
Transportation to site of erection			
4 - frames per load (includes loading and unloading)	1.50		
Total			1.50

COST ESTIMATE (Cont.)

ITEM	LABOR	MATERIAL	TOTAL
Erection			
Assembly of peak and columns on ground (welding considered in fabrication cost)	$ 3.50		
Setting entire frame in place and bolting	6.50		
Total	10.00		$ 10.00
Painting 580 □' (Large members 1-½ hrs.)	8.10	$ 3.50	11.60
Total Labor Cost	51.45		
Total Cost of Labor and Material			429.96
Equipment ownership expense			10.00
Overhead, including all insurance, power etc. @ 40% of 429.96			172.00
Total Cost			$611.96

11. Comparison of Costs. The cost differential between the two types of structures was $112.20 in favor of the truss and column bent. This higher cost of the rigid-frame was due entirely to its heavier members and, consequently, a higher material cost. It should be noted that the cost differential could be reduced by other possible rigid-frame design. The preceding design could be modified in several respects to give a lower first cost. It will be noted, from the tabulation of maximum unit stresses, that a smaller section could be used for the girder portion of the frame. This would reduce the weight and cost of material. Many steel fabricators produce a rigid-frame with a cross-section that varies with

the stress requirements which uses the minimum of material. Also, it may be possible to lessen the column height of the rigid-frame due to its greater overhead clearance and still allow for the installation of an overhead crane that can clear a standard gage box car.

These are only several of the possibilities that should be investigated in a more thorough redesign of the building.

CHAPTER IV

GENERAL DISCUSSION AND CONCLUSION

12. General Discussion of the Two Types of Structures. The column and truss, or mill building type of structure is the oldest and most commonly used for industrial buildings of nominal spans. It is economical and very practical when used in structures where overhead clearance and appearance is not critical. It is simple to design and easy to fabricate even in the smaller steel shops. The total weight of the trussed structure tends to be less than that of a rigid-frame when used for nominal spans and ordinary roof loadings. In fact one author states that the truss and column type of structure remains the most economical for short spans and ordinary loadings.[9]

The rigid-frame is becoming more popular and economical with the advance of arc welding procedures. The rigid-frame is ideal for structures where overhead clearance and appearance is essential. They have been used considerable in churches, gymnasiums, auditoriums, two storied industrial buildings and other such structures. In warehouse structures, its advantages over truss and column bents are questionable since the overhead clearance

9. L.E. Grinter, Design of Modern Steel Structures p.307

and appearance is not usually critical. Its design is a little more complex since it is an indeterminate structure. However, most designing engineers should be familiar with the design principles. The fabrication requires a better equipped shop since cutting and bending of larger sections is necessary.

13. Conclusions. The truss and column type of structure proved to be about 13% more economical in first cost than the rigid-frame. The rigid-frame design considered may not be the most economical. Many rigid-frames are designed with a cross-section that varies with the stress requirements at each point around the frame. However, the extra cost of fabrication will tend to reduce the savings of cost resulting from the reduction of weight. It is possible that the truss and column structure will still have the lowest first cost even when a rigid-frame with a varying section is used. However, when all factors are considered it will be found, that, even on a first cost basis, the rigid-frame is highly competitive.

14. Summary. The important points of the paper can be summarized as follows:

(a) The truss and column design was investigated and the columns were found to have unit stresses exceeding

the allowable by approximately 10%.

(b) The principles of the Maxwell-Mohr Method of Work were stated and the design procedures outlined.

(c) A constant section rigid-frame with a circular knee was designed.

(d) A cost estimate was made of both structures.

(e) The truss and column structure was found to have the lowest first cost which agrees with the conclusions of several authors. However, a more complete study should investigate other possible rigid-frame design which may reduce its first cost.

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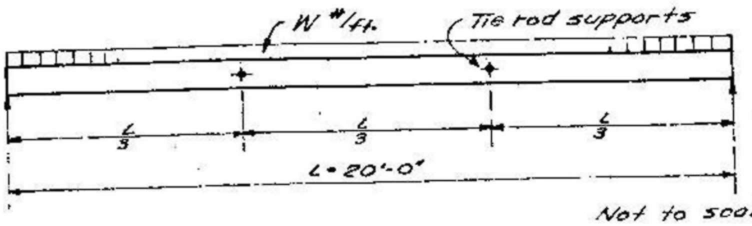
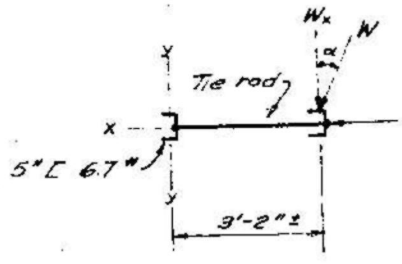
APPENDIX

COMPUTATIONS



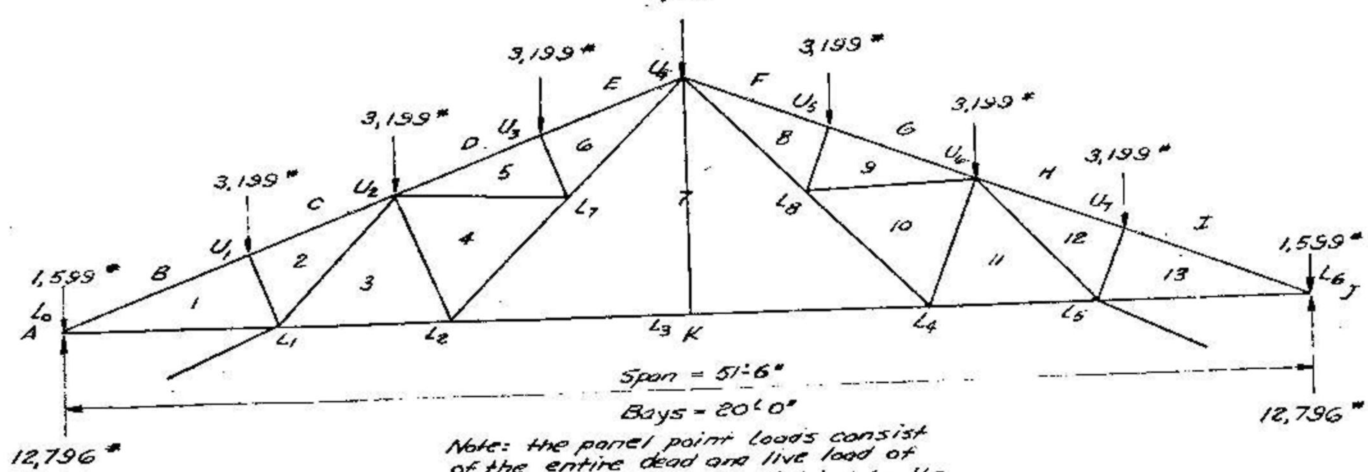


Not to scale

Type of Load	Wt. per Foot	Wx lbs/ft	Wy lbs/ft.	Mx in-lbs	My in-lbs	I/c x-x axis	I/c y-y axis	Fibre Stress x-x axis	Fibre Stress y-y axis	End Reactions
Dead	10	9.3	3.6	5590	216	3.0 in³	0.38 in³	1860 #/□"	570 #/□"	200 #
Live	59	55	21.3	33000	1280	3.0	0.38	11000	3650	1190
										1380
Wind	30	30	----	18000	---	3.0	0.38	6000	----	620
							Totals	18860 #/□"	4220 #/□"	2000 #

Note: allowable bending unit stress is, 20,000 x .33⅓ = 26,700 #/□".

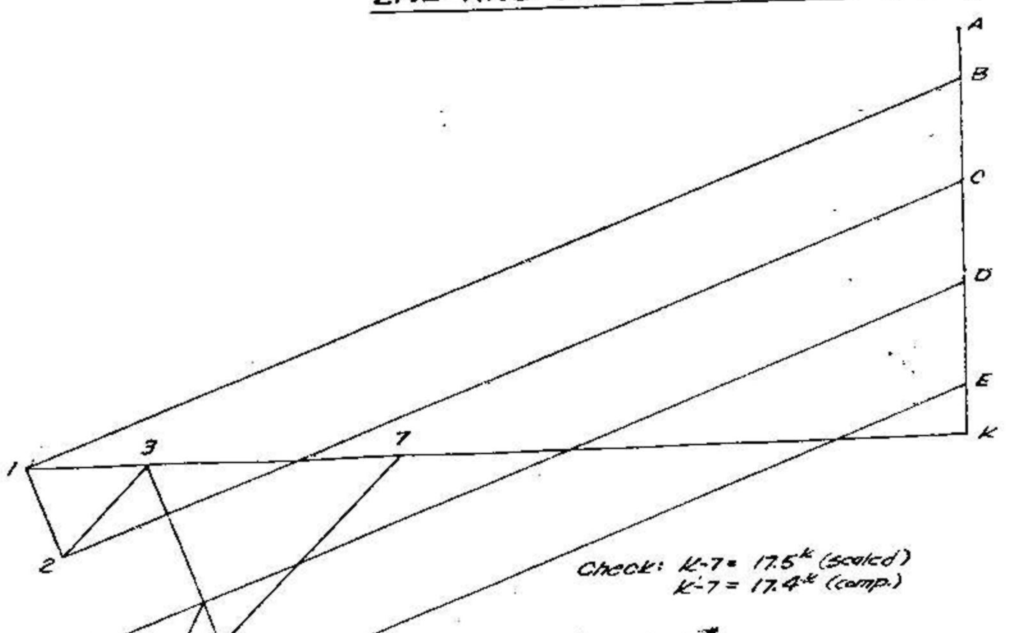
PURLIN AND UNIT STRESSES



Note: the panel point loads consist of the entire dead and live load of the roof and truss divided by the number of panels.

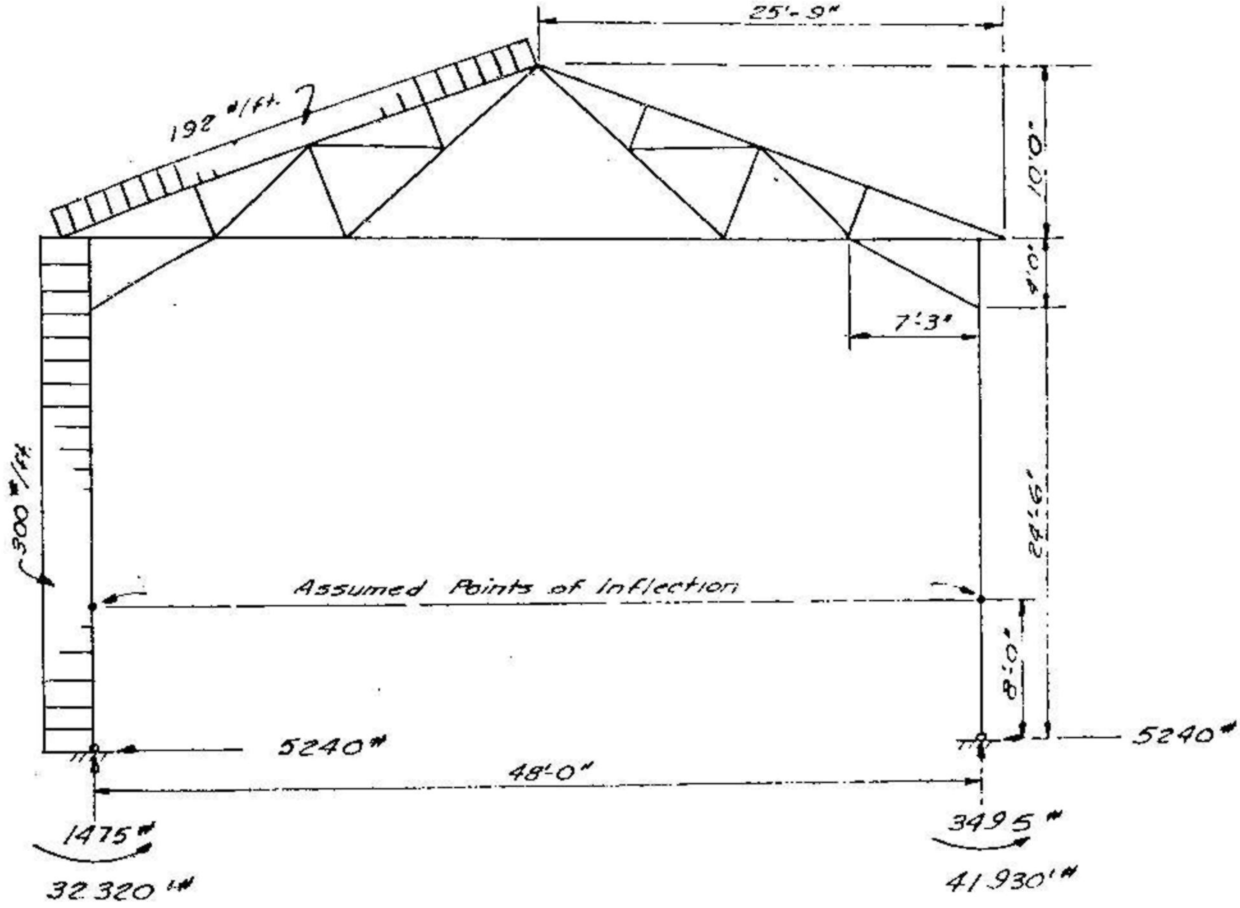
Scale: 1/8" = 1'-0"

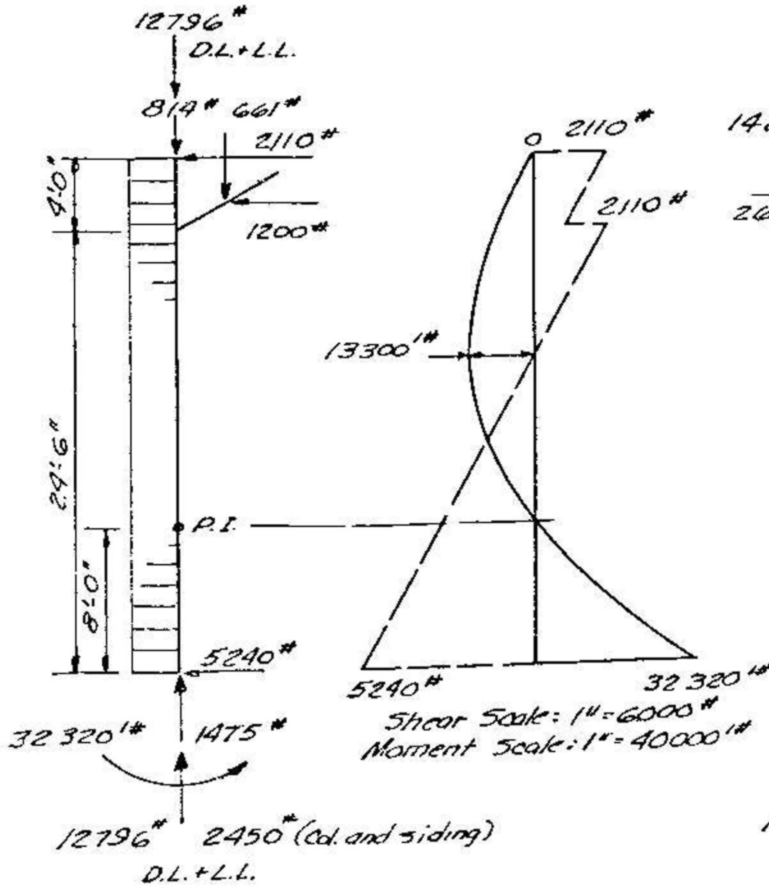
LIVE AND DEAD LOADS ON TRUSS

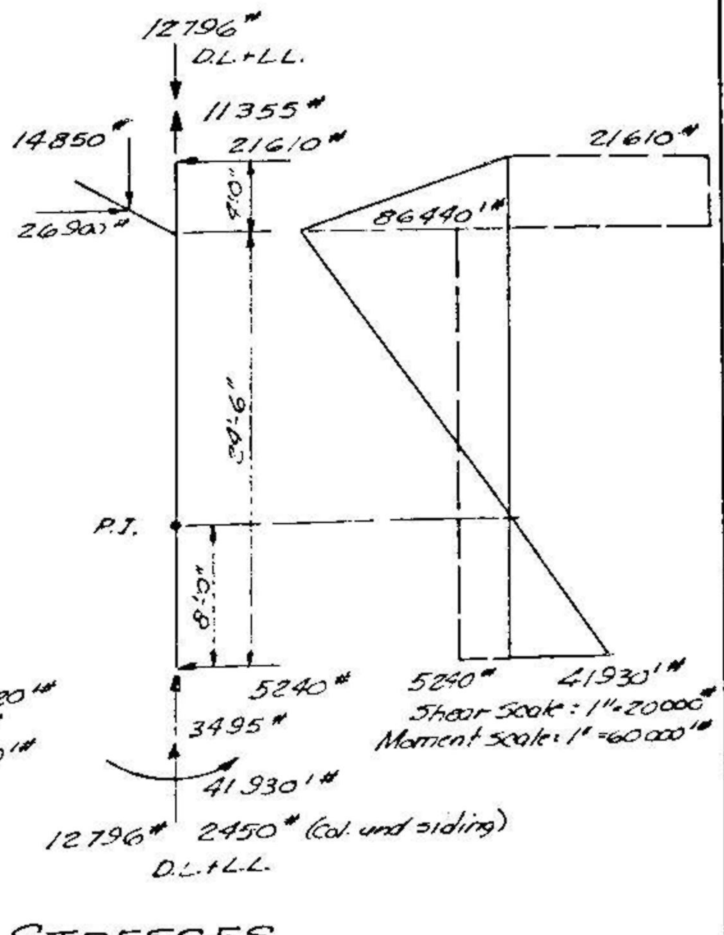


MEMBER	STRESS
L_0-U_1 & L_6-U_7	-31,000 #
U_1-U_2 & U_6-U_7	-29,900
U_2-U_3 & U_6-U_5	-28,800
U_3-U_4 & U_5-U_4	-27,600
L_0-L_1 & L_5-L_6	+28,900
L_1-L_2 & L_4-L_5	+25,400
L_2-L_3 & L_3-L_4	+17,500
L_1-U_1 & L_5-U_7	-3,000
L_1-U_2 & L_5-U_6	+3,700
L_2-U_2 & L_4-U_6	-6,100
L_2-L_7 & L_4-L_8	+8,000
L_7-U_2 & L_8-U_6	+3,700
L_7-U_3 & L_8-U_5	-3,000
L_7-U_4 & L_8-U_4	+11,800
L_3-U_4	0

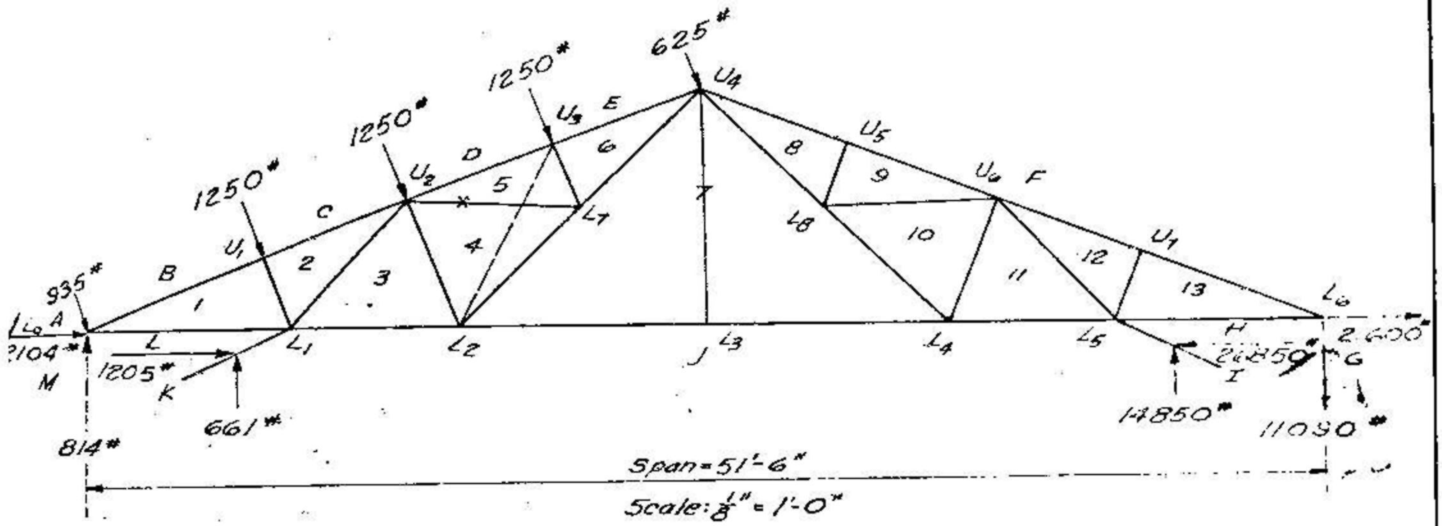
STRESS DIAGRAM FOR LIVE AND DEAD LOAD



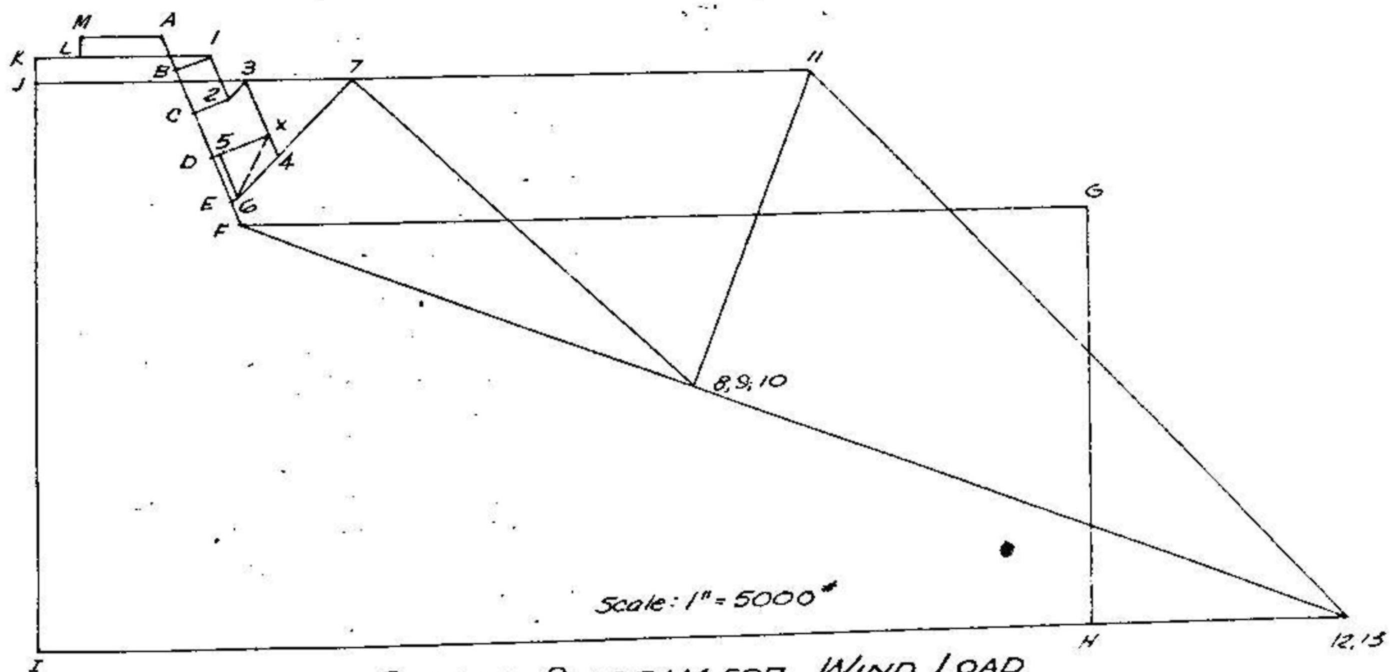




COLUMN STRESSES

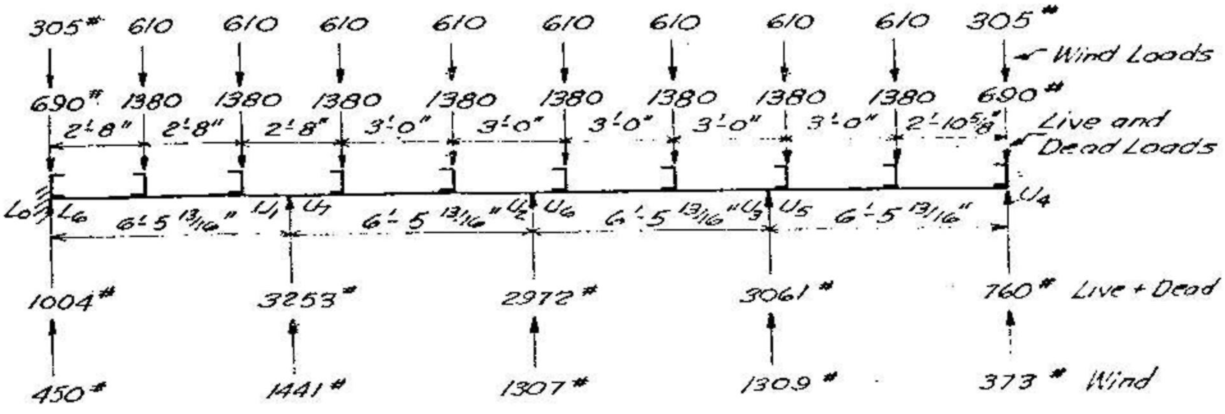


WIND LOAD ON TRUSS



STRESS DIAGRAM FOR WIND LOAD

Member	Stress	Member	Stress
L_0-U_1	+ 920#	L_6-U_7	+30450#
U_1-U_2	+ 920	U_6-U_7	+30450
U_2-U_3	+ 180	U_6-U_5	+12250
U_3-U_4	+ 180	U_5-U_4	+12250
L_0-L_1	− 3300	L_5-L_6	− 6700
L_1-L_2	− 5410	L_4-L_5	−19900
L_2-L_3	− 8300	L_3-L_4	− 8300
L_1-U_1	− 1250	L_5-U_7	0
L_1-U_2	+ 700	L_5-U_6	−20200
L_2-U_2	− 2190	L_4-U_6	+ 8850
L_2-L_7	+ 2900	L_4-L_8	−11700
L_7-U_2	+ 1550	L_8-U_6	0
L_7-U_3	− 1250	L_8-U_5	0
L_7-U_4	+ 4500	L_8-U_4	−11700
L_3-U_4	0	Right Brace	−30700
		Left Brace	− 1370

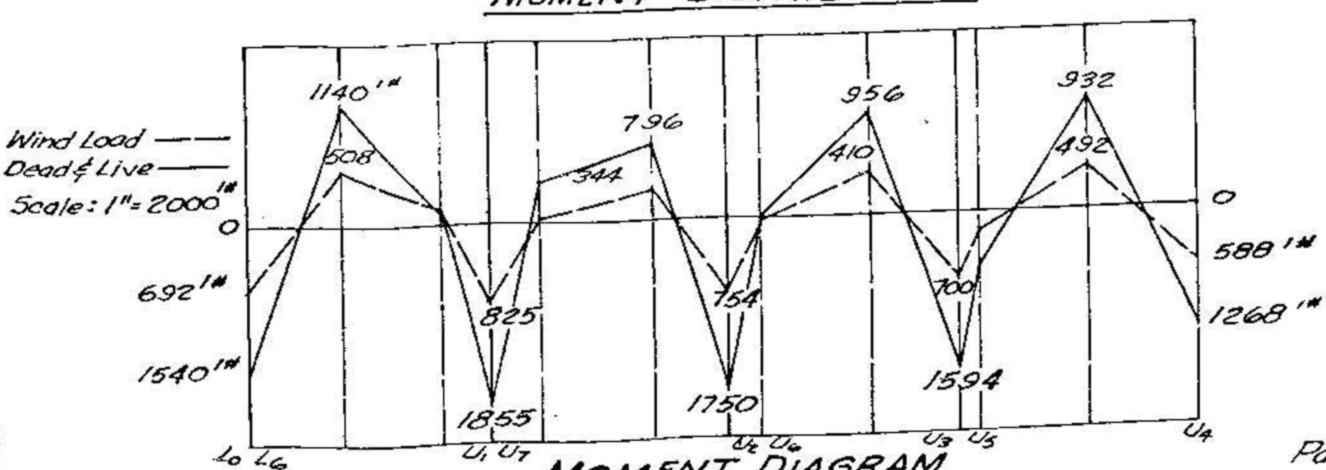


LOADS ON TOP CHORD

Dead and Live Loads

Division of unbalanced mom.	1	1/2	1/2	1/2	1/2	1/2	1/2	1
Carry-over Factor	1/2 →	← 1/2	1/2 →	← 1/2	1/2 →	← 1/2	1/2 →	← 1/2
Fixed-end Mom.	+1499	-1915	+1798	-1703	+1853	-1534	+1607	-1261
Balance	0	+79	+79	-75	-75	-37	-37	0
Carry-over	+39	0	-37	+39	-19	-37	0	-19
Balance	0	+18	+19	-10	-10	+18	+19	0
Carry-over	+9	0	-5	+9	+9	-5	0	+9
Balance	0	+3	+2	-9	-9	+3	+2	0
Carry-over	+1	0	-4	+1	+2	-4	0	+1
Balance	0	+2	+2	-2	-1	+2	+2	0
Carry-over	+1	0	-1	+1	+1	-1	0	+1
Balance	0	0	0	-1	-1	0	0	0
Σ	+1540	-1855	+1854	-1749	+1752	-1595	+1593	-1268
Wind Load								
Fixed-end Mom.	+674	-861	+800	-747	+797	-655	+736	-571
Balance	0	+31	+31	-25	-25	-40	-41	0
Carry-over	+15	0	-12	+15	-20	-12	0	-20
Balance	0	+6	+6	+3	+2	+6	+6	0
Carry-over	+3	0	+2	+3	+3	+2	0	+3
Balance	0	-1	-1	-3	-3	-1	-1	0
Carry-over	0	0	-1	0	0	-1	0	0
Balance	0	0	0	0	0	0	0	0
Σ	+692	-825	+824	-754	+754	-701	+700	-588

MOMENT DISTRIBUTION



MOMENT DIAGRAM

Member	Cross-sect. Area	Direct Stress							Least $\frac{L}{r}$	Allow. Col.** Unit Stress	Allow. Ten.** Unit Stress	Remarks
		Dead Load*	Live Load	Wind Load	Max. Stress	Min. Stress	Max. Unit Stress	Min. Unit Stress				
L_0-U_1	4.18 □"	- 7000#	- 24000#	+ 920#	- 6080#	- 31000#	- 1450#/□"	- 7400	52	- 15740#/□"	26700#/□"	See table below
U_1-U_2	4.18	- 6730	- 23170	+ 920	- 5810	- 29900	- 1390	- 7150	57	- 15420	26700	"
U_2-U_3	4.18	- 6480	- 22320	+ 180	- 6300	- 28800	- 1510	- 6880	57	- 15420	26700	"
U_3-U_4	4.18	- 6200	- 21400	+ 180	- 6120	- 27600	- 1460	- 6600	52	- 15740	26700	"
L_0-L_1	2.38	+ 6500	+ 22400	- 3300	+ 28900	- 3200	+ 12100	- 1340	88	- 13240	26700	O.K.
L_1-L_2	2.38	+ 5610	+ 19790	- 5410	+ 25400	+ 200	+ 10680	+ 80	104	- 11750	26700	O.K.
L_2-L_3	2.38	+ 3940	+ 13560	- 8300	+ 17500	- 4360	+ 7350	- 1830	200	- 5600	26700	O.K.
L_1-U_1	2.12	- 680	- 2320	- 1250	- 680	- 4250	- 321	- 2000	31	- 22200	26700	O.K.
L_1-U_2	1.06	+ 860	+ 2840	+ 700	+ 4400	+ 860	+ 4150	+ 810	81	- 18400	26700	O.K.
L_2-U_2	2.12	- 1370	- 4730	- 2190	- 1370	- 8290	- 650	- 3110	69	- 19600	26700	O.K.
L_2-L_7	2.12	+ 1800	+ 6200	+ 2900	+ 10900	+ 1800	+ 5150	+ 850	95	- 12640	26700	O.K.
L_7-U_2	1.06	+ 860	+ 2840	+ 1550	+ 5250	+ 860	+ 4950	+ 810	81	- 13800	26700	O.K.
L_7-U_3	2.12	- 680	- 2320	- 1250	- 680	- 4250	- 320	- 2000	31	- 22200	26700	O.K.
L_7-U_4	2.12	+ 2660	+ 9140	+ 4500	+ 16300	+ 2660	+ 7690	+ 1250	92	- 12900	26700	O.K.
L_3-U_4	1.06	0	0	0	0	0	0	0	193	- 5600	26700	O.K.
L_6-U_7	4.18	- 7000	- 24000	+ 30450	+ 23450	- 31000	+ 5610	- 7400	52	- 15740	26700	See table below
U_6-U_7	4.18	- 6730	- 23170	+ 30450	+ 23720	- 29900	+ 5670	- 7150	57	- 15420	26700	"
U_6-U_5	4.18	- 6480	- 22320	+ 12250	+ 5770	- 28800	+ 1380	- 6880	57	- 15420	26700	"
U_5-U_4	4.18	- 6200	- 21400	+ 12250	+ 6050	- 27600	+ 1450	- 6600	52	- 15740	26700	"
L_5-L_6	2.38	+ 6500	+ 22400	- 6700	+ 28900	- 200	+ 12100	- 84	88	- 13240	26700	O.K.
L_4-L_5	2.38	+ 5610	+ 19790	- 19900	+ 25400	- 14290	+ 10680	- 5970	104	- 11750	26700	O.K.
L_3-L_4	2.38	+ 3940	+ 13560	- 8300	+ 17500	- 4360	+ 7350	- 1830	200	- 5600	26700	O.K.
L_5-U_7	2.12	- 680	- 2320	0	- 680	- 3000	- 320	- 1420	31	- 16560	26700	O.K.
L_5-U_6	1.06	+ 860	+ 2840	- 20200	+ 3700	- 19340	+ 3500	- 18200	81	- 18400	26700	O.K.
L_4-U_6	2.12	- 1370	- 4730	+ 8850	+ 7480	- 6000	+ 3530	- 2830	69	- 14700	26700	O.K.
L_4-L_8	2.12	+ 1800	+ 6200	- 11700	+ 8000	- 9900	+ 3770	- 4660	95	- 12640	26700	O.K.
L_8-U_6	1.06	+ 860	+ 2840	0	+ 3700	+ 860	+ 3490	+ 81	81	- 13240	26700	O.K.
L_8-U_5	2.12	- 680	- 2320	0	+ 680	- 3000	+ 320	- 1420	31	- 16,560	26700	O.K.
L_8-U_4	2.12	+ 2660	+ 9140	- 11700	+ 11800	- 9040	+ 5560	- 4260	92	- 12900	26700	O.K.
Right Brace	2.38	0	0	- 30700	0	- 30700	0	- 12900	104	- 15700	26700	O.K.
Left Brace	2.38	0	0	- 1370	0	- 1370	0	- 580	104	- 15700	26700	O.K.

* Dead load stresses are 22 ½ % of the dead + live load stresses.
** Increased by 33⅓ % when dead, live and wind loads are acting simultaneously.

Direct Unit Stress of Truss Members

Member	Combined Stresses			$\frac{f_a}{F_a}+\frac{f_b}{F_b}$	Combined Stresses			$\frac{f_a}{F_a}+\frac{f_b}{F_b}$	Combined Stresses			$\frac{f_a}{F_a}+\frac{f_b}{F_b}$
	Max. Direct Unit Stress	Moment	Bend. Unit Stress		Min. Direct Unit Stress	Moment	Bend. Unit Stress		Unit Stress (All Forces)	Moment	Bend. Unit Stress	
L_0-U_1	- 1450#/□"	1233'#	± 6180#/□"	0.30	- 7400#/□"	1855'#	± 9280#/□"	0.71	- 7200#/□"	2680'#	± 13400#/□"	0.85
U_1-U_2	- 1390	1233	± 6180	0.30	- 7150	1855	± 9280	0.70	- 6940	2680	± 13400	0.84
U_2-U_3	- 1510	1148	± 5740	0.28	- 6880	1750	± 8760	0.67	- 6850	2504	± 12500	0.81
U_3-U_4	- 1460	1059	± 5300	0.27	- 6600	1594	± 7970	0.62	- 6550	2294	± 11470	0.75
U_4-U_5	+ 1450	1059	± 5300	0.25	- 6600	1594	± 7970	0.62	- 131	2294	± 11470	0.43
U_5-U_6	+ 1380	1148	± 5740	0.26	- 6880	1750	± 8760	0.67	- 131	2504	± 12500	0.47
U_6-U_7	+ 5670	1233	± 6180	0.44	- 7150	1855	± 9280	0.70	- 3960	2680	± 13400	0.69
U_7-L_6	+ 5610	1233	± 6180	0.44	- 7400	1855	± 9280	0.71	- 3660	2680	± 13400	0.68

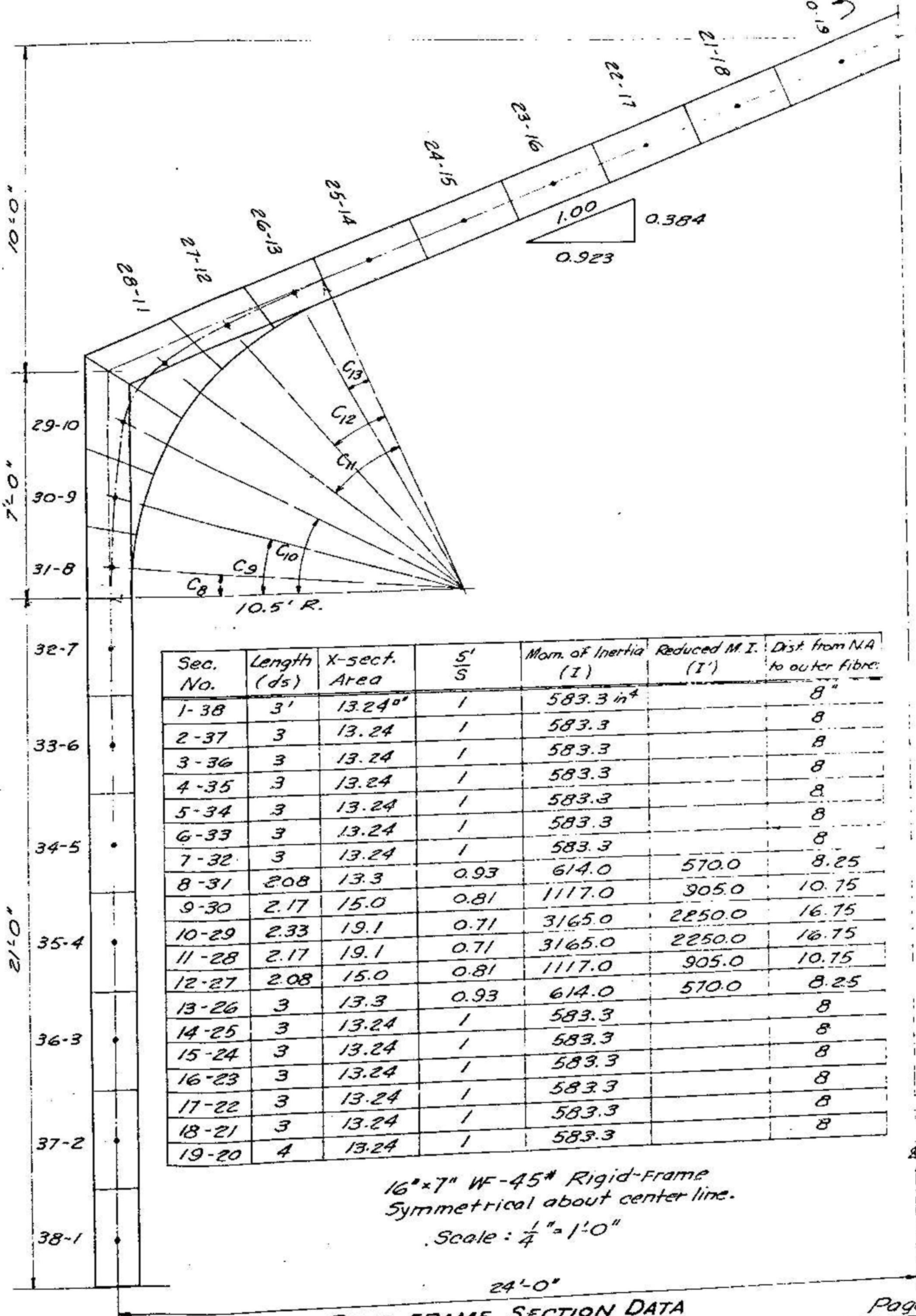
Chord member is composed of 2 ∠s - 4 × 3 5/16, $A = 4.18$ □" and $\frac{I}{c} = 2.4$.
All members are O.K. since $\frac{f_a}{F_a}+\frac{f_b}{F_b} < 1$ in all cases.

Combined Unit Stress in Top Chord

Col.*	Direct Stress	Max. Moment	Direct Unit Stress	Bend. Unit Stress	$\frac{L}{r}$	Allow. Col.** Unit Stress	Allow. Bend** Unit Stress	$\frac{f_a}{F_a}+\frac{f_b}{F_b}$	Remarks
Left	15265#	13330'#	1675#/□"	5830#/□"	85	17980#/□"	26700#/□"	.311	O.K.
Right	16641	86500	1825	37900	85	17980	26700	1.52	N.G.

* 8" WF-31#; $A = 9.12$ □", $r_{xx} = 3.47$, $(\frac{I}{c})_{xx} = 27.4$
** Increased by 33⅓ % in accordance with specs.

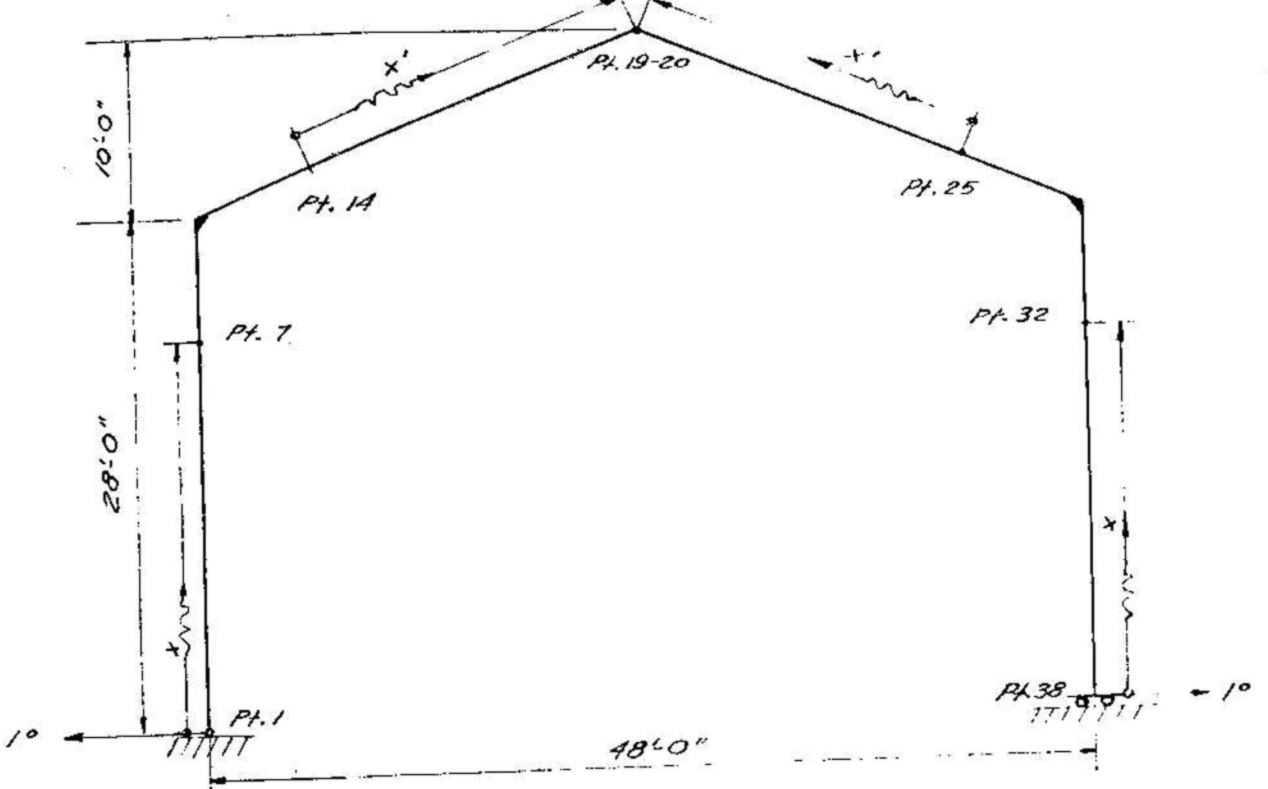
Column Unit Stresses



Sec. No.	Length (ds)	X-sect. Area	S'/S	Mom. of Inertia (I)	Reduced M.I. (I')	Dist. from N.A. to outer fibre.
1-38	3'	13.24□"	1	$583.3\ in^4$		8"
2-37	3	13.24	1	583.3		8
3-36	3	13.24	1	583.3		8
4-35	3	13.24	1	583.3		8
5-34	3	13.24	1	583.3		8
6-33	3	13.24	1	583.3		8
7-32	3	13.24	1	583.3		8
8-31	208	13.3	0.93	614.0	570.0	8.25
9-30	2.17	15.0	0.81	1117.0	905.0	10.75
10-29	2.33	19.1	0.71	3165.0	2250.0	16.75
11-28	2.17	19.1	0.71	3165.0	2250.0	16.75
12-27	2.08	15.0	0.81	1117.0	905.0	10.75
13-26	3	13.3	0.93	614.0	570.0	8.25
14-25	3	13.24	1	583.3		8
15-24	3	13.24	1	583.3		8
16-23	3	13.24	1	583.3		8
17-22	3	13.24	1	583.3		8
18-21	3	13.24	1	583.3		8
19-20	4	13.24	1	583.3		8

16"×7" WF-45# Rigid-Frame
Symmetrical about center line.
Scale: $\frac{1}{4}$" = 1'-0"

RIGID-FRAME SECTION DATA

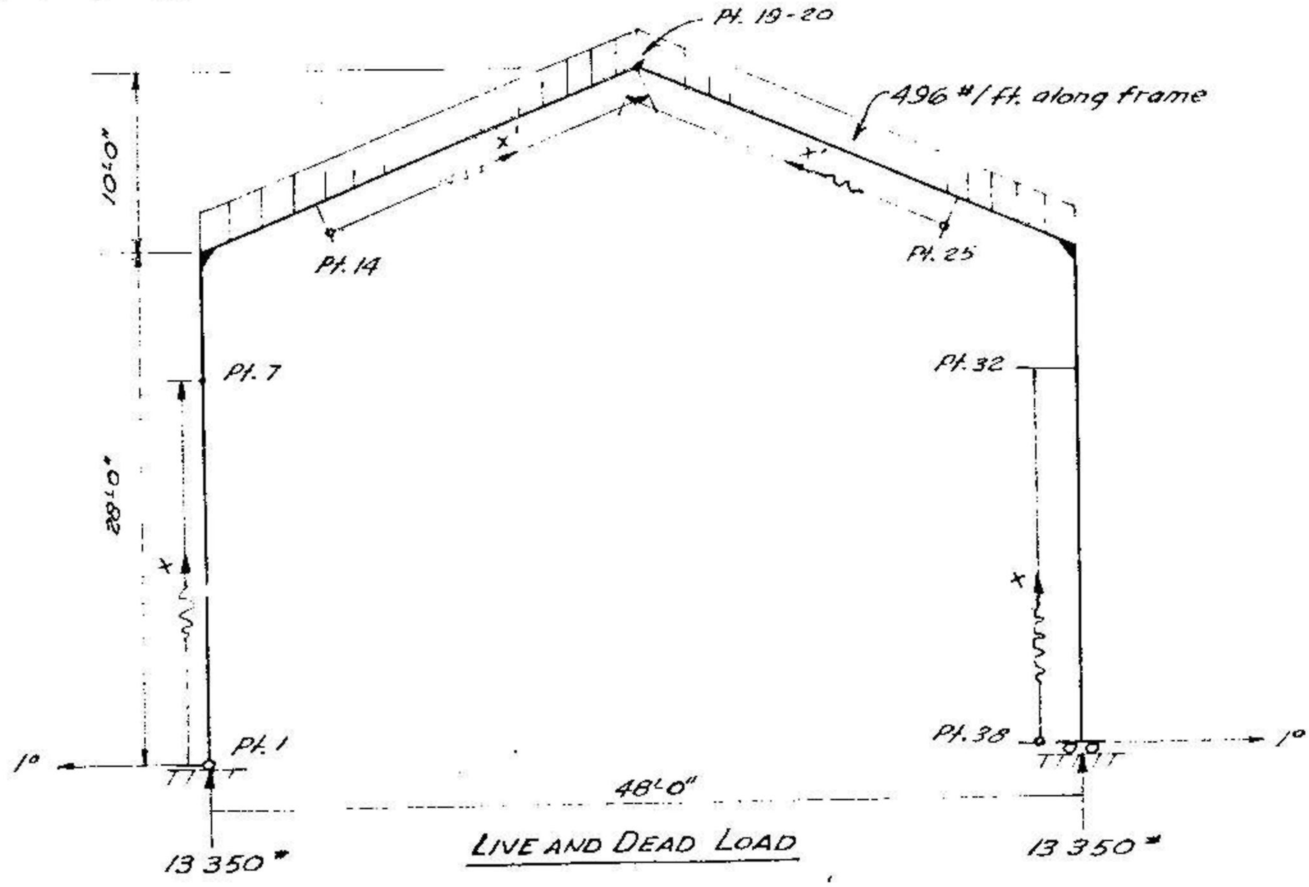


LOADING

FOR VIRTUAL UNIT LOAD

Sect. No.	Dummy Load Moment ~ (m')	$(m')^2\,ds$	$\int \frac{(m')^2\,ds \times 144}{EI}$
1 thru 7	(1)(x)	$\int_0^{21} x^2\,dx$	0.000025550
8	(1)(22.0)	$(22.0)^2(2.08)$	0.000008620
9	(1)(24.06)	$(24.06)^2(2.17)$	0.000006680
10	(1)(26.5)	$(26.5)^2(2.33)$	0.000003230
11	(1)(28.25)	$(28.25)^2(2.33)$	0.000004150
12	(1)(29.31)	$(29.31)^2(2.17)$	0.000009870
13	(1)(30.31)	$(30.31)^2(2.08)$	0.000016380
14 thru 19	(1)(28.0+0.384x')	$\int_{7.0}^{26.0}(28.0+0.384x')^2\,dx'$	0.000186000
20 thru 25	(1)(28.0+0.384x')	$\int_{7.0}^{26.0}(28.0+0.384x')^2\,dx'$	0.000186000
26	(1)(30.31)	$(30.31)^2(2.08)$	0.000016380
27	(1)(29.31)	$(29.31)^2(2.17)$	0.000009870
28	(1)(28.25)	$(28.25)^2(2.33)$	0.000004150
29	(1)(26.5)	$(26.5)^2(2.33)$	0.000003230
30	(1)(24.06)	$(24.06)^2(2.17)$	0.000006680
31	(1)(22.0)	$(22.0)^2(2.08)$	0.000008620
32 thru 38	(1)(x)	$\int_0^{21} x^2\,dx$	0.000025550
	Deflection, $d_{10} = \Sigma \int \frac{(m')^2\,ds}{EI}$	=	0.000520960 ft.

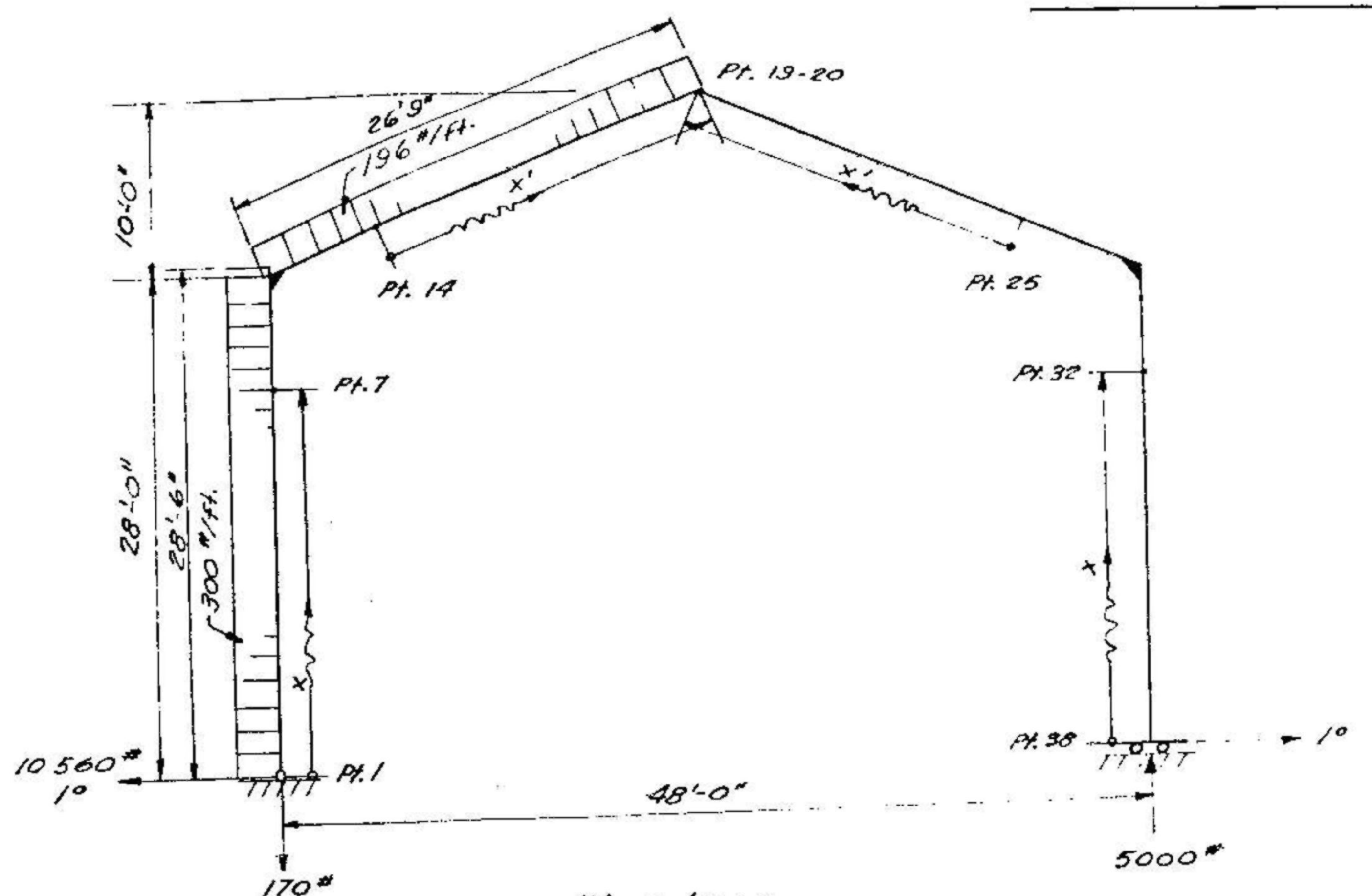
DEFLECTION DUE TO DUMMY LOAD



LIVE AND DEAD LOAD

Sect. No.	Dead and Live Load Moment (M)	Dummy Load Moment (m')	$\int \frac{M m' ds \times 144}{EI}$
1 thru 8	0	$(1)(x)$	0
9	$(13350)(0.25)$	$(1)(24.06)$	0.000924
10	$(13350)(0.50)$	$(1)(26.5)$	0.000894
11	$(13350)(1.86)-(4.96)(2.5)(1.25)$	$(1)(28.25)$	0.003270
12	$(13350)(3.86)-(4.96)(4.9)(2.45)$	$(1)(29.31)$	0.015500
13	$(13350)(5.66)-(496)(6.75)(3.1)$	$(1)(30.4)$	0.034800
14 thru 19	$(13350)(0.923x')-(496)(x)(\frac{0.923x'}{2})$	$(1)(27.75+0.384x')$	0.731000
20 thru 25	$(13350)(0.923x)-(496)(x)(\frac{0.923x'}{2})$	$(1)(27.75+0.384x')$	0.731000
26	$(13350)(5.66)-(496)(6.75)(3.1)$	$(1)(30.4)$	0.034800
27	$(13350)(3.86)-(496)(4.9)(2.45)$	$(1)(29.31)$	0.015500
28	$(13350)(1.86)-(496)(2.5)(1.25)$	$(1)(28.25)$	0.003270
29	$(13350)(0.50)$	$(1)(26.5)$	0.000894
30	$(13350)(0.25)$	$(1)(24.06)$	0.000924
31 thru 38	0	$(1)(x)$	0
	Deflection, $\delta_{L+D} = \Sigma \int \frac{M m' ds}{EI}$ =		1.572776 ft.

DEFLECTION DUE TO LIVE AND DEAD LOAD



WIND LOAD

Sect. No.	Wind Load Moment (M)	Dummy Load Mom. (m')	$\int \frac{Mm'ds \times 144}{EI}$
1 thru 7	(10560)(x)-(300)(x)(x/2)	(1)(x)	0.208000
8	(10560)(22)-(300)(22)(11)	(1)(22)	0.063000
9	(10560)(24.06)-(300)(24.06)(12.0)	(1)(24.06)	0.046200
10	(10560)(26.5)-(300)(26.5)(13.25)	(1)(26.5)	0.023000
11	(10560)(28.25)-(8550)(14.25)-(170)(1.83)-(196)(2)	(1)(28.5)	0.024700
12	(10560)(29.31)-(8550)(15.1)-(170)(3.75)-(196)(4.3)(2.2)	(1)(29.31)	0.060200
13	(10560)(30.31)-(8550)(16.1)-(170)(5.66)-(196)(6.6)(3.3)	(1)(30.31)	0.098500
14 thru 19	(10560)(27.75+.384x')-(8550)(13.87+.384x') -(170)(.923x')-(196)(x')(x'/2)	(1)(27.75+.384x')	0.817000
20 thru 25	(5000)(0.923x)	(1)(28.0+.384x)	0.325000
26	(5000)(5.67)	(1)(30.31)	0.016900
27	(5000)(3.75)	(1)(29.31)	0.006220
28	(5000)(1.83)	(1)(28.25)	0.001230
29	(5000)(0.50)	(1)(26.5)	0.000315
30	(5000)(0.25)	(1)(24.06)	0.000346
31 thru 38	0	(1)(x)	0
	Deflection, $d_W = \Sigma \int \frac{Mm'ds}{EI}$ =		1.690611 ft.

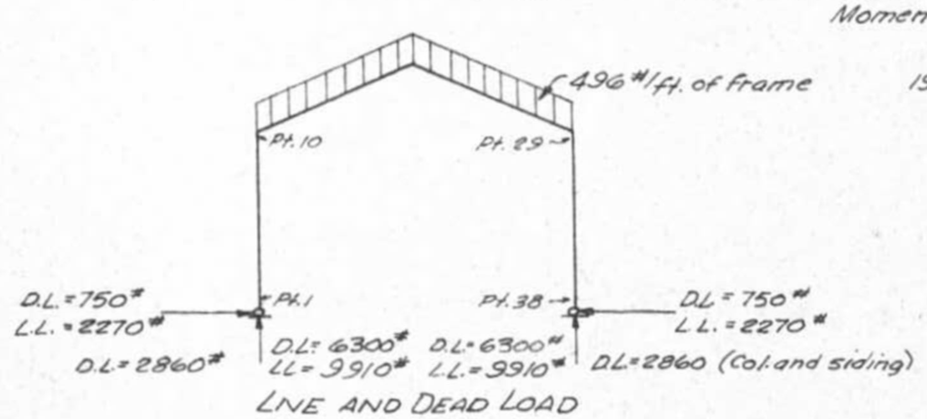
DEFLECTION DUE TO WIND LOAD

Sect No.	X-Sect Area	Direct Stresses: Dead Load	Live Load	Wind Load	Max. Stress	Min. Stress	Total Stress	(A) - Max. Unit Stress	(B) - Min. Unit Stress	(C) - Total Unit Stress	Case A: Moment	Bend. Unit Stress	$\frac{f_a}{F_a}+\frac{f_b}{F_b}$	Case B: Moment	Bend. Unit Stress	$\frac{f_a}{F_a}+\frac{f_b}{F_b}$	Case C: Moment	Bend. Unit Stress	$\frac{f_a}{F_a}+\frac{f_b}{F_b}$
1	13.24	- 6150#	- 9910#	+ 170#	- 5980#	- 16060#	- 15890#	- 450#/□"	- 1210#/□"	- 1200#/□"	- 9660"	± 1600#/□"	0.11	+ 4530"	± 750#/□"	0.11	- 6010"	± 995#/□"	0.10
2	13.24	- 5850	- 9910	+ 170	- 5680	- 15760	- 15590	- 430	- 1190	- 1180	- 26860	± 4450	.25	+ 13600	± 2250	.19	- 15960	± 2640	.16
3	13.24	- 5550	- 9910	+ 170	- 5380	- 15460	- 15290	- 410	- 1170	- 1150	- 40770	± 6750	.37	+ 22600	± 3760	.26	- 23260	± 3850	.20
4	13.24	- 5250	- 9910	+ 170	- 5080	- 15160	- 14990	- 380	- 1150	- 1130	- 51480	± 8500	.45	+ 31700	± 5250	.33	- 27950	± 4630	.23
5	13.24	- 4950	- 9910	+ 170	- 4780	- 14860	- 14690	- 360	- 1120	- 1110	- 61200	± 10120	.57	+ 40800	± 6770	.40	- 29800	± 4940	.25
6	13.24	- 4650	- 9910	+ 170	- 4480	- 14560	- 14390	- 340	- 1100	- 1080	- 67400	± 10980	.57	+ 49800	± 8250	.48	- 29100	± 4820	.24
7	13.24	- 4350	- 9910	+ 170	- 4180	- 14260	- 14090	- 320	- 1080	- 1060	- 71000	± 11750	.61	+ 58800	± 9750	.56	- 25500	± 4220	.22
8	13.3	- 4100	- 9910	+ 170	- 3930	- 14020	- 13850	- 300	- 1060	- 1040	- 71700	± 12400	.64	+ 66500	± 11510	.64	- 20500	± 3390	.18
9	15.0	- 3990	- 9910	+ 170	- 3820	- 13900	- 13730	- 250	- 930	- 920	- 70900	± 10100	.52	+ 72800	± 10400	.58	- 14800	± 24700	.13
10	19.1	- 3650	- 9910	+ 170	- 3480	- 13560	- 13390	- 180	- 710	- 700	- 68500	± 6120	.32	+ 80000	± 7150	.40	- 7000	± 11600	.08
11	19.1	- 2010	- 5520	- 1672	- 2010	- 9202	- 9202	- 100	- 480	- 480	+ 79500	± 7110	.41	- 21900	± 1960	.09	- 21900	± 1960	.09
12	15.0	- 1800	- 5210	- 1700	- 1800	- 8710	- 8710	- 120	- 580	- 580	+ 76690	± 10980	.56	- 38280	± 5410	.23	- 38280	± 5410	.23
13	13.3	- 1690	- 4890	- 2070	- 1690	- 8650	- 8650	- 130	- 650	- 650	+ 74380	± 12900	.65	- 52030	± 9030	.37	- 52030	± 9030	.37
14	13.24	- 1610	- 4570	- 2310	- 1610	- 8510	- 8510	- 120	- 640	- 640	+ 72190	± 11920	.60	- 63070	± 10450	.43	- 63070	± 10450	.43
15	13.24	- 1410	- 4140	- 2760	- 1410	- 8310	- 8310	- 110	- 630	- 630	+ 69630	± 11510	.59	- 74850	± 12400	.49	- 74850	± 12400	.49
16	13.24	- 1270	- 3710	- 3160	- 1270	- 8140	- 8140	- 100	- 610	- 610	+ 68300	± 11300	.58	- 78050	± 12930	.51	- 78050	± 12930	.51
17	13.24	- 1140	- 3290	- 3560	- 1140	- 7990	- 7990	- 90	- 600	- 600	+ 67500	± 11180	.57	- 79300	± 13140	.52	- 79300	± 13140	.52
18	13.24	- 1000	- 2840	- 4020	- 1000	- 7860	- 7860	- 80	- 590	- 590	+ 68200	± 11300	.57	- 71900	± 11900	.48	- 71900	± 11900	.48
19	13.24	- 850	- 2350	- 4490	- 850	- 7690	- 7690	- 60	- 580	- 580	+ 70100	± 11600	.56	- 57400	± 9520	.39	- 57400	± 9520	.39
20	13.24	- 850	- 2350	- 4910	- 850	- 8110	- 8110	- 60	- 610	- 610	+ 70100	± 11600	.56	- 35500	± 5880	.25	- 35500	± 5880	.25
21	13.24	- 1000	- 2840	- 4910	- 1000	- 8750	- 8750	- 80	- 660	- 660	+ 68200	± 11300	.57	- 21300	± 3530	.16	- 21300	± 3530	.16
22	13.24	- 1140	- 3290	- 4910	- 1140	- 9340	- 9340	- 90	- 700	- 700	+ 67500	± 11180	.57	- 3700	± 610	.05	- 3700	± 610	.05
23	13.24	- 1270	- 3710	- 4910	- 1270	- 9890	- 9890	- 100	- 750	- 750	+ 6830	± 11300	.58	+ 19750	± 3280	.16	+ 19750	± 3280	.16
24	13.24	- 1410	- 4140	- 1410	- 1410	- 10460	- 10460	- 110	- 790	- 790	+ 69630	± 11510	.59	+ 42750	± 7090	.33	+ 42750	± 7090	.33
25	13.24	- 1610	- 4570	- 4910	- 1610	- 11090	- 11090	- 120	- 840	- 840	+ 72190	± 11920	.60	+ 73350	± 12180	.51	+ 73350	± 12180	.51
26	13.3	- 1690	- 4890	- 4910	- 1690	- 11490	- 11490	- 130	- 870	- 870	+ 74380	± 12900	.65	+ 97470	± 16900	.67	+ 97470	± 16900	.67
27	15.0	- 1800	- 5210	- 4910	- 1800	- 11920	- 11920	- 120	- 790	- 790	+ 76690	± 10920	.56	+ 121370	± 17300	.69	+ 121370	± 17300	.69
28	19.1	- 2010	- 5520	- 4910	- 2010	- 12440	- 12440	- 100	- 650	- 650	+ 79500	± 7110	.41	+ 147200	± 13180	.53	+ 147200	± 13180	.53
29	19.1	- 3650	- 9910	- 5000	- 3650	- 18560	- 18560	- 190	- 970	- 970	+ 19700	± 760	.04	+ 168000	± 15010	.60	+ 168000	± 15010	.60
30	15.0	- 3990	- 9910	- 5000	- 3990	- 18900	- 18900	- 260	- 1260	- 1260	+ 18100	± 2580	.15	+ 153000	± 21800	.88	+ 153000	± 21800	.88
31	13.3	- 4100	- 9910	- 5000	- 4100	- 19010	- 19010	- 310	- 1430	- 1430	+ 16500	± 2860	.16	+ 139500	± 24180	.94	+ 139500	± 24180	.94
32	13.24	- 4350	- 9910	- 5000	- 4350	- 19260	- 19260	- 330	- 1450	- 1450	+ 14600	± 2420	.14	+ 123500	± 20480	.84	+ 123500	± 20480	.84
33	13.24	- 4650	- 9910	- 5000	- 4650	- 19560	- 19560	- 350	- 1480	- 1480	+ 12400	± 2250	.13	+ 104600	± 17350	.72	+ 104600	± 17350	.72
34	13.24	- 4950	- 9910	- 5000	- 4950	- 19860	- 19860	- 370	- 1500	- 1500	+ 10100	± 1670	.10	+ 85500	± 14180	.60	+ 85500	± 14180	.60
35	13.24	- 5250	- 9910	- 5000	- 5250	- 20160	- 20160	- 400	- 1520	- 1520	+ 7900	± 1310	.10	+ 66500	± 11010	.48	+ 66500	± 11010	.48
36	13.24	- 5550	- 9910	- 5000	- 5550	- 20460	- 20460	- 420	- 1550	- 1550	+ 5620	± 930	.08	+ 47500	± 7880	.37	+ 47500	± 7880	.37
37	13.24	- 5850	- 9910	- 5000	- 5850	- 20760	- 20760	- 440	- 1560	- 1560	+ 3380	± 560	.06	+ 28500	± 4720	.25	+ 28500	± 4720	.25
38	13.24	- 6150	- 9910	- 5000	- 6150	- 21060	- 21060	- 460	- 1590	- 1590	+ 1120	± 180	.04	+ 9500	± 1570	.13	+ 9500	± 1570	.13

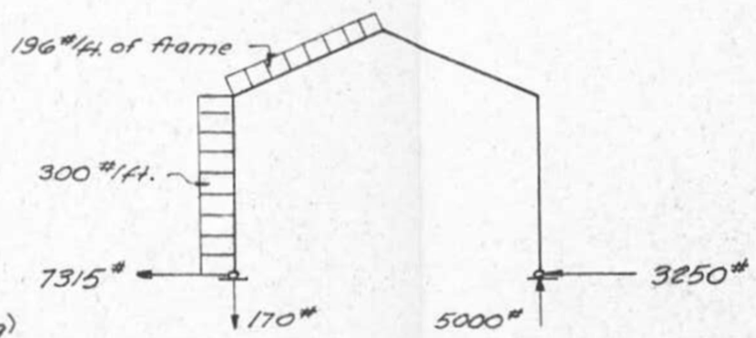
* Includes moment due to thermal expansion (direct stress insignificant).

Constant section rigid frame: 16×7 WF-45#; $\frac{I}{c}$ = 72.4; r = 6.64.

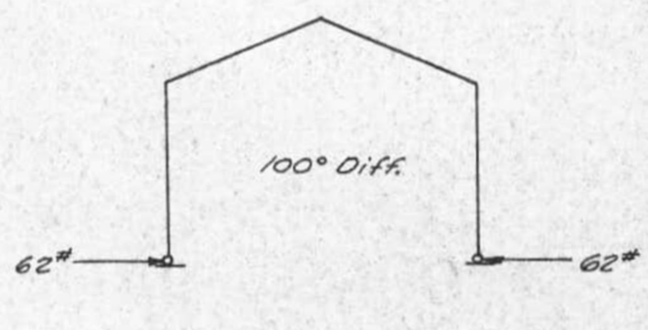
Moment designation: tension in outer fibre ~ positive moment.



LNE AND DEAD LOAD

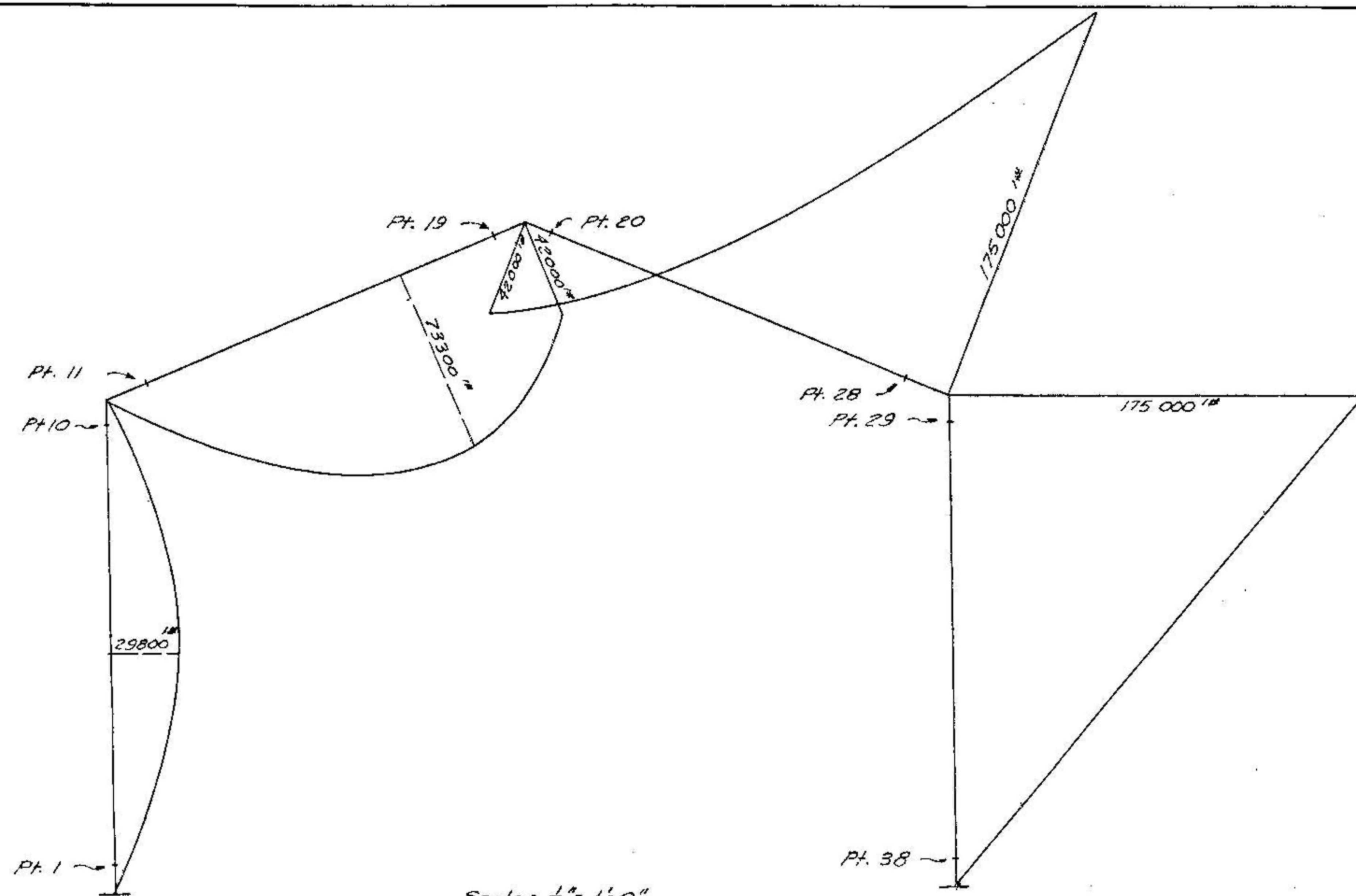


WIND LOAD



THERMAL EXPANSION

RIGID FRAME REACTIONS AND UNIT STRESSES



Scale : $\frac{1}{8}$" = 1'-0"
Moment Scale : 1" = 60,000 '#

MOMENT DIAGRAM FOR COMBINED LOADING