

RODER UK

FILE NAME PARTY TENT UP TO

10M RODER FRAME

TENT

EVALUATION

THE 10 METER RODER FRAME TENT EVALUATION

F.T.L./Happold, Incorporated

Prepared for
Mr. Richard Martin, Roder U.S.A.
June 17, 1996

EVALUATION SUMMARY AND RECOMMENDATIONS

This report, based on the technical background information provided by Roder U.S.A., covers the structural evaluation of their 10 meter Aluminum Frame Tent in accordance with the U.S. Building Code Requirements. The specifications outlined in the American National Standards Institute / American Society of Civil Engineers (ANSI/ASCE) 7-93: "Minimum Design Loads for Buildings and Other Structures" were followed in determining the integrity of the structure. This document was intended to serve as a basis for the acceptability of these "temporary structures" under the standard design wind loads at varying levels of exposure (terrain and wind velocities): Table 0-1 presents the wind pressures (loads) under which the 10 meter frame was tested.

F.T.L./Happold, Incorporated compiled this report based on the existing tent system with reference to the applicable building codes in the U.S. This report includes the loading used in the analysis and gives an indication as to what wind exposure the structure is suitable for. Certification of this document only shows that the Professional Engineer of that particular state is in agreement with the report's contents. It does not, however, imply that the structure is generally suitable for use within that state, or that every installation is covered by the report.

Computer-aided structural frame analyses were involved in the course of the investigation. Different load combinations were considered to find the critical aspects of the design. Member and detail checks were established to derive the conclusions for the entire report.

With such, F.T.L./Happold, Incorporated arrived at the following conclusions and recommendations:

1. The 10 meter Frame Tent and its elements (main flexural) are adequate based on the structural analysis performed in accordance with the nationally recognized wind speeds and exposure classes (as shown in Table 0-1). The 10 meter version was successfully rated up to 70 mph in a Class C exposure provided that the 94/48/3 aluminum profile is used.
2. The frames should be anchored securely to the ground to prevent the possibility of uplift in the event of wind suction, especially when the wind hits the structure parallel to its ridge. Considerations should also be given to the resulting horizontal reactions (Part 7.2.5: Base Plates).

Maximum Uplift : 0.73 K (Class C, 70 mph)
Maximum Shear : 0.72 K (Class C, 70 mph)

The figures given are per base plate.

3. The structure should be set on firm and unyielding ground. This ground should sufficiently contain the bearing pressures of the base plates as well as the tractive forces of the rod anchors.

Maximum Pressure : 0.41 K (Class C, 70 mph)

The figure given is per base plate.

It is thereby understood that the responsibility of proper installation according to the plans rests upon the contractor. Such includes, but is not limited to, ensuring the following:

1. that the cables are always held taut,
2. that the fabric is stretched tight enough to prevent the development of pockets and to maintain the prescribed roof gradient, and
3. that the purlins are installed securely against the rafters so as to resist the calculated loads.

Finally, reference is made to the condition that the live load allowances (Part 4.2):

left rafter centerspan : 125 lbs
 ridge : 250 lbs
 right rafter centerspan : 125 lbs

must not be exceeded at any time for the conclusions of this report to remain valid.

10 METER RODER FRAME (13.35 FT MAXIMUM HEIGHT)
 WIND PRESSURES, q (psf)

Exposure	60 mph	70 mph	80 mph	90 mph
Class A	1.55	2.11	2.75	3.48
Class B	3.34	4.54	5.93	7.51
Class C	5.77	7.85	10.26	12.98
Class D	7.54	10.26	13.41	16.97

The 10 meter version is suitable for up to a Class C, 70 mph wind exposure.

Refer to Part 8 of this report.

The stricken values are the unsuitable pressures for the 10 m Roder Frame Tent.

Table 0-1 10 m Roder Frame Tent Allowable Exposure Chart

where, according to ANSI/ASCE 7-93, p. 14:

Exposure A

Large city centers with at least 50% of the buildings having a height in excess of 70 feet. Use of this exposure category shall be limited to those areas for which terrain representative of Exposure A prevails in the upwind direction for a distance of at least one-half mile or 10 times the height of the building or structure, whichever is greater. Possible channeling effects or increased velocity pressures due to the building or structure being located in the wake of adjacent buildings shall be taken into account.

Exposure B

Urban or sub-urban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger. Use of this exposure category shall be limited to those areas for which terrain representative of Exposure B prevails in the upwind direction for a distance of at least 1500 feet or 10 times the height of the building or structure, whichever is greater.

Exposure C

Open terrain with scattered obstructions having heights generally less than 30 feet. This category includes flat, open country and grasslands.

Exposure D

Flat, unobstructed areas exposed to wind flowing over large bodies of water. This exposure shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 1500 feet or 10 times the height of the building or structure, whichever is greater.

TABLE OF CONTENTS

Evaluation Summary and Recommendations

Table of Contents

Part 1 Summary and Background Information

- 1.1 Nature of the Evaluation
- 1.2 Objectives

Part 2 Structural Framing Plan

Part 3 Section and Material Properties

- 3.1 Material Distribution
- 3.2 Section Properties
- 3.3 Material Properties and Design Criteria

Part 4 Load Assumptions

- 4.1 Dead Loads
- 4.2 Live Loads
- 4.3 Wind Loads
- 4.4 General Load Cases and Combinations

Part 5 Structural Analysis and Results

- 5.1 Computer Model
- 5.2 ROBOT V6 Results

Part 6 Critical Loads

Part 7 Structural Investigations

- 7.1 Primary Elements
- 7.2 Secondary Elements

Part 8 Acceptability Under Variable Exposure and Wind Conditions

- 8.1 Relative Stress Levels
- 8.2 Allowable Exposure Chart

Appendix

- A Profiles
- B ANS/ASCE 7-93 Wind Load Specifications
- C ROBOT V6 Listings

Part 1

Introduction

1.1 Nature of the Evaluation

The 10 meter Roder Frame Tent consists of a series of pin-supported, double joint frames spanning the tent hall width. These frames are the main supporting elements and are made up of custom-designed hollow profiles of structural aluminum alloy (Al Mg Si - 1F28). An equivalent American alloy, the 6061-T6, is detailed in the U.S. Aluminum Construction Manual and shall be the basis of material property and allowable stress determinations. The structure is systematically reinforced with steel inserts at the ridge and eaves locations. These moment resisting elements as well as the other connection components of the frame are made of galvanized steel (St 37 and St 52).

These tents are classified as temporary structures. Their installation and use are restricted to certain seasons and environmental conditions. In lieu of this, snow loads are neglected. If such occurs, acceptable means of snow melting or removal, and interior heating shall be immediately employed. Further, the tent should be maintained closed at all unused times to prevent the possibility of an internal pressure build-up which is not considered in the succeeding stability calculations of this report. It is also assumed that adequate pressure leakage at the side walls is always available.

In general, five (5) load cases and eight (8) load combinations corresponding to a Class C exposure, 70 mph wind velocity are investigated in this evaluation. These are outlined in Table 4-2. Since ANSI/ASCE 7-93 rules out the possibility of windward roof suction for the 20 degree rafter inclination, the DL + LL + WSUCTION and DL + WSUCTION load combinations are disregarded. In the course of the structural checks the following frame components are given consideration:

- A. Primary Elements
 - 1. Interior Frame Elements: Rafter Sections
 - 2. Interior Frame Elements: Column Sections
 - 3. Gable Frame Elements: Interior Strut Sections

- B. Secondary Elements
 - 1. Purlins
 - 2. Beams
 - 3. Roof Cables
 - 4. Wall Braces
 - 5. Base Plates

The technical background information, design drawings and material properties were made available to F.T.L./Happold, Incorporated by Mr. Richard Martin of Roder U.S.A. to facilitate the evaluation of the structure according to the more general U.S. wind loading requirements. ANSI/ASCE 7-93 code regulations will be utilized for this purpose in lieu of its more general coverage regarding temporary structures, an advantage it has over the other U.S. codes (Part 4.3 and Appendix B).

1.2 Objectives

The ultimate goal of this analysis is to provide a sound basis for the structural acceptability of the 10 meter Roder Frame Tent in the light of the minimum U.S. Building Code requirements.

This evaluation is also aimed at developing a suitability chart for the structure based on the various terrain and wind conditions. In this regard, a stability report on the relevant conditions is presented.

Part 2

Structural Framing Plan

Presented in Figure 2-1 are the typical interior and gable end aluminum frames with their corresponding dimensions. It is understood that the length of the structure may be extended when necessary. As recommended, the minimum hall length would be 19.69 ft (6 m, 2 bays). At hall lengths of over 78.74 ft (24 m), additional wind bracing fields are to be arranged so that there would be 6 bracing-free fields (18 m) at the most between the wall bracings. The structure in contention is the extrusion box section tent spanning the full 32.81 ft (10 m) width and the primary structural assemblages are the aluminum frames.

The schematic elevations show the presence of steel inserts at the ridge and eaves and pinned connections at the base support joints. The roof and walls are clad in non-prestressed fabric skin connected to the aluminum frames by edge ropes slid through the aluminum extrusions (Appendix A). Since this fabric is not attached to the purlins, it transmits its forces directly to the supporting frames. Moreover, the structure rests on base plates anchored securely to the ground against uplift. The interior frame has a 32.81 ft span (10 m) and is considered more critical than the intermediately strutted gable end frame.

Finally, for longitudinal stability, high strength cross bracing cables and aluminum tubes are utilized for the roof and walls, respectively. In addition, the purlins, ridge and eaves beams transmit longitudinal forces to the intermediate frames of the structure.



made by/date

Figure 2-1 Structural Framing Plan

checked/date

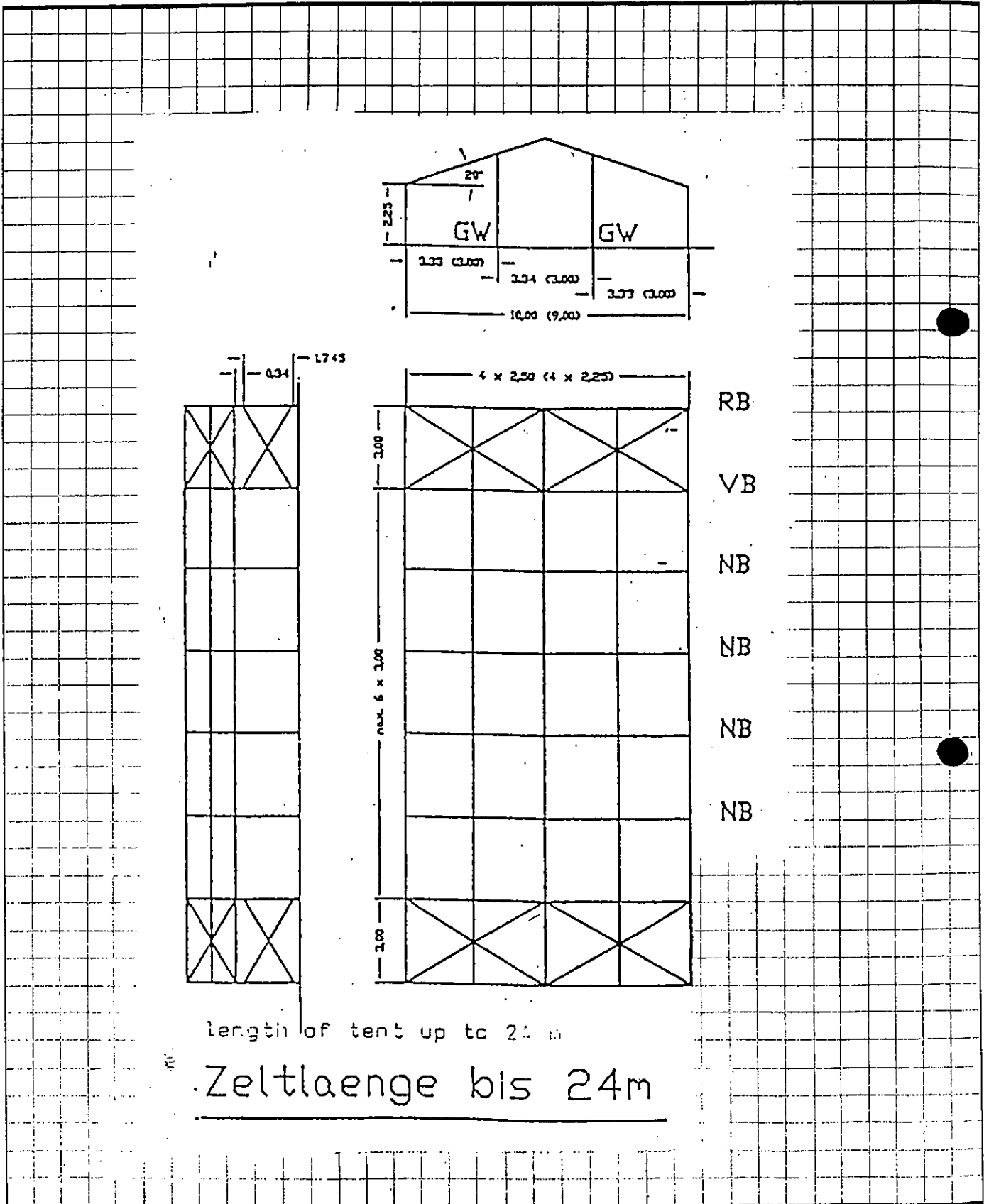
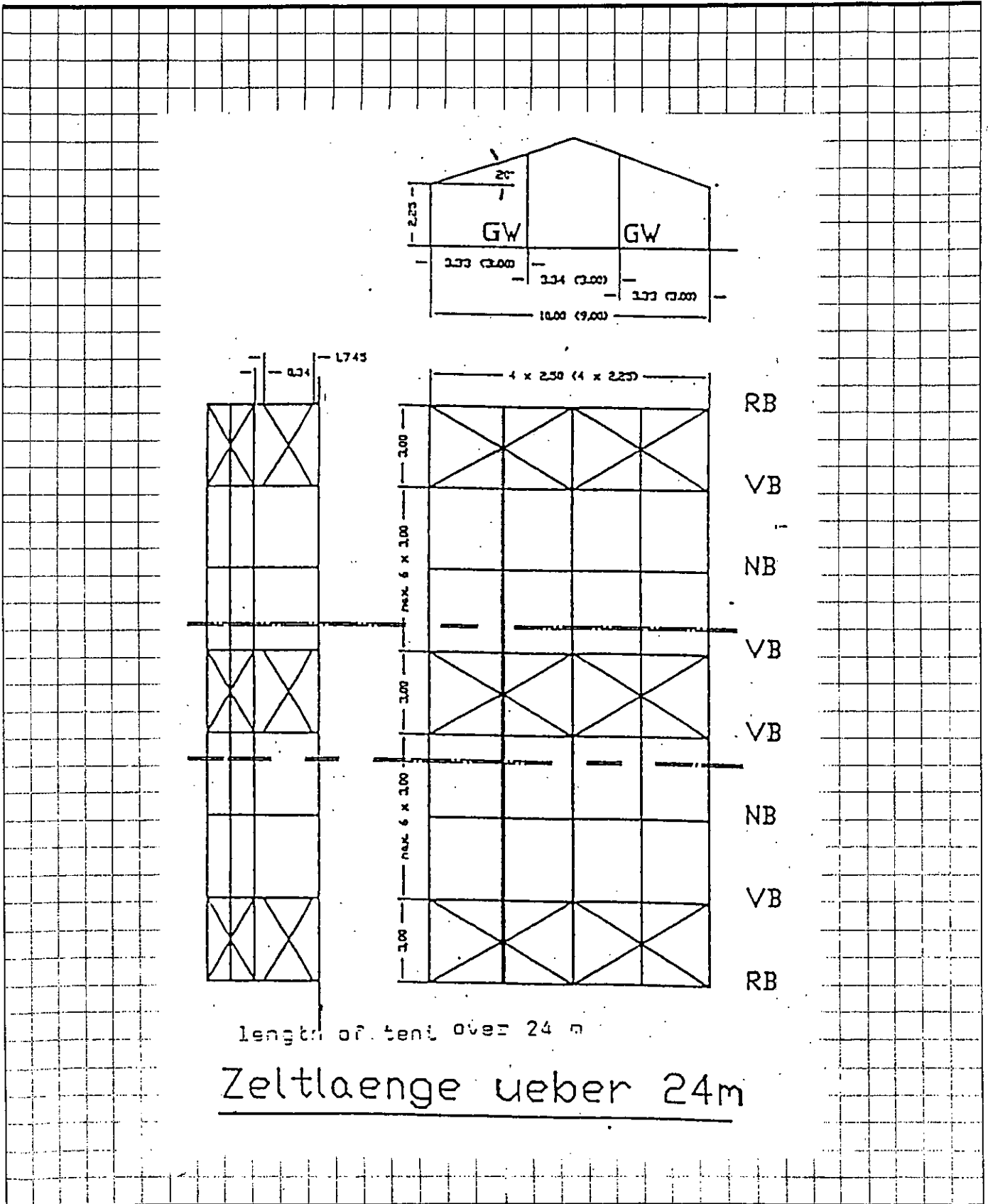




Figure 2-1 Structural Framing Plan

made by/date
checked/date



Part 3

Section and Material Properties

3.1 Material Distribution

Figure 3-1 shows the material assignments for 10 meter Roder Frame Tent. The diagram is accompanied by Table 3-1 outlining the section properties relevant to the analysis.

3.2 Section Properties

A tabulation of section properties for the Roder Frame Tent is presented as Table 3-1 in both English and Metric units. More detailed calculations based on component areas, as well as the cross section illustrations of the various profiles, are shown in Appendix A.

3.3 Material Properties and Design Criteria

The frames are assembled using structural aluminum box sections of Al Mg Si - 1F28 alloy conforming to DIN 4113. This is the European aluminum alloy 6082-T6 which has an American equivalent in the form of the 6061-T6. Section 2.2.1.1.2 (b) of the British Standard 8118 states that:

"An alternative alloy to 6082 is 6061 (Al Mg 1 Si Cu) of durability rating B which has very similar properties with slight improvement in formability and surface finish. It is available in extruded tubular form and is mainly used for structures."

as a reference.

Table 3-2 presents a rough property comparison of equivalent European and U.S. materials for the 10 m Roder Frame Tent.

F T L

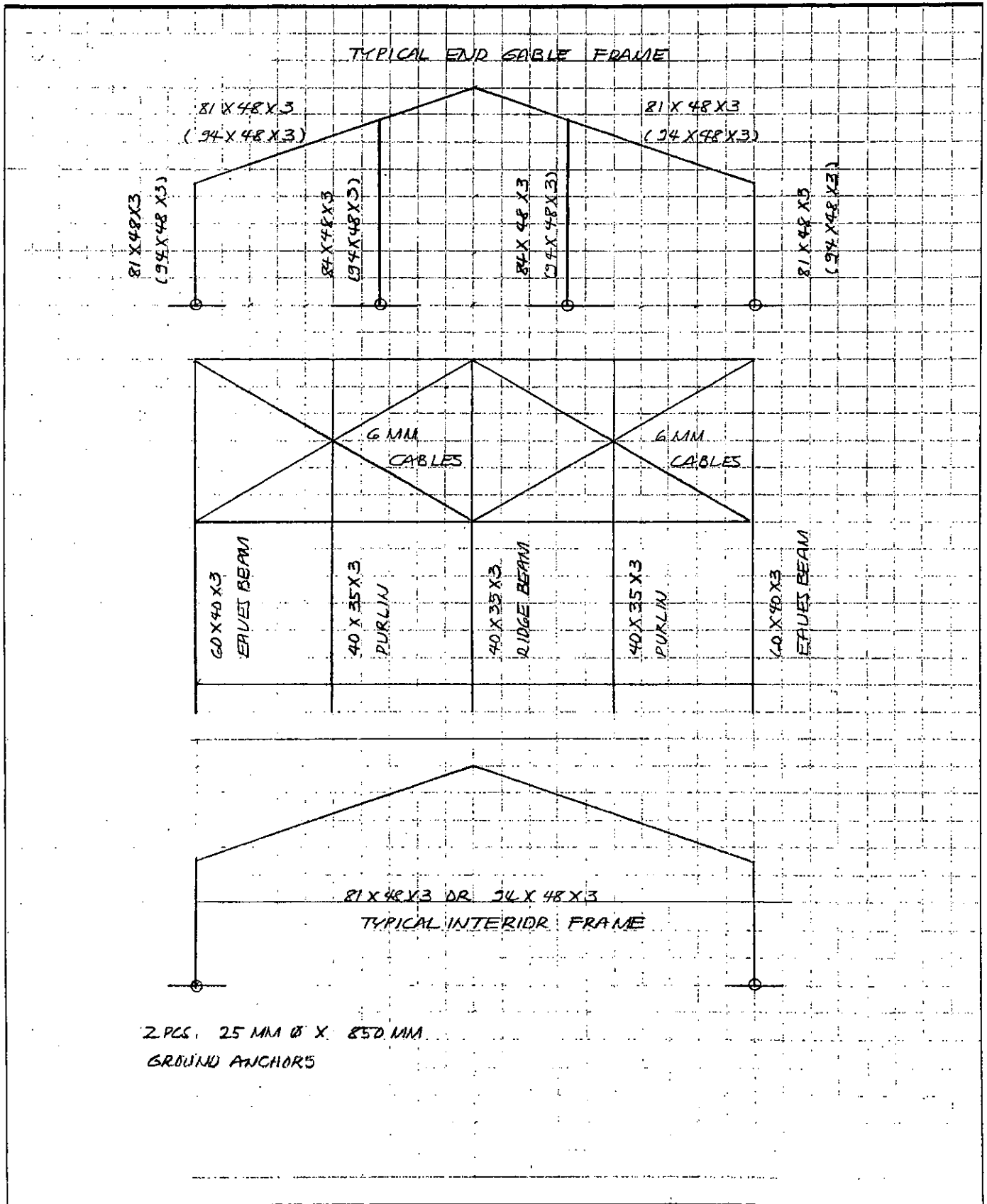
Happold

3.1 Material Distribution

made by/date

Figure 3-1 Material Distribution

checked/date



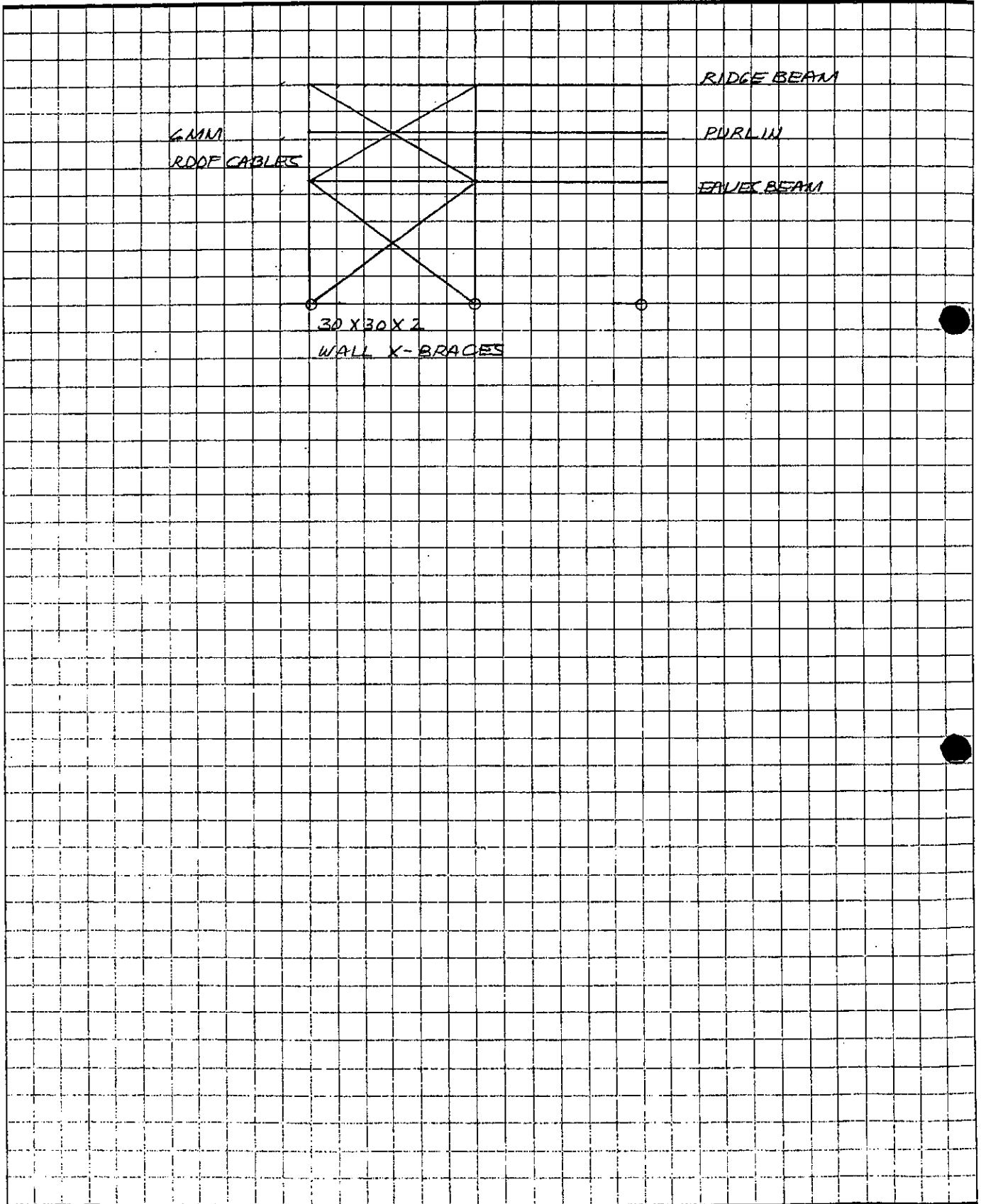
F T L

Happold

made by/date

Figure 3-1 Material Distribution

checked/date





3.2 Section Properties

Table 3-1 Section Properties (English Units)
Section Properties (Metric Units)

made by/date

checked/date

SECTION PROPERTIES

English System

Section	Aluminum Type	Axx in ²	Iyy in ⁴	Syy in ³	ryy in	Izz in ⁴	Szz in ³	rzz in	Weight plf
30/30/2	F22	0.35	0.07	0.12	0.45	0.07	0.12	0.45	0.408
40/35/3	F22	0.64	0.22	0.28	0.59	0.18	0.26	0.53	0.755
60/40/3	F22	0.87	0.34	0.44	0.63	0.66	0.56	0.87	1.028
81/48/3	F28	1.29	1.71	(T) 1.16 (B) 0.99	1.15	0.69	0.73	0.73	1.515
94/48/3	F28	1.79	3.16	(T) 1.56 (B) 1.89	1.33	0.81	0.85	0.67	2.107

The properties were lifted from the Test Report by TUV Bayern.
Verifications were done in Appendix A.

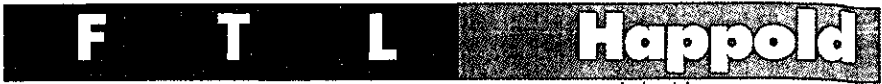
Metric System

Section	Aluminum Type	Axx cm ²	Iyy cm ⁴	Syy cm ³	ryy cm	Izz cm ⁴	Szz cm ³	rzz cm	Weight kN/m
30/30/2	F22	2.24	2.94	1.96	1.15	2.94	1.96	1.15	0.006
40/35/3	F22	4.14	9.17	4.58	1.49	7.38	4.22	1.34	0.011
60/40/3	F22	5.64	14.31	7.16	1.59	27.39	9.13	2.20	0.015
81/48/3	F28	8.31	70.97	(T) 19.00 (B) 16.30	2.92	28.61	11.90	1.86	0.022
94/48/3	F28	11.56	131.39	(T) 25.50 (B) 30.90	3.37	33.71	14.00	1.71	0.031

General Notes:

- Figures are based on the following unit weights:
 - Aluminum: 169.34 pcf (or 26611.83 N/m³)
 - Steel: 490.00 pcf (or 77003.63 N/m³)
- Refer to the tabulation of cross sectional properties for the corresponding areas.

3.3 Material Properties and Design Criteria



made by/date

Table 3-2 Design Criteria

checked/date

DESIGN CRITERIA

Criteria	Germany	U.S.	Germany	U.S.	Germany	U.S.
Material	Aluminum	Aluminum	Steel	Steel	Cable	Cable
Type	Al Mg Si -1F28 DIN 4113 Aluminum Alloy	6061-T6 Aluminum Box Sections	ST 37 Hollow Section	A-36 Structural Tubing	FE/1770 DIN 3066	Steel Structural Wire Rope (F.O.S. = 2.5)
Fy	29.0 Ksi @ 0.2%	35.0 Ksi	33.0 Ksi	36.0 Ksi		
Fu	39.9 Ksi	42.0 Ksi	53.1 Ksi	58.0 Ksi		
E	10100 Ksi	10100 Ksi	29000 Ksi	29000 Ksi		16000 Ksi
w	165 pcf	169 pcf	490 pcf	490 pcf		550 pcf
F all	19.1 Ksi (Flexure)	19.0 Ksi (Flexure)		Section 1.5 AISC Specs.	1.32 K (6 mm) Minimum Breaking Load	2.04 K (6 mm) Minimum Breaking Load

General Notes:

1. The German Aluminum (Al Mg Si - 1F28) Alloy conforming to DIN 4113 is also equivalent to the following:
 - a. the 6082-T6 European Aluminum Alloy, and
 - b. the 6061-T6 American Aluminum Alloy (Fy = 35.0 Ksi and Fu = 42.0 Ksi).
It has the following properties: Fy = 29.0 Ksi @ 0.2% and Fu = 39.9 Ksi.
2. The German Steel (ST-37) is likewise similar to the American Grade A36 Structural Steel.

Part 4

Load Assumptions

A typical interior frame was considered in determining the loads for analysis. The load calculations for the 10 meter Roder Frame Tent were done with the aid of a spreadsheet program and will mainly consist of a combination of the following: Dead Loads, Live Loads (Snow and Fixtures) and Wind Loads. The output is presented hereafter.

4.1 Dead Loads

The structure dead loads are of two types:

1. a distributed element load, and
2. point loads at the ridge and eaves.

The rafter, purlin, roof fabric and minor component weights account for the former while the ridge and eaves beams, and wall fabric weights make up the latter. The column weight was neglected since it would normally be transmitted directly to the foundations. Nonetheless, it will be incorporated in the analysis of the base plates.

4.2 Live Loads

Due to the temporary nature of the structure and its seasonal installation, snow is neglected in the load considerations. This will be forgone under the condition that measures be provided to ensure snow removal or melting in such an unlikely event. Furthermore, the prescribed gradient of the roof fabric should be maintained to allow for smooth drainage and the prevention of ponding.

Seismic, moving and additional live loads (during the construction stage) are also beyond the scope of the analysis. It is assumed that no onerous stresses will be imposed on the lightweight, fabric-clad frame structure during its installation and subsequent use. Only the electrical and mechanical fixtures (lighting, HVAC, suspended items, etc.) totalling 500 lbs per frame are accounted for. The total weight is assumed to be distributed accordingly:

1. left rafter centerspan (125 lbs),
2. ridge (250 lbs), and
3. right rafter centerspan (125 lbs).

4.3 Wind Loads

The ANSI/ASCE 7-93 wind loading provisions for various areas around the U.S. shall be used as the guidelines for wind pressure (and load) calculations. ANSI/ASCE 7-93 standards are recognized by all model building codes including the:

1. Building Officials and Code Administrators (BOCA),
2. Standard Building Code (SBC),
3. South Florida Building Code (SFBC), and
3. Uniform Building Code (UBC).

The following ANSI/ASCE 7-93 factors for the wind pressure calculations apply to the 10 meter version of the Roder Frame Tent:

- I = Importance Factor, which is related to the structure's intended occupancy and use,
 Gh = Gust Response Factor, which is dependent upon the structure height and exposure class, and
 Kz = Velocity Pressure Exposure Coefficient, which depends on similar factors as the above mentioned Gh.

The importance factor, I, is set at 0.77 instead of 1.00 since the structures are only temporary and the design winds come with a reduced return period, that is, from 50 years to less than 2 years. It should be noted that the importance factor values of 1.07 and 0.95 are specifically associated with annual probabilities of being exceeded of 0.01 and 0.04 (mean wind recurrence intervals of 100 and 25 years - ANSI/ASCE 7-93 Commentary, p. 60), respectively.

It is considered grossly impractical from an engineering point of view to design for a wind occurrence that has a probability of 0.02 (once in 50 years) especially if the structure has only a short service life. ANSI/ASCE 7-93 gives an allowance for this by reducing the I to 0.77. This is equivalent to a 23% reduction in the design wind speed. This practice is also documented in British codes in the form of S3, the statistical factor for temporary structures [British CP3: "Code of Basic Data for the Design of Buildings", Chapter V, Part 2, Wind Loading].

Refer to the attached calculations for the constant values (Table 4-1). These figures were used in conjunction with varying exposure classes and wind speeds to derive the maximum design wind pressures according to the ANSI/ASCE 7-93 formula. Finally, for a bay width of 9.84 ft (3 m), the uniform wind loads were calculated. These were then subjected to the geometry dependent Cp factors (Table B-1 and Figure B-2) to produce the distributed frame loads utilized in the computer analyses. Figure 4-2 shows the different coefficients for the main wind-force resisting system. An internal pressure coefficient (Cpi) of zero is assumed at this stage.

Furthermore, the structure has been analyzed with the assumption that no dominant windward or leeward openings are present during its exposure to wind. Locally high wind pressures brought about by uncommon topography are also neglected.

Appendix B contains more information on the subject of wind loading.

4.4 General Load Cases and Combinations

The five (5) load cases (the WSUCTION scenario will later be excluded), are graphically illustrated in Figure 4-2 and the eight (8) combinations are summarized in Table 4-2. The frames will be initially analyzed using the C-70 (Class C, 70 mph) wind pressure and will subsequently be tested on the other pressures, if warranted.

F T L

Happold

made by/date

checked/date

DATA FOR THE FRAME LOADS**Structure Geometry**

Bay Width 9.84 ft

Frame Geometry

Total Frame Width 32.81 ft

Total Frame Height 13.35 ft

Column Height 7.38 ft

Rafter (94/48/3 Aluminum Profile)

Horizontal Span 16.40 ft

Inclination Angle 20.00 deg

Cosine(Angle) 0.94

Length 17.46 ft

Linear Weight 2.11 plf

Ridge Beam (40/35/3 Aluminum Profile)

Span 9.84 ft

Linear Weight 0.76 plf

Eaves Beam (60/40/3 Aluminum Profile)

Span 9.84 ft

Linear Weight 1.03 plf

Purlin (40/35/3 Aluminum Profile)

Span 9.84 ft

Quantity per Rafter 1.00 pc

Linear Weight 0.76 plf

Fabric

Tributary Width 9.84 ft

Tributary Height 7.38 ft

Weight 0.208 psf

4.1 Dead Loads
4.2 Live Loads



made by/date

Figure 4-1 Dead and Live Loads (Spreadsheet Output)

checked/date

DEAD LOADS	
Uniform Load Distribution on the Inclined Main Members	
Weight of the Main Member	2.11 plf
Weight of the Purlins	0.76 plf * 9.84 ft = 7.43 lbs
	7.43 lbs / 0.94 = 7.94 lbs
Weight of the Fabric	2.05 plf * 9.84 ft = 20.28 lbs
Weight of Minor Components, Cables, Bolts, etc.	3.00 plf * 9.84 ft = 29.52 lbs
Total	7.58 plf
Point Load on the Ridge	
Weight of the Ridge Beam	7.43 lbs
Total	7.43 lbs
Point Load on the Eaves	
Weight of the Eaves Beam	10.12 lbs
Weight of the Fabric	1.03 plf * 9.84 ft = 10.13 lbs
Total	15.11 lbs
Total	25.23 lbs
LIVE LOAD	
Uniform Load Distribution on the Inclined Main Members	
Weight of Snow	0.00 plf
Point Loads on the Inclined Main Members	
Weight of the Fixtures	125.00 lbs
Left Rafter Midlength	250.00 lbs
Ridge	125.00 lbs
Right Rafter Midlength	125.00 lbs
Total	500.00 lbs

4.3 Wind Loads

F T L

Happold
made by/date

Table 4-1 Applied Wind Pressures

checked/date

I Factor	0.77	M/L		0.32	
Gh Factor	h = z	Exp A	Exp B	Exp C	Exp D
	0.0	2.36	1.65	1.32	1.15
	15.0	2.36	1.65	1.32	1.15
Kz Factor	h = z	Exp A	Exp B	Exp C	Exp D
	0.0	0.12	0.37	0.80	1.20
	15.0	0.12	0.37	0.80	1.20
Roof Cp's	M/L	15.0	20.0	degrees	
0.3e-h/L	-0.5	0.32	-0.9	0.12	Cps

10.37 ft
9.84 ft
60.00 mph = V1
70.00 mph = V2
80.00 mph = V3
90.00 mph = V4

Mean Structure Height, h
Tributary Width, b
Wind Pressure Equation from ANSI/ASCE 7-93
q = 0.00256 (Gh Kz)^2 V^2
where V, the wind velocities are:

EQUATION CONSTANTS

Exposure	q	Gh	Kz	I
Class A	0.00042985 V^2	2.98	0.12	0.77
Class B	0.00092663 V^2	1.65	0.37	0.77
Class C	0.00160282 V^2	1.32	0.80	0.77
Class D	0.00209460 V^2	1.15	1.20	0.77

WIND PRESSURES, q (psf)

Exposure	60 mph	70 mph	80 mph	90 mph
Class A	1.55	2.11	2.75	3.48
Class B	3.34	4.54	5.93	7.51
Class C	5.77	7.85	10.26	12.88
Class D	7.54	10.26	13.41	16.97

DISTRIBUTED WIND LOADS, P (psf)

Exposure	60 mph	70 mph	80 mph	90 mph
Class A	15.23	20.73	27.08	34.27
Class B	32.83	44.69	58.37	73.68
Class C	56.78	77.30	100.97	127.78
Class D	74.22	101.02	131.94	166.99

WINDWARD ROOF SUCTION / PRESSURE COEFFICIENT, Cr

Cp 0.12

LOAD CASES (C<70 EXPOSURE)

Case	q (psf)	P (psf)	w = CpwxP	Cpw	w = CpwxP	Cpr	w = CpwxP	Cpr	w = CpwxP	Cpr
WSUCTION (Same as WPRESSURE)	7.85	77.30	61.84	0.80	9.59	0.12	-54.11	-0.70	-38.65	-0.50
WPRESSURE	7.85	77.30	61.84	0.80	9.59	0.12	-54.11	-0.70	-38.65	-0.50
WPARALLEL	7.85	77.30	-54.11	-0.70	-54.11	-0.70	-54.11	-0.70	-54.11	-0.70

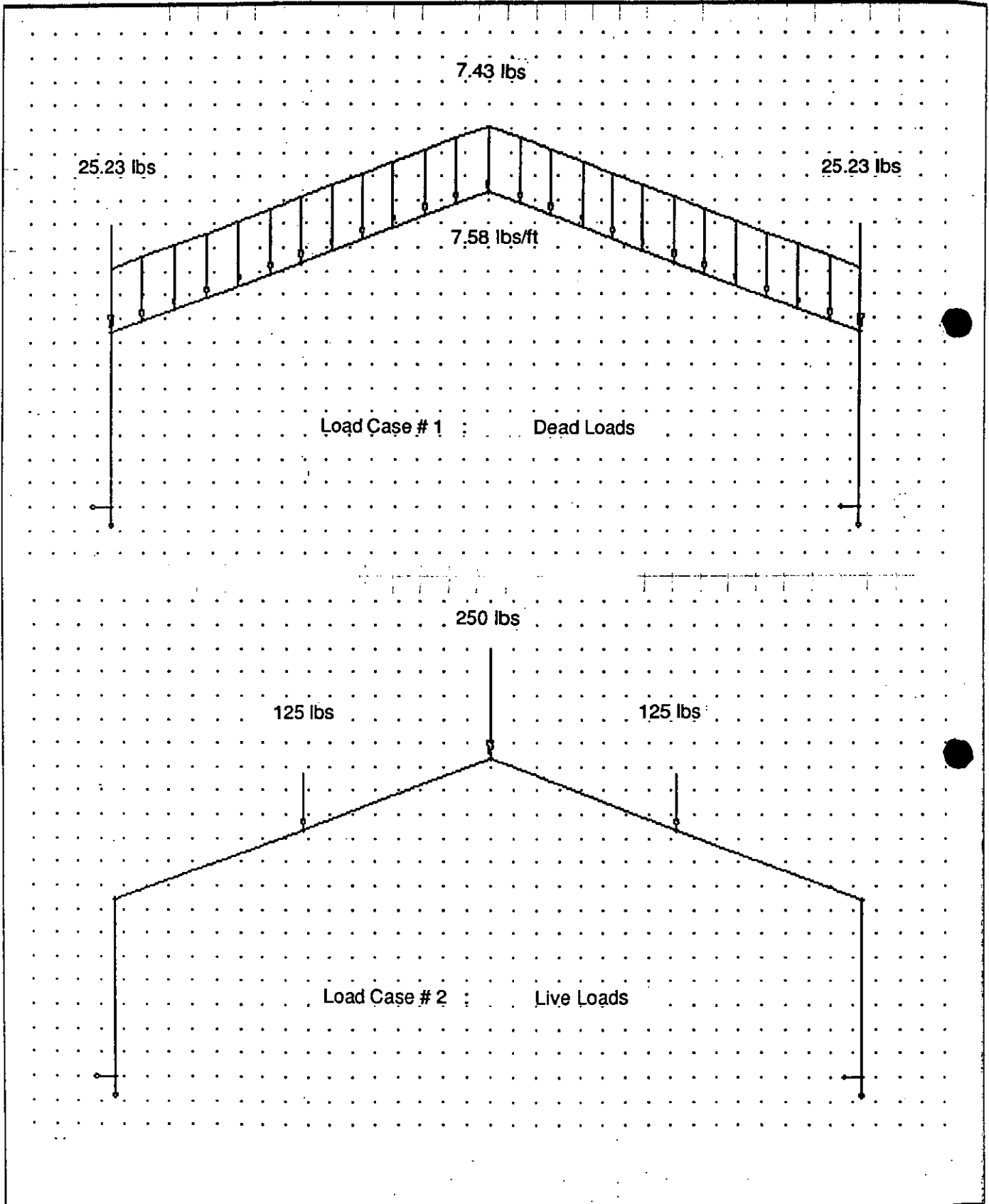
4.4 General Load Cases and
Combinations



made by/date

Figure 4-2 Load Cases

checked/date



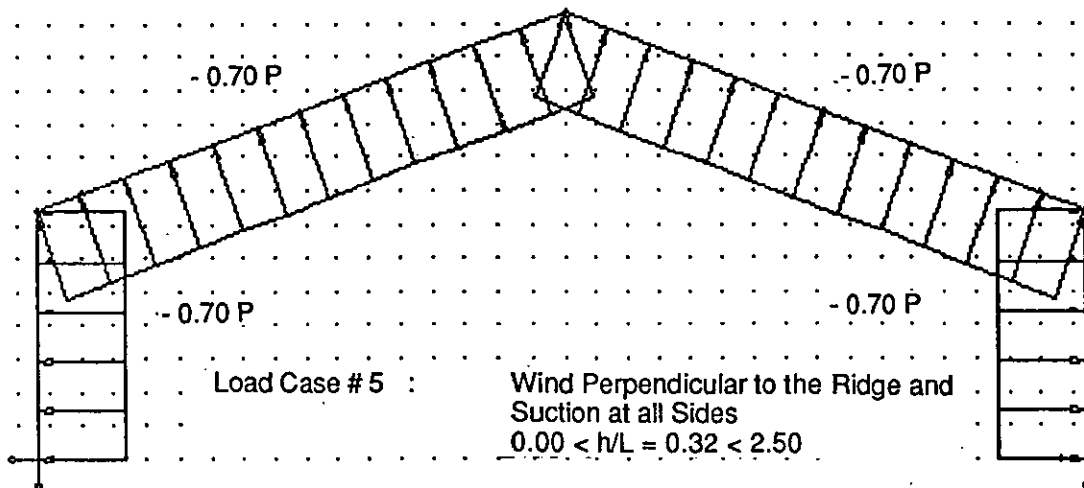
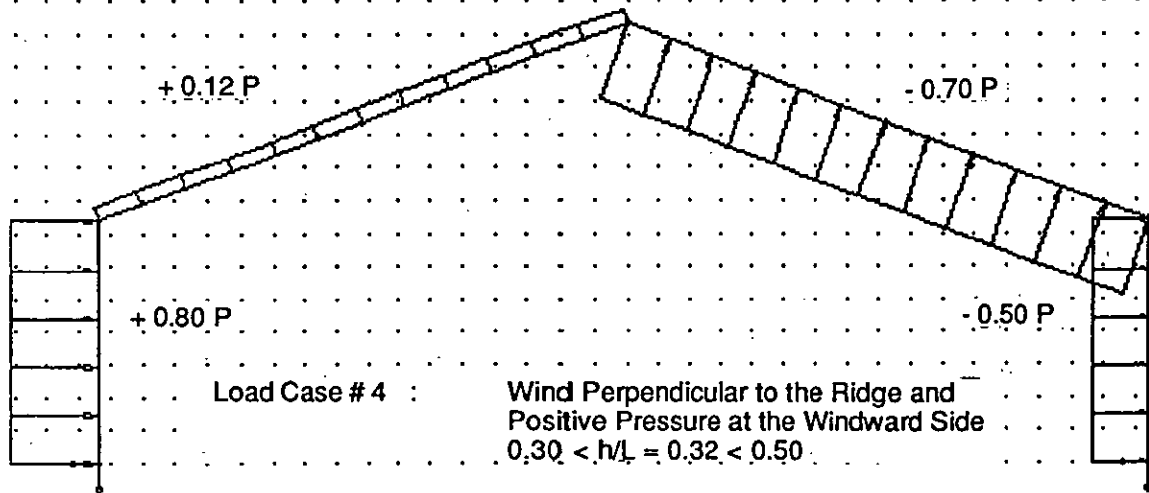
F T L

Happold

made by/date

Figure 4-2 Load Cases

checked/date





made by/date

Table 4-2 Load Case and Combination Summary

checked/date

LOAD CASE SUMMARY

Case No.	Description
1	Dead Loads
2	Live Loads
3	Wind Load Perpendicular to the Ridge Suction on the Windward Side
4	Wind Load Perpendicular to the Ridge Positive Pressure on the Windward Side
5	Wind Load Parallel to the Ridge Suction on All Sides

LOAD COMBINATION SUMMARY

Case No.	Description
6	DL
7	DL + LL
8	DL + LL + WSUCTION
9	DL + LL + WPRESSURE
10	DL + LL + WPARALLEL
11	DL + WSUCTION
12	DL + WPRESSURE
13	DL + WPARALLEL

Part 5

Structural Analysis and Results

5.1 Computer Model

The Roder Frame is modeled as an assemblage of beam and column segments systematically connected with moment resisting inserts. Releases are set where the components are pinned or bolted to accurately simulate a zero moment condition. These occur at the column bases.

Shown in Figure 5-1 is the typical interior frame with its node and element labels. To facilitate the model set-up, coordinates are assigned to each node (corresponding to its relative point in space) and element connectivity data are introduced. Support or boundary conditions (pinned supports) are then set to finish the system geometry.

This model is then input into ROBOT V6, a graphical user interfaced finite element analysis program, together with the material and cross section properties, load cases and combinations (Figure 5-2). At this stage, the computer model is ready for analysis.

5.2 ROBOT V6 Results

The graphical results involving the frame displacements, moments and internal forces for the resulting six load combinations are shown as Figure 5-3. Also indicated are the extreme values and their relative locations. These should be viewed in conjunction with the listings provided in Appendix C.



5.1 Computer Model

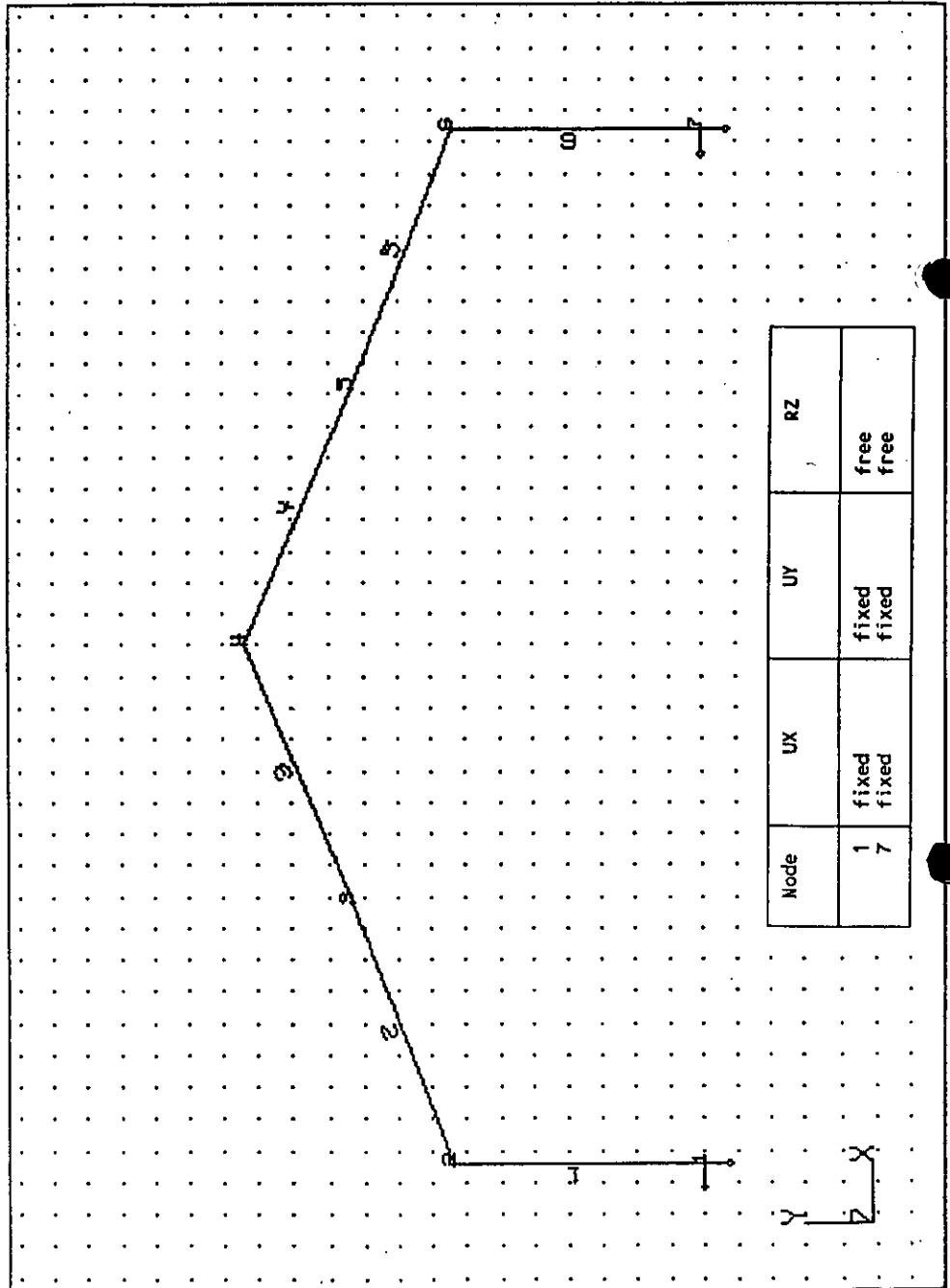
made by/date

Figure 5-1 Node and Element Data

checked/date

Element	Node1	Node2	Length (ft)	Materials
1	1	2	7.380000	ALUM
2	2	3	8.728121	ALUM
3	3	4	8.724701	ALUM
4	4	5	8.724701	ALUM
5	5	6	8.728121	ALUM
6	6	7	7.380000	ALUM

Node	X (ft)	Y (ft)
1	-16.400000	0.0
2	-16.400000	7.380000
3	-8.200000	10.370000
4	0.0	13.350000
5	8.200000	10.370000
6	16.400000	7.380000
7	16.400000	0.0



Node	UX	UY	RZ
1	fixed	fixed	free
7	fixed	fixed	free



Table 5-1 Coordinate Data

made by/date

checked/date

SPREADSHEET GENERATION OF THE FRAME LOADS

96.6.5.14:58

Roder 10m Run Number 1

COORDINATE DATA (METRIC UNITS)

Description	Global X (m)	Global Y (m)
Left Pinned Support	-5.000	0.000
Left Eaves	-5.000	2.250
Left Live Load Application Point	-2.500	3.160
Ridge Point	0.000	4.070
Right Live Load Application Point	2.500	3.160
Right Eaves	5.000	2.250
Right Pinned Support	5.000	0.000

Slope 0.364
Span 10.000 m
Bay Width 3.000 m

COORDINATE DATA (ENGLISH UNITS)

Description	Global X (ft)	Global Y (ft)
Left Pinned Support	-16.40	0.00
Left Eaves	-16.40	7.38
Left Live Load Application Point	-8.20	10.37
Ridge Point	0.00	13.35
Right Live Load Application Point	8.20	10.37
Right Eaves	16.40	7.38
Right Pinned Support	16.40	0.00

Slope 0.36
Span 32.81 ft
Bay Width 9.84 ft



made by/date

Figure 5-2 ROBOT V6 Input File

checked/date

06/05/1996 17:05		06/05/1996 17:05	
Filename: R0010		Filename: R0010	
Page 1		Page 2	
<p>'This file was generated by r3D on 5.06.1996 at 12:56:50</p>			
<p>ROBOT</p>			
<p>FRAME PLANE</p>			
<p>NODES 7 ELEMENTS 6</p>			
<p>UNITS</p>			
<p>LENGTH=ft force=lbs</p>			
<p>NODES</p>			
N°	X	Y	
1	-16.40	0	
2	-16.40	7.38	
3	-8.20	10.37	
4	0	13.35	
5	8.20	10.37	
6	16.40	7.38	
7	16.40	0	
<p>ELEMENTS</p>			
N°	ORIG	END	
1	1	2	
2	2	3	
3	3	4	
4	4	5	
5	5	6	
6	6	7	
<p>PROPERTIES</p>			
<p>ALUM</p>			
<p>RO=169 34</p>			
<p>1106 USER94 x 48 x 3</p>			
<p>SUPPORTS</p>			
<p>Label - free</p>			
<p>No label - blocked</p>			
<p>1 7 RZ</p>			
<p>LOADS</p>			
<p>CASE # 1 DEAD LOAD</p>			
<p>2 6 FY=-25.23</p>			
<p>4 FY=-7.43</p>			
<p>ELEMENTS</p>			
<p>2105 PY=-7.58</p>			
<p>CASE # 2 LIVE LOAD</p>			
<p>3 5 FY=-125</p>			
<p>4 FY=-250</p>			
<p>CASE # 4 WIND PRESSURE</p>			
<p>ELEMENTS</p>			
<p>1 PX=61.84</p>			
<p>2 3 PY=-9.59 Local</p>			
<p>4 5 PY=54.11 Local</p>			
<p>6 PX=38.65</p>			
<p>CASE # 5 WIND PARALLEL</p>			
<p>COMBINATION # 6 DL</p>			
<p>1 1</p>			
<p>COMBINATION # 7 DL+LL</p>			
<p>1 1 2 1</p>			
<p>COMBINATION # 9 DL+LL+PRESSURE</p>			
<p>1 1 2 1 4 1</p>			
<p>COMBINATION # 10 DL+LL+PARALLEL</p>			
<p>1 1 2 1 5 1</p>			
<p>COMBINATION # 12 DL+PRESSURE</p>			
<p>1 1 4 1</p>			
<p>COMBINATION # 13 DL+PARALLEL</p>			
<p>1 1 5 1</p>			
<p>END</p>			

F T L

Happold

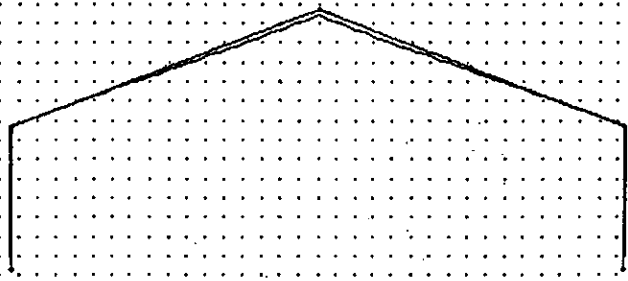
5.2 ROBOT V6 Results

Figure 5-3 Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 6 : DL

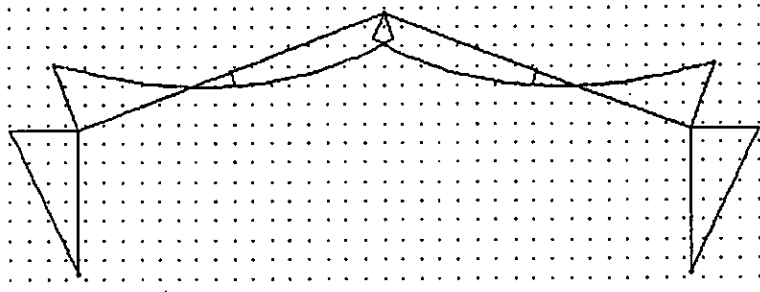
made by/date

checked/date

Scale		Extremes		
Scale factor		[kip, kipft, in]		
	3	Dis	Elem:	Pos:
		1.02	3	1
		1.08	3	1
		0		
		1.08	3	1



Extremes		
[kip, kipft, in]		
MZ	Elem:	Pos:
0.524	6	0



Extremes		
[kip, kipft, in]		
FXc	Elem:	Pos:
0.161	6	0

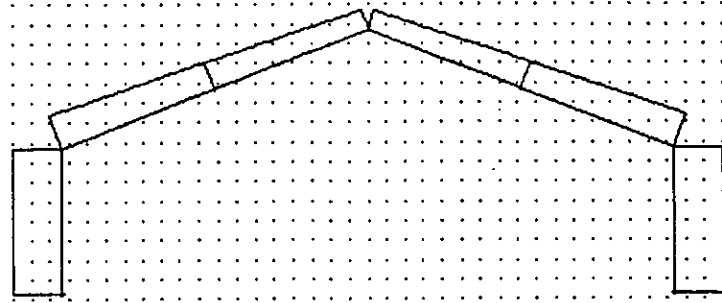




Figure 5-3

Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 7 : DL + LL

made by/date

checked/date

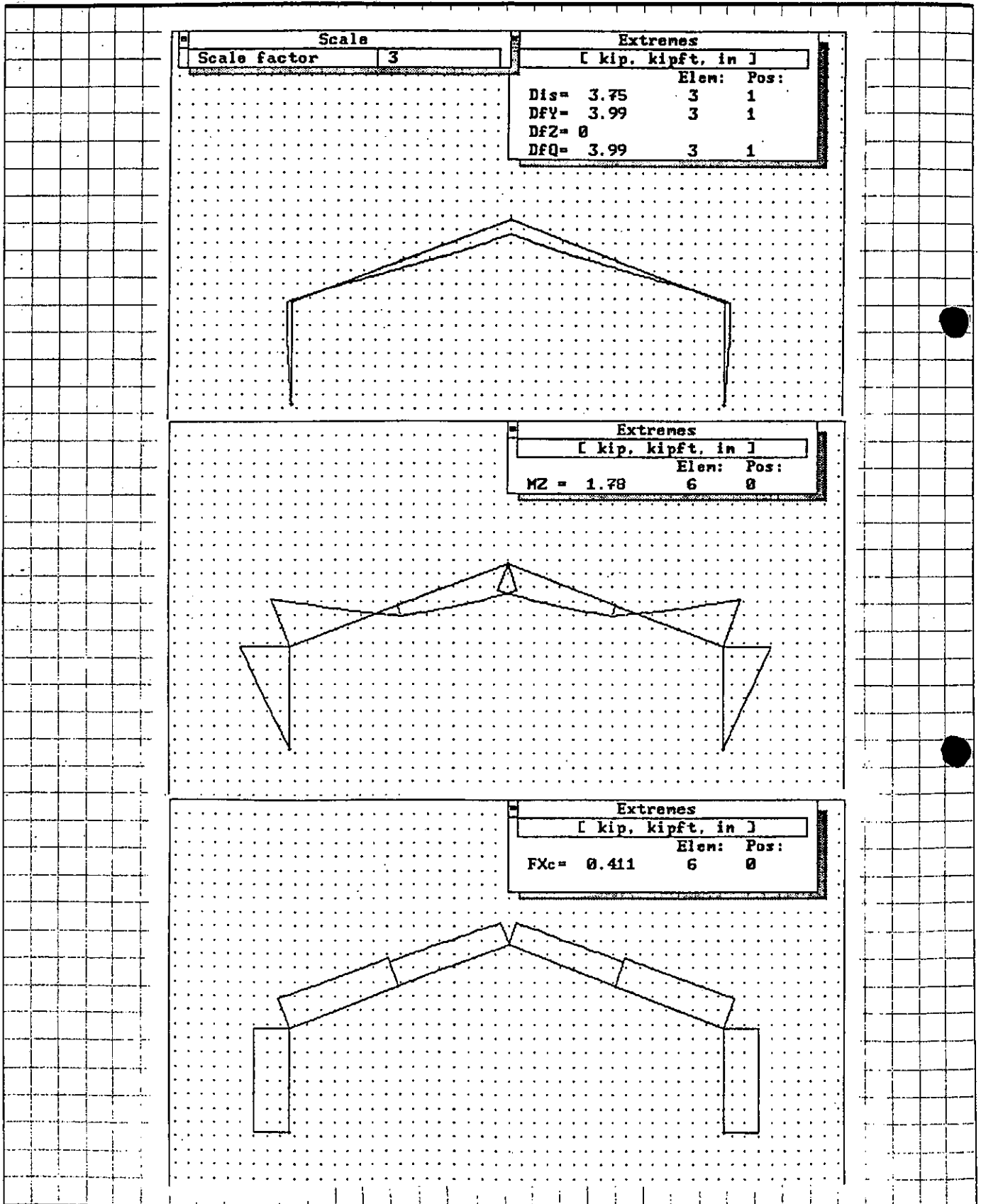




Figure 5-3

Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 9 : DL + LL + WPRESSURE

made by/date

checked/date

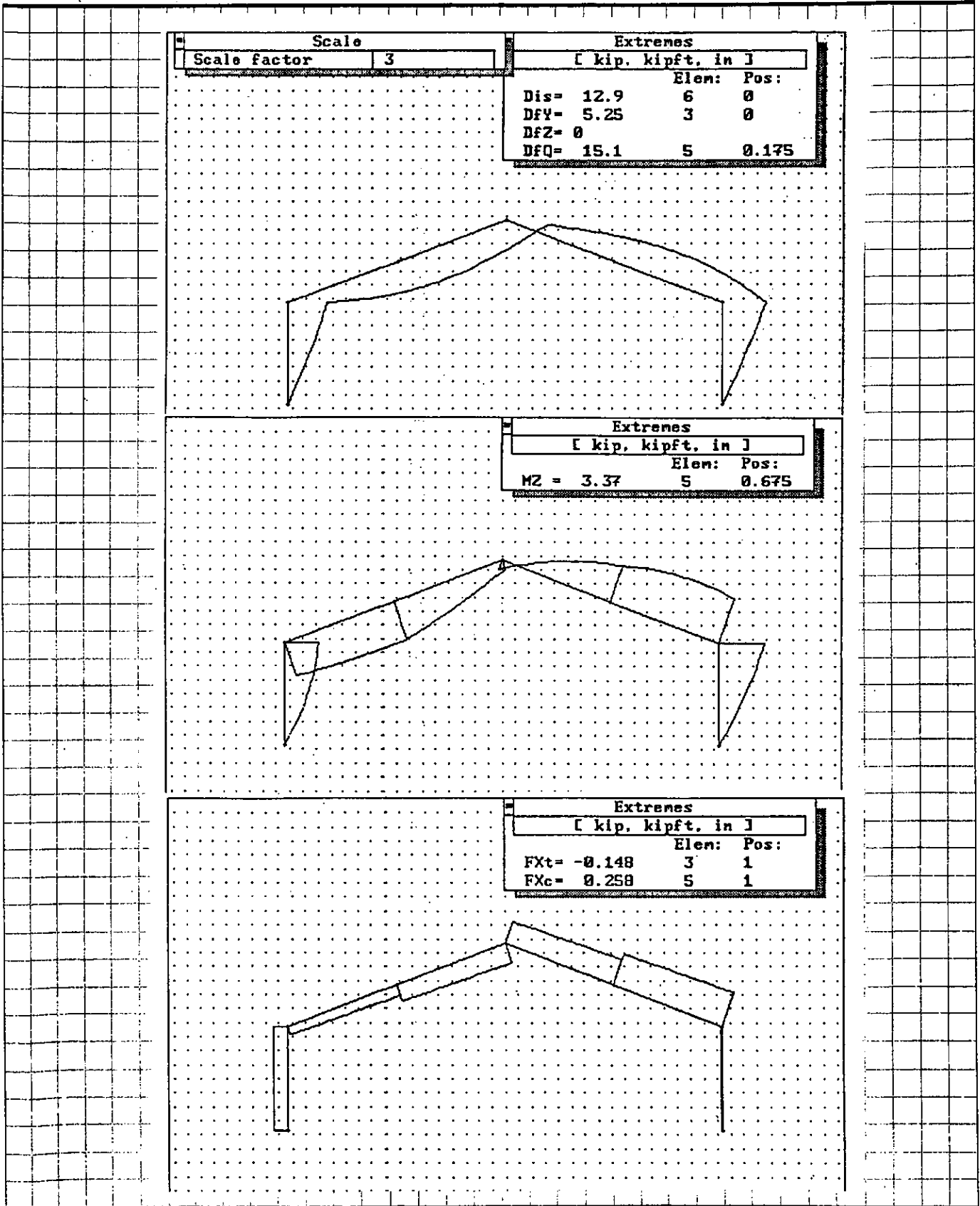




Figure 5-3

Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 10 : DL + LL + WPARALLEL

made by/date

checked/date

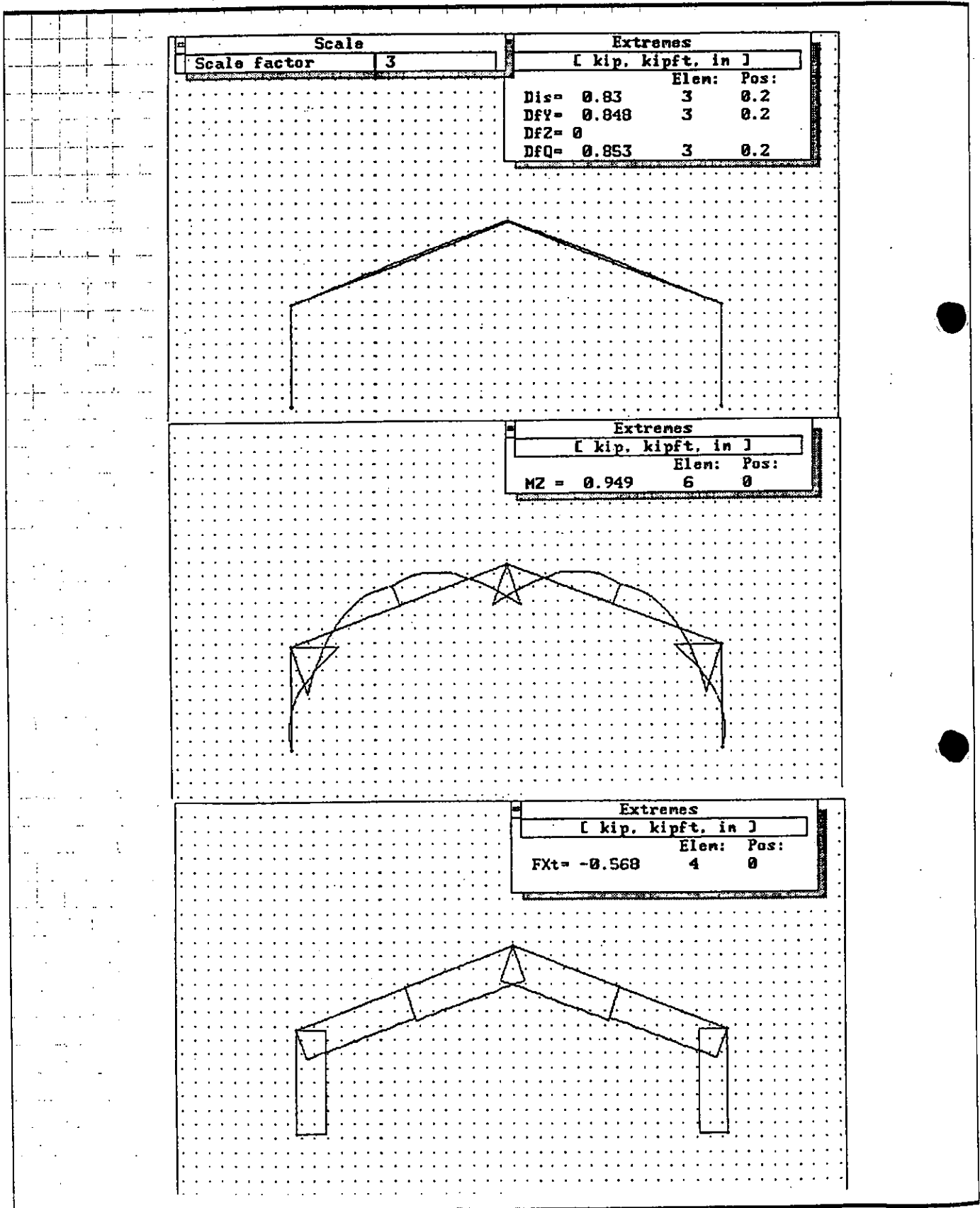




Figure 5-3 Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 12 : DL + W/PRESSURE

made by/date

checked/date

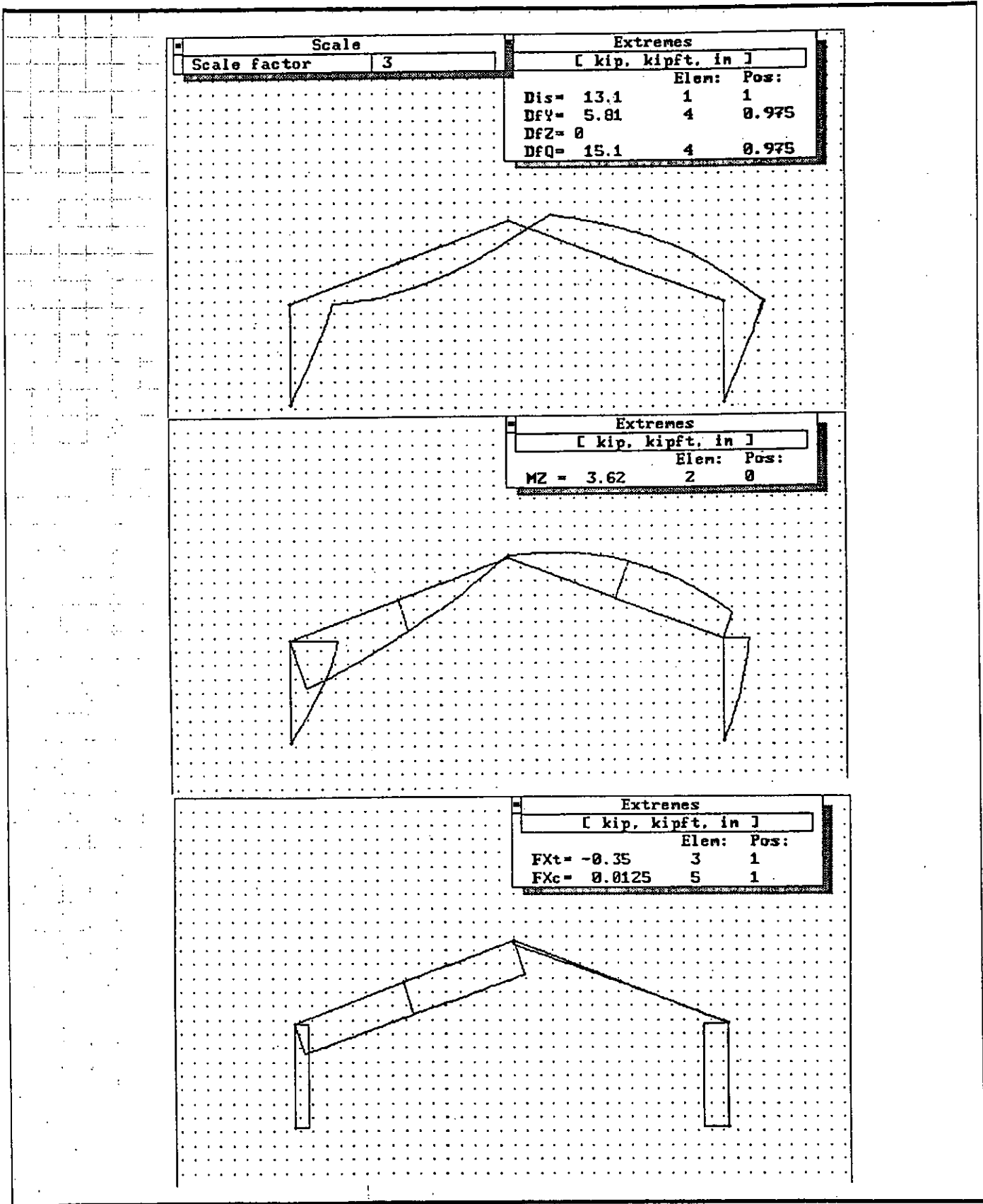
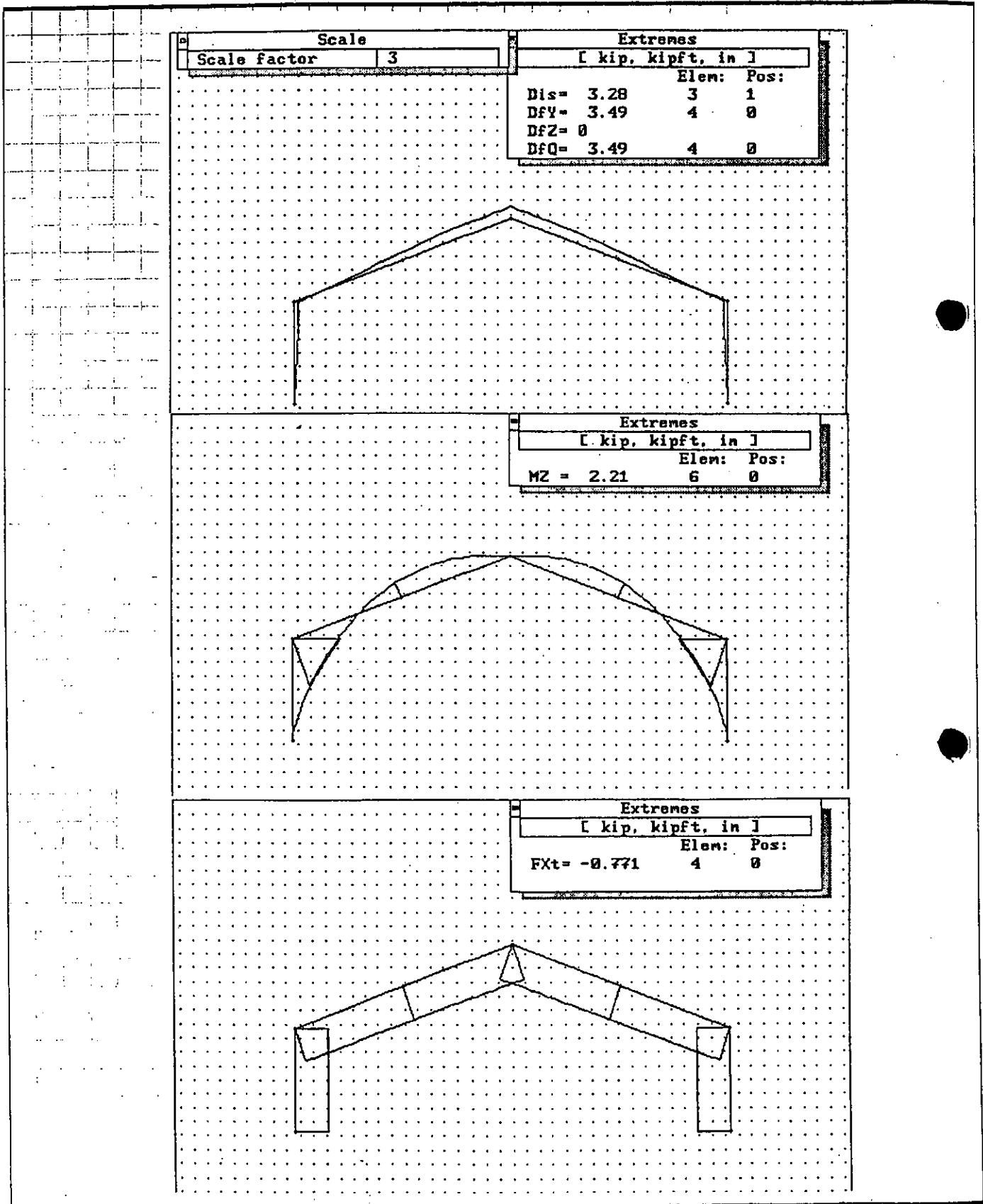




Figure 5-3 Diagrams of Displacements, Moments and Internal Forces
Load Combination No. 13 : DL + WPARALLEL

made by/date

checked/date



Part 6

Critical Loads



Table 6-1 Critical Applied Loads

made by/date

checked/date

CRITICAL APPLIED LOADS

FRAME ELEMENT	LOAD COMBINATION	MEMBER	MAX P (Kips)	MAX V (Kips)	MAX M (Kip-ft)
Flexural Members (Rafters)	DL + WPRESSURE	2	0.31	-0.07	-3.62
Frame Column Segments	DL + WPRESSURE	1	0.15	0.26	-3.62
Supports or Foundations	DL + LL	1,6	0.41	(+/-) 0.24	0.00
	DL + WPRESSURE	1	-0.15	-0.72	0.00
	DL + WPARALLEL	1,6	-0.73	(-/+) 0.10	0.00

Note:

The other elements will be tested using $q = 7.85$ psf corresponding to a Class C, 70 mph wind exposure.

Part 7

Structural Investigations

This section covers the evaluation of key structural elements, connections and details that may prove critical to the frame stability under the applied wind loads. In the course of the analyses, constant reference is made to the following:

1. Section properties (Part 3 or Appendix A),
2. Failure criteria established for the frame version, and
3. The Aluminum and Steel Construction Manuals (U.S.).

Frame Loading Provisions

On the subject of external loading and allowable stresses, the following important provisions are lifted from the Aluminium Construction Manual:

Section 2.3.1: Dead Load

The dead load to be used in the design of the structure includes its self-weight and the weight of all materials permanently attached to and supported by it.

Section 2.3.2: Live Load

Static and dynamic live loads, as well as snow, ice, ponding, and wind loads shall be based on the appropriate building codes. Where building codes do not apply, requirements shall be established from performance specifications of the structure.

In computing allowable stresses, the values provided in these specifications may be increased by one-third (1/3) when stresses are produced by wind or seismic loading, acting alone or in combination with the design dead and live loads.

However, these sections shall not be less than that required for the dead and other live loads acting alone. In the case of wind or ice loads, the form of the structure and any of its exposed components (e.g. increased area exposed to wind due to icing) shall be considered.

Likewise, from the manual of the American Institute of Steel Construction:

Section 1.5.6: Wind and Seismic Stresses

Allowable stresses may be increased 1/3 above the values otherwise provided when produced by wind or seismic loading, acting alone or in combination with the design dead and live loads, provided that the required section computed on this basis is not less than that required for the design dead and live load and impact (if any), computed without the 1/3 stress increase, and further provided that the stresses

are not otherwise required to be calculated on the basis of reduction factors applied to design loads in combinations.

10 Meter Roder Frame Tent Version

The 10 meter version of the Roder Frame Tent, with an external column height of 7.38 ft, was analyzed with design loads corresponding to a C-70 (Class C exposure, 70 mph) wind. The following were calculated in Part 4.3:

$$q = 7.85 \text{ psf}$$
$$P = 77.30 \text{ plf}$$

according to the ANSI/ASCE 7-93 guidelines for wind loading.

The analyses using the other wind pressures will be deferred until Part 8: Acceptability Under Variable Exposure and Wind Conditions. This section will only investigate the effects of the C-70 forces to facilitate the determination of the most critical frame element.

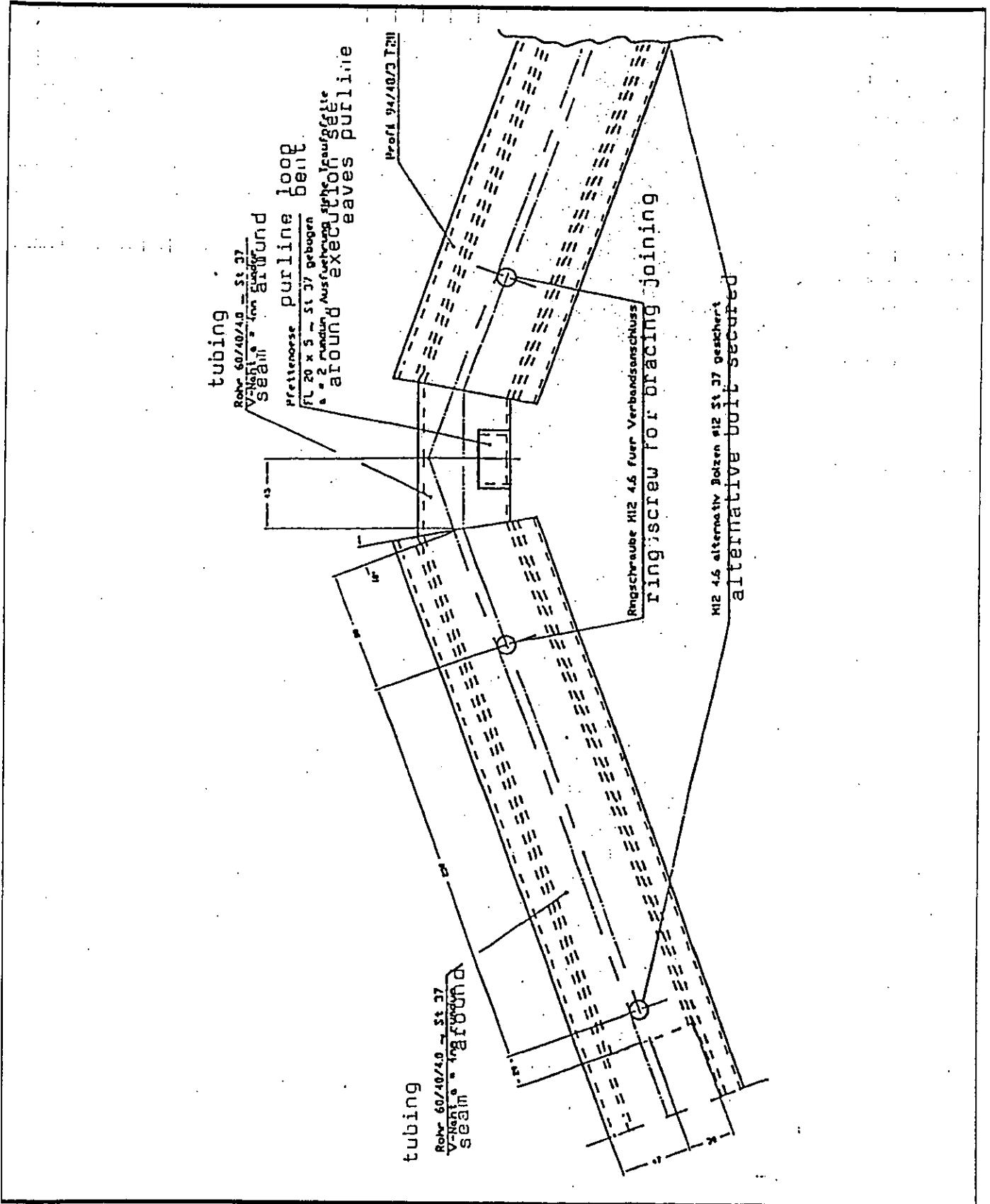
F T L

Happold

made by/date

Figure 7-1 Rafters

checked/date



7.1 Primary Elements



made by/date

7.1.1 Interior Frame Elements: Rafter Sections

checked/date

MAIN EXTERIOR FRAMEWORK MEMBERS 9/14/13 RAFTER SECTIONS

A. MAXIMUM APPLIED LOADS

COMBINATION	DESCRIPTION	W/P	NO. P	P (K)	W (K')	LENGTH (FT)
9/12	DL + W/PRESSURE	2	2	10.31	3.62	17.46

B. FAILURE CRITERION

LIMIT STATE FOR COMBINED AXIAL & BENDING STRESSES

C. ALLOWABLE STRESSES

COMPRESSION

BUCKLING AXIS LATERALLY UNRESTRICTED L (FT) L/r S_x

MINOR 17.46 8.73' 8.73/12 = 0.73 156.36 > CC

2 0.67

MAJOR 17.46' 17.46 x 12 = 209.52 157.53 > CC GOVERNS

1.33

THE HIGHER L/r GOVERNS, AND, SINCE IT IS GREATER THAN CC,

$$F_a = \frac{51 \times 10^3}{(L/r)^2} = \frac{51 \times 10^3}{(157.53)^2} = 2.06 \text{ KSI} \quad (F_t = 19 \text{ KSI FOR TENSION)}$$

BENDING

ON THE EXTREME TENSION FIBER OF BEAMS

$$F_t = 19 \text{ KSI}$$

ON THE EXTREME COMPRESSION FIBER OF BEAMS

$$L/r = \frac{(8.73 \times 12)(1.56)}{I_y} = \frac{163.78}{0.81} = 201.76$$

THE RESULT IS MORE THAN 146 BUT LESS THAN 1700

$$F_c = 23.0 - 0.24 (L/r)^2 = 23.0 - 0.24 (201.76)^2 = 20.49 \text{ KSI}$$

THE LOWER F_b GOVERNS, IE

$$F_b = 19 \text{ KSI}$$

D. WORKING STRESSES

TENSION

$$f_t = \frac{P}{A} = \frac{0.31}{1.73} = 0.17 \text{ KSI}$$

BENDING

SLAB LENGTH = 30 + 250 + 20 = 300' = 1.18'

30% B

M (ASSUMING A LINEAR DISTRIBUTION) = $\frac{(17.46 - 1.18) \times 3.62}{17.46} = 3.38 \text{ K'}$

$$f_b = \frac{M}{S} = \frac{3.38 \times 12}{1.83} = 21.93 \text{ KSI} \quad (\text{BOTTOM FLANGE})$$

F T L

Happold

made by/date

checked/date

$$f_b = \frac{M}{S} = \frac{3.38 \times 12}{1.52} = 25.96 \text{ ksi (TOP FLANGE)}$$

E. UNITY CHECKS & CONCLUSION

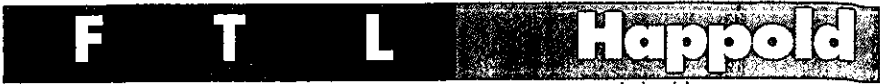
BOTTOM FLANGE (TENSION)

$$\frac{0.17}{1.33 (19)} + \frac{26.43}{1.33 (19)} = \underline{0.85 < 1.00 \text{ GOOD!}}$$

TOP FLANGE (COMPRESSION)

$$\frac{-0.17}{1.33 (19)} + \frac{25.96}{1.33 (20.49)} = \underline{0.94 < 1.00 \text{ GOOD!}}$$

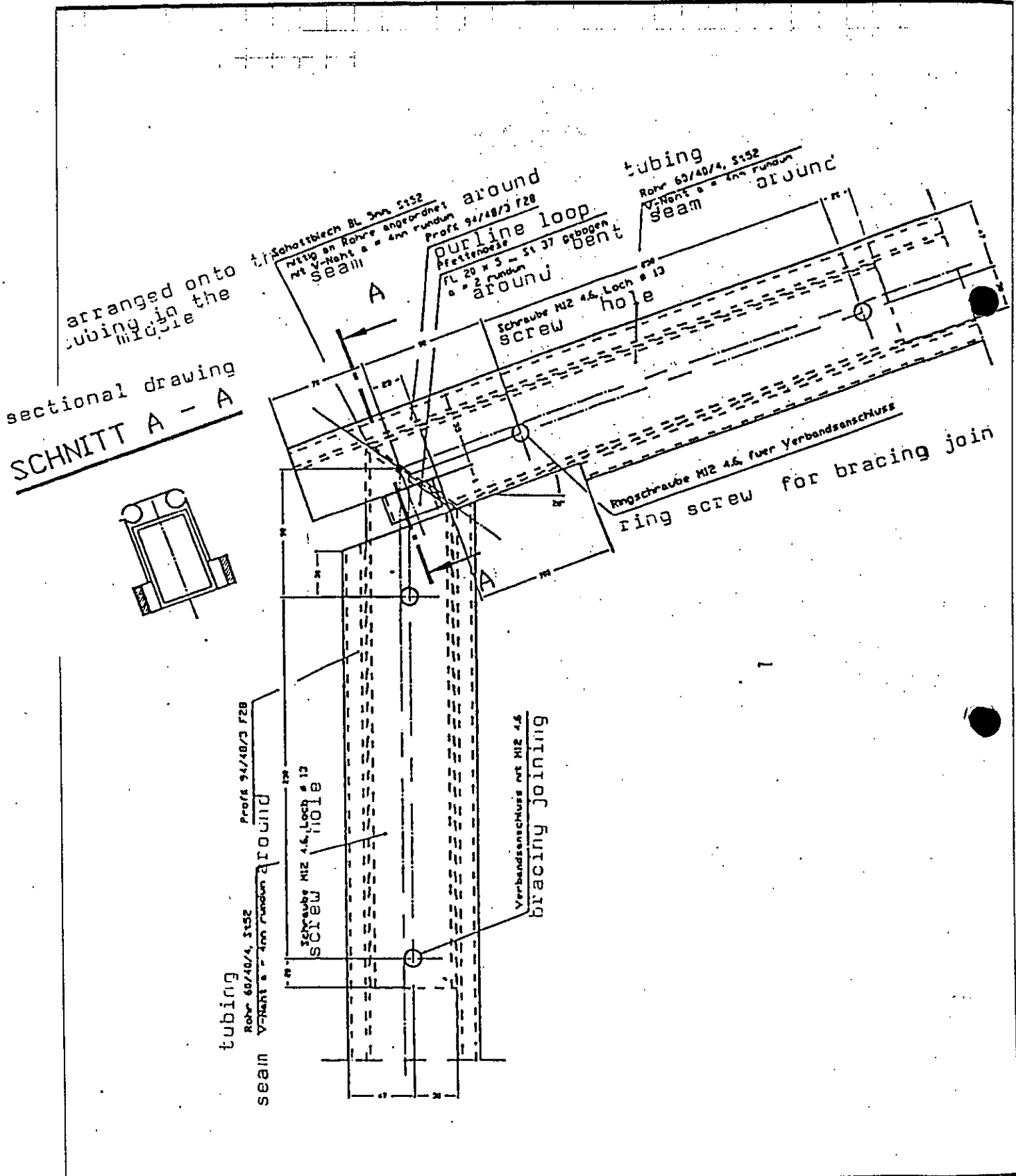
SINCE THE RESULTS ARE LESS THAN UNITY, THE 9R/4R/3 RAFTER SECTION IS ACCEPTABLE.



made by/date

Figure 7-2 Exterior Columns

checked/date



F T L

Happold

made by/date

7.1.2 Interior Frame Elements: Column Sections

checked/date

WOOD FRAMEWORK TRUSS MEMBERS, 28/88/3 COLUMN SECTIONS

A. MAXIMUM APPLIED LOADS

COMBINATION	DESCRIPTION	EL#	NO#	P(K)	M(K')	LENGTH (FT)
#12	D1 + WINDSUCK	1	2	+0.15	-2.62	7.38

B. FAILURE CRITERIA

UNITED CHECK FOR COMBINED TENSION & BENDING STRAINS

C. ALLOWABLE STRESSES

COMPRESSION

BUCKLING FORM	LATERALLY UNSUPPORTED L (FT)	L/C	S2
MINOR	7.38'	$7.38 \times 12 = 132.18$	> 60 GOVERNS
MAJOR	7.38'	$7.38 \times 12 = 66.50$	> 60

THE HIGHER L/C GOVERNS, AND, SINCE IT IS GREATER THAN 60,

$$F_c = \frac{51 \times 10^3}{(L/C)^2} = \frac{51 \times 10^3}{(132.18)^2} = 2.92 \text{ KSI} \quad (F_c = 19 \text{ KSI IN TENSION)}$$

BENDING

ON THE EXTREME TENSION FIBER OF BEAMS

$$F_c = 19 \text{ KSI}$$

ON THE EXTREME COMPRESSION FIBER OF BEAMS

$$L_b S_c = \frac{(7.38 \times 12)(1.52)}{I_y} = 170.50$$

$I_y = 0.81$

THIS RESULT IS LOWER THAN 140 BUT LESS THAN 1700, THUS

$$F_c = 23.9 - 0.24 (L_b S_c / I_y)^{1/2} = 23.9 - 0.24 (170.50)^{1/2} = 20.77 \text{ KSI}$$

THE LOWER F GOVERNS, I.E.

$$F_c = 19 \text{ KSI}$$

D. WORKING STRESSES

TENSION

$$f_c = \frac{P}{A} = \frac{0.15}{1.70} = 0.08 \text{ KSI}$$

BENDING

$$SLEWB LENGTH = \frac{70 + 250 + 20}{304.8} = 1.18'$$

M. (ASSUMING A LINEAR DISTRIBUTION) = $\frac{(7.38 - 1.18)}{7.38} \times 3.62 = 3.04 \text{ K}'$

$$f_b = \frac{M}{S} = \frac{3.04 \times 12}{1.80} = 19.31 \text{ KSI} \quad (\text{BOTTOM FLANGE})$$

F T L

Happold

made by/date

checked/date

$$f_u = \frac{M}{s} = \frac{2.04 \times 12}{1.52} = 23.33 \text{ kN} \quad (\text{TOP FLANGE})$$

B. LIMITS CHECKS & CONCLUSIONS

BOTTOM FLANGE (TENSION)

$$\frac{2.04}{1.33(15)} + \frac{12.31}{1.33(10)} = 0.77 < 1.00 \text{ GOOD!}$$

TOP FLANGE (COMPRESSION)

$$\frac{-0.08}{1.33(19)} + \frac{23.32}{1.33(20.77)} = 0.84 < 1.00 \text{ GOOD!}$$

SINCE THE RESULTS ARE LESS THAN UNITY, THE GULF/3 COLUMN SECTION IS ACCEPTABLE.

F T L

Happold

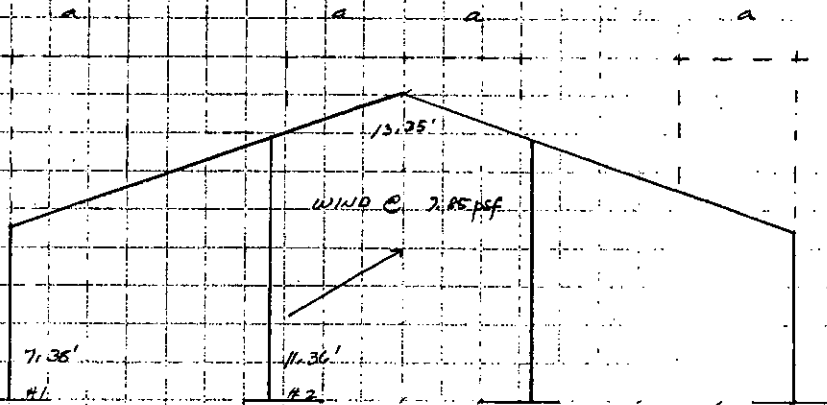
made by/date

Load Assumptions: Gable Frame Elements

checked/date

LOAD DISTRIBUTION FOR A CASE AS WIND (PARALLEL TO THE RIDGE)

GABLE END ELEVATION



TOTAL WIND FORCE, F , ON THE GABLE FRAME

$$A = 32.81 \times (7.36 + 11.36) = 340.13 \text{ ft}^2$$

$$F = 0.69 q_h A = 0.69 (7.85) A = 5.95 (0.69) A$$

ASSUME THAT THE STRUCC SHALE THIS LOAD ACCORDING TO THEIR TRIBUTARY AREAS.

REFER TO THE TABULATION OF STRUT LOADS BASED ON FIGURE 3-A OF ANSI-SPICE 7-92.

ALSO ASSUME THAT F IS DIVIDED AMONG THE CORRESPONDING RESISTING MEMBERS.

REFER TO THE TABULATION OF BRAC & PULLIN LOADS WHERE THE $+P$ 'S ARE THE TENSION FORCES DUE TO NEGATIVE PRESSURES.

UPLIFT FORCES PER LINEAR FOOT OF THE RAFTER

REFER TO THE TABULATION OF UPLIFT LOADS FOR WELLS WITHIN & BEYOND a .



Table 7-1 Wind Load Distribution, Struts
 Table 7-2 Wind Load Distribution, Beams and Purlins
 Table 7-3 Uplift Forces, Rafter

made by/date

checked/date

WIND LOAD DISTRIBUTION, STRUTS

Strut No.	Strut Height (ft)	Tributary Area (sq ft)	+GCp	-GCp	q (psf)	Lateral Load (+) (lbs)	Strut Shear (lbs)	Lateral Load (-) (lbs)	Strut Shear (lbs)
1	7.38	45.81	1.24	-1.65	6.95	338.98	169.49	-449.44	-224.72
2	11.36	124.26	1.14	-1.24	5.95	844.16	422.08	-918.06	-459.03
		340.13	sq ft			2366.29	1183.15	-2735.00	-1367.50

Note : For A < 10 sq ft, GCp = Ci
 For A > 500 sq ft, GCp = Cf
 ANSI/ASCE 7-93 Figure 3 (a)
 (Figure B-2 in Appendix B)

WIND LOAD DISTRIBUTION, BEAMS AND PURLINS

Component	Corresponding Strut No.	Cross Section Area (sq in)	Compressive Capacity (K)	Tensile Capacity (K)	Compression (K)	Tension (K)	Remarks
Eaves Beam	1	0.87	1.30	16.53	-0.58	0.68	ok
Purlin	2	0.64	0.68	12.16	-0.42	0.46	ok
Ridge Beam	na	0.64	0.68	12.16	-0.63	0.69	ok

Note : Fa, the allowable compressive stress, was derived according to slenderness restrictions.
 Ft, the allowable tensile stress for the aluminum profiles, is 19 Ksi.

UPLIFT FORCES, RAFTER

Region	-GCp	Ws (plf)
Ridge & Eaves	-2.07	-60.46
Beyond "a"	-1.11	-32.58

where a < 3.28 ft
 4.15 ft
 but > 1.31 ft
 3.00 ft

A = 85.91 sq ft
 85.91 sq ft

Note : For A < 10 sq ft, GCp = Ci
 For A > 100 sq ft, GCp = Cf
 ANSI/ASCE 7-93 Figure 3 (b)
 (Figure B-2 in Appendix B)

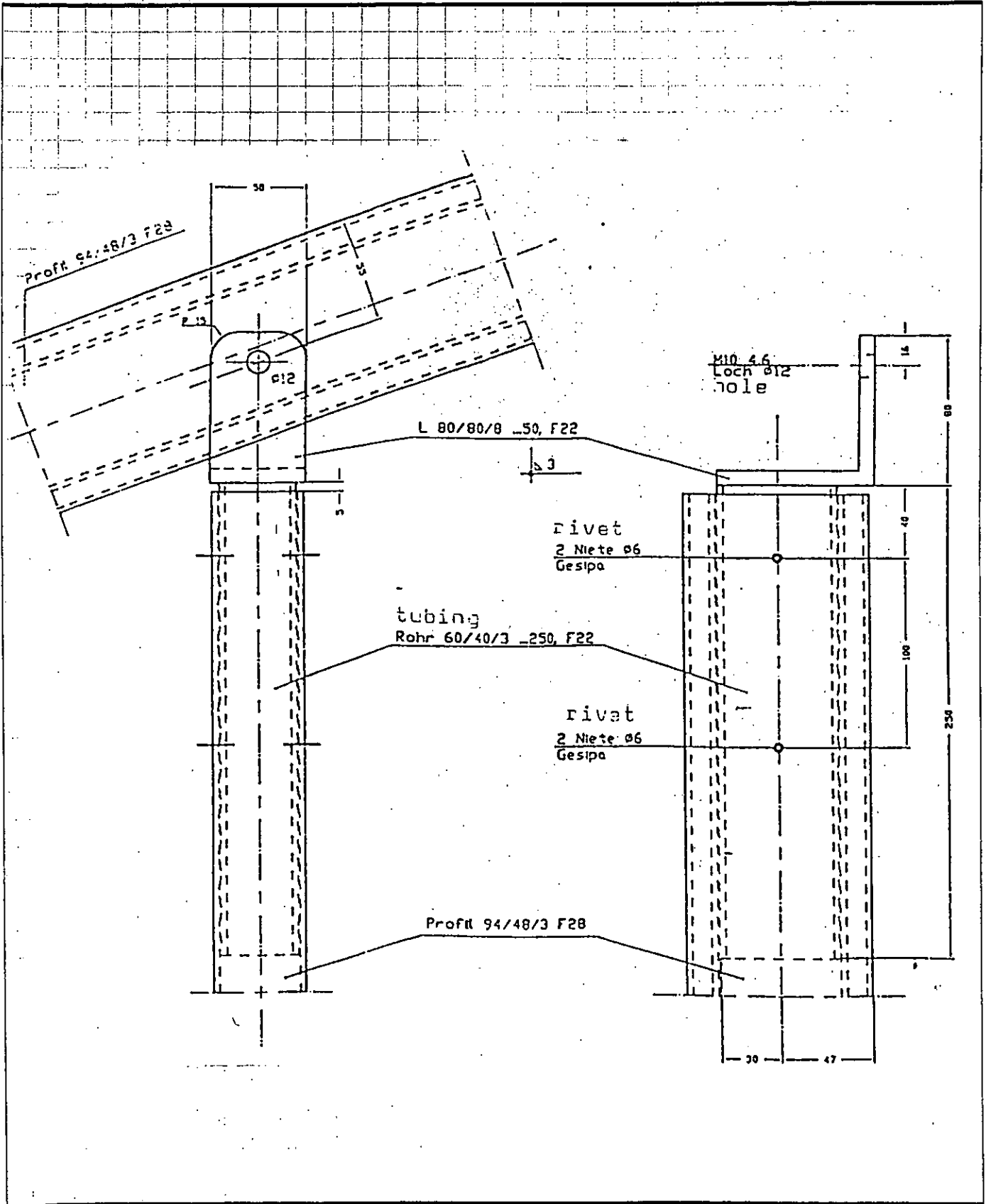
F T L

Happold

made by/date

Figure 7-3 Interior Strut Sections

checked/date





made by/date

7.1.3 Gable Frame Elements: Interior Strut Sections

checked/date

END GABLE FRAME INTERIOR STRUTS 24/48/3 STRUT SECTIONS

A. MAXIMUM APPLIED LOADS

COMBINATION	DESCRIPTION	STRUT	PH. (K)	V. (K)	LENGTH (ft)
#5	UPARALLEL	#2	0.63	0.46	11.36

B. FAILURE CRITERION

UNITY CHECK FOR COMBINED AXIAL & BENDING STRESSES
THE STRUTS ARE BEHAVED AS SIMPLY SUPPORTED BEAMS WITH THEIR STRONGER AXIS ORIENTED ALONG THE UPWARD DIRECTION.

C. ALLOWABLE STRESSES

TENSION
 $F_t = 10 \text{ ksi}$
 ON THE AXIAL NET SECTION OF ANY TENSION MEMBER

BENDING
 ON THE EXTREME COMPRESSION FIBER OF BEAMS

$$L_e S_x = (11.36 \times 12) (1.89) = 318.08$$

$$I_y = 0.81$$
 THE RESULT EXCEEDS 144 BUT IS LESS THAN 1700, THEREFORE

$$F_c = 22.3 = 0.24 (L_e S_x)^{1/2} = 23.0 = 0.24 (318.08)^{1/2} = 13.62 \text{ ksi}$$

$$I_y$$
 ON THE EXTREME TENSION FIBER OF BEAMS
 $F_t = 10 \text{ ksi}$
 THE LOWER F_t GOVERNS, 10

D. WORKING STRESSES

TENSION

$$F_t = \frac{P}{A} = \frac{0.63}{1.75} = 0.35 \text{ ksi}$$

BENDING

$$M = \frac{V L}{4} = \frac{0.46 (11.36)}{4} = 1.31 \text{ K'}$$

$$F_b = \frac{M}{S} = \frac{1.31 \times 12}{1.54} = 10.05 \text{ ksi}$$

E. UNITY CHECK & CONCLUSION

$$\frac{F_t}{1.33 F_t} + \frac{F_b}{1.33 F_b} = \frac{0.35}{1.33 (10)} + \frac{10.05}{1.33 (10)} = 0.41 < 1.00 \text{ GOOD!}$$

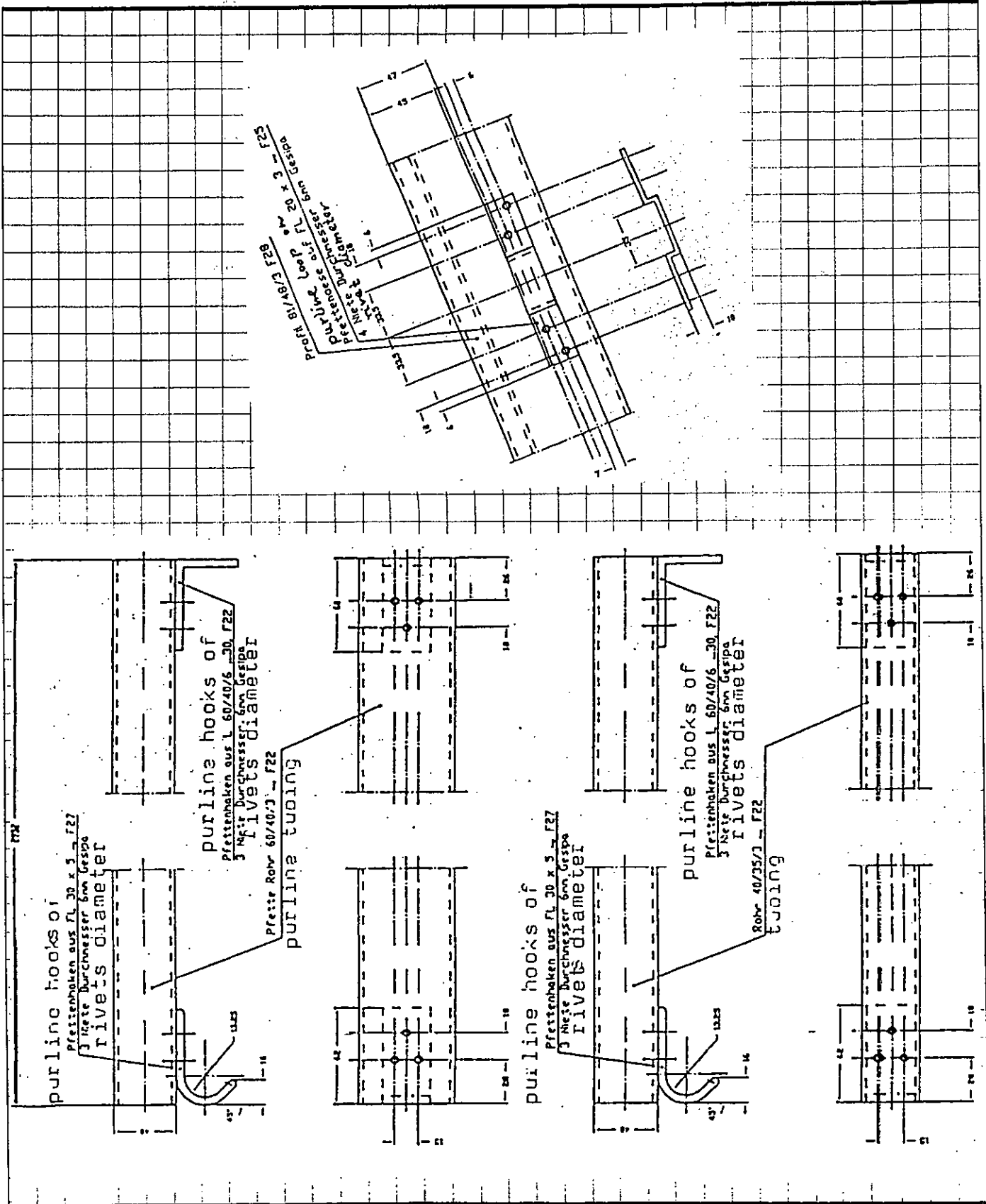
THE RESULT IS LESS THAN UNITY, THEREFORE,
 THE 24/48/3 STRUT SECTION IS ACCEPTABLE.



made by/date

Figure 7-4 Beams and Purlins

checked/date



F T L

Happold

made by/date

Table 7-4 Total Compressive Capacity, Beams and Purlins

checked/date

TOTAL COMPRESSIVE CAPACITY OF THE LONGITUDINAL FORCE-RESISTING MEMBERS

Criteria	Units	Purlins	Eaves Beams	Ridge Beam	Remarks
Section		40/35/3	60/40/3	40/35/3	F28 Aluminum
Quantity	(pcs)	2.00	2.00	1.00	
Length	(ft)	9.69	9.69	9.69	Net Length
Ax	(sq in)	0.64	0.87	0.64	
r min	(in)	0.53	0.63	0.53	
Slenderness	$KL / r \text{ min}, K = 1$	219.28	184.48	219.28	> 66
Fa	$51E3 / (KL/r)^2$	(Ksi) 1.06	1.50	1.06	Compression
P	Ax Fa	(K) 0.68	1.30	0.68	
Sum P	P x Quantity	(K) 1.36	2.61	0.68	
Total					4.64



7.2 Secondary Elements
7.2.1 Purlins
7.2.2 Beams

made by/date
checked/date

PURLIN & BEAMS, 40/35/3 & 60/40/3 ALUMINIUM SECTIONS

A. MAXIMUM APPLIED LOADS

COMBINATION	DESCRIPTION	TOTAL P (K)	REMARKS
#5	UP PARALLEL	1.15	DUE TO NORMAL WIND LOAD

REFER TO TABLE 7-1 FOR THE TOTAL MAXIMUM COMPRESSIVE LOAD ON THE ROOF

B. FAILURE CRITERION

TOTAL CAPACITY OF THE AXIALLY LOADED ELEMENTS

C. ALLOWABLE CAPACITY

FROM THE LOAD ASSUMPTIONS

$$\leq P_c = 4.04 \text{ K}$$

D. WORKING LOAD

FROM PREVIOUS CALCULATIONS

$$P = 1.15 \text{ K}$$

E. LOAD CHECK & CONCLUSION

THERE IS ENOUGH CAPACITY IN THE SYSTEM TO RESIST THE CALCULATED WIND LOADS. SINCE THE TOTAL CAPACITY IS NOT EXCEEDED, I.E.,

$$P = 1.15 \text{ K} < 4.04 \text{ K} = \leq P_c$$

THEN THE ASSEMBLAGE OF AXIAL ELEMENTS IS ACCEPTABLE.

F. ADDITIONAL CHECKS

REFER TO TABLE 7-2 FOR THE ACTUAL LOAD VS. CAPACITY CHECKS FOR A CASE #5 WIND LOAD DISTRIBUTION.



made by/date

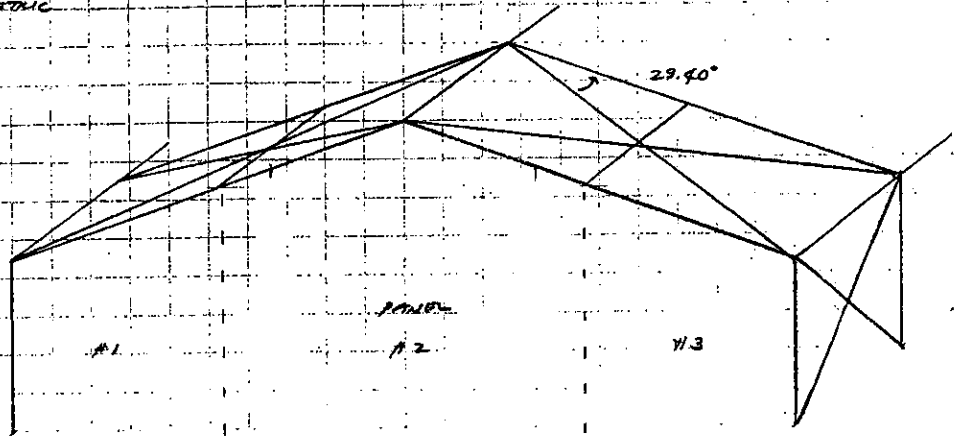
Load Assumptions: Roof and Wall Cables

checked/date

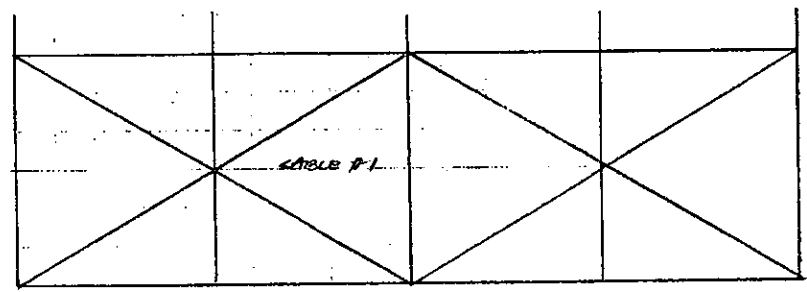
LOAD DISTRIBUTION FOR A CASE OF WIND (WIND PARALLEL TO THE RIDGE)

ROOF & WALLS

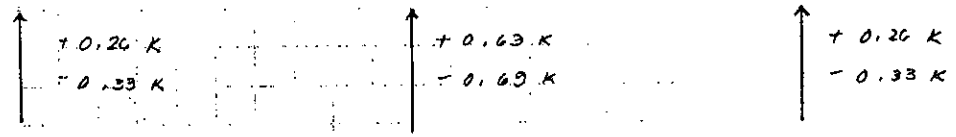
HAPPOLD



PLAN

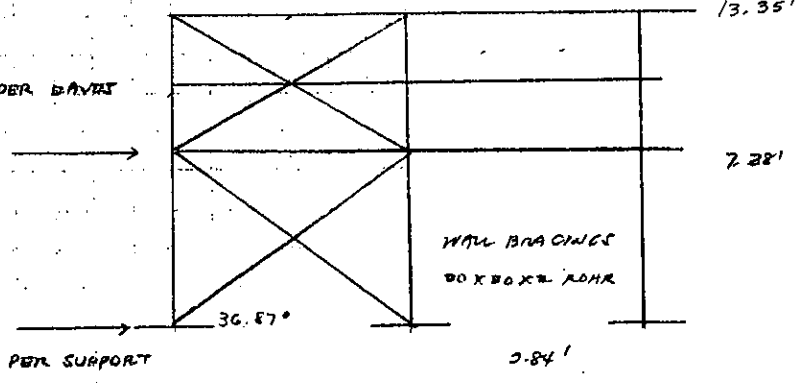


6 MM CABLES



ELEVATION

F/4 PER BAY





made by/date

Table 7-6 Cable Forces

checked/date

CABLE FORCES

Panel No.	Mean Panel Ht. (ft)	Tributary Area (sq ft)	+G _{Cp}	-G _{Cp}	q (psf)	Lateral Load (+) (lbs)	Lateral Load (-) (lbs)	P (+) (lbs)	P (-) (lbs)
1	8.87	72.79	1.20	-1.54	5.95	518.17	-668.08	259.09	-334.04
2	11.86	194.55	1.10	-1.20	5.95	1268.66	-1384.36	634.33	-692.18
3	8.87	72.79	1.20	-1.54	5.95	518.17	-668.08	259.09	-334.04

340.13

2305.00

-2720.51

1152.50

-1360.26

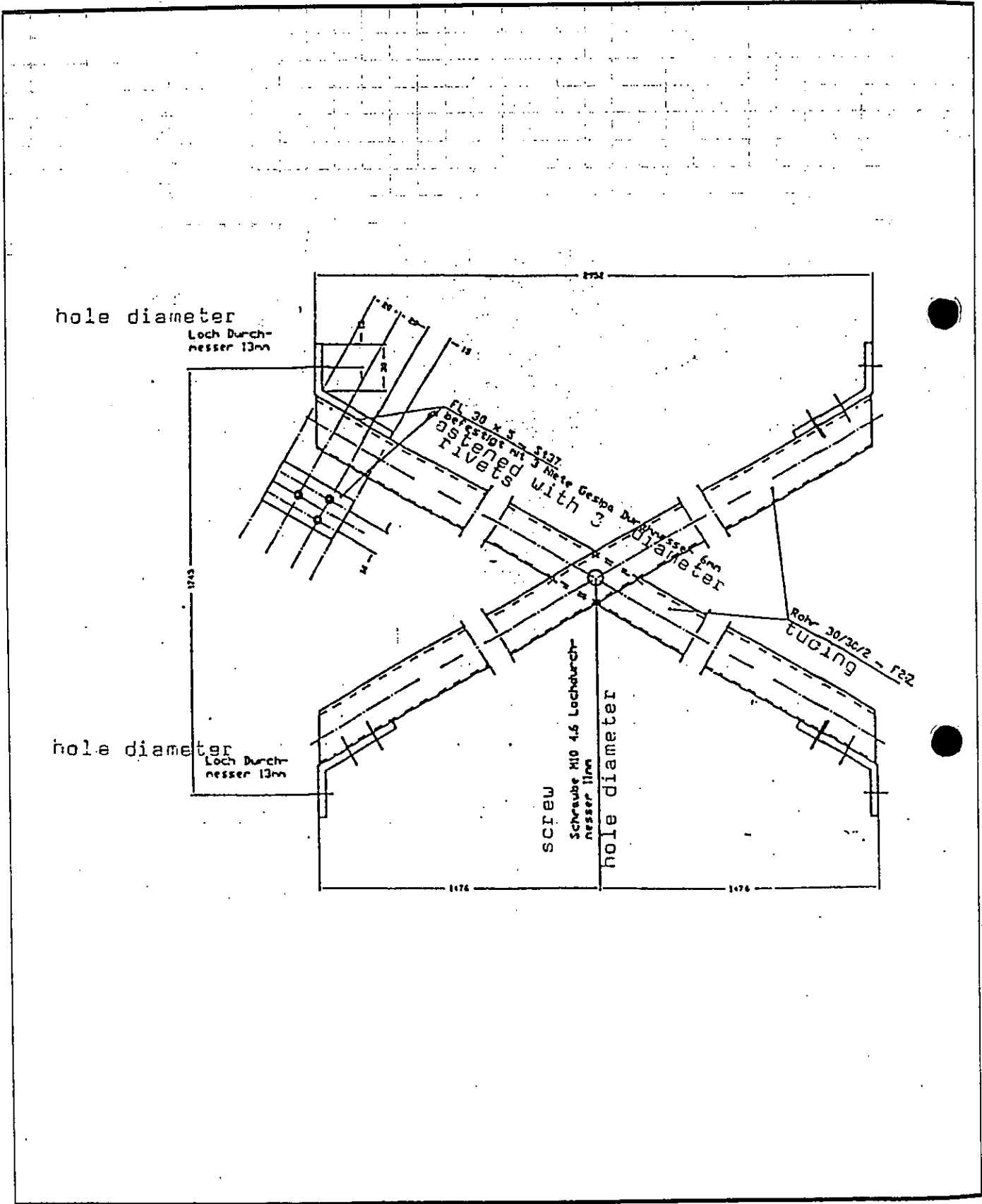
Note :
 For A < 10 sq ft, G_{Cp} = C_i
 For A > 500 sq ft, G_{Cp} = C_f
 ANS/ASCE 7-93 Figure 3 (a)
 (Figure B-2 in Appendix B)



made by/date

Figure 7-6 Wall Braces

checked/date



F T L

Happold

7.2.3 Roof Cables

made by/date

7.2.4 Diagonal Column Braces

checked/date

ROOF AND WALL CABLE TENSIONSPressure Loads

Cable No.	No. of Bays	Diameter (mm)	Capacity F.O.S = 2.5 (k)	Cable Inclination (deg)	Wind Load (Pressure) (K)	Beam / Purlin Compression (K)	Aggregate Cable Tension (K)	Relative Stress (%)
1: Roof	1	6.00	2.04	60.59	0.63	0.63	0.65	31.68
2: Wall	1	30.00	7.56	36.87	0.26	0.58	0.72	9.53

$$F / 4 = 0.58$$

$$F = 2.31$$

Suction Loads

Cable No.	No. of Bays	Diameter (mm)	Capacity F.O.S = 2.5 (k)	Cable Inclination (deg)	Wind Load (Suction) (K)	Beam / Purlin Tension (K)	Aggregate Cable Tension (K)	Relative Stress (%)
1: Roof	1	6.00	2.04	60.59	-0.69	0.69	0.70	34.57
3: Wall	1	30.00	7.56	36.87	-0.33	0.68	0.85	11.25

$$F / 4 = -0.68$$

$$F = -2.72$$

Note:

1. The aggregate cable tension needed to resist the applied wind load is distributed among the bays with cables.
2. The cable inclination is taken from the beams or purlins.
3. The relative stress is the ratio of the actual load per bay to the cable capacity.

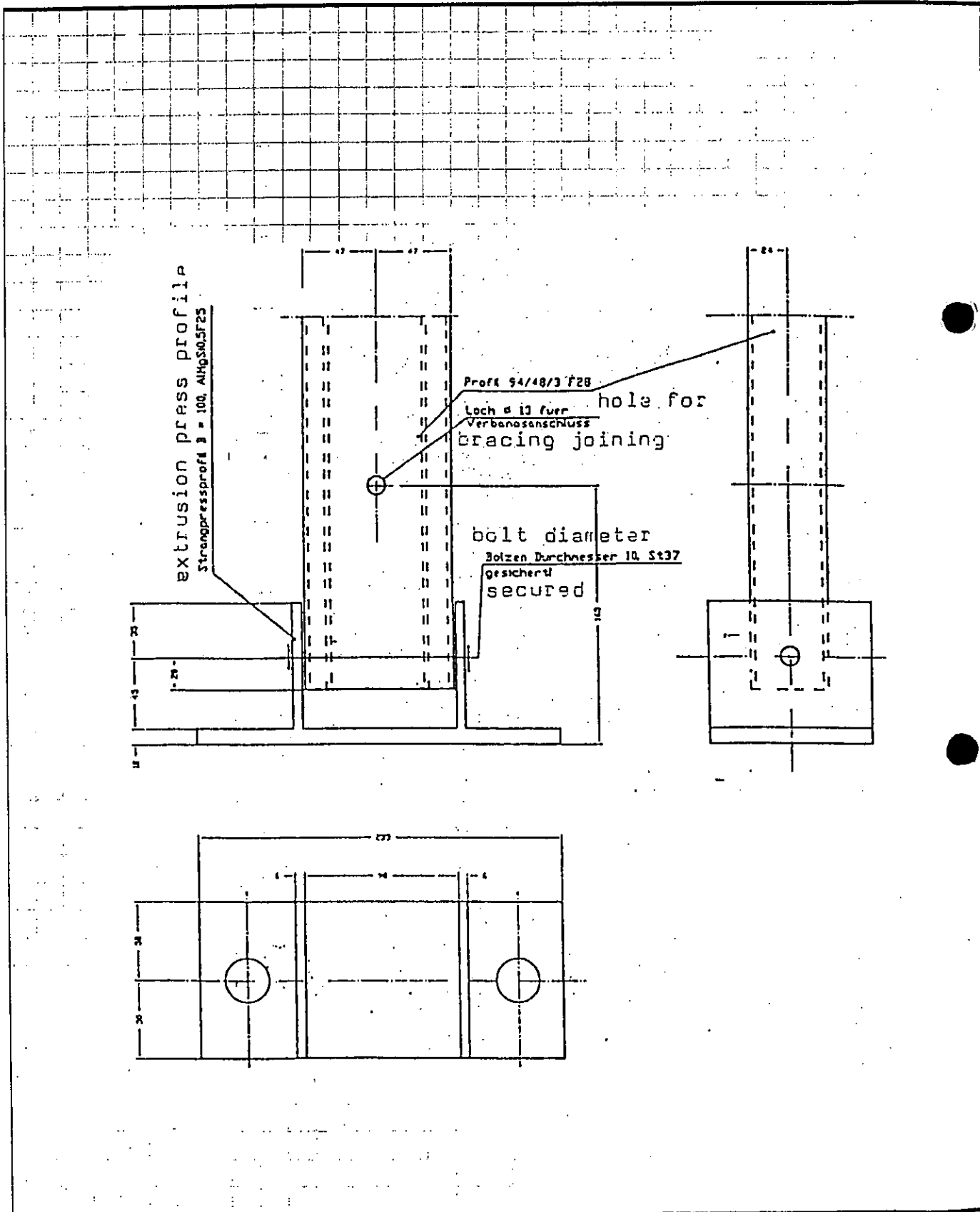
F T L

Happold

made by/date

Figure 7-7 Base Plates

checked/date



F T L

Happold

made by/date

7.2.5 Base Plates

checked/date

EXTERNAL BASE PLATES, 233 X 100 X 1/4 P25

A. MAXIMUM APPLIED LOADS

COMBINATION	DESCRIPTION	P (K)	V (K)	REMARKS
#7	DL + LL	+ 0.91	± 0.24	MAXIMUM PRESSURE
#12	DL + WINDPRESSURE	- 0.15	- 0.72	MAXIMUM SHEAR
#13	DL + UPTURNWIND	- 0.73	+ 0.10	MAXIMUM UPLIFT

B. FAILURE CRITERIA

A_{req} , THE MINIMUM REQUIRED BASE PLATE AREA

F_v , ON THE GRAVITY PEGS (ST 37)

THE EFFECTS OF UPLIFT SHOULD BE COUNTERACTED BY OTHER ACCEPTABLE MEANS.

C. REQUIRED BASE PLATE AREA & ALLOWABLE SHEAR STRESS

$$A_{req} = \frac{0.91 + (7.38 \times 2.11)}{3 \text{ Ksf}} \cdot 10^{-3} = 0.19 \text{ ft}^2$$

$$F_v = 10 \text{ KSI} \quad \text{AISC MANUAL}$$

D. ACTUAL BASE PLATE AREA & SHEAR STRESS

BEARING AREA

$$A_{prov} = (233 \times 100) / (304.8)^2 = 0.25 \text{ ft}^2$$

SHEAR STRESS ON EACH OF THE 2 Ø25 MM PEGS

$$f_{v25} = \frac{0.72}{2 \cdot 10.76} = 0.34 \text{ KSI}$$

$$f_{v25} = \frac{0.72}{2 \cdot 10.76} = 0.34 \text{ KSI}$$

E. CHECKS & CONCLUSION

SINCE

$$A_{prov} = 0.25 \text{ ft}^2 > 0.19 \text{ ft}^2 = A_{req}$$

$$f_{v25} = 0.34 \text{ KSI} < 10 \text{ KSI} = F_v$$

THAT THE 233 X 100 X 1/4 EXTERNAL BASE PLATE IS ACCEPTABLE.

IN IMPLEMENTING THE FOUNDATION AND/OR ANCHORAGE SYSTEMS, CONSIDERATIONS SHOULD BE GIVEN TO THE MAXIMUM UPLIFT & SHEAR LOAD COMBINATIONS.

F T L

Happold

made by/date

checked/date

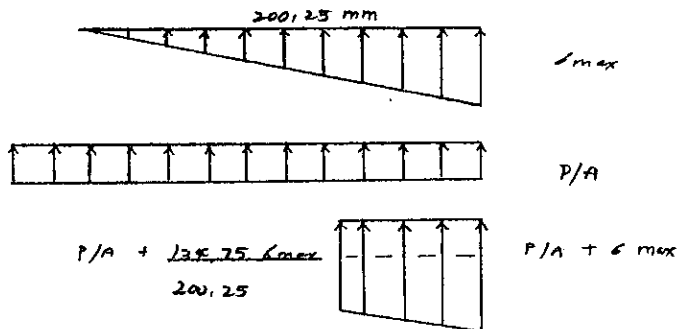
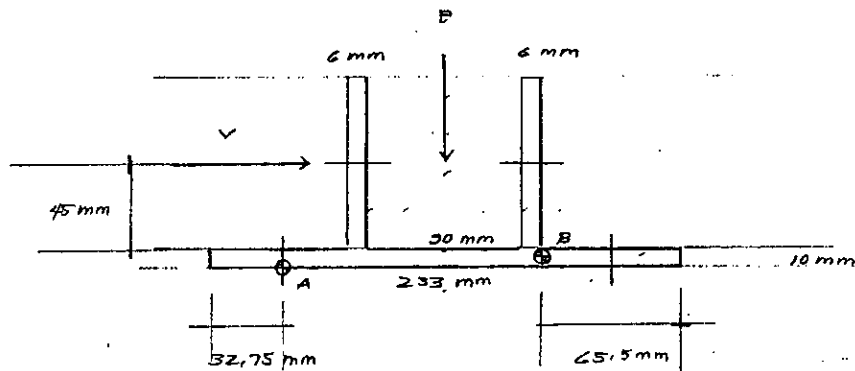
EXTENSION BASE PLATE, 233 x 100 x 10 F25 ALUMINIUM PLATE BEARING

A. MAXIMUM APPLIED LOADS
SAME AS PREVIOUS

B. FAILURE CRITERION
 F_b , THE ALLOWABLE BENDING STRESS OF ALUMINIUM

C. ALLOWABLE STRESS
 $F_b = 13 \text{ ksi}$

D. WORKING STRESS



SECTION PROPERTIES

$$A = 233 \times 100 = 23300 \text{ mm}^2 \sim 36.12 \text{ in}^2$$

$$S = \frac{bd^2}{6} = \frac{100(10)^2}{6} = 1666.67 \text{ mm}^3 \sim 0.10 \text{ in}^3$$

2 M FROM POINT A

$$55V = 0.5 \text{ Gmm} \frac{(200.25)(100)(2)(200.25)}{2}$$

$$\sigma_{max} = \frac{V}{24303.07} \text{ kN/mm} = \frac{V}{37.67} \text{ ksi}$$

F T L

Happold

made by/date

checked/date

Σ F ALONG THE VERTICAL

$$\frac{P}{A} = \frac{P}{23300} \text{ ksi/mm} = \frac{P}{36.12} \text{ ksi}$$

MAXIMUM MOMENT @ B

$$\frac{M}{100} = \left(\frac{P}{23300} + \frac{136.75}{200.25} \frac{V}{24303.07} \right) \frac{65.5^2}{2} + 0.15 \left(\frac{65.5}{200.25} \right) \frac{V}{24303.07} \frac{2}{3} (65.5)^2$$

$$M = 2.21 P + 7.86 V \quad \text{kmm}$$

$$= 0.36 P + 0.31 V \quad \text{kin}$$

BENDING STRESS

$$f_b = \frac{M}{S} = \frac{2.21 P + 7.86 V}{0.10} = 3.52 P + 3.07 V \quad \text{ksi}$$

REFER TO TABLE 7-5 FOR THE RESULTS.

E. STRESS CHECK & CONCLUSION

SINCE

$$f_b = 2.13 \text{ ksi} < 19 \text{ ksi} = F_b$$

THAT THE 233 X 100 X 16 EXTERIOR BASE PLATE IS ACCEPTABLE.

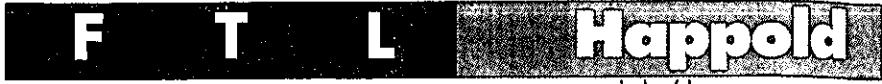


Table 7-7 Plate Bending

made by/date

checked/date

PLATE BENDING

Maximum Pressure			
P/A =	P / 23300.00	Ksmm	
	P / 36.12	Ksf	
	P / 0.25	Ksf	
Maximum Stress			
S maximum =	V / 24303.07	Ksmm	
	V / 37.67	Ksf	
	V / 0.26	Ksf	
Maximum Moment			
M maximum =	9.21 P +	7.86 V	Kmm
	0.36 P +	0.31 V	Kin
Bending Stress			
fb =	3.56 P +	3.04 V	Ksf

Tabulation

Criteria	Units	Load Comb. # 7	Load Comb. # 12	Load Comb. # 13
P	K	0.41	-0.15	-0.73
P/A	Ksf	1.63	-0.60	-2.91
V	K	0.24	-0.72	-0.10
S maximum	Ksf	0.92	2.75	0.38
M	Kin	0.22	0.17	-0.23
fb	Ksf	2.19	1.66	-2.00

Note : Disregard the negative M's and fb's.

F T L

Happold

made by/date

checked/date

EXTENSION PLATE PLATE, 6 X 100 VERTICAL CURVED

A. MAXIMUM APPLIED LOADS
SAME AS PREVIOUS

B. FAILURE CRITERIA

AISC SPECIFIED ALLOWABLES

- F_c COMPRESSIVE STRESS
- F_t TENSILE STRESS
- F_v SHEAR STRESS
- F_b BENDING STRESS

C. ALLOWABLE STRESSES

COMPRESSION

$$r_{min} = \left(\frac{I}{A} \right)^{1/2} = \left(\frac{1800}{600} \right)^{1/2} = 1.73 \text{ mm}$$

$$\frac{KL}{r_{min}} = \frac{1.0 (45)}{1.73} = 26.01 < 60$$

$$F_c = 20.2 - 0.126 (4/r) = 20.2 - 0.126 (26.01) = 16.92 \text{ ksi}$$

TENSION

$$F_t = 19 \text{ ksi}$$

ON THE NET SECTION OF ANY TENSION MEMBER

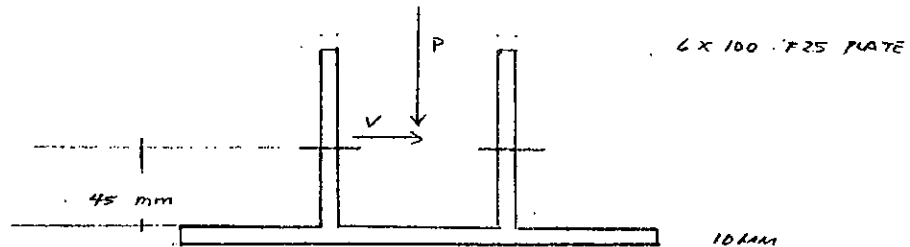
SHEAR

$$F_v = 12 \text{ ksi}$$

BENDING

$$F_b = 19 \text{ ksi}$$

D. WORKING STRESSES



WHERE

$$f_c = \frac{P}{2A_{gross}} = \frac{P}{2(0.93)} \text{ ksi}$$

$$A_{net} = 0.81 \text{ in}^2 \text{ ONLY}$$

$$f_v = \frac{V}{2A_{net}} = \frac{V}{2(0.84)} \text{ ksi}$$

F T L

Happold

made by/date

checked/date

$$f_b = \frac{M}{S} = \frac{0.5 \times V (95)(3)}{1500} = \frac{V}{26.67} \text{ K/mm} = 24.13 \text{ V K/ci}$$

REFER TO THE SUCCESSING TABULATION FOR THE RESULTS.

E, STRESS CHECKS & CONCLUSION

SINCE NONE OF THE ALLOWABLE STRESSES IS EXCEEDED,
THEN THE 6 X 100 VERTICAL PLATE IS ACCEPTABLE.

F T L

Happold

made by/date

Table 7-8 Support Stresses

checked/date

VERTICAL SUPPORTS

Criteria	Units	Load Comb. # 7	Load Comb. # 12	Load Comb. # 13	Allowables
P	K	0.41	-0.15	-0.73	
fa gross (C)	Ksi	0.22			16.92
ft net (T)	Ksi		0.09	0.44	19.00
ft gross (T)	Ksi		0.08	0.39	19.00
V	K	0.24	-0.72	-0.10	
fv	Ksi	0.14	0.43	0.06	12.00
M	Kin	0.43	1.28	0.18	
fb	Ksi	5.81	17.42	2.42	19.00
fb total	Ksi	6.03	17.50	2.81	19.00

Part 8

Acceptability Under Variable Conditions

8.1 Relative Stress Levels

Table 8-1 shows the frame components' loading condition under the standard C-70 wind. It is from this tabulation that the most stressed element is discovered and consequently tested against additional, higher loadings.

8.2 Allowable Exposure Chart

Since it was already proven that the frame can sufficiently withstand C-70 wind pressures, minor calculations can be performed to determine its adequacy for a slightly more severe exposure. This is made valid and possible by the principle of superposition. Reference is made to Appendix C: ROBOT V6 Listings for internal force and moment values at the same node of the critical element or segment for both the static (DL or DL+LL) and the dynamic loading conditions.

8.1 Relative Stress Levels



made by/date

Table 8-1 Relative Stress Levels

checked/date

**RELATIVE STRESS LEVELS (AS PERCENTAGES OF THE ALLOWABLE STRESSES)
MAIN AND SECONDARY MEMBERS**

ELEMENT	AXIAL (%)	BENDING (%)	SHEAR (%)	BEARING (%)
Rafters				
Tension Flange	0.67	84.59		
Compression Flange	-0.67	95.02		
Columns				
Tension Flange	0.32	76.22		
Compression Flange	-0.32	84.46		
Gable Struts	1.38	39.67		
Purlins and Beams	19.07			
Roof Cables	34.57			
Wall Cables	11.25			
Base Plates				
Bearing Plate		11.53		56.00
Vertical Support Plates	(C) 1.30 (T) 2.32	69.08	3.58	
Ground Pegs			4.70	

8.2 Allowable Exposure Chart

F T L

Happold

made by/date

checked/date

ACCEPTABILITY UNDER VARIABLE WIND PRESSURES (EXPOSURE CLASS D & WIND SPEED C)

FROM THE TABLE OF RELATIVE STRESS LEVELS & BS INSPECTION, THE CRITICAL FRAME SEGMENT IS ELEMENT # 2 (RAFTER SECTION). THIS IS TRUE FOR A DESIGNATED C-70 WIND DELIVERING A PRESSURE OF 7.85 psf.

ELEMENT # 2 WITH A MAXIMUM MOMENT AT NODE # 2

COMBINATION	DESCRIPTION	FL #	NDR	PCF	M (K')	LENGTH (FT)	MAX (P)
# 12	DL + WINDPRESSURE	2	2	+0.31	-3.62	17.46	0.0L
# 6	DL	2	2	-0.11	+0.52	17.46	0.0L

MAXIMUM MOMENT AT q = 10.26 psf (CLASS D, 70 MPH OR CLASS C, 80 MPH)

$$\text{MAX } M = \left(-3.62 - 0.52 \right) \frac{10.26}{7.85} + 0.52 = -4.89 \text{ K'}$$

MAXIMUM AXIAL FORCE AT q = 10.26 psf (CLASS D, 70 MPH OR CLASS C, 80 MPH)

$$\text{MAX } P = \left(+0.31 - -0.11 \right) \frac{10.26}{7.85} + -0.11 = -0.44 \text{ K}$$

AXIAL & BENDING STRESSES

$$F_t = \frac{P}{A} = \frac{-0.44}{1.73} = -0.25 \text{ KSI}$$

$$M \text{ (ASSUMING A LINEAR DISTRIBUTION)} = \frac{(17.46 - 1.18)}{17.46} \times -4.89 = -4.56 \text{ K'}$$

$$f_b = \frac{M}{S} = \frac{-4.56 \times 12}{1.89} = -28.25 \text{ KSI} \quad \text{BOTTOM FLANGE}$$

$$f_c = \frac{M}{S} = \frac{4.56 \times 12}{1.52} = 35.07 \text{ KSI} \quad \text{TOP FLANGE}$$

UNITY CHECKS WHERE F_t & F_b ARE LIFTED FROM PREVIOUS (SECTION 7.1.1) CALCULATIONS

BOTTOM FLANGE (TENSION)

$$\frac{f_t}{1.33 F_t} + \frac{f_b}{1.33 F_b} = \frac{0.25}{1.33(12)} + \frac{28.25}{1.33(12)} = 1.15 > 1.00 \text{ NOT GOOD!}$$

TOP FLANGE (COMPRESSION)

$$\frac{f_t}{1.33 F_t} + \frac{f_c}{1.33 F_c} = \frac{-0.25}{1.33(12)} + \frac{35.07}{1.33(20.42)} = 1.27 > 1.00 \text{ NOT GOOD!}$$

THE RESULTS ARE GREATER THAN UNITY.

THEFORE, THE MEMBER WILL FAIL AT EITHER A CLASS D, 70 MPH OR CLASS C, 80 MPH OR WORSE EXPOSURE.

SHOWN HEREAFTER IS A WIND PRESSURE CHART FOR THE 10 M RODER FRAME TENT, ELIMINATING THE PRESSURES AT WHICH THE STRUCTURE IS NOT ADEQUATELY DESIGNED FOR.

F T L

Happold

made by/date

Table 8-2 Allowable Exposure Chart

checked/date

10 METER RODER FRAME (13.35 FT MAXIMUM HEIGHT)
WIND PRESSURES, q (psf)

Exposure	60 mph	70 mph	80 mph	90 mph
Class A	1.55	2.11	2.75	3.48
Class B	3.34	4.54	5.93	7.51
Class C	5.77	7.85	10.26	12.98
Class D	7.54	10.26	13.41	16.97

The 10 meter version is suitable for up to a Class C, 70 mph wind exposure.
The stricken values are the unsuitable pressures for the 10 m Roder Frame Tent.