PACIFIC
ENGINEERING &
DESIGN, P.L.L.C.
CONSULTING ENGINEERS

December 17, 1998

Mr. Dave Schott, Architect
The DOH Associates, P.S.
7 N. Wenatchee Ave., Suite 500
Wenatchee, WA 98801

RE: *Building No. 401 and No. 408*
Temporary Structural Repairs
PE & D Project #98019BS

Dear Dave,

As requested, we are in the process of developing temporary structural repairs for the bowstring roof trusses at Building No. 401 and No. 408 located at the Port of Moses Lake in Moses Lake, Washington. We anticipate completion of repair drawings for Building No. 408 by December 23, 1998 and completion of repair drawings for Building No. 401 by January 11, 1999.

It is imperative that the risks of continued use of these structures be understood. Due to the overall poor condition of the existing truss bottom chords, truss repair beyond that detailed in the Construction Documents or total truss replacement is required (See Building Structural Assessment Reports dated May 13 and May 15, 1998 for Building 408 and Building 401, respectively).

The temporary repairs detailed on the construction documents will <u>reduce</u> the risk of building collapse. In the event of further truss damage or deflection, the safety posts to be installed in Building 408 should alert the Port of Moses Lake personnel designated to monitor the building situation. This should allow sufficient time to evacuate the occupants of the structure.

These temporary repairs address only the most urgent repair requirements and do not eliminate the life safety risks associated with continued use of the structures. Please feel free to call if you wish to discuss this or if we can be of any further assistance.

Sincerely,

Kyle C. Rumble, P.E.
Structural Engineer

7 N. WENATCHEE AVE., SUITE 405 • WENATCHEE, WASHINGTON 98801 • (509) 662-1161

BUILDING STRUCTURAL ASSESSMENT

PORT OF MOSES LAKE
BUILDING NO. 401
MOSES LAKE, WASHINGTON

FOR

THE DOH ASSOCIATES
500 DONEEN BUILDING
WENATCHEE, WA 98801

BY

KYLE C. RUMBLE
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES JANUARY 31, 2000

Pacific Engineering & Design, P.L.L.C.
7 North Wenatchee Avenue, Suite 405
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Kyle C. Rumble, P.E.
Structural Engineer

PE & D Project #98019AS
May 15, 1998

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ATTACHMENTS

1.0 INTRODUCTION:

Pacific Engineering & Design, P.L.L.C. was retained by the Port of Moses Lake to inspect, investigate, analyze, and report on: structural defects, necessary repairs to upgrade the facility, formulate an opinion as to the general structural integrity of the structure, and to prepare an Engineer's Estimate of Probable Construction Costs to repair and upgrade the structure of Building 401 located at the Port of Moses Lake, Grant County Airport.

1.1 REPORT SCOPE:

The scope of this inspection/report is as follows:

1. A general inspection of the each bowstring truss was conducted at the site on April 2, 1998. The inspection included photographing and noting signs of deflection, cracking, splitting, deterioration, and other indicators that would suggest a structural problem. It is not within the scope of this report to map out and inspect every piece, member, structural component, or call out every potential structural problem within the structure. Some portions of the structure were inaccessible for inspection due to access limitations (height), existing framing, and sheathing. Also, many of the structural components (i.e. split ring connectors) are within the structure's construction and are not accessible for inspection.

2. Attending the April 2, 1998 site visit were Kyle C. Rumble, P.E., Mike R. Rolfs, E.I.T. (both of pacific Engineering & Design), and Lyle VanWinkle (maintenance person from the Port of Moses Lake). A hydraulic manlift was utilized to inspect the various portions of the structure.

3. Structural calculations were prepared to determine allowable load capacities.

4. A report was prepared summarizing the results of the site inspection, structural analysis, and assessment. Recommendations are summarized in general terms. It is advised that any repair recommendations be designed, detailed, and construction supervised by a licensed structural engineer.

5. The lean-to structures to the east, west, and south were not inspected. The office extension to the south was also not inspected.

1.2 LOCALLY REQUIRED DESIGN DATA:

The following loads and conditions are locally required design loads as determined by the local Building Official and the Uniform Building Code.

Roof Live (Snow) Load	: 20 psf (plus snowdrift)
Wind	: 70 mph Exposure C
Seismic	: Zone 2B
Frost Depth	: 18"

1.3 REFERENCE DRAWINGS:

The following original construction drawings for the Air Force Hangar and Building Numbers 401 and 408 were made available to Pacific Engineering & Design for the preparation of this report.

Sheet 1 of 13	- Footing and Foundation Plan	(November 1942)
Sheet 3 of 13	- Elevations "A" and "C"	(November 1942)
Sheet 5 of 13	- Sections	(November 1942)
Sheet 10 of 13	- Roof Trusses	(November 1942)
Sheet 11 of 13	- Roof Bracing Details	(November 1942)
Sheet 1 of 2	- Hangar Truss Details	(November 1942)
Sheet 2 of 2	- Hangar Truss Details	(November 1942)
Sheet 1 of 1	- Air Installation	(February 1956)
Sheet 1 of 1	- Floor Plan	(June 1962)

2.0 STRUCTURE DESCRIPTION:

Building Number 401 was constructed in approximately 1942. The structure was constructed by the Corps of Engineers to be utilized as an airplane hangar at Moses Lake Airfield.

The Building consists of a Hangar Structure and a Lean-To Structure attached to the east, west, and south. The overall structure is 200' ± wide by 140' ± long (this does not include the office extension to the south). See Photos #1, #2, #3, #4, and Sheet S1 - Attachment B.

2.1 HANGAR STRUCTURE:

The Hangar Structure is 160'-0" ± wide by 140'-0" ± long and 36'-2" high (inside clear). The Hangar Structure consists of the following (See Sheets S1 and S2 - Attachment B).

2.1.1 ROOF STRUCTURE:

The roof structure consists of 1x sheathing (perpendicular to joists). The sheathing is supported by 2 x 12 roof joists spaced at 24" on center. The roof joists bear on timber bowstring trusses spaced at 20' on center and spanning approximately 160'. The radius of curvature of the bowstring trusses is approximately 160'. The north bowstring truss supports the adjustable door guide system. The roof structure has bottom chord wind bracing in each end bay and horizontal struts at interior bays (See Photos #61, #64, #66, and Sheets S1 and S2 - Attachment B).

2.1.1.1 TRUSS CONSTRUCTION:

Bowstring trusses are so named because they resemble a hunter's bow. The bow portion (top chord) is primarily in compression. The string portion (bottom chord) is primarily in tension. Truss construction consists of spaced double 5¼" x 16½" glu-laminated top chords that appear to be site built (as no top chord splices were observed). Bottom chords consist of double 6 x 12 timbers. The bottom chords are spliced at six locations on each side of the trusses Splice 1 (SP1) through Splice 6 (SP6). Forces are transferred through splices via 6" timber spacers, 3" timber side plates, and bolted 4" ϕ split ring connectors (See Photo #15). Top chord forces are transferred into bottom chords at heel joints via 3/8" steel side plates, 3/4" bearing plates, and 1¼" ϕ bolts in two rows (See Photos #25, #26, and #27). Web members consist of 6 x 6 timbers attached at top and bottom chord joints with bolted 4" ϕ split rings, except diagonals adjacent to the truss king post (i.e., centerline vertical web member). These diagonals are 6 x 8 timbers also attached at top and bottom chord joints with bolted 4"ϕ split rings (See Photos #9, #10, #29, and #43).

2.1.2 BUTTRESS/WALL STRUCTURE:

The bowstring trusses bear on concrete buttresses within the east and west wall structure. (Note: The buttresses that support the bowstring trusses and roof framing for the Lean-To Structure are incorporated and poured as an integral part of the wall structure). The east and west wall structure consists of reinforced concrete spandrel beams and columns with masonry wall infill (See Photos #55 and #67). The south wall is a wood frame wall with seven 2 x 14 built-up posts at 20' on center with 2 x 6 at 24" on center stud infill framing. Exterior sheathing is 1x shiplap with metal siding (See Photos #9, #69, and #70). The north wall consists of a sectional hangar door that recesses into a concrete pocket (See Photos #1 and #2).The hangar doors bear and roll on steel rails embedded in the concrete slab. See Sheets S1 and S2 - Attachment B.

2.1.3 FOUNDATION AND FOOTINGS:

Per existing drawings, the buttresses are founded on reinforced concrete spread footings. The wall structure at the east and west are founded on reinforced concrete foundation walls and unreinforced concrete continuous footings.

2.2 LEAN-TO STRUCTURE:

The Lean-To Structure is a single-level structure approximately 20'-0" ± wide and full length at the Hangar Structure on each side of the building (east, west, and south). The Lean-To Structure is 13'-0" ± high (inside clear). See Photos #1, #2, #3, and #4. See Sheet S5 - Attachment B.

2.2.1 ROOF STRUCTURE:

The roof structure consists of 1x sheathing supported on site built trusses (or braced rafters) spaced at 24" on center. The trusses consist of 2 x 10 top chords (rafters), 2 x 8 bottom chords (ceiling joists), 2 x 6 diagonal webs (braces), and 1 x 6 vertical webs (verts).

The trusses bear on the reinforced concrete spandrel beams and columns with masonry infill at the exterior of the Hangar Structure (east and west) and on the wood frame wall at the south.

2.2.3 FLOOR STRUCTURE:

The floor structure consists of concrete slab on grade (See Sheets S1 and S2 - Attachment B). It is not known whether or not the concrete slab on grade is reinforced.

2.2.4 WALL STRUCTURE:

The exterior wall structure of the Lean-To Structure consists of metal siding over 1x sheathing over 2x stud walls at the exterior east, west, and south (See Sheet S5 - Attachment B).

2.2.5 FOUNDATIONS:

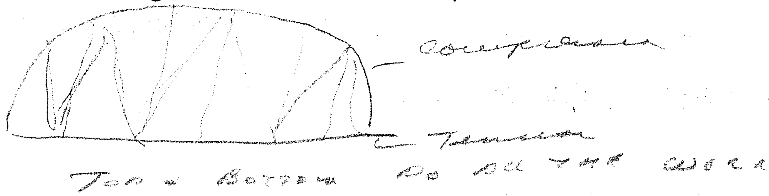
Per existing drawings, the buttresses (as previously stated) bear on reinforced concrete spread footings. The reinforced concrete spandrel beam and column wall structure (with masonry wall infill) are founded on unreinforced concrete continuous footings.

The exterior lean-to wood frame walls are founded on a reinforced concrete foundation wall. The foundation wall bears on an unreinforced concrete continuous footing (See Sheets S5 - Attachment B).

3.0 OBSERVATIONS:

The following observations were noted during the course of the inspection.

3.1 HANGAR STRUCTURE:



3.1.1 ROOF STRUCTURE:

The following observations were noted during the course of the inspection.

A. Broken and cracked joists were observed at five locations throughout the roof structure (See Photo #56).

B. Top chord stitch bolts were observed to be loose. This was a typical condition found in all the trusses (See Photo #5).

C. Top chords appeared to be in fair condition. No large splits or checks were observed. Glue lines were brittle on the exterior surface of the top chords. 5-16-26 AFTER

D. Truss web members appeared to be in good condition with a few exceptions. Some members had large longitudinal drying checks (See Photos #13 and #14). Also, at a few locations, members have twisted (probably prior to construction) and split ring connectors are nearly disengaged (See Photos #29 and #38).

E. Bottom chord splits near the east and west heel connections were observed in many of the trusses. Splits are in close proximity to one or both of the line of bolts at the heel connection (See Photos #7, #16, #17, #18, #19, #24, #25, #26, #28, #36, #37, #46, #47, #48, #53, and #54). Some of these splits are small and others appear to be large enough to indicate the initial stage of truss yielding at the heel connection.

F. Loose side plate bolts were observed at many of the truss bottom chord splices (See Photo #6). At some locations, a gap has developed between the bottom chord/splice plate interface. Split ring connectors were observed within these gaps (See Photos #8, #22, #23, #30, #32, #45, and #49).

G. Side plate splits were observed in many of the splice plates that transfer bottom chord tension forces across joint interfaces (See Photos #15, #20, #21, #22, #31, #34, #35, #39, #41, #42, #43, and #52). Size of splits range from small and insignificant (Photo #31) to large and continuous (See Photo #20).

H. Bottom chord cracks and checks were observed at many locations throughout the trusses (See Photos #9, #10, #11, #12, #31, #33, #35, #40, #42, #43, #44, #50, #51, and #52).

I. Anchor bolts securing the truss heel assembly to the concrete wall at each end were found to be loose at several locations throughout the structure (See Photo #27).

J. The top chords have been skewed to an "S" configuration at all of the trusses (See Photos #57 and #59). Bottom chords have also bowed in a similar manner (See Photos #58, #60, and Sheet S3 - Attachment B).

K. "X" bracing between Truss A and B and Truss F and G was observed to be racked (See Photos #61, #62, and #63).

L. Wind bracing struts were observed to have cracked and failed at several locations between Truss A and B (See Photos #67, #68, #69, and #70).

M. Wall studs near the southwest corner of the building were observed to have cracked near the bottom chord of Truss A (See Photos #71 and #72). Note: the tenants of the building reported that they had heard cracking of wood in the vicinity of the southwest corner recently during a windstorm.

N. Anchor bolts that secure the wind truss (between Trusses A and B and Trusses F and G) were observed to be loose at some locations (See Photo #65).

O. A separation was observed between the chimney and south wall of the building near the top of the truss (See Photo #4).

The following table summarizes the bottom chord observations photographed and recorded for each truss during the course of the inspection.

BUILDING 401
TRUSS BOTTOM CHORD INSPECTION SUMMARY

TRUSS ID #	BOTTOM CHORD SPLIT AT EAST HEEL	BOTTOM CHORD SPLITS	SPLICE PLATE SPLITS	BOTTOM CHORD SPLIT AT WEST HEEL
A	----	----	----	----
B	Yes	Yes	Yes	Yes
C	Yes	----	Yes	Yes
D	Yes	Yes	Yes	Yes
E	Yes	Yes	Yes	Yes
F	Yes	Yes	Yes	Yes
G	----	----	----	----

Measurements were taken from the bottom chord to the top of the slab at the west heel and truss centerline on trusses at Grid B, C, D, E, F, and G. The following is a summary of these measurements.

BUILDING 401
TRUSS BOTTOM CHORD MEASUREMENTS

GRID	HEEL ELEVATION	TRUSS CENTERLINE ELEV.
A	----	----
B	36'-3⅞"	36'-2"
C	36'-4⅝"	36'-2⅛"
D	36'-3⅜"	36'-1½"
E	36'-4"	36'-2" ±
F	36'-5¼"	36'-2" ±
G	36'-4⅜"	36'-2" ±

The above information indicates that the trusses were constructed with an upward bottom chord camber and it remains to this day.

3.2 LEAN-TO STRUCTURE:

3.2.1 ROOF STRUCTURE:

The Lean-To roof structure was not inspected, however, no major signs of structural distress were observed in a similar roof structure in Building 408.

4.0 ANALYSIS:

The structure of the Hangar and Lean-To were analyzed. The following is a summary of the results of the analysis.

4.1 HANGAR STRUCTURE:

The drawings note that the structure was designed in accordance with the following criteria:

4.1.1 LIVE LOADS:

Basic Roof Load	30 lbs. per sq. ft.
Basic Wind Load	20 lbs. per sq. ft.
(Modified for curved surfaces by Duchemin's Formula)	

The above criteria satisfies current Uniform Building Code (UBC) requirements except for wind load requirements. Wind loads for UBC 70 mph wind Exposure C approach 22 psf total or 13 psf for the windward wall and 9 psf for the leeward wall. Note that the locally required live (snow) load is 20 psf.

4.1.2 ALLOWABLE STRESSES:

Timber

Extreme Fiber in Bending	1200 lbs. per sq. in.
Horizontal Shear	100 lbs. per sq. in.
Compression Parallel to Grain	1000 lbs. per sq. in.
Tension Parallel to Grain	800 lbs. per sq. in.
(With l/d reduction)	

It was customary during the time that this structure was constructed to utilize allowable tension parallel to grain stresses equal to extreme fiber in bending stresses. Studies made in the intervening years have indicated that these values may be incorrect. We have therefore utilized a lower (800 psi) value in the analysis.

It should be noted that the drawings listed allowable stress values for timber only. Generally, this would apply to the timber bottom chords and webs. Most likely, the truss top chord (built-up glu-laminated) and joists' allowable stress values for that time period would be equal to or exceed the following higher values.

Allowable Bending Stress (Repetitive)	:	1450 psi
Allowable Shear Stress	:	100 psi
Modulus of Elasticity	:	1700 ksi

The load carrying capacity of the roof joists and truss top chord are based on these higher values.

4.1.3 ROOF DEAD LOAD SUMMARY:

A detailed analysis of the structural components was conducted. The following dead loads were utilized in the analysis.

ROOF JOIST:

Roofing (two layers)	:	4.4 psf
1x Sheathing	:	2.3 psf
2 x 12 at 24" o.c.	:	2.2 psf
Total (Use 10 psf)	:	8.9 psf

BOWSTRING TRUSSES:

TOP CHORD:

Roofing	:	4.4 psf
1x Sheathing	:	2.3 psf
2 x 12 at 24" o.c.	:	2.2 psf
Top Chord	:	2.1 psf
Total	:	11.0 psf

BOTTOM CHORD:

Bottom Chord	:	2.0 psf
Misc.	:	2.0 psf
Total	:	4.0 psf

4.1.4 ROOF STRUCTURE FINDINGS:

Based on the structural analysis, the following table summarizes the results of the analysis.

BUILDING 401
STRUCTURAL ANALYSIS SUMMARY

ITEM	DESCRIPTION	LOAD CAPACITY		REQUIRED CODE CAPACITY (LIVE) (PSF)[1]
		LIVE (PSF)	DEAD (PSF)	
1.	Roof Sheathing	30+	10+	20
2.	Roof Joists	41	11	20
3.	Bowstring Trusses			
	Top Chord	30±	11	20
	Bottom Chord	20±[2]	4	20
	Webs	30±	15	20
	Connections	30±	15	20

FOOTNOTES:

1. Structural component capacity based on applied roof live (snow) load in psf.

2. This lower value reflects the 33 percent reduction of allowable tensile stress (1200 psi to 800 psi) that was utilized for the analysis and as previously discussed.

Based on the structural analysis, the top chord is adequate for a 30 psf roof live (snow) load except near the heels where actual stresses slightly exceed allowable stresses. The top chord allowable stresses utilized in the analysis are probably low and therefore, the top chord is most likely adequate.

The bottom chord is inadequate for a 30 psf live load and barely adequate for a 20 psf roof live (snow) load. The allowable stresses utilized are based on good lumber without large splits and checks. Therefore, the bottom chord is of concern since it is barely adequate and the condition of the lumber is poor.

4.1.5 BUTTRESS/WALL STRUCTURE:

The buttress/wall structure was not analyzed, however, the structural components are well sized and reinforced (per existing drawings). On the basis of experience and observation, the buttress/wall structure is likely adequate to support the required loads.

4.1.6 FOUNDATION:

The drawings list the allowable soil bearing capacity as 6000 psf for the soil at the site. Analysis indicates that a maximum soil pressure of 2500 psf occurs at buttress spread footings.

4.2 LEAN-TO STRUCTURE:

The drawings note that the structure was designed in accordance with the following criteria:

4.2.1 LIVE LOADS:

Basic Roof Load . . . . 30 lbs. per sq. ft.
Basic Wind Load . . . . 20 lbs. per sq. ft.

The above criteria satisfies current Uniform Building Code (UBC) requirements except for wind load requirements. Wind loads for UBC 70 mph wind Exposure C approach 22 psf total or 13 psf for the windward wall and 9 psf for the leeward wall. Note that the locally required roof live (snow) load is 20 psf.

4.2.2 ALLOWABLE STRESSES:

Allowable Bending Stress (Repetitive)	: 1450 psi
Allowable Shear Stress	: 100 psi
Modulus of Elasticity	: 1700 ksi

4.2.3 ROOF DEAD LOAD SUMMARY:

A detailed analysis of the structural components was conducted. The following dead loads were utilized in the analysis.

ROOF JOIST:

Roofing (two layers)	: 4.4 psf
1x Sheathing	: 2.3 psf
2 x 10 at 24" Joists	: 1.9 psf
Misc.	: 1.4 psf
Total	: 10.0 psf

CEILING JOISTS:

1x Sheathing	: 2.3 psf
2 x 8 at 24" o.c.	: 1.5 psf
Misc.	: 1.2 psf
Total	: 5.0 psf

ITEM	DESCRIPTION	LOAD CAPACITY		REQUIRED CODE CAPACITY (LIVE)
		LIVE	DEAD	
1.	Roof Sheathing	30+	10+	20
2.	Roof Joists	30+	10+	20
3.	Ceiling Joists	0	30	10 (Dead)

Based on the above information, it appears that the structural capacity of the building's roof structure satisfies current building code requirements (except for snowdrift) as long as the structural components are not damaged or deteriorated and if additional roofing materials do not include more than two layers of roofing membrane.

4.2.4 FOUNDATION:

The drawings list the allowable soil bearing capacity as 6000 psf for the soil at the site. Analysis indicates that the maximum soil pressure is 1800 psf and therefore, it appears that the foundation is adequately designed.

5.0 ASSESSMENT:

The following assessments are summarized for the Hangar Structure and the Lean-To Structure. These assessments are based on inspection and analysis.

5.1 HANGAR STRUCTURE:

The following is a list of assessment comments for the Hangar Structure.

A. Roof joists are adequate to support the required roof live (snow) and dead loads. The observed cracked joists were few and isolated and are probably the result of defective joists.

B. The bowstring trusses (if in good condition) are adequate to support the required roof live (snow) and dead loads. Due to the many observed deficiencies, the existing condition of the roof trusses creates a potential risk of structural failure under normal snow load. The critical structural component of the trusses is the segmented timber bottom chord which transfers large tension forces from top chord connections at the heel at each end. This component is only as strong as its weakest link. Given the numerous bottom chord and splice plates splits in conjunction with loose bolts and split ring connectors not fully engaged, a normal snow load could eventually cause bottom chord failure and possibly building collapse. The building has successfully supported snow loads for the past 55 years, however, eventually the condition of the bottom chords will degrade to a point that will result in failure (unless repaired or replaced).

C. The bottom chord splice plates have splits and cracks at many locations. The cause of these cracks can be attributed to the shrinkage in the depth of the splice plate members. As the member dries out, it experiences volumetric shrinkage. The two rows of split rings prevent the wood from reducing in depth and the end result is a longitudinal crack between the rows of split rings. These splits are the result of drying and checking of the timber members. It is likely that tension loads have enlarged these cracks over the course of the life of the structure. These splits <u>reduce</u> the strength of the splice plates due to reduced split ring bearing area and at some locations, create a weakened plane where the member could split.

D. The bottom chord timbers have splits and cracks at many locations. Many of these cracks are the result of drying and checking of the timber members. At some locations, cracks may have enlarged due to tensile stresses. The presence of these cracks in the bottom chord <u>reduces</u> the structural capacity. This load carrying reduction is dependent upon the depth of the crack and angle of grain to the longitudinal axis of the bottom chord.

E. The lateral load resisting system for the structure has failed. This is evidenced by the deflected "S" configuration of the bowstring truss top and bottom chords, (See Sheet S3 - Attachment B), racked, and broken "X" bracing, cracked struts in the wind truss, broken studs at the southwest corner of the structure, and separation of the south wall from the chimney.

F. The wood frame walls of the structure appear to be in good condition. An exception to this is the broken studs at the southwest corner.

G. The reinforced concrete spandrel beam/column with masonry infill appears to be in good condition.

H. The reinforced concrete buttresses that support the bowstring trusses appear to be in good condition. An exception to this is the surface concrete that is exposed to weather. It appears that the manner in which the buttresses were formed and poured resulted in rock pockets and voids in the exterior sloping surfaces. Originally, this was repaired by filling the voids and rock pockets with a sand/cement mixture. Over the course of the life of the structure, this sand/cement fill and surface material has degraded and is spauling off. This surface damage is non-structural and should be cosmetically repaired to prevent further degradation of the concrete.

I. The foundation and footings were not accessible for inspection, however, since the footings are adequately designed, it is likely that the foundations are in good condition.

J. The bolted heel connection with rows of bolts closely spaced is utilized very seldom in modern timber construction due to the obvious weakness in the joint. Current codes require larger bolt spacings and reduction in allowable loads for these types of connections. The observed bottom chord splits indicate that this failure has started to occur at most of the trusses in the roof structure.

K. The cracked wall studs at the southwest corner of the building are the result of stresses from movement of the roof structure (diaphragm). The stud wall is balloon framed and no allowance for movement was considered in the construction.

6.0 RECOMMENDATIONS:

6.1 HANGAR STRUCTURE:

The structure's failed lateral resisting system in conjunction with the condition of the truss bottom chords and other components requires that the roof structure on this building either be repaired or replaced.

The following is a brief description of the construction procedure for repair (Repair Alternative A) and replacement (Replacement Alternative B). Following is an Engineer's Estimate of Probable Construction Cost for each alternative.

REPAIR ALTERNATIVE A:

Scope:

A. Repair truss bottom chords.

B. Re-align trusses.

C. Establish new lateral resisting system utilizing plywood diaphragm and shear walls.

Note: Assumes glue in top chord is found to be structurally sound.

Procedure:

1. Temporarily shore each truss (three places).
2. Remove existing roofing and 1x sheathing.
3. Remove existing "X" braces and secure.
4. Re-align truss top and bottom chords and secure.
5. Establish lateral transfer system at eave.
6. Install new plywood and roofing.
7. Repair truss bottom chords.
8. Remove existing wind truss each end.
9. Install new "X" braces and bridging.
10. Repair buttresses.
11. Misc. and contingencies.

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST

ITEM	DESCRIPTION	UNIT	RATE	AMOUNT
1.	Shoring	20,000 lbs.	$2/lb.	40,000.00
2.	Demo. Roof	20,000 s.f.	$2/s.f.	40,000.00
3.	Demo. Braces	160 MH	$40/MH	6,400.00
4.	Re-align Trusses	LS	-----	40,000.00
5.	Lateral System	LS	-----	6,000.00
6.	Plywood and Roof	20,000 s.f.	$5/s.f.	100,000.00
7.	Repair Trusses	7	30,000	210,000.00
8.	Demo. Wind Trusses	320 MH	$40/MH	12,800.00
9.	New "X" Bracing	LS	-----	24,000.00
10.	Buttress Repair	12	2000	24,000.00
11.	Misc. and Contingencies	LS	-----	52,800.00
TOTAL				**$556,000.00**

The above estimate assumes no hazardous waste, does not include Washington State Sales Tax, electrical work, mechanical work, architectural and engineering fees, and Port of Moses Lake Administration costs. The above estimate also does not include tenant relocation and storage costs.

Prior to the repair of the trusses, a core sample of the top chord should be taken and tested to verify that the glue is in good condition and has a strength of no less than 50 psi.

REPAIR ALTERNATIVE B:

Scope:

1. Demolish existing roof structure.
2. Install new pitched long span steel joists, purlins, deck, and roofing.
3. Utilize steel roof deck.

Procedure:

1. Demo roof, roof sheathing, joists, and trusses.
2. Install new long span trusses at 20' on center.
3. Install new roof purlins at 5' on center.
4. Install galvanized steel roof deck.
5. Install recovery board, rigid insulation, and roofing.
6. Install door track support.
7. Repair buttresses.
8. Misc. and contingencies.

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST

ITEM	DESCRIPTION	UNIT	RATE	AMOUNT
1.	Demo.	20,000 s.f.	$6/s.f.	120,000.00
2.	Joists (Long Span)	7	20,000	140,000.00
3.	Purlins	3500 l.f.	$5/l.f.	17,500.00
4.	Deck	20,000 s.f.	$2/s.f.	40,000.00
5.	Insulation and Roofing	20,000 s.f.	$5/s.f.	100,000.00
6.	Door Track Support	160 l.f.	$125/l.f.	20,000.00
7.	Buttress Repair	12	2,000	24,000.00
8.	Misc. and Contingencies	LS	-----	40,000.00
TOTAL				**$501,500.00**

The above estimate assumes no hazardous waste, does not include Washington State Sales Tax, electrical work, mechanical work, architectural and engineering fees, and Port of Moses Lake Administration costs. The above estimate also does not include tenant relocation and storage costs.

In regards to the above cost estimates, conservative values were utilized from Means Construction Cost Data - 1998. However, due to the unique nature of this project and since the engineer has no control over the cost of labor, materials, equipment, or services furnished by others; the contractor's method of determining prices; or the contractor's competitive bids or market conditions, the engineer cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from estimates of probable construction costs noted. If the Port of Moses Lake needs greater assurance as to the probable construction cost, an independent cost estimator for the project should be employed.

6.2 LEAN-TO STRUCTURE:

Although the Lean-To Structure was not inspected, it is assumed that any costs incurred to correct structural deficiencies will be minimal.

7.0 SUMMARY:

The structure is in generally sound condition from the foundation to the bottom of the truss bearing locations. The bowstring trusses are in poor condition. Unfortunately, the bowstring trussed roof structure is a significant part of the overall structure due to the long span condition and height of the structure and is therefore very expensive to repair or replace.

The structure's failed lateral resisting system and condition of the segmented timber truss bottom chords present some risk to life and property safety.

The roof structure is approximately 55 years old and may be approaching the end of its service life. Segmented chord bowstrings, more than other types of roof structures, may require more maintenance as they near or exceed their normal service life. It is our experience that segmented chord bowstring trusses of this vintage have a history of requiring remediation, ongoing maintenance, or possible replacement.

One problem with the repair of segmented bowstring trusses is that a member can be replaced; however, damage may occur to a different member (one that is not replaced) in the same truss at some point in the future. That is why we have recommended extensive bottom chord repairs or replacement of the roof structure.

Even though the truss glu-laminated top chords appeared to be in good condition, the segmented bottom chords were found to be in poor condition. This is often due to propagation of structural splits in normal drying checks or where the wood grain is not orientated parallel to the axial dimension of the member. The large timber sections utilized in the bottom chords of these trusses is not adequately dried prior to truss construction and quite often drying checks develop after the trusses are installed. These drying checks vary in depth and thickness. At some locations, the drying checks reduce the section properties of the member and failure (cracks or splits) may occur when the truss is loaded.

This is not the situation with the glu-laminated wood sections utilized in the top chords. The small sections of wood used to construct laminated wood members are much dryer and drying checks are typically very small or non-existent.

Due to the condition of the roof structure of Building 401, the ongoing occupancy of the building presents some risk to property and life safety. The structure could be severely damaged and collapse in either a heavy snowfall or a high wind event. It is our recommendation that the Port of Moses Lake communicate the findings of this report to the occupants so that they are aware of the risk.

With regards to time frame as to when repair or replacement should be executed, a risk does exist with the structure's current state. The observed problems have developed over time. The structure may continue to degrade for some time before extensive damage or collapse occurs. On the other hand, a high wind or heavy snowfall may be the event that triggers further damage or collapse.

Due to the unknowns regarding future weather events, this firm cannot accept any liability for the continued use of the structure.

Regardless of what course of action the Port of Moses Lake chooses to take in regards to the structure, the cracked struts in the wind truss should be repaired to prevent portions of the wind truss from falling and causing injury or damage. Also, all bolts should be tightened. If tightening of the bottom chord bolts results in damage to the wood, immediate repairs should be undertaken.