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Structural Evaluation of Aircraft Hangar 46, Corpus Christi Army Depot

Ghassan K. Al-Chaar, Daniel J. Randolph,
Gregory E. Lamb, and Pramod J. Desai

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Foreword

This study was conducted for the Facilities Engineering and Management Division, Corpus Christi Army Depot (CCAD), TX, under Military Interdepartmental Purchase Request (MIPR) 0J3W000107, dated 22 June 2000. The technical monitor was Luis H. Salinas, SIOCC-DS-FE.

The work was performed by the Materials and Structures Branch (CF-M) of the Facilities Division (CF), Construction Engineering Research Laboratory (CERL). The CERL Principal Investigator was Dr. Ghassan K. Al-Chaar. The technical editor was Gordon L. Cohen, Information Technology Laboratory – Champaign. Martin J. Savoie is Chief, CF-M, and L. Michael Golish is Chief, CF. The Acting Technical Director of the Facility Acquisition and Revitalization business area is Dr. Paul A. Howdyshell, CV-ZT. The Acting Director of CERL is William D. Goran.

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Contents

List of Figures and Tables

Figures

Tables

1 Introduction

Background

Recent hurricanes have demonstrated the vulnerabilities of older structures to hurricane-level wind loads. Many aged aircraft hangars and other structures built with steel-truss-type roofs have been damaged or totally collapsed in recent hurricanes. Wind-related damage observed in hangars has included the over-stressing of structural members, the blowing of hangar door systems out of their frames, and the compromising of structural member connections. In many observed cases progressive structural failure has begun with the failure of hangar doors or windows, immediately followed by a sudden, high variation between internal and external wind pressures that resulted in a total collapse.

In the past, structures were built according to less stringent building codes. Over the years, many such structures have been modified to support additional loads in excess of the original design specifications. Also, over time, environmental factors have reduced the capacities of structural members.

A number of steel truss aircraft hangars at Corpus Christi Army Depot (CCAD) are similar in design and construction to ones that have performed poorly during recent hurricanes in other parts of the nation. Engineering analyses of hangars currently in use can identify structural vulnerabilities and provide engineers with the information necessary to develop retrofit schemes for reducing or eliminating these vulnerabilities to severe wind loads. The Construction Engineering Research Laboratory, U.S. Army Engineer Research and Development Center, was tasked by the CCAD Facilities Engineering and Management Division to conduct such an engineering analysis of a representative steel truss aircraft hangar on the installation.

Objective

The objective of this study was to evaluate the structural adequacy of Hangar 46 at CCAD by performing structural analysis using the most recent code guidelines.

Approach

State-of-the-art research methods on the behavior of steel structures under dynamic loads were utilized. The current condition of Hangar 46 was evaluated. Structural deficiencies and overstressed members and joints were identified, and retrofit methods to meet the requirements of current codes were developed.

The work was conducted in the following three phases:

1. *Inspection of Hangar 46.* A detailed field inspection of the structure under consideration was performed during this task. Weak links and structural deficiencies were identified. Data needed to carry out structural evaluations were collected. Repairs of deficient members were proposed to restore their strength to the original design specifications.
2. *Execution of structural analyses.* Thorough structural analyses were conducted, including combined gravity, wind uplift, and equivalent lateral static analyses. Wind load values were based on ANSI/ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers, 1998).
3. *Development of retrofit schemes.* Upgrades were developed for structural members subjected to failure, as identified in Task 2. Technical drawings were provided along with general notes on the upgrade requirements.

Units of Weight and Measure

U.S. customary units of measure are used throughout this report. A table of conversion factors for International System (SI) of units is provided below.

SI conversion factors		
1 in.	=	2.54 cm
1 ft	=	0.305 m
1 sq in.	=	6.452 cm^2
1 sq ft	=	0.093 m^2
1 cu ft	=	0.028 m^3
1 cu yd	=	0.764 m^3
1 lb	=	0.453 kg
1 psi	=	6.89 kPa
1 psf	=	0.048 kPa
1 kip	=	453 kg
1 ton	=	906 kg

2 Hangar Inspection

Five types of trusses were identified in Hangar 46, as shown in Figures 2-1 and 2-2. Every member was labeled for each type of truss, and can be located by defining the hangar number, truss type number, truss location on the plan drawing, section of the truss, and a unique identifying label. Photographs of damaged members and other hangar elements may be found in Appendix A, and supplementary diagrams are shown in Appendix B.

The most severe problems were found in the door pockets and the bottom chords connecting Truss T1. Three typical problems were identified in the door pockets: (1) the base beams of some columns were severely corroded, (2) the flanges and webs of some columns at the top of concrete walls were severely corroded, and (3) diagonal braces were either bent or loose. Based on the structural analysis reported in Chapter 5, repair recommendations for returning damaged members to their original condition (when necessary) were developed (see Chapter 7).

The following notes on specific deficiencies were recorded from observations made during the inspection. These notes are related to strength reduction and in many cases indicate the need for repairs. For location of members on a plan view of the hangar, see Figure 2-3. Notes in parentheses indicate member size.

Notes 1 through 13 are for door pockets:

1. Tie rod (7/8 in. diameter) located at column P5.5, is bent. See Appendix A, Figure A-1.
2. Tie rods (7/8 in. diameter) located at columns P0.5, P1.5, P4.5, and P5.5 are loose.
3. Horizontal brace (10-I-21) located at second floor, between columns P5.5 and N5.5, is buckling. See Figure A-2.
4. Hangar door wheels are rusting and deteriorating. See Figure A-3.
5. Vertical members by handles on hangar doors are dented, likely resulting from opening doors with hooks and chains.
6. Concrete is spalling at column N5.5 (8-H-31). See Figure A-4.
7. Spalled concrete allowed major corrosion of column A0.5 (8-H-31), resulting in an estimated strength reduction of up to 30 percent. See Figure A-5.
8. Minor corrosion was found at the base of column AA0.5 (12-I-25).

9. Unsupported wood column (4x4 post) near column AA2 is a safety hazard. See Figure A-6.
10. A makeshift wood column (4x4 post) near column AA2 provides inadequate support of second floor beam between columns AA2 and AA1.5. See Figure A-7.
11. A beam is under torsional stress at the second floor between columns AA4 and AA4.5 (6-H-15.5).
12. Major corrosion of the web and flange at the base of column A5 (18-I-47) has resulted in an estimated 30 percent strength reduction. See Figure A-8.
13. Minor corrosion of the web and flange at the base of column A5.5 (8-H-31) has resulted in an estimated 10 percent strength reduction. See Figure A-9.

Notes 14 through 17 are for deficiencies in the main hangar area:

14. Entire hangar is losing paint. See Figure A-10.
15. Minor buckling of diagonal braces (single angle, 3½ x 2½ x ¼) has occurred at the following locations:
    - between columns G2.4 and H2.5; see Figure A-11
    - between columns L2.5 and M3; see Figure A-12.
16. Buckling of bottom braces (single angle, 3½ x 2½ x ¼) has occurred at the following locations:
    - between columns L2.4 and M2.4; see Figure A-13
    - between columns H2.5 and J2.5; see Figure A-14
    - between columns C2.5 and E2.5; see Figure A-15
    - between columns E3.1 and F3.1; see Figure A-16
    - between columns K3.1 and L3.1; see Figure A-17
    - between columns H3.2 and J3.2; see Figure A-18
    - between columns L3.4 and M3.4; see Figure A-19
    - between columns E3.4 and F3.4; see Figure A-20.
17. A column located at K2 (18-I-85) has been damaged from welding and gouging, resulting in an estimated 5 percent strength reduction. See Figure A-21.

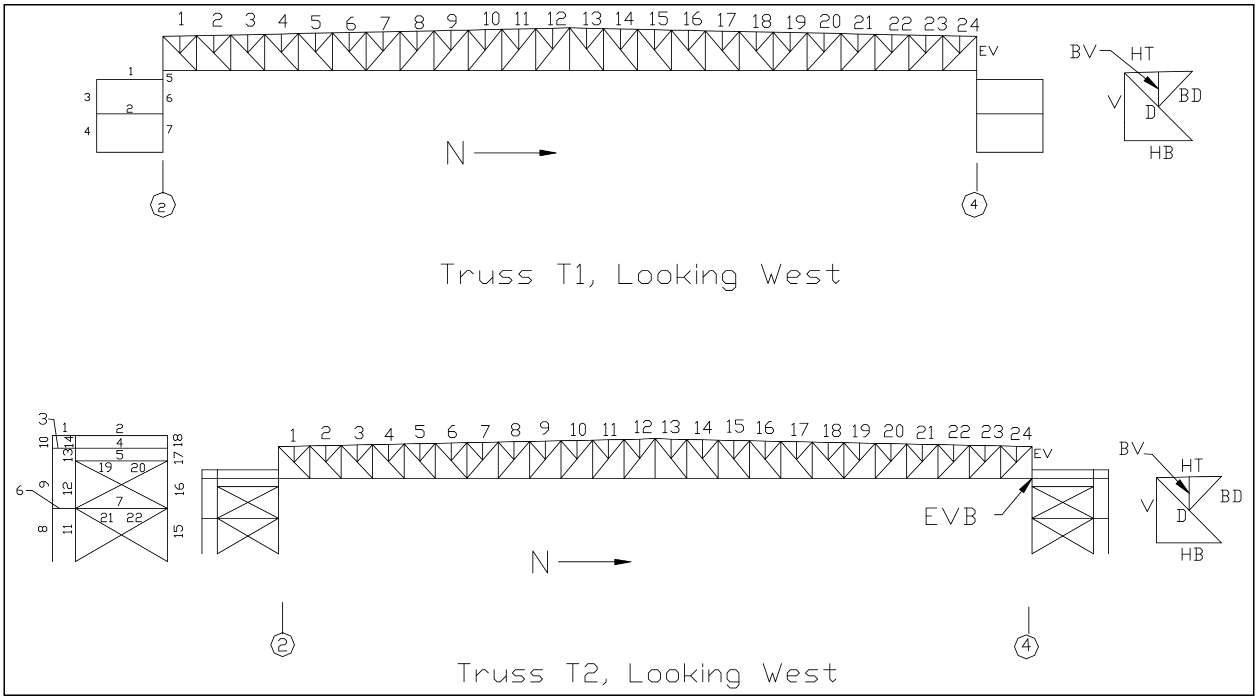

Figure 2-1. Truss T1 and T2 identification numbers.

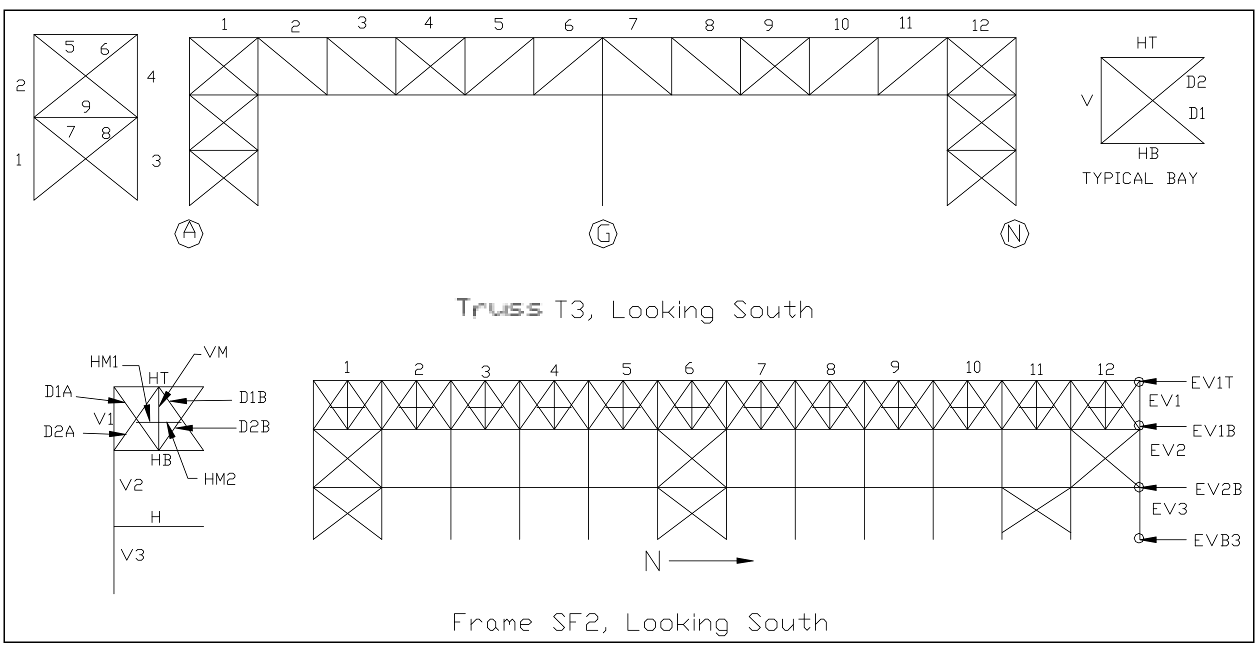

Figure 2-2. Truss T3 and Frame SF2 member identification numbers.

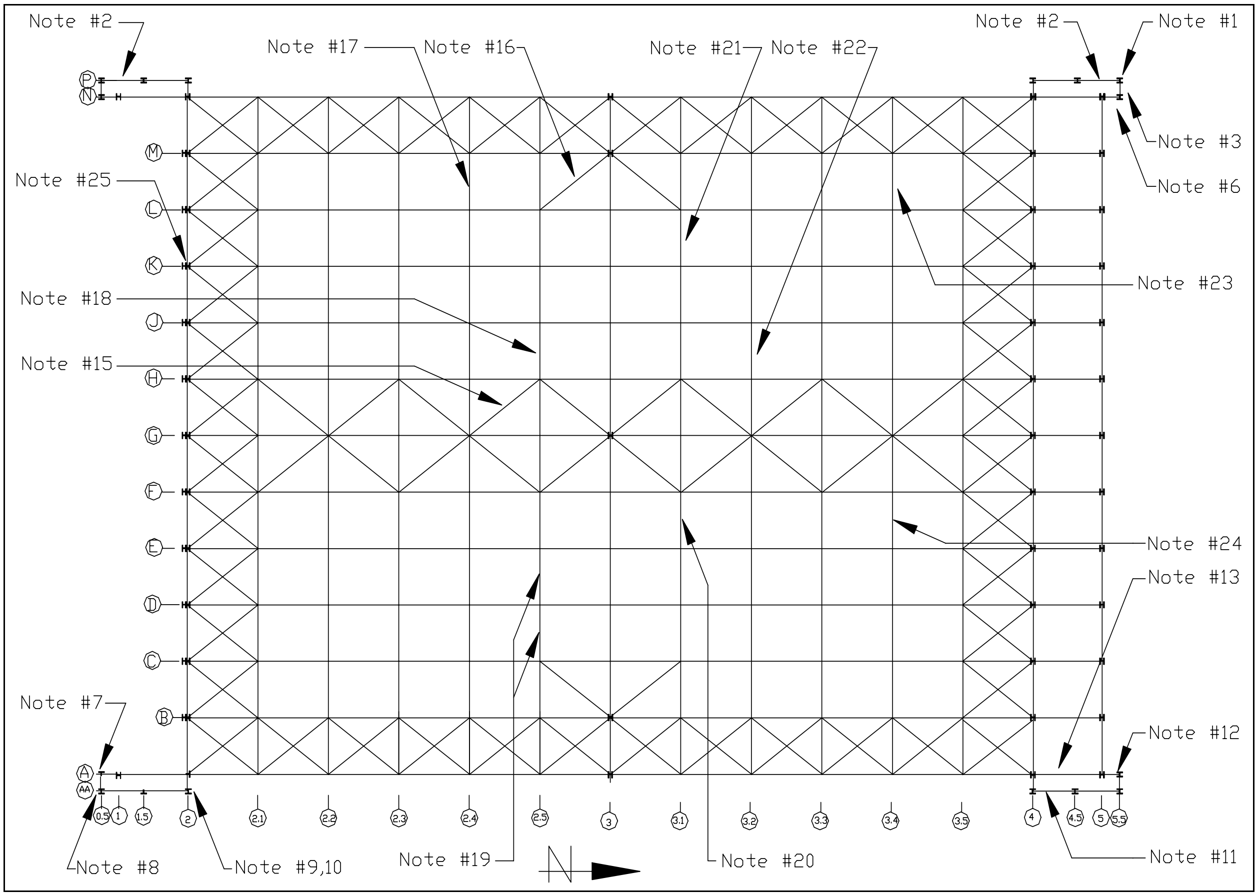

Figure 2-3. Locations of deficient structural members.

3 Loading

Dead Load

Roofing:	Decking (1 5/8 in. Yellow Pine)	44 pcf (AISC ASD 89)	6 psf
	BUR (3-ply composition)		3 psf
	Total:		9 psf

T1 Tributary area 20 ft x 6.67 ft = 133 sq ft
9 psf x 267 sq ft = 1.2 kips at each joint along the top chord

T2 Tributary area 10 ft x 6.67 ft = 67 sq ft
9 psf x 133 sq ft = 0.6 kips at each joint along the top chord

T3 Tributary area 20 ft x 160 ft = 3200 sq ft
9 psf x 3200 sq ft = 28.8 kips at each joint along the top chord

SF Tributary area 20 ft x 80 ft = 1600 sq ft
9 psf x 1600 sq ft = 14.4 kips at each joint along the top chord

Crane Load

Crane Loads per Joint:	Dead	Live
Bridge	50 ppf x 13.3 ft	-
Legs	50 ppf x 10 ft	-
Header	50 ppf x 20 ft	-
End Trucks	-	0.75 kip
Hangar Assembly	-	0.05 kip
Crane Capacity	-	5 kip
Total	2.165 kip/joint	5.8 kip/joint

Hangar 46 currently has one crane, and construction of a new crane has been proposed. It is assumed that the proposed crane would be constructed with similar materials and in a similar fashion to the current crane. Calculated dead loads act on every joint in the structure; live loads act only on the two joints above the crane during use. To ensure that the worst-case scenario was covered in the analysis, different crane loads were used with different wind loads.

Attached as Appendix C is a memorandum pertaining to the crane installation in Hangar 46, including a diagram of the current and proposed crane locations and a photograph of the current system in Hangar 46. It was submitted by the project team's structural engineer as an initial opinion on the safety of expanding the crane system. The recommendations were based on the assumption that the existing crane system is safe according to current codes. However, after structural analysis it was determined that Hangar 46 requires some structural rehabilitation because the old crane system puts stresses on trusses T1 and T3 that exceed AISC ASD 89 allowable limits. Therefore, retrofit as proposed in Chapter 8 is necessary.

The self-weight of the truss is included in the analysis by including the weight per foot of each member in the SAP2000 program. The weight of each member is computed by the program and included in analysis. Reactions about the middle support of truss T1 were applied to truss T3 at the connection locations.

Live Load

Roof: 15 psf (from NAVFAC drawing 1353177)

T1: Tributary area 20 ft x 6.67 ft = 133 sq ft
15 psf x 133 sq ft = 2.0 kips at each joint along the top chord

T2: Tributary area 10 ft x 6.67 ft = 67 sq ft
15 psf x 67 sq ft = 1.0 kips at each joint along the top chord

T3: Tributary area 20 ft x 160 ft = 3200 sq ft
15 psf x 3200 sq ft = 48 kips at each joint along the top chord

SF: Tributary area 20 ft x 80 ft = 1600 sq ft
15 psf x 1600 sq ft = 24 kips at each joint along the top chord

Truss: 8 psf (from NAVFAC drawing 1353177)

T1: Tributary area 20 ft x 13.3 ft = 267 sq ft
8 psf x 267 sq ft = 2.13 kips at each joint along the top chord

T2: Tributary area 10 ft x 13.3 ft = 133 sq ft
8 psf x 133 sq ft = 1.07 kips at each joint along the top chord

T3: Tributary area 20 ft x 160 ft = 3200 sq ft
8 psf x 3200 sq ft = 25.6 kips at each joint along the top chord

SF: Tributary area 20 ft x 80 ft = 1600 sq ft
8 psf x 1600 sq ft = 12.8 kips at each joint along the top chord

Roof live loads were considered not to be a factor during the type of storm that creates the design wind forces, so they were not included in the load combinations. No truss live loads were included because wind pressures by their nature create uplift. Because a live load would counteract wind pressure loading, it is therefore conservative to leave live load out of the calculations. The crane load was also assumed not to be a factor during a storm that would create the design wind loads. For crane load analysis, the roof live load and wind load were not taken into account.

Point Load

A point load at each joint was considered to include the effects of objects hanging from the trusses, such as water pipes, radiant heaters, and lighting. A load of 0.5 kips was used at each panel point along the T1 and T2 trusses. This load was determined from information gathered during the inspection. This load represents a slightly conservative maximum.

Wind Load

The following shows a typical wind analysis calculation using ANSI/ASCE 7-98.

Basic Hangar Data

Location: Corpus Christi, Texas
Terrain: Coastal area
Dimensions: 320 ft x 240 ft in plan
Eave height of 45.3 ft
Roof slope of 1.2 degrees (flat)
Ridge height is 48.7 ft

Exposure and Structure Classification

The structure is located in a coastal region (open water).
Use **Exposure Category D.**

The structure function is industrial-military and used as an essential facility. Use Category IV, with an **Importance Factor (I) = 1.15** (Table 1-1).

Basic Wind Speed

Wind speed was selected per Section 6.5.4 of the standard and Figure 6-1.
Basic Wind Speed (V) = 130 mph

Velocity Pressures

The velocity pressures are computed using:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ psf} \qquad \textbf{Eq. 6-13 (ANSI/ASCE 7-98)}$$

K_z = velocity pressure exposure coefficient from Table 6-5, ANSI/ASCE 7-98
K_{zt} = topographic factor (Section 6.5.7) = 1.0
K_d = wind directionality factor from Table 6-6 = 1.0 (not employing ANSI/ASCE 7-98 load combinations)
I = 1.15
V = 130 mph.

Note: since θ (angle of plane of roof from horizontal) **< 10 degrees**, use the eave height for the mean roof height (**h_m = 45.3 ft**)

The values of velocity pressure vary over the height of the structure with respect to K_z and are summarized in Table 3-1.

Table 3-1. Velocity pressures, psf.

Height, ft	K_z	q_z, psf
0-15	1.03	51.3
20	1.08	53.7
30	1.16	57.7
Horizontal Truss Ht. 32	1.17	58.2
40	1.22	60.7
Eave Ht. 45.3	1.24	61.7
Ridge Ht. 48.7	1.26	62.7

Design Wind Pressures

The design wind pressures for the main wind-force-resisting system (MWFRS) are calculated using Equation 6-15 in ANSI/ASCE 7-98:

$$\mathbf{p = q *(GC_p) - q_i*(GC_{pi})}$$

Where:

q	=	q_z for windward walls at height *z* above the ground
q	=	q_h for leeward walls, side walls, and roofs
q_i	=	q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings
G	=	gust effect factor = 0.85 for Exposure D (Section 6.5.8)
C_p	=	external pressure coefficient from Figure 6-3 or Table 6-8
(GC_{pi})	=	internal pressure coefficient from Table 6-7.

External Pressure Coefficient, C_p

The external wall pressure coefficients are found from Figure 6-3, ANSI/ASCE 7-98 (see Table 3-2 below), and are functions of the L/B ratio. C_p for the windward wall and for the side walls are 0.80 and -0.70, respectively, for all L/B ratios. The leeward wall pressure coefficient varies depending on the wind direction. For wind normal to the ridge, L/B = 320/240 = 1.33 and Figure 6-3 gives the leeward wall pressure coefficient as -0.43. For flow parallel to the ridge, L/B = 240/320 = 0.75; hence, the value of C_p is -0.50.

Table 3-2. Wall C_p.

Surface	Wind Direction	L/b	C_p
Windward Wall	All	All	0.80
Leeward Wall	Normal to Ridge Parallel to Ridge	1.33 0.75	-0.43 -0.50
Side Wall	All	All	-0.70

The roof pressure coefficients are also from Figure 6-3 and are functions of the h/L ratio. However, since h/L is less than 0.5 for all wind directions and θ < 10 degrees, the external roof pressure coefficients resolve to the values in Table 3-3.

Table 3-3. Roof C_p.

Surface	Wind Direction	Distance*	C_p
Windward and Leeward Roof	All	0-h	-0.9
		h-2h	-0.5
		> 2h	-0.3
* horizontal distance along roof from windward edge			

Internal Pressure Coefficient, GC_{pi}

The internal pressure coefficients are taken from Table 6-7, ANSI/ASCE 7-98. Since the hangar qualifies as an enclosed building, two cases were considered to determine the critical load requirements. A positive value of GC_{pi} = 0.18 and a negative value of GC_{pi} = -0.18 were applied to all internal surfaces. The structure would experience both positive and negative internal pressure along with wind directions either perpendicular or parallel to the ridge. Therefore, a total of four wind load cases result. Using these inputs for Equation 6-15 in the standard, the four wind loading cases were calculated; the values are summarized in Tables 3-4 and 3-5 and are illustrated in Figures 3-1 through 3-8:

Table 3-4. Wind Load I (wind normal to ridge with negative internal pressure).

Surface	Height (z), ft	P, psf
Windward Wall	0-32	51
	32-45.3	53
Windward and Leeward Roof	0-h*	-36
	h-2h*	-15
	> 2h	-5
Leeward Wall	All	-11
Side Wall	All	-26
* horizontal distance along roof from windward edge		

Table 3-5. Wind Load II (wind normal to ridge with positive internal pressure).

Surface	Height (z), ft	P, psf
Windward Wall	0-32	28
	32-45.3	31
Windward and Leeward Roof	0-h*	-58
	h-2h*	-37
	> 2h	-27
Leeward Wall	All	-34
Side Wall	All	-48
* horizontal distance along roof from windward edge		

When the wind is parallel to the ridge (Wind Load III and IV), the design wind pressures are the same as above (Wind Load I and II) for the windward wall, side wall, and windward and leeward roofs. However, the leeward wall design wind pressures differ, and are shown in Tables 3-6 and 3-7.

Table 3-6. Wind Load III (wind parallel to ridge with negative internal pressure).

Surface	Height (z), ft	P, psf
Leeward Wall	All	-15
- values differing from Wind Load I		

Table 3-7. Wind Load IV (wind parallel to ridge with positive internal pressure).

Surface	Height (z), ft	P, psf
Leeward Wall	All	-37
- values differing from Wind Load II		

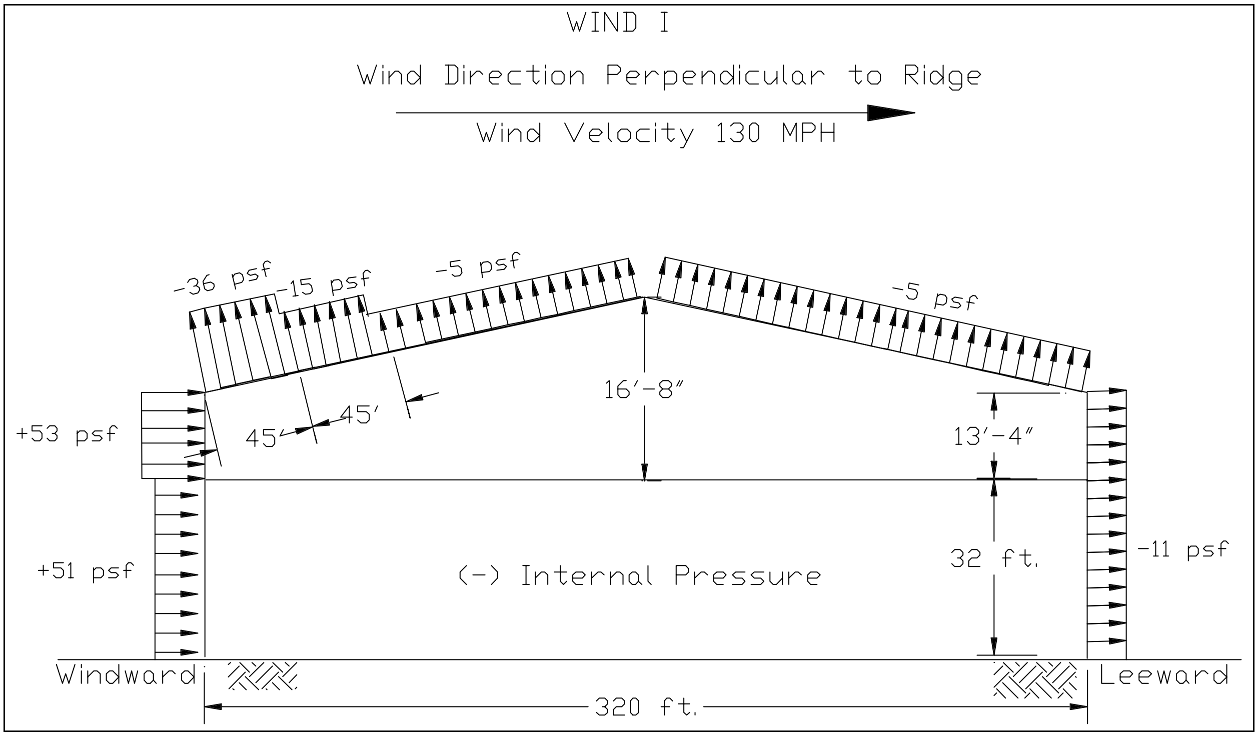

Figure 3-1. Wind Load I pressure distribution looking parallel to ridge.

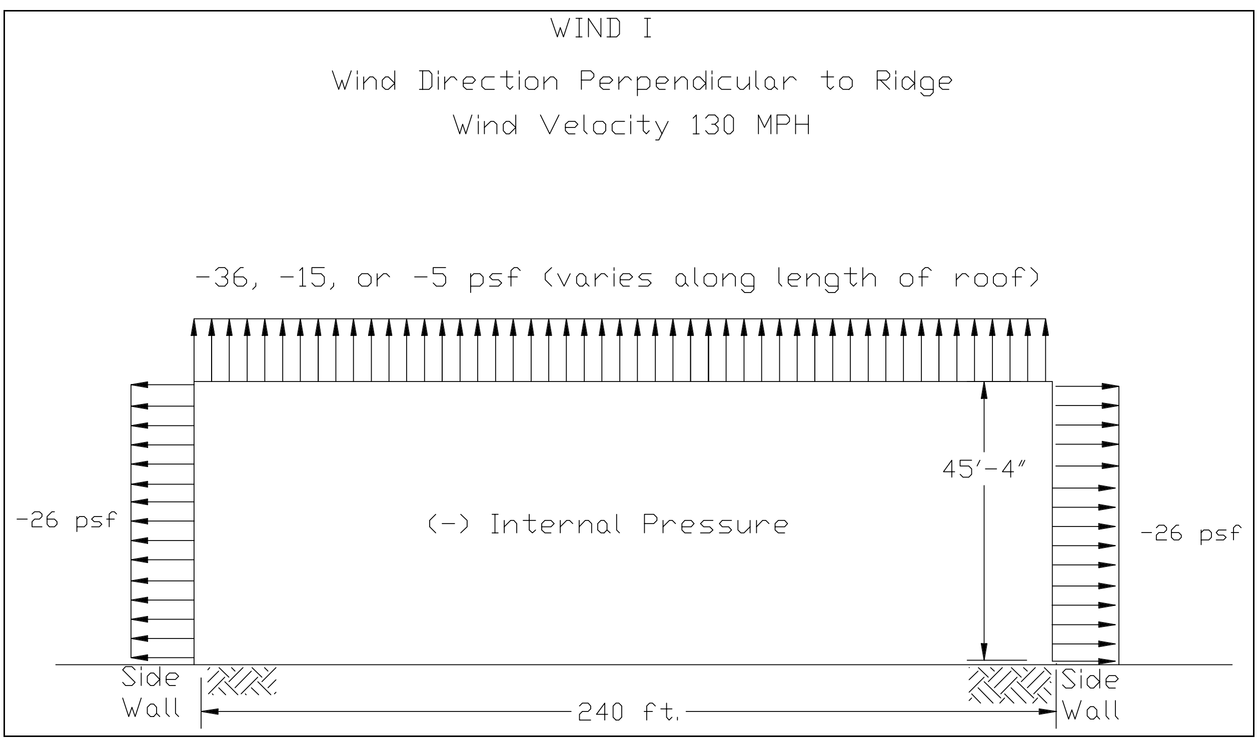

Figure 3-2. Wind Load I pressure distribution looking perpendicular to ridge.

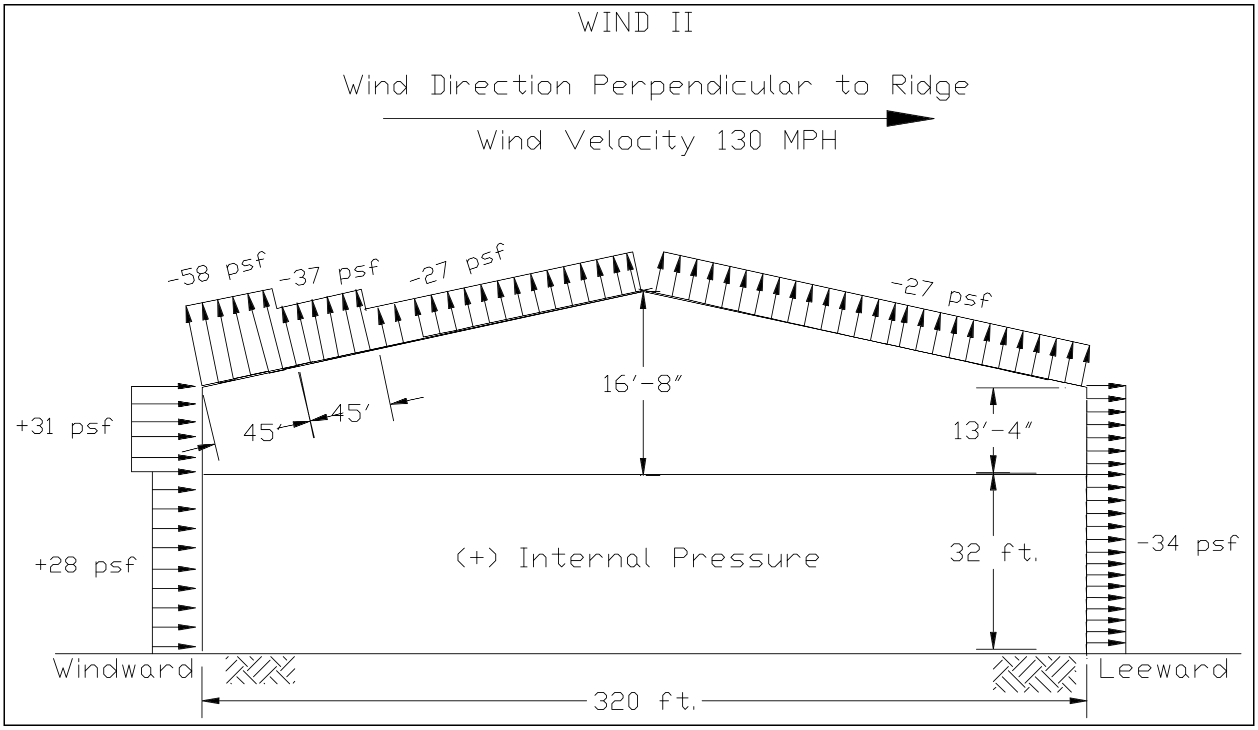

Figure 3-3. Wind Load II pressure distribution looking parallel to ridge.

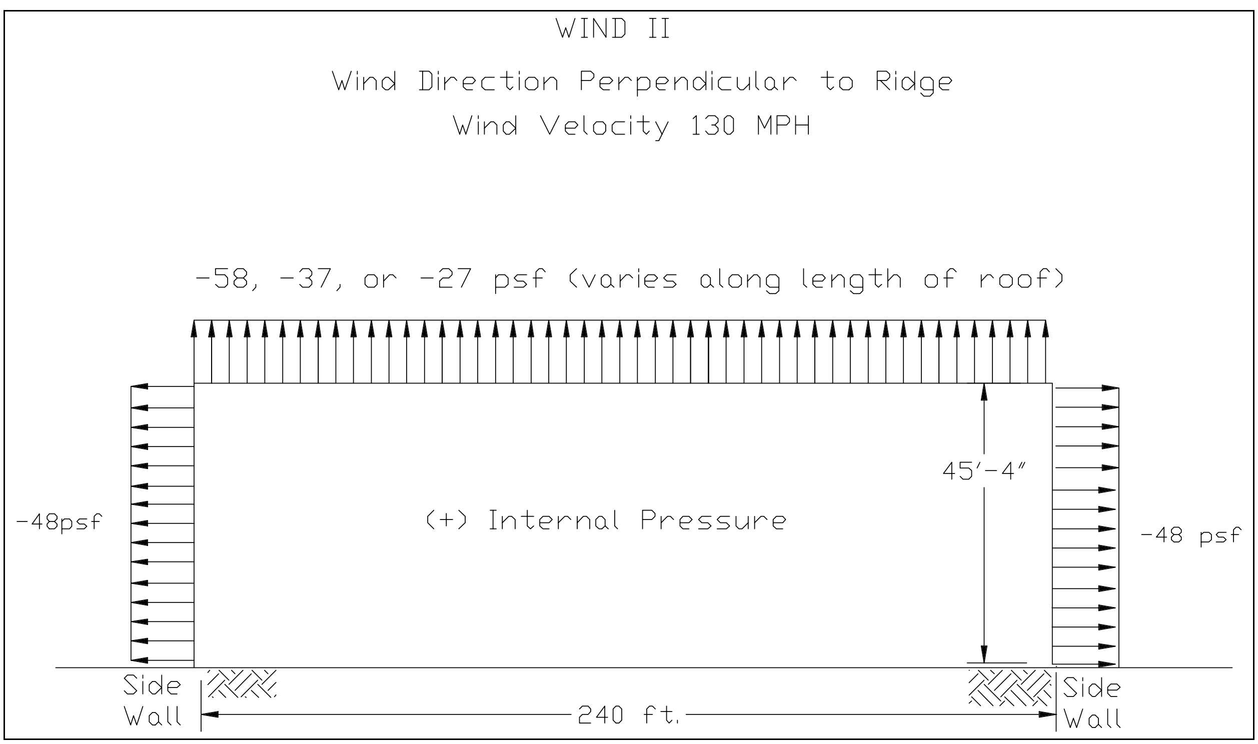

Figure 3-4. Wind Load II pressure distribution looking perpendicular to ridge.

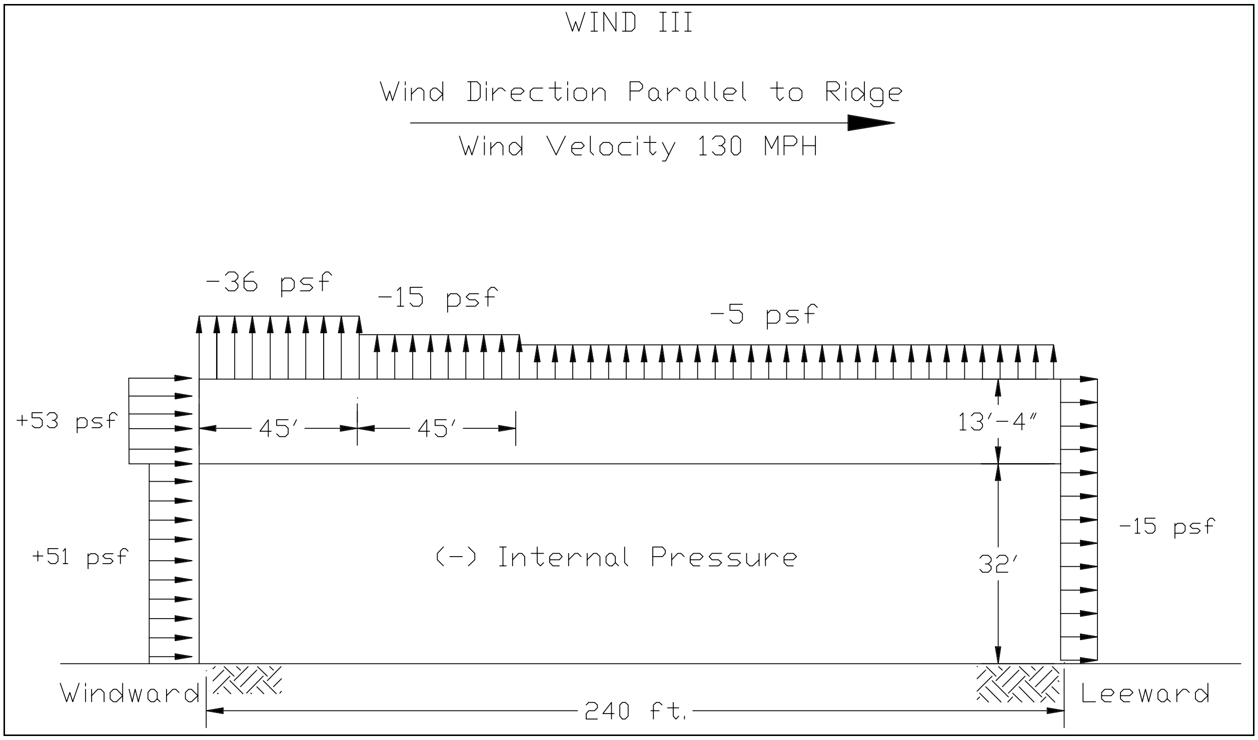

Figure 3-5. Wind Load III pressure distribution looking perpendicular to ridge.

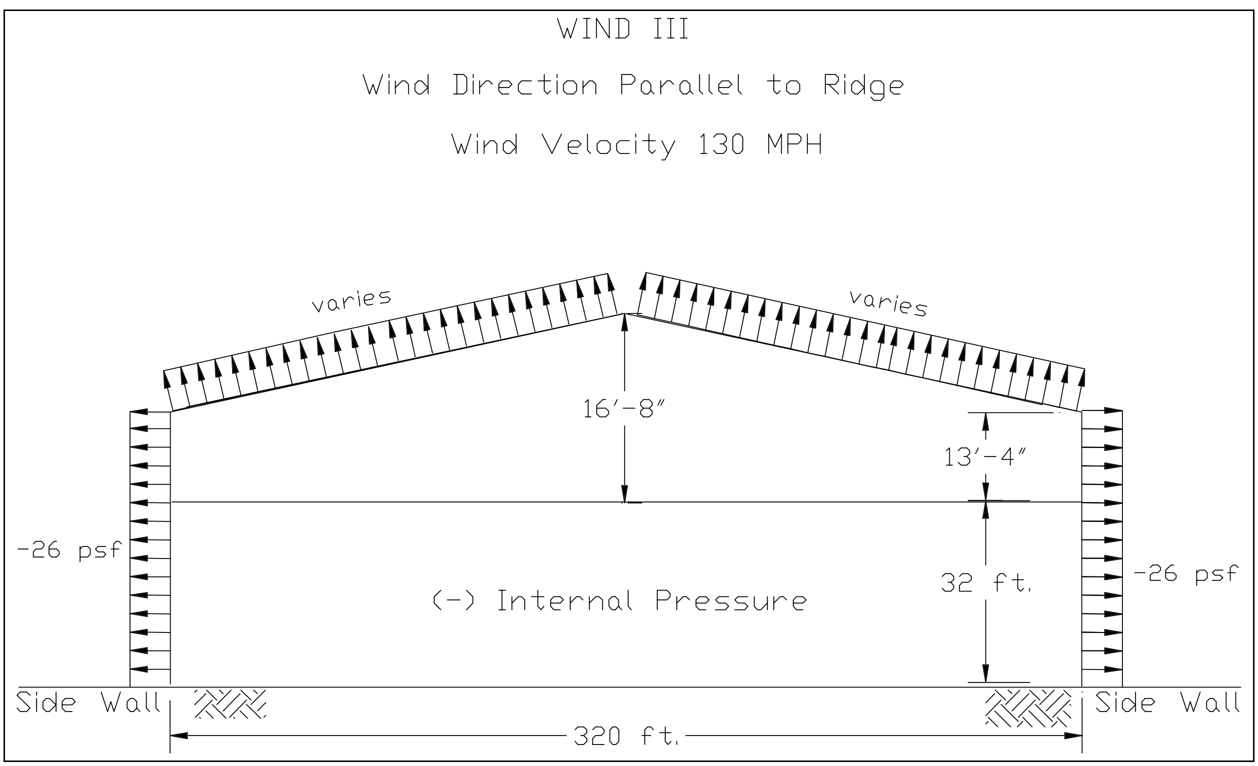

Figure 3-6. Wind Load III pressure distribution looking parallel to ridge.

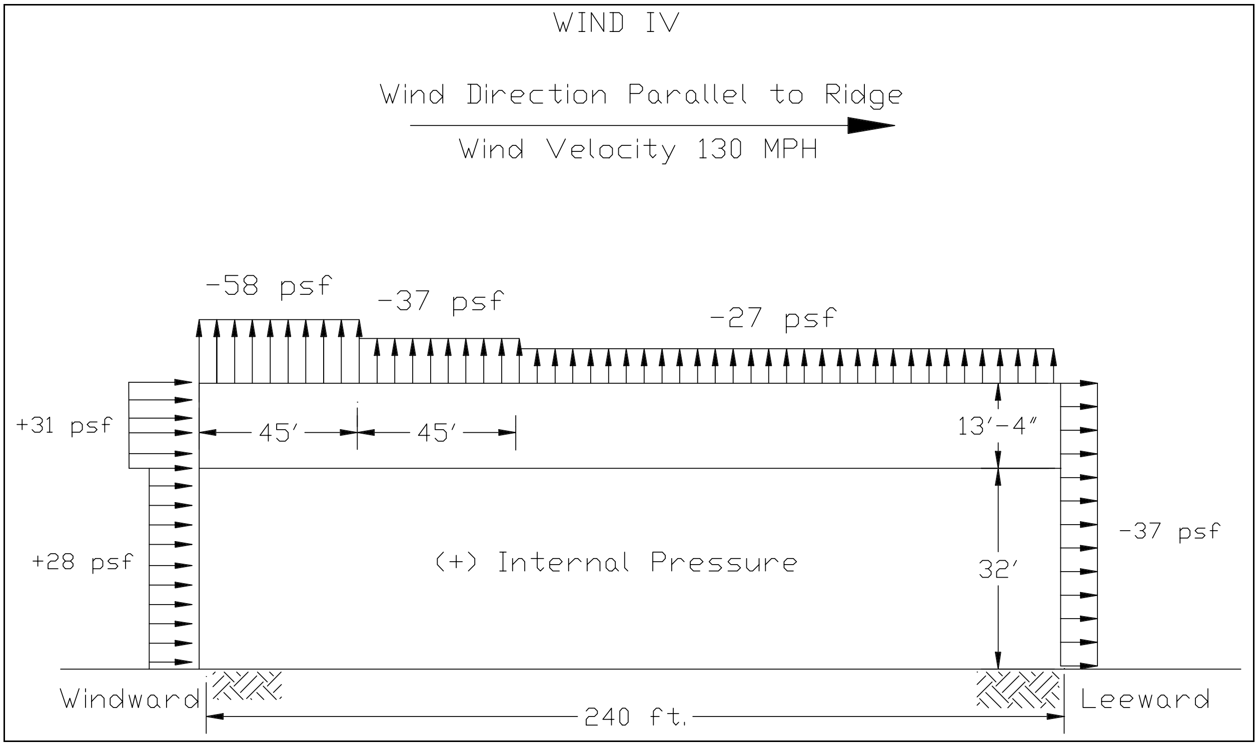

Figure 3-7. Wind Load IV pressure distribution looking perpendicular to ridge.

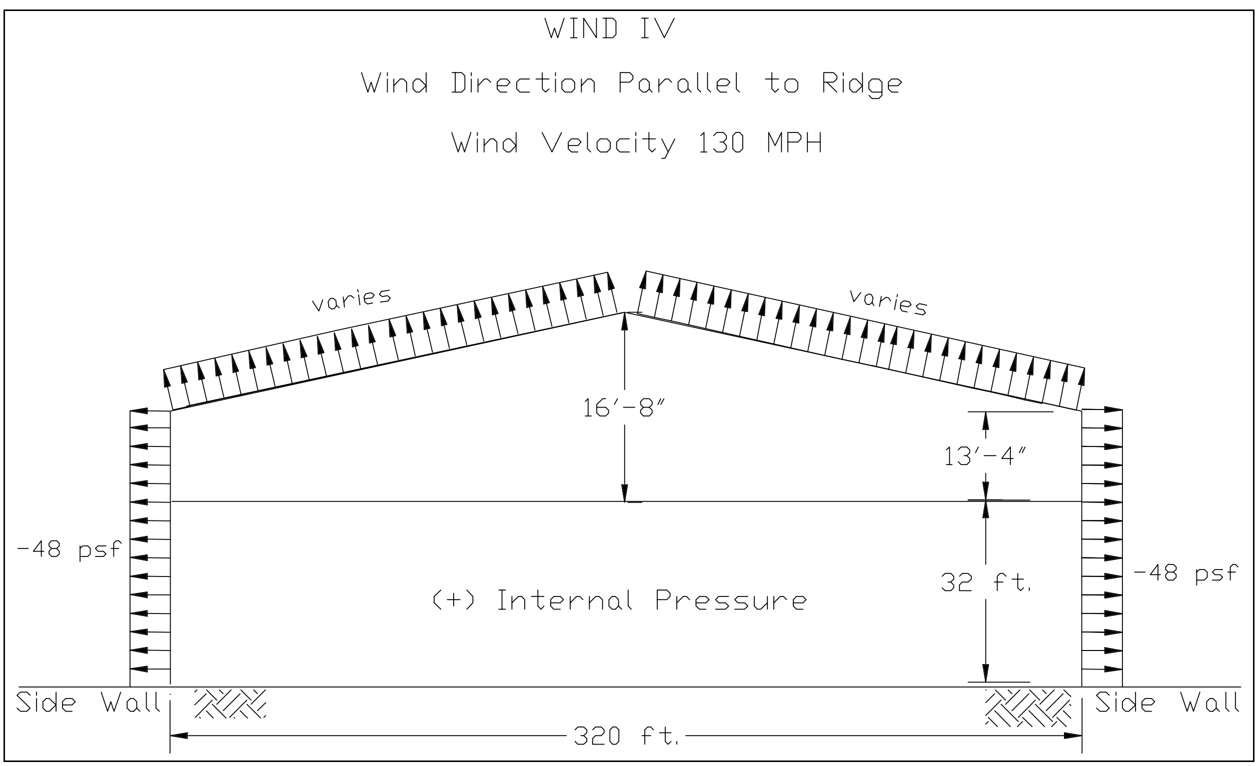

Figure 3-8. Wind Load IV pressure distribution looking parallel to ridge.

4 Modeling

Computer Modeling of Trusses

There are five different typical trusses in Hangar 46. The locations of the trusses in plan for the hangar were taken from NAVFAC drawing 1353171. There are 11 trusses called T1 in the main interior that run north-south. Two type-SF trusses, one at each end, and one type T3 truss run perpendicular to the T1 type trusses. The T1 trusses span from SF at the north and south ends of the hangar area to the center truss called T3. There are two T2 trusses, again spanning from SF to T3, on the exterior edges above the hangar doors. The north end of T1 and T2 trusses are braced with moment-resisting frames. T3 vertically supports all T1 and T2 trusses at their center and provides lateral bracing. The SF trusses support the T1 and T2 trusses at each end, vertically and laterally in the east-west direction. The T1 and T2 trusses are laterally braced at the bottom chord members by horizontal diagonal bracing and channel sections, as seen in NAVFAC drawing 1353171. There are two types of SF trusses, SF-I and SF-II. Both the loading and structure of each type is distinct. SF-I is found at the north end, where there are braced bays with offices and storerooms; SF-II is found at the south end, where there are no braced bays, just an outside wall. Figure 4-1 shows the plan view of Hangar 46.

All columns are anchor-bolted into the foundation at the base and are assumed pinned. All connections are riveted and assumed pinned unless otherwise noted. Each truss was modeled two-dimensionally using the SAP2000 structural analysis program that calculates interaction stress ratios for each member.

The top and bottom chord members of trusses T1 and T2 are continuous between T3 and SF supports, where they are assumed pinned. All other truss members are pinned at both ends except for the main diagonals of each bay. These are continuous through the center joint and pinned at the top and bottom chords. The diagonal bracing members of the end-braced bays of truss T2 are either double or single angle sections, as taken from NAVFAC drawing 1353177. These braces are assumed pinned at both ends and continuous through the intersection. The braces are also assumed to be significant only in tension. Compressive stresses were calculated, but braces overstressed in compression were not considered for retrofit. The horizontal and vertical sections of the end bracing bays are assumed to have moment-resisting connections to one another, as seen in NAVFAC drawing 1353177. The second floor horizontal members in the north end bays of these trusses act in composite with a concrete slab and are therefore continuously braced against out of plane motion.

The joints of the bottom and top chords of all T1 and T2 trusses were restrained from out of plane motion in the 2D analysis. These joints are supported laterally either by the roofing and roof purlins or by the lateral bracing in the plane of the bottom chord. The columns of the end bracing bays are braced laterally along their length by walls.

All connections in the T3 trusses are assumed rigid. Joints along the top and bottom chords were restrained from out of plane motion in the 2D analysis. This is justified by the fact that the top or bottom chord of a T1 or T2 truss supports each joint.

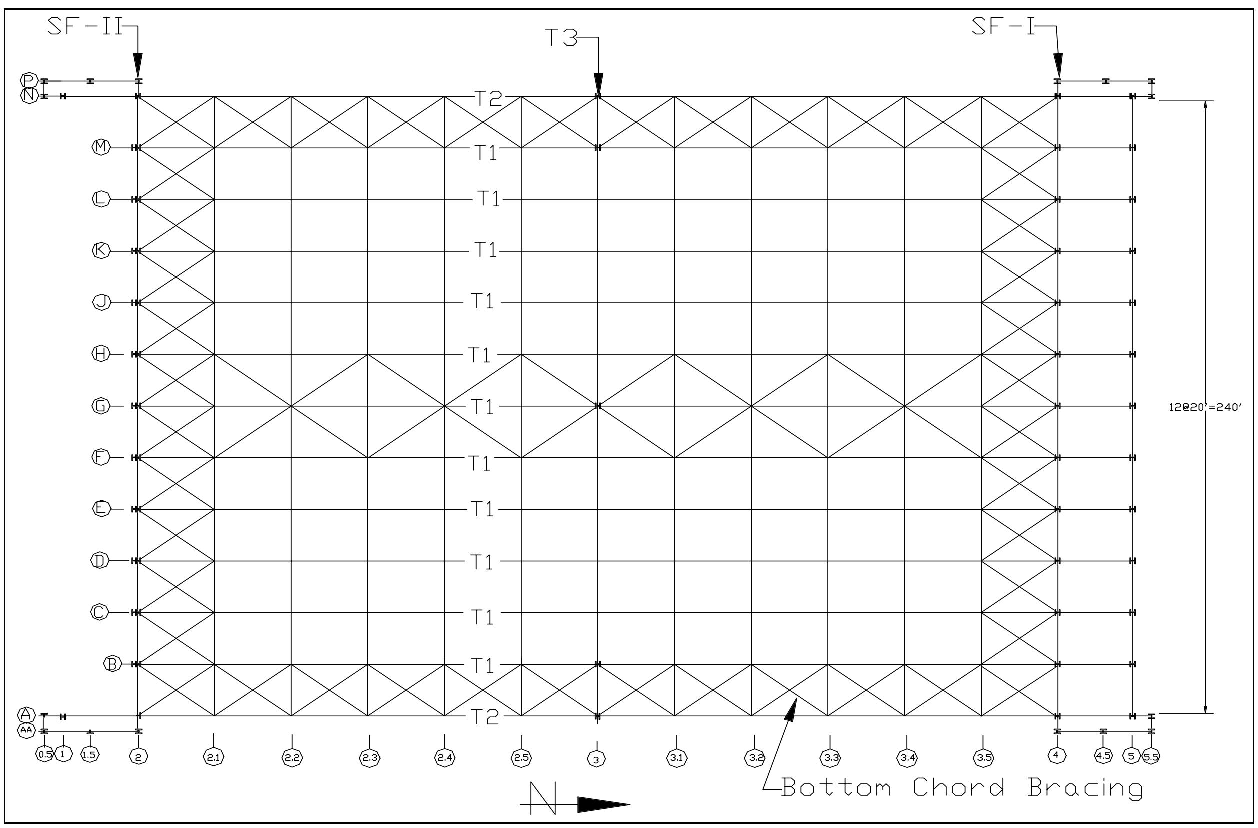

Figure 4-1. Plan view and location of truss types.

Dimensions, location, and member sections for trusses T1 and T2 were taken from NAVFAC drawing 1353177. Dimensions, location, and member sections for truss T3 were taken from NAVFAC drawing 1353174. Member section properties were entered using the AISC database for standard sections. Properties of members with compound multiple sections were calculated by hand and entered into SAP2000 as general, non-compact sections. Members that did not have exact matches in the database were substituted with the closest match from the database. Table 4-1 shows the original member sections and the members that were used for analysis from the AISC database for each truss type. No considerations were made for modifications to the original structure.

Table 4-1. List of substituted members.

Truss Number	Section as in Drawings	Section Used for Analysis*
T1	2L 8x6x7/16	2L 8x6x1/2
T1	18-I-85	W18x86
T3	2 C12x25	TS 12x6x3/8
T3	14-I-87	W14x82
T3	14-I-142	W14x145
T3	33-I-200	W33x201
T3	14-I-43	W14x43
T3	14-I-61	W14x61
T3	2 C12x30	TS 12x6x1/2
T3	14-H-78	W14x74

* Sections are standard AISC rolled shapes.

Material Properties

All steel members were assumed to have a yield stress of 36 ksi and a modulus of elasticity of 29,000 ksi. All rivets were assumed to be ASTM A325 steel.

Member Labels

Line drawings of the horizontal bracing system and the two trusses analyzed are shown in Figures B-1 through B-7. Three drawings include selected member numbers, which are used as labels for Table 5-1 in Chapter 5.

Load Combinations

See CERL Technical Report (TR) 99/27, page 26 (Al-Chaar et al. February 1999).

Analysis of Allowable Stresses

The *ASD Steel Design Manual* (9th edition) was used to calculate interaction stress ratios for each member of the trusses in the SAP2000 post processor. In each analysis all the wind loads were applied as separate loading combinations and SAP2000 calculated the maximum compressive and tensile stress ratios for each member of all combinations used. In addition, the factors of safety were removed from the equations to yield actual member stresses.

The ASD interaction equations for combined axial compression and bending are as follows:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \le 1.0 \qquad (H1-1)$$

$$\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0 \qquad (H1-2)$$

If,

$$f_a / F_a \le 0.15$$

Then instead of (H1-1) or (H1-2), (H1-3) can be used,

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0 \qquad (H1-3)$$

where

F_a = axial compressive stress that would be permitted if the axial force alone existed, ksi

F_b = compressive bending stress that would be permitted if bending moment alone existed, ksi

$$F'_e = \frac{12\pi^2 E}{23\left(K l_b / r_b\right)^2}$$

= Euler stress divided by a factor of safety, ksi. (In the expression for F'_e, l_b is the actual unbraced length in the plane of bending and r_b is the corresponding radius of gyration. K is the effective length factor in the plane of bending, taken as one for beams and braces.)

f_a = computed axial stress, ksi

f_b = computed compressive bending stress at the point under consideration, ksi

C_m = Coefficient representing distribution of moment along member length and is assumed to be 1.0 for all cases except for columns in unbraced frames when they are taken as 0.85

The ASD interaction equation for combined axial tension and bending is:

$$\frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad (H2-1)$$

where

F_t = axial compressive stress that would be permitted if the axial force alone existed, ksi

F_b = tensile bending stress that would be permitted if bending moment alone existed, ksi

f_a = computed axial tensile stress, ksi

f_b = computed tensile bending stress at the point under consideration, ksi

The allowable axial compressive stress is determined as follows. For $Kl / r < C_c$, where Kl / r is the largest effective slenderness ratio:

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right]Fy}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \qquad (E2-1)$$

where

$$C_c = \sqrt{2\pi^2 E / F_y}$$

For $Kl/r > C_c$,

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \qquad (E2-2)$$

the allowable tensile axial stress is:

$$F_t = 0.60F_y \qquad (D1)$$

The allowable bending stresses are calculated as follows. For the out-of-plane unbraced length l, if

$$l < \frac{76b_f}{\sqrt{F_y}} < \frac{20{,}000}{\left(d/A_f\right)F_y} \qquad (F1-2)$$

then for *compact* sections:

$$F_b = 0.66F_y \qquad (F1-1)$$

and for *non-compact* sections:

$$F_b = 0.60F_y \qquad (F1-5)$$

If l exceeds the limits above, then for *compact* and *non-compact* sections:

$$F_b = \frac{12\times10^3 C_b}{ld/A_f} \leq 0.60F_y \qquad (F1-8)$$

where

l = distance between cross sections braced against twist or lateral displacement of the compression flange, in inches

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad (F1.3)$$

where

M_1 = the smaller bending moment at the end of the unbraced segment

M_2 = the larger bending moment at the end of the unbraced segment

The allowable shear stress is calculated as follows:

$$F_v = 0.40F_y \qquad (F4-1)$$

where

$$\frac{f_v}{F_v} \leq 1.0 \qquad (F4)$$

The factor of safety used for axial compression was calculated as follows:

For $Kl/r<C_c$,

$$FOS = \frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c} \qquad (E2-1)$$

For $Kl/r>C_c$,

$$FOS = \frac{23}{12} \qquad (E2-2)$$

The factor of safety used for axial tension is,

$$FOS = 1/0.60 \qquad (D1)$$

The factor of safety used for bending is,

$$FOS = 1/0.66 \qquad (F1-1)$$

The factor of safety used for shear is,

$$FOS = 1/0.40 \qquad (F4-1)$$

5 Structural Analysis of Members

Structural analysis of Hangar 46 was conducted in two stages. The first stage was comparing Hangar 46 to Hangars 44 and 45, which were studied previously and documented in CERL TR-99/27. Wind loadings on all the trusses were either similar or more conservative in TR-99/27 than were the loadings used for Hangar 46. New analysis of the trusses and sway frames under wind loading was not conducted on Hangar 46; Hangar 46 is assumed to behave in a manner similar to Hangars 44 and 45 because all are identical in construction and location. For analysis data and retrofit schemes of the trusses due to wind loadings, see TR-99/27.

The second stage was running computer analyses on structural members that were not covered in the previous report. First, the T1 truss was analyzed using SAP2000 with newer loads resulting from proposed crane construction. Second, the effect of the crane loading was tested on truss T3. Third, the horizontal lateral bracing system was checked under worst-case wind loading, namely Wind Load III. Members structurally deficient in any loading are presented in Table 5-1 with corresponding stress ratios. Listed stress ratios reflect maximum ratios to that particular loading. Members not listed are structurally sound according to ASD 89. See Chapter 4 for calculation of allowable stresses and the factors of safety. Listed double angles are designed with long sides back-to-back, unless noted otherwise. Member numbers correspond to element numbering system shown in Appendix B. The member names correspond to those used for the inspection, as included in Chapter 2, but equal only half of the total members when the truss is symmetrical; all values for one member are applicable to its counterpart across the centerline.

Analysis of Horizontal Bracing System Under Wind Load III

The horizontal bracing system in Hangar 46 is used as a lateral brace for the bottom chords of trusses T1 and T2. It is made of mostly slender single angles that are not effective in compression. A SAP2000 analysis was conducted on the bottom plane when subjected to Wind Load III; single angle bracing members acting in compression were removed from analysis and the rest of the members were checked for structural efficiency.

Results of the test are shown in Table 5-1. Values of allowable and demand stresses are given for members whose demand exceeds allowable stress. Members that are structurally sound are not listed since they do not need retrofit consideration. In the bracing system, 24 members had demands exceeding capacities when the factor of safety was considered. Upon removal of the factor of safety the number of failing members dropped to 12; any member that did not have a generous factor of safety was considered for retrofit. In all, 12 members were proposed for retrofit.

Analysis of T1 Under Crane Loading

When the proposed crane loading of T1 was investigated, each analysis showed four members that would fail. Allowable stresses and demands for the failing members are shown in Table 5-1. All four members were proposed for retrofit.

Analysis of T3 Under Crane Loading

Truss T3 supports truss T1 at its centerline, so crane loadings on truss T1 induce an extra load on truss T3. SAP2000 showed 15 members failing, including the center column. Instead of retrofitting all the failed members, an entirely different retrofit scheme was developed which includes two new columns and eight retrofitted members. Stress ratios for the failed members are given in Table 5-1.

Table 5-1. Maximum stress ratios of original and retrofitted members.

(Note: ASD factors of safety are included in calculations.)

Horizontal Bracing Frame

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
L1	L4x3x1/4	2139	398			
L2	L3.5x2.5x1/4	1	240			
L3	2L5x3.5x3/8	2209	398			
S2	2 C7x9.8	2146	240	1.02	1.02	0.00
T1	2L6x6x3/8	2258	319	1.66	1.64	0.02
T2	2L6x6x1/2	2135	319	1.16	1.05	0.11
T3	W14x74	2529	240			
SF2A	2L5x3.5x3/8	2145	240	2.18	2.18	0.00

Horizontal Bracing Frame - Retrofitted

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
T1_retro	+2L6x6x3/8	2258, 2462, 2239, 2440	319	0.68	0.66	0.02
SF2A_retro	+2L5x3.5x1/2	2145, 2195,2399,2349,2156,2206,2410,2360	240	0.82	0.82	0.00

Truss T1 (Crane Loading)

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
2-12 BD	2L3x2x1/4	310	113			
1-12 BV	2L3x2x1/4	299	80			
4,5,8 D	2L3x2x5/16	266	226			
1 BD	2L3x2.5x1/4	311	113			
3,9 D	2L3.5x2.5x5/16	269	226			
6,7,10 D	2L4x3x5/16	260	226			
2,11 D	2L5x3x5/16	272	226			
1,12 D	2L5x3.5x5/16	275	226			
1-9 V	2L6x3.5x5/16	273	160			
10-12 V	2L6x4x3/8	243	160			
1-4,9-12 HB	2L6x6x3/8	276	320	3.29	3.25	0.04
5-8 HB	2L6x6x1/2	279	320			
1-12 HT	2L8x6x1/2	217	160			

Truss T1 Retrofitted (Crane Loading)

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
11HB Retro	+2L 6x6x3/8	286	320	1.08	0.91	0.2
12HB Retro	+4L 6x6x3/8	287	320	0.88	0.72	0.2

Truss T3 (Crane Loading)

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
1, D1, D2	2L 7x4x5/8	33	311			
1, HB	2-C12x25	16	240			
Col A3	w14x82	1	192			
Col B3	w14x145	17	192			
Col G3	w33x201	37	384	1.58	1.58	0.0
2D1, 6D2	w14x74	20	311	1.64	1.44	0.2
3D1, 5D2	w14x43	22	311	1.49	1.40	0.1
4V, 5V	w14x43	23	192	1.63	1.25	0.4
3V, 6V	w14x61	21	192			
2-6HT	w14x82	5	240	1.28	1.02	0.3
1HT	2-C12x30	4	240			
2-6HB	w14x74	15	240	1.98	1.24	0.7
4D1,4D2	2L5x3x5/16	26	311	3.66	3.59	0.1

Truss T3 Retrofitted (Crane Loading)

Name	Shape	#	Length (in.)	Stress Ratio	AXL	B33
5D2 Retro	+2PL 1/2"	29	311	0.71	0.59	0.12
5V Retro	+PL 1/2"	25	192	0.82	0.74	0.08
4D1,2 Retro	+2C12x30+2PL	26	311	0.76	0.59	0.17

6 Structural Analysis of Connections

[Editor's note: Connections were not analyzed specifically for Hangar 46 because they are identical to those found in Hangars 44 and 45. The analyses of Hangars 44 and 45 are published in CERL TR-99/27; the connection details below pertain to Hangar 45, and are excerpted from pages 36 and 37 of TR-99/27.]

Thirty-eight connection types were investigated, several from each truss type and some from the horizontal bracing of the bottom chords of the trusses. The locations of the connection types can be found in Figures 6.1 through 6.5 [of TR-99/27]. No detailed drawings of the connections could be located. All connection data was taken during inspection from Hangar 45. Some of the connections were actually measured while most were drawn from the ground using comparisons with measured connections to determine spacing and sizes of the rivets. Photographs of the connections were also used to aid in the detailing of the connections. Drawings of each connection investigated can be found in Figures 6.6 through 6.15 [of TR-99/27].

Rivet heads were measured because no data could be found about the rivet sizes in the connections. To determine the rivet shaft diameter, the following equation (from AISC ASD 9th edition) was used:

$$HD=1.5D + 1/8$$

where HD is the diameter of the driven rivet head in inches. No detailed analysis was done for the connections due to the lack of reliable information.

The forces used to evaluate the connections in the four truss types came from the structural analysis of each truss type in Hangar 45. Each connection type represents a group of connections on each truss. The forces used for evaluation represent the maximum forces for all loading conditions at that location, as marked in Figures 6.1 through 6.5 [of TR-99/27]. These forces, however, are not the maximum forces on any connection in the connection type. Table 6.1 [see TR-99/27] summarizes the connection types, the members framing into the connection, the tension forces on each member, and the member's maximum stress interaction ratio from the analysis of the trusses.

Table 6.1 [see TR-99/27] also includes the ratio of computed tension forces to the total shear capacity of the rivets. The shaft diameter of each rivet was calculated by taking the head diameter of the rivet and applying the equation shown above. The shear capacity of each rivet was taken from Table 1-A of AISC ASD 9^{th} edition according to its estimated size. The rivets were investigated for both A307 and A325 steels. It is most likely that the rivets are A325. The shear capacity of each rivet was then multiplied by the number of bolts connecting each member to get the total shear capacity of the rivets. For A325 bolts, no connections exceeded a ratio of 1.0. The highest ratio calculated was 0.771 for member #7741 of connection type 34. For A307 bolts, five members exceeded a capacity ratio of 1.0. However, the high probability that the rivets are A325 steel, taken together with the fact that all the member interaction stress ratios are low, leads to the judgment that the connections are strong enough to resist the analysis loads used in this investigation.

7 Repair of Deficient Members Observed During Inspection

Based on observations made during the inspection of Hangar 46, this chapter outlines immediate actions needed to restore damaged structural members to their original condition. Proposed repair methods for all deficient members are presented here. These repair methods conform to general retrofit notes and guidelines originally published in Figure D-1 of Appendix D in CERL TR-99/27, previously cited. Appendix D of that report includes technical illustrations that also pertain to the current inspection; drawings from that report are referenced below as necessary.

Notes 1 through 13 are for the door pockets:

1. Replace the bent tie rod (7/8 in. diameter) attached to column P5.5 with identical or larger member that has the same connection details with same configuration of bolts and welds.
2. Tighten or replace loose tie rods (7/8 in. diameter) attached to columns P0.5, P1.5, P4.5, P5.5.
3. The buckling horizontal brace (10-I-21) on the second floor, between columns P5.5, N5.5, is not designed for compression, but has been acting as a compression member. This brace is considered to be a secondary member. It would be possible to increase stiffness and strength through retrofit, but this would make connections more vulnerable to failure. Therefore, no retrofit is advised.
4. Deteriorated hangar door wheels should be replaced to ensure the strength of the door system under wind loading.
5. Dented vertical members near handles on hangar doors can result in loss of structural integrity of doors. Do not open doors with chains; use original design method for opening doors.
6. Remove spalling concrete from column N5.5 (8-H-31), sandblast rust off of the column structural steel, repair steel as shown in Figure D-4 of TR-99/27, and re-cast the concrete.
7. Remove spalling concrete from column A0.5 (8-H-31), sandblast rust off of the column structural steel, repair steel as shown in Figure D-5 of TR-99/27, and re-cast the concrete.

8. If the base and anchorage of column AA0.5 (12-I-25) are sufficient, repair as shown in Figure D-4 of TR-99/27; if not, repair as shown in Figure D-5 of TR-99/27.
9. Remove unsupported wood column (4x4 post) at column AA2.
10. Remove makeshift wood column (4x4 post) near column AA2 or securely brace the column to the beams.
11. Retrofitting beam 6-H-15.5 on the second floor, to stiffen and strengthen it between columns AA4 and AA4.5, would make connections vulnerable to failure. Therefore, no retrofit is advised.
12. Repair column A5 (18-I-47) as shown in Figure D-5 of TR-99/27.
13. Repair column A5.5 (8-H-31) as shown in Figure D-5 of TR-99/27.

Notes 14 through 16 are for deficiencies in the main hangar area.

14. Any unpainted steel needs to be painted; use as listed in Figure D-1 of TR-99/27.
15. No retrofits are necessary for buckling bracing members listed in notes 15 and 16 of Chapter 2; these angles are intended for tension only.
16. No retrofit is necessary for column K2 (18-I-85), but further welding damage and gouging of the sort identified here should be avoided.

8 Retrofit Schemes for Members Failed in Analysis

This chapter discusses retrofit for the entire model with respect to all loadings. As noted in Chapter 5, analytical data for wind loading of the trusses are given in CERL TR-99/27 and are not reprinted here. Please consult that technical report for specific details.

Retrofit of Truss T1

Knee braces, double angles 8x8x1/2, will be added to the truss. They are placed under the first and last bays of the truss and extend from the first panel point of the truss' bottom chord at about a 30 degree angle to connect to the column 95 in. below the level of the bottom chord. Details for the knee braces are shown in Figure 8.3 in CERL TR-99/27. Figures E1 and E2 in TR-99/27 show the element numbering system and location of knee braces; Figures E6 and E7 in TR-99/27 show the joint numbering system.

Members 11HB and 14HB will have two 6x6x3/8 angles added onto them. Connection details and drawings are shown in Figure K of Sheet 4B, IJO No. 5-5712. That drawing shows 4x4x3/8 angles; use 6x6x3/8 angles with the same connection details as shown. Members 12HB and 13HB will have four 6x6x3/8 angles added. Connection details and drawings are shown in Figure L of Sheet 4B, IJO No. 5-5712. These retrofits are needed for the proposed 5 ton crane system. A heavier crane capacity would require more retrofit.

Retrofit of Truss T2

Knee braces similar to those used in the truss T1 retrofit are added to the truss models. Four knee braces are used, two at the outer ends of the truss and two in the middle bays. The outer two are placed under the first and last bays from the panel points of the bottom chord down on a 30 degree angle to the outer columns 95 in. below the height of the bottom chord. The middle two braces are placed under bays 12 and 13 from the panel points of the bottom chord to the center column, connecting at 95 in. below the height of the bottom chord. Details for the knee braces are shown in Figure F-5 in TR-99/27. Figures E3 and E4 in TR-99/27 show the element numbering system and location of knee braces. Figures E8 and E9 in TR-99/27 show the joint numbering system.

Retrofit of Truss T3

First, W33x201 columns were added to the truss at locations E3 and J3. Details for the columns are shown in Drawing D 3 of 5. This drawing shows a W 24x146 column; substitute in its place a W 33x201 column. All other connections remain as shown. These columns alleviate much of the load and reduce the number of retrofits necessary on the truss. However, some members remain overstressed. Members 5D2, 8D1, 5V, and 9V are retrofitted with two 0.5 in. plates welded to the flanges. Details are shown in Drawing H of Sheet 4A, IJO No. 5-5712. The diagonals 4D1, 4D2, 9D1, and 9D2 are to be enclosed by a makeshift tube made of two C12x30 channels and plates welded together. Details are shown in Drawing E3 of Sheet 4A, IJO No. 5-5712.

Retrofit of Horizontal Bracing Frame

Of the 12 members requiring retrofit, four are bottom chords of the T1 truss. These members have already been discussed earlier in this chapter. The remaining eight members are SF2A members, double angles 5x3.5x3/8, labeled as numbers 2145, 2195, 2399, 2349, 2156, 2206, 2410, and 2360. They are to be retrofitted by adding double angles 5x3.5x1/2 to the underside, with welding details as shown in Drawing L of Sheet 4B, IJO No. 5-5712. Single angle bracing, although showing signs of slight buckling, does not need retrofit. These members are intended for tension only and were not integral to the compressive stiffness of the bracing system.

9 Conclusions

Structural analyses of Hangar 46 were performed for the most dominant loading combinations. A significant number of members did not meet the allowable design stresses per AISC ASD 89 code. However, many of these members had demand-to-capacity ratios just slightly greater than 1:1. When the ASD factors of safety are changed to 1 these members show capacities much greater than demand. Only members that did not have a reasonable margin of safety were considered for retrofit.

Retrofit of the hangar was given in four parts: Truss T1, Truss T2, Truss T3, and the horizontal bracing frame. Assumptions made during analysis are as follows. During a high-wind storm the crane will not be used and the hangar doors will be closed and secured. In the event of live load on the roof (i.e., snow) the crane will not be used. Finally, crane capacity will not exceed 5 tons.

This report revealed that buckling of single-angle bracing members in the plane of the lower chords of the trusses is not a structural problem and with slight retrofits the bracing is adequate. The trusses, with the proposed retrofit, are just adequate to sustain a 5 ton crane as proposed. Larger cranes or cranes of unusual construction should not be installed in Hangar 46 without further analysis or by means of a 'built-up' frame.

Visual inspection of the hangar revealed structural deficiencies that need to be fixed immediately. Retrofit of these deficiencies were considered and proposed. Items of importance are attention to the columns in the door pockets and the replacement of hangar door wheels.

In addition to its specific applicability to Hangar 46 at Corpus Christi Army Depot, this report is also offered as a case study on the structural analysis of existing hangars vulnerable to wind loads.

References

AISC *ASD Steel Design Manual*, 9th Edition.

Al-Chaar, Ghassan K., *Case Study: Structural Evaluation of Steel Truss Aircraft Hangars at Corpus Christi Army Depot, Texas*, Technical Report 99/27/ADA361006 (U.S. Army Construction Engineering Research Laboratory [CERL], February 1999).

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-98.

Drawing IJO No. 5–5712, Bridge Crane System Hangar 47 South, CCAD, Foundation Plan, Truss Elevations, Column Footing Details, Spread Footing Details and Section Details, Total 3 Sheets, 12 – 11 – 96.

NAVFAC Drawing Number 1353171, Paint Structural and Miscellaneous Steel in Hangars, Bottom Chord Framing and Location Map, Buildings 43, 44, 45 and 47, 5 – 18 – 72.

NAVFAC Drawing Number 1353174, Paint Structural and Miscellaneous Steel in Hangars, Framing Elevations, Door Details and Color Key, Buildings 43, 44, 45 and 47, 5 – 18 – 71.

NAVFAC Drawing Number 1353177, Paint Structural and Miscellaneous Steel in Hangars, Truss Elevations and Details, Buildings 43 and 47, 5 – 18 – 71.

SAP2000 Non-linear, CSI.

Acronyms

AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
CCAD	Corpus Christi Army Depot
CERL	Construction Engineering Research Laboratory
ERDC	Engineer Research and Development Center
MWFRS	main wind-force-resisting system
NAVFAC	Naval Facilities Engineering Command
SAP	Structural Analysis Program

Appendix A: Photographs of Structural Elements From Inspections

Figure A-1. Buckled tie rod (7/8 in. diameter) near Column P5.5.

Figure A-2. Buckled horizontal brace (10-I-21), second floor between Columns P5.5 and N5.5.

Figure A-3. A typical deteriorated hangar door wheel.

Figure A-4. Spalling concrete at the base of Column N5.5.

Figure A-5. Major corrosion sample retrieved from base of Column AO.5 (8-H-31).

Figure A-6. Loose timber near Column AA2.

Figure A-7. Makeshift support near Column AA2.

Figure A-8. Major corrosion at base of Column A5 (18-I-47).

Figure A-9. Minor corrosion at the base of Column A5.5 (8-H-31).

Figure A-10. Typical example of elements needing fresh paint.

Figure A-11. Buckled diagonal between Columns G2.4 and H2.5.

Figure A-12. Buckled diagonal between Columns L2.5 and M3.

Figure A-13. Buckled bottom chord between Columns L2.4 and M2.4.

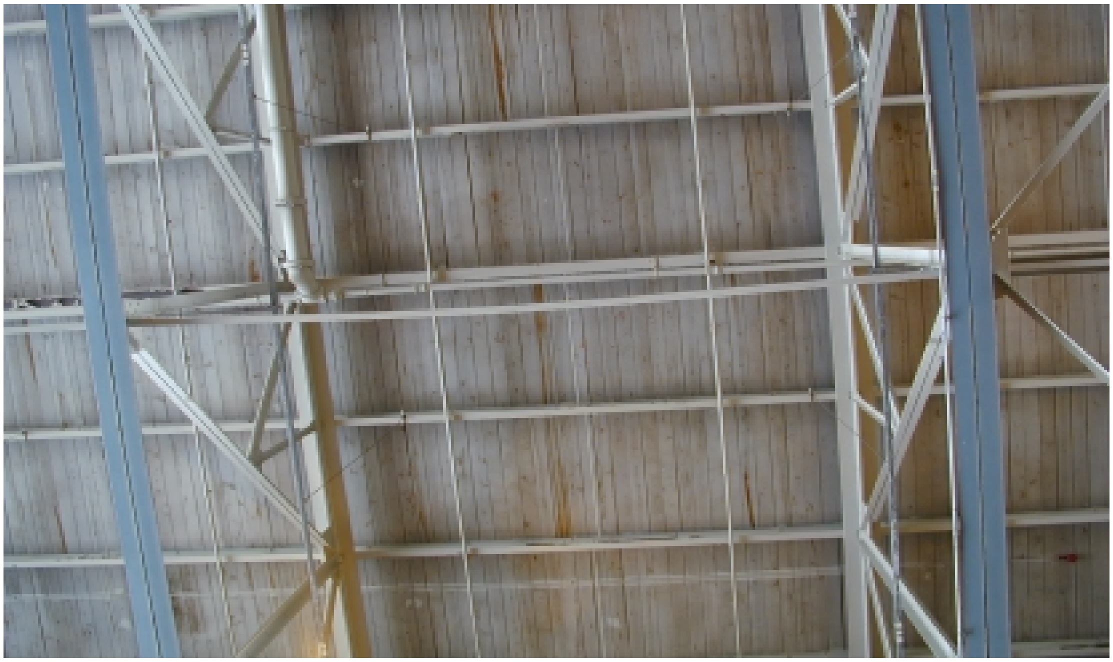

Figure A-14. Buckled bottom chord between Columns H2.5 and J2.5.

Figure A-15. Buckled bottom chord between Columns C2.5 and E2.5.

Figure A-16. Buckled bottom chord between Columns E3.1 and F3.1.

Figure A-17. Buckled bottom chord between Columns K3.1 and L3.1.

Figure A-18. Buckled bottom chord between Columns H3.2 and J3.2.

Figure A-19. Buckled bottom chord between Columns L3.4 and M3.4.

Figure A-20. Buckled bottom chord between Columns E3.4 and F3.4.

Figure A-21. Damage to Column K2 (18-I-85).

Appendix B: Truss Diagrams and Element Numbering

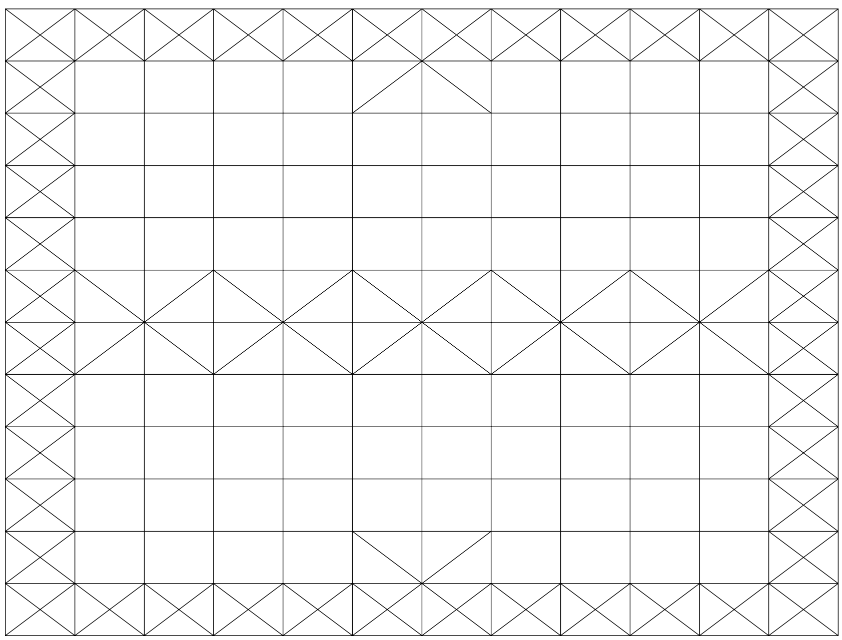

Figure B-1. Plan view of horizontal bracing system with all members shown.

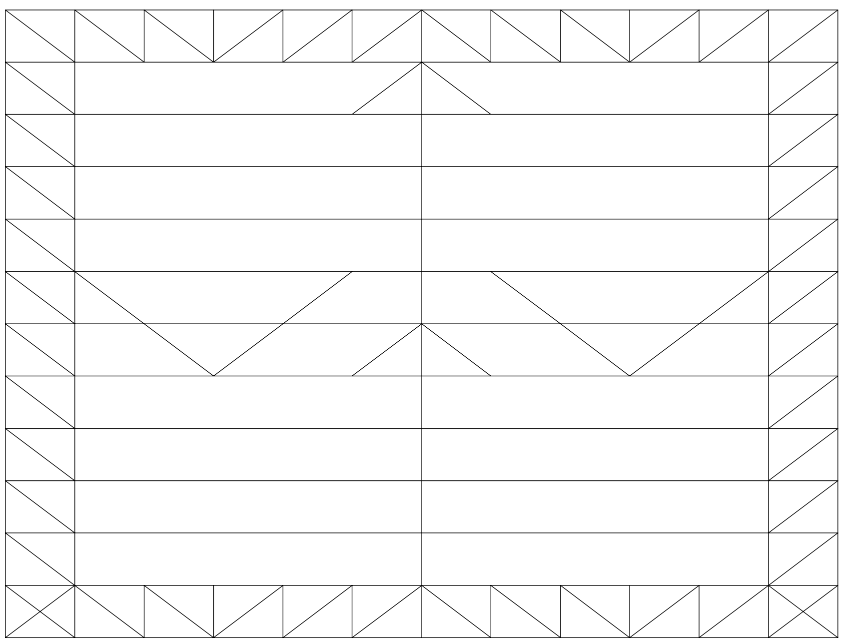

Figure B-2. Horizontal bracing system with compression members removed.

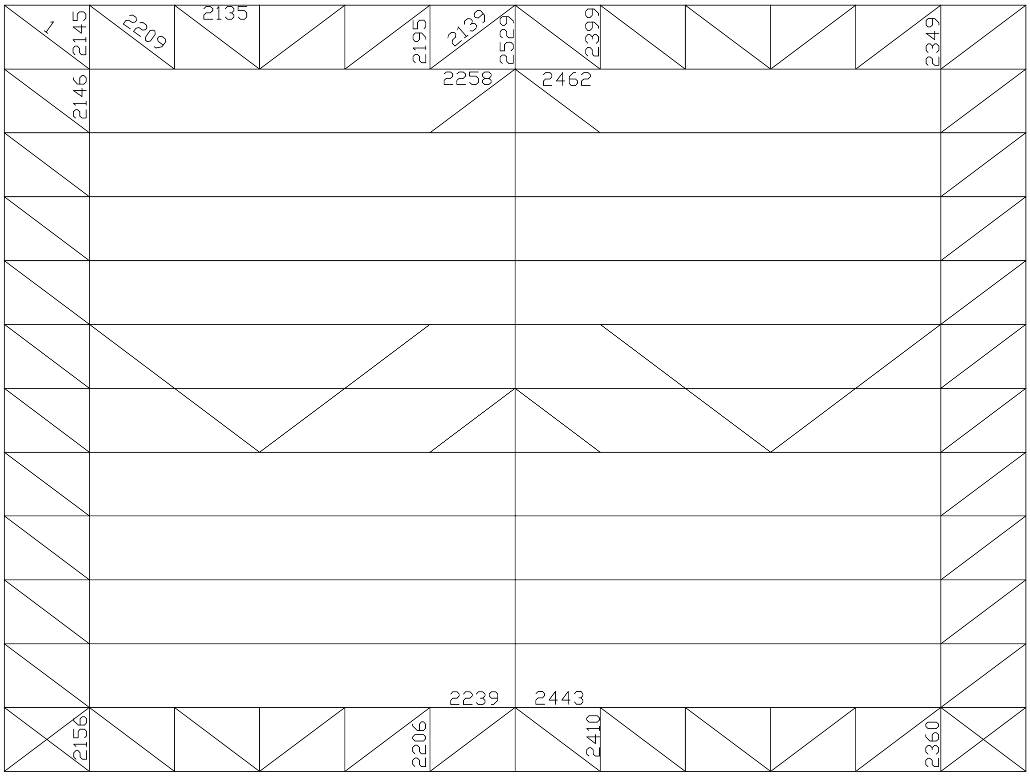

Figure B-3. Horizontal bracing frame with selected member numbers shown.

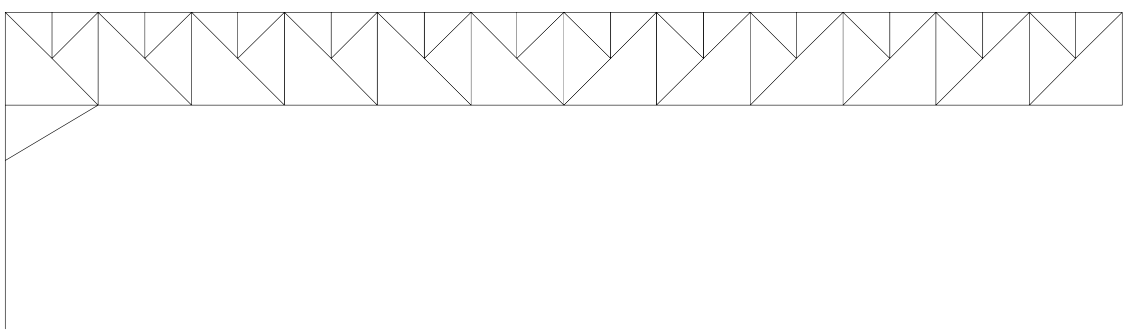

Figure B-4. Truss T1 with knee brace included.

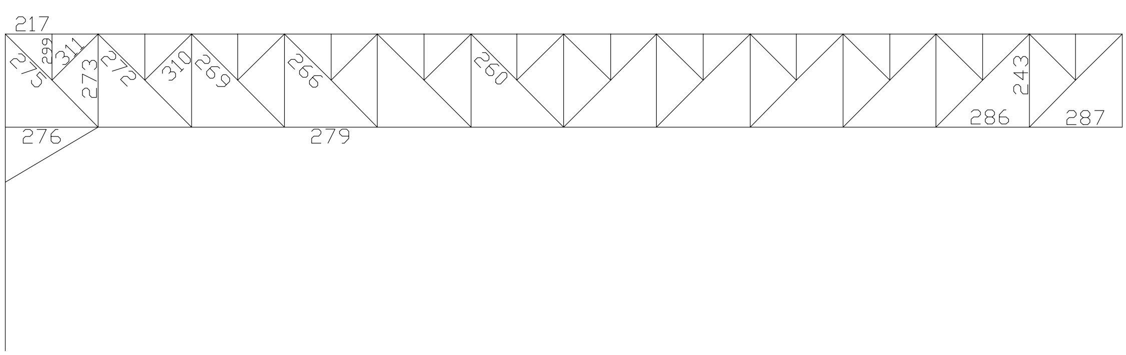

Figure B-5. Truss T1 with selected member numbers shown.

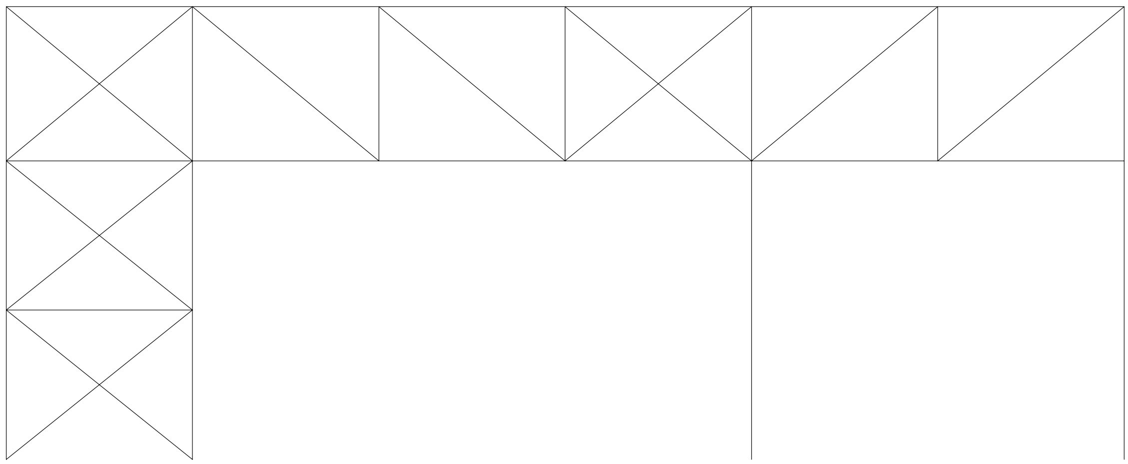

Figure B-6. Truss T3 with retrofit column included.

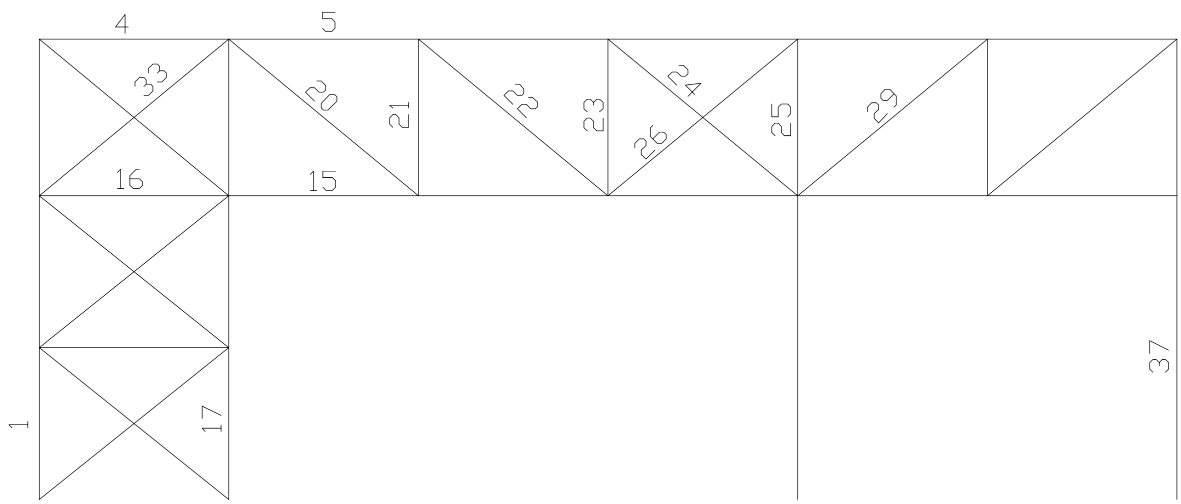

Figure B-7. Truss T3 with selected member numbers shown.

Appendix C: Memorandum on Crane Installation

1 August 2000

CEERD-CF-M (70-1y)

MEMORANDUM FOR Commander, SIOCC-ES-PS, ATTN: Mr. Pramod Desai,
Corpus Christi Army Depot, TX 78419-5260

SUBJECT: Recommendation for installation of an additional 5-ton crane system in Hangar 46, Corpus Christi Army Depot.

Detailed structural evaluation for Hangar 46 has not been carried out as of this date but is scheduled to be complete as of December 2000. If there is an urgent need to install an additional crane to serve the south half of the hangar before the completion of the evaluation, I recommend the following:

A new 5-ton crane system can be installed using principles from the existing system such that the new crane will not contribute a point load more than 5,000 lbs at any of the truss joints. This assumes that the crane is not in operation during heavy winds.

The attached drawing represents a plan view of Hangar 46. The existing crane is shown as blue with thick lines, with 5,000 lb point loads at the large circles. The proposed addition is shown in red with somewhat thinner lines, with 5,000 lb point loads at the small circles. Construction of the additional crane is recommended to follow this design, such that point load on the attached trusses from the crane does not exceed 5,000 lb at any joint at any one time.

This recommendation is based on the fact that the trusses proposed to support the additional crane are identical to the trusses supporting the existing crane. The stresses in the trusses due to the new crane will not exceed the stresses in the trusses supporting the existing crane.

If for any reasons higher crane capacity or different configurations are required, please furnish such data to me as soon as possible to incorporate in the final structural evaluation.

If you have any questions, I can be reached at (217) 373-7247, or FAX (217) 373-6763.

/ S /

Encl

Ghassan Al-Chaar, PhD

Structural Engineer

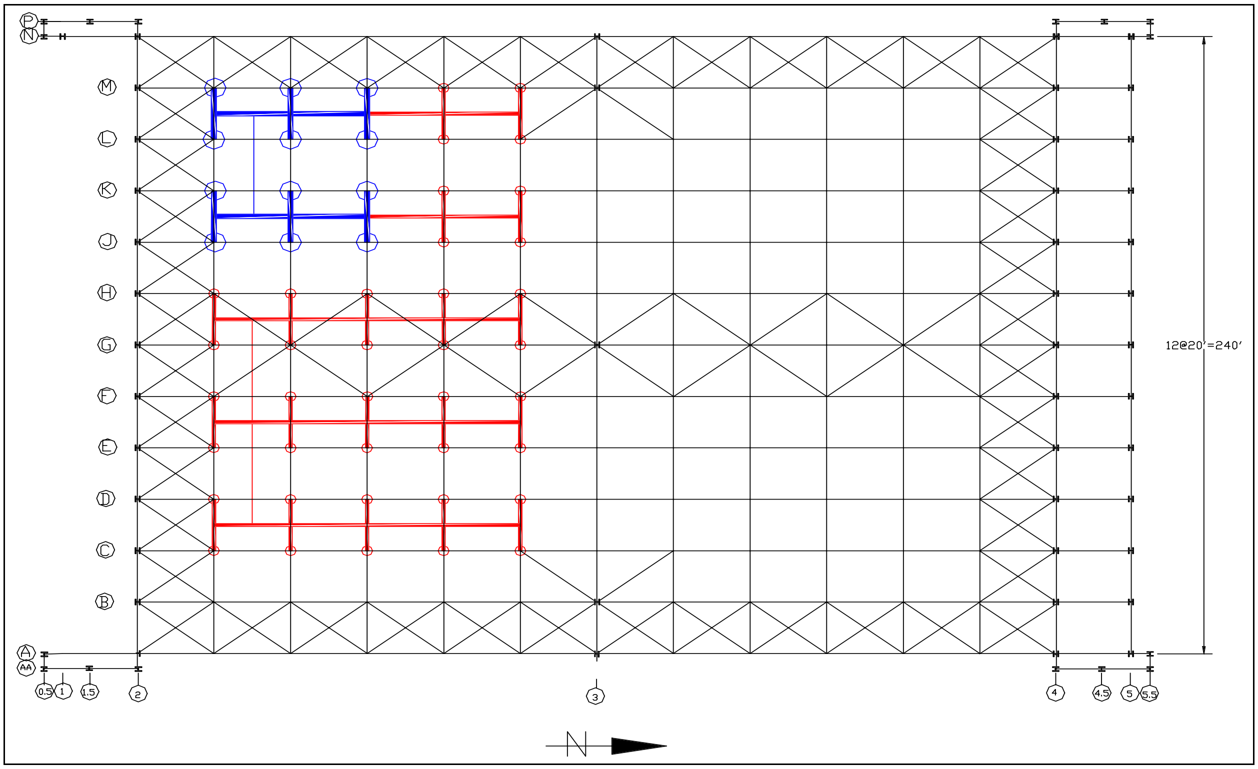

Figure C-1. Location of current and proposed railway crane.

Figure C-2. Ten thousand pound capacity crane in Hangar 46.

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14. ABSTRACT

A number of steel truss aircraft hangars at Corpus Christi Army Depot (CCAD) are similar to those that have performed poorly during recent hurricanes in other parts of the country. Engineering analysis of such structures currently in use can identify structural vulnerabilities, and retrofit methods may be developed to reduce or eliminate these vulnerabilities to severe wind loads.

The objective of this work was to evaluate the structural adequacy of one steel truss aircraft hangar at CCAD by conducting structural analyses using the most recent building code guidelines. The current condition of Hangar 46 was evaluated. Structural deficiencies and overstressed members and joints were identified, and retrofit methods to meet the requirements of current codes were developed.

This report may be considered a companion report to CERL TR-99/27. A number of technical details are cross-referenced to that report since many design elements and engineering details are similar to those of four CCAD hangars evaluated in the earlier study.

15. SUBJECT TERMS
Corpus Christi Army Depot, TX, wind loads, steel structures, structural engineering, aircraft hangars, steel trusses, structural reliability

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