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May 12, 2008

Mr. John Sibold
Director of Aviation
Aviation Division WSDOT
P.O. BOX 3367
Arlington, WA 98223

RE: Structural Examination of Woodland State Airport

Dear John:

At the request of Paul Wolf, I visited the Woodland State Airport in Woodland Washington, on Thursday, April 17, 2008. The purpose of the visit was to examine the three existing hanger structures for their current condition and structural adequacy. Mike Smith, from our office and you were present at the time of my visit.

You should understand that this report and the conclusions contained herein are the results of a visual examination of the property. No calculations, measurements, test, etc. other than those described below have been made. No interior finishes were removed, and no demolition or excavation was conducted to locate hidden areas of damage. As a result, the report and its conclusions are circumscribed by the inherent limitations of the methods used.

Building Descriptions

The primary structures at the woodland airport consist of three separate buildings with interior separated hangar stalls that are leased out to private individuals for the purpose of aircraft storage. Buildings A and B contain hangar stalls 1 through 8. Building C contains hanger stalls 9 through 12. The front hangar openings are all typically 40' wide to accommodate the aircraft wingspans. Individual hangers have had numerous and various modifications and repairs to include additions of doors, interior enclosures and storage mezzanines made to them by individual tenants over the lifespan of the building. (See Site Map)

Buildings A and B are located end to end on the east side of the airfield and may or may not have been constructed at or about the same time, but appear to be approximately more than 30 years old. Building A is 32' wide x 120' long. Building B is 32' wide x 160' long. Both buildings measure 14-1/2' to the gable peak with 9-1/2' walls. Each building has a pole frame type construction with three post lines supporting a gable-truss type, wood framework that was constructed to be post supported at the center as well as the front and rear post lines. Roof purlins consist of 2x6 members at 2' to 2'-6" on center single spanned up to 20' with a galvanized corrugated metal roof. The exterior posts generally consist of creosote treated double 3x6 members and interior posts consist of pressure treated 6x6 members. The posts are embedded into the ground and are situated at an alternating 10' and 20' on center spacing with the exterior post lines enclosed by galvanized corrugated metal siding. (See Photos #1 and #2)

Building C is located on the west side of the airfield and appears to have been constructed in the early to mid 80's; measuring 30' wide x 280' long, with 14-1/2' to the gable peak and 9-1/2' walls. This building also has a pole frame type construction with three post lines supporting double pre-manufactured trusses situated at an alternating 12' and 16' on center spacing. The roof purlins consist of 2x6 members single spanned up to 16' at ±2'-6" on center with a galvanized corrugated metal roof. All of the posts generally consist of pressure treated 6x6 members embedded into the ground with the exterior post lines enclosed by galvanized corrugated metal siding. (See Photo #3)

Structural Observations

Buildings A and B have been typically modified at the hangar bay openings by removing exterior supporting posts to accommodate the required bay openings. A few of the hangar bay openings have door systems of different construction installed and many others are open to the elements. In some locations, replacement 4x4 posts were installed with hinges at the exterior truss bearing point to apparently allow the post to be moved up out of the way and then returned to a supporting position. However, each of the posts installed in this manner have been tied up in the open position and provide no support. In a few locations, the original posts have been cut off above the ground and spliced to replacement posts. In building A, all of the hangar openings and missing posts are located along the west wall line. In building B, the hangar openings and missing posts are alternated between the west and east wall lines. (See Photo #4)

The gabled, wood framework supporting the roof joists generally consist of creosote treated 3x6 wood members framed into the interior bearing post from each side and connected at the eaves to form a truss type configuration. At locations where the exterior post was removed or modified, the tails of the truss- type framework is cantilevered out 16' with a 16' back span. An intermediate vertical member is present at each side near mid-span and a diagonal member is typically installed on the cantilevered side. Connections at the tails and panel points vary, but generally consist of plywood gusset plates at one or both sides with 3/8"-1/2" through bolts and varied nailing patterns. Many interior panel connection points for the vertical and diagonal members have no gusset plates or obvious connection. (See Photo #5)

In several areas the truss type frame work has been modified further with additional vertical and diagonal scab members and or strapping in various configurations to provide additional rigidity. On the cantilevered end side of the framework, additional diagonal knee braces have been installed in various configurations. These braces typically consist of single or double 2x6's attached to the interior post and out to near mid span of the framing. These connections also vary with through bolts in some locations and minimal nailing in others. Many of the installed knee braces are buckling out of plane up to 1" or more and several nailed connections are beginning to separate. At a few locations, 1/2" steel tie rods with angle clips, between the top and bottom chords, were installed in place of knee braces. The angle clips are bending slightly and crushing at the back of the through bolt connection. (See Photos #5 and #6)

On the exterior of the building, obvious and excessive deflection is evident at the ridge roof line and along the eaves, but most predominant at the hangar bay openings where the roof framing is cantilevered. Obvious sagging can be seen with a measured downward deflection of up to 2-1/2". Many of the rim joists are scabbed and spliced in these areas. In building A, hangar 2, sagging roof purlins have been reinforced with an intermediate truss running between the framing over the post lines and the sag in the roof is still apparent. (See Photo #7)

In building B, hangar 4, an interior post has been cut just below the roof framing support. Support for the cut post has been provided by a steel W4x13 steel member that is spanning 30'. This is a cantilever support location for the roof framing. The steel member is deflecting downward 3", outward 1-1/2" and is bearing at each end over scabs nailed to existing posts. Connections at the bearing points consist of bent over 16d nails and there is no noted connection to the cut post above. (See Photo #8)

Galvanized metal siding and roofing has been cut away and or replaced in several areas with varied degrees of connection. Wall siding, in the areas present, is typically supported with 2x6 wind girts at 2'-6" to 3' on center. No obvious clips or connections to provide for resistance against wind uplift were noted.

Building C has been typically modified at the hangar bay openings by removing or omitting exterior supporting posts to accommodate the required bay openings. A few of the hangar bay openings have door systems of different construction installed and many others are open to the elements. In at least one location, a 4x4 post was installed with a hinge at the exterior truss bearing point to allow the post to be moved up out of the way and then returned to a supporting position. Again, the post has been tied up in the open position and provides no support. In another location 2x6 knee braces were installed in the line of the door opening from the existing posts up to the rim joist. All of the hangar openings and missing posts are located along the east wall.

Roof joists, on a 2'-6" on center spacing, span up to 16' between lines of double pre-manufactured trusses and sheathed walls. Trusses generally consist of 2x6 top chords with 2x4 bottom chords and web members that are connected at panel points with pressed plates. At hangar bay openings, the trusses bear at the rear wall and at nailer blocks to the interior posts, but cantilever up to 15' out to the hangar bay openings. Vertical web members at the center of the trusses also appear to be nailed to the interior post. No lateral truss bracing is present. (See Photo #9)

Interior partition walls have been sheathed and are supported with 2x6 wind girts at 2'-6" on center. Framing above the interior walls, up to the roof purlins, appears to consist primarily of plywood sheathing up to a 2x6 rafter running approximately 15' from the ridge down to the eave. At locations where the exterior posts were removed or omitted, the tails of the trusses have been reinforced with wood blocking in some areas and small plywood gusset plates.

In hangar 13, wood framed storage mezzanines have been constructed at each side of the bay running from the front to the rear wall and are accessed by ladders. On the south side of the bay, there is an enclosed storage or office space supported by the mezzanine. Framing for the mezzanines consists of 3/4" plywood over 2x6 and 2x6 rough sawn members spanning up to 15' between wall lines and wood beams. The beams are attached to the existing posts with field bent Simpson Strong Tie brackets. (See Photo #10)

Exterior wall siding is typically supported with 2x6 flat wind girts at 3' on center. Roof purlins are attached at the trusses with joist hangers and all other connections of trusses or wind girts to posts appear to be made directly with nailed connections.

Discussion

The structures at the woodland airport consist of three separate buildings with slightly different methods and materials in construction, as well as various states of repair. However, all of the buildings are essentially constructed as pole supported structures with posts designed to be embedded into the ground. Typically, this type of framing requires that the posts and connections to the posts transfer and carry both, vertical roof loads with snow, and lateral loads due to wind and earthquake. Any diaphragm capacity in the corrugated metal roofing or shear wall type resistance in the metal siding is very minimal. Wall framing will typically carry its own weight, but must be designed to resist wind loads out of plane.

This is a light weight type of framing with a comparatively small amount of dead load from the structure itself. As a result, seismic forces on the structure, based on the building framing and interior partition walls, are not large compared to other types of framing. However, the configuration, lightweight nature and use of the structure make it very susceptible to high wind loads.

The individual hangers have had numerous and various repairs and modifications to include additions of doors, interior enclosures and storage mezzanines. To accommodate the wingspan of aircraft, modifications to remove or omit supporting posts from the construction have been made. Additionally, several remaining posts have been cut out just above the ground and replaced with splice connections.

Removing or omitting the posts has impacted the structures by reducing the capacity to carry vertical loads safely and to resist lateral loads applied to them. Cutting out and splicing posts has further reduced lateral resistance to wind or earthquake loads. Additionally, the construction of interior enclosures and mezzanines attached to the building has further increased the effect of seismic forces on the structure.

A previous site examination and calculations from May of 2002 indicate that the roof framing at structures A and B was determined to be overstressed in both bending and allowable deflection.

Buildings A and B are the older two of the three structures and are in a poor state of repair. The numerous modifications have compromised the ability of the structure to safely support required design vertical loads. The calculated vertical roof dead load for this structure is approximated to be in the area of 3 psf. The typical required design snow load is an additional 25 psf. Reviewing and checking against previous calculations from May of 2002 indicates that the roof purlins alone are more than 90%, and possibly as much as 300%, overstressed under a full snow load depending on the grade of wood used for construction.

The gable truss-type framework supporting the roof joists are not supported or constructed as true trusses. Typical truss design for hand built trusses requires careful attention to connections at the joined members. The observed geometry and existing connections do not even closely appear to be adequate. This is evident in the necessity of the numerous additional supporting knee braces, tie rods and scabs that have been installed to the framework over time. The design and connections of which are also not entirely sufficient as several are failing. As a result, the roof framing is deflected and sagging in many areas under dead load alone. Increased or full required design vertical loading could be expected to cause full or partial collapse in many areas.

Lateral resistance to wind or earthquake on these two structures was not calculated for this report primarily due to the current condition and loss of capacity to resist required design vertical loads alone. However, the numerous additions attached to the structure and post removals, as well as repairs to many of the remaining posts, are anticipated to have significantly affected the structures ability to resist required wind and earthquake forces.

During examination of the structures, it was noted that in hanger 4 of building B, the interior supporting post had been cut out and a steel member had been installed to carry the vertical load. The W4x13 steel section, as installed, spanned 30' unsupported with no connection to the cut post and under dead load alone, was deflected downward beginning to buckle. Follow-up communication at that time indicated possible imminent partial collapse and required that the hanger be evacuated and closed off to public access until temporary repairs, providing adequate support, could be installed. This requirement is still in effect until a designed repair is obtained and installed.

Building C is the newer of the three structures and in a fair state of repair with little outward sign of obvious duress. However, numerous modifications or omissions in construction have impacted the structure capacity to safely support required design vertical and lateral loads.

The trusses provided at the support for the roof purlins were most likely not originally designed to carry cantilevered loads. Although they appear to be holding up adequately under dead load only at this time, with no obvious or noted problems, as pre-manufactured trusses, they were most likely designed to be fully supported at each end and in the center with a snow load of 20 or 25 psf.

Calculations and analysis of the truss configuration, with 15' of cantilever span, indicate that the trusses may be capable of carrying the existing dead load only, depending on the grade of wood used for construction. This would require that the wood members used for construction be of high structural grade (DF Select Structural or MSR 2250). Based on the observed condition and performance from the time of construction, it is reasonable to believe that they are adequate for supporting the present dead load, but the grade of lumber used in the trusses should be verified.

However, analysis indicates that the existing installation of the trusses and connection to the supporting posts are inadequate to support or resist snow and wind loading. Inadequate bracing and connections to the posts make collapse due to loss of support or buckling of compression members very possible under heavy loading.

Additional analysis of the 2x6 purlins, spanning 16' indicates that they too are adequate for the existing dead load, but are possibly as much as 160% overstressed for full snow loading depending on the grade of lumber used.

A complete lateral analysis of the building, using full wind and seismic loads, in both directions was not conducted. However, an abbreviated seismic analysis of the building down the long axis was made to estimate the level of loading to the post lines. Diaphragm action and shear resistance of the corrugated metal was discounted.

Properly constructed, this type of building is more accurately analyzed as a pole supported structure in which all vertical and lateral forces are resisted by posts embedded into the ground.

Three lines of pressure treated 6x6 posts run in the long axis of the building. Along the rear wall line and down the center there are approximately 22 posts at each line. Down the front wall line of the building, with hangar bay openings, there are only 8 posts. Approximated distribution of forces to each line and to each post along the front wall line, indicate that the posts may be as much as 60% overstressed in bending and compression according to current code. Deflection was not calculated and the effect of constructed mezzanines or interior structures was not considered. This limited analysis did not take into account adequacy of connections to the posts for force transfer nor did it address adequacy of the post condition or installation below ground.

On the interior of hanger 13, the recently wood framed mezzanines do not outwardly appear to meet current code requirements. Typical framing for this purpose is usually required to be designed for loads in the area of 50 to 100 psf. to accommodate office or light storage loads.

Recommendations and Conclusion

The existing hangars at the woodland airport are essentially pole supported structures built at different times and are in various structural conditions and states, of repair. Methods of original construction, modifications and inadequate repairs over the years have created problems that in some cases greatly limit or reduce the structural capacity of the buildings.

Vertical supporting capacity is primary in the structural design of a building to make it capable of supporting expected loads, as these are always present in some degree and can directly result in collapse. Additionally, a building's ability to resist wind and earthquake loads must also be considered. Observation and evaluation of the existing hangar construction indicates that all of the buildings are seriously limited in vertical, and or lateral capacity, or are already beginning to fail.

Buildings A and B are already showing signs of roof framing failure under dead load alone and past modifications indicate that this has been an ongoing problem. More recent modifications with the post removal in hangar 4 have created a situation requiring closure of the hangar bay. This hangar bay must remain closed until proper support has been restored.

The lateral capacity of buildings A and B for wind and seismic forces have not been directly evaluated but current condition, modifications and past repairs have significantly reduced the structural capacity to resist these forces. In the event of design level earthquake, it can likely be expected that this structure may experience some degree of collapse.

Designed repairs, retrofitting, replacement or just removal or of the these two buildings, which contain hangar stalls 1 through 8, is necessary to prevent eventual failures that will result in partial or full collapse.

Building C is generally in good condition without obvious sign of distress. However, analysis of the roof framing and support indicates that the existing 2x6 roof purlins are overstressed for design snow load requirements. Furthermore, the installed configuration of the trusses and connections to the posts are at risk for collapse due to loss of support, or buckling of truss compression members due to snow and or wind loading.

Field observation and rough analysis do not immediately indicate reason for significant concern over wind and earthquake forces. No verification has been made as to the subsurface condition of the posts or depth of embedment. It is not uncommon that older buildings will often be overstressed to some degree when compared to current code requirements. The 2006 IBC provides further guidelines for existing structures and acceptable comparison values.

Designed repair and retrofitting needs to be provided to insure that the structural integrity of the roof is maintained and able to support the necessary loads.

Any and all repairs or retrofits to the structures should be designed and detailed by a licensed professional engineer. If necessary or desired the capacity for resistance to design level wind and earthquake loads may also be further evaluated.

If you have any questions, or if I may be of further help, please call me at (360) 754-9339.

Sincerely,

GREGORY A. SEIPEL P.E.
Senior Lead Bridge Inspector
WSDOT Bridge & Structures Office

GREGORY A SEIPEL
STATE OF WASHINGTON
40895
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PROFESSIONAL ENGINEER
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Attachments: Site Map
Photos

Washington State Department of Transportation

Bridge & Structures Branch
Design Calculations

Project				Sheet No. of Sheets
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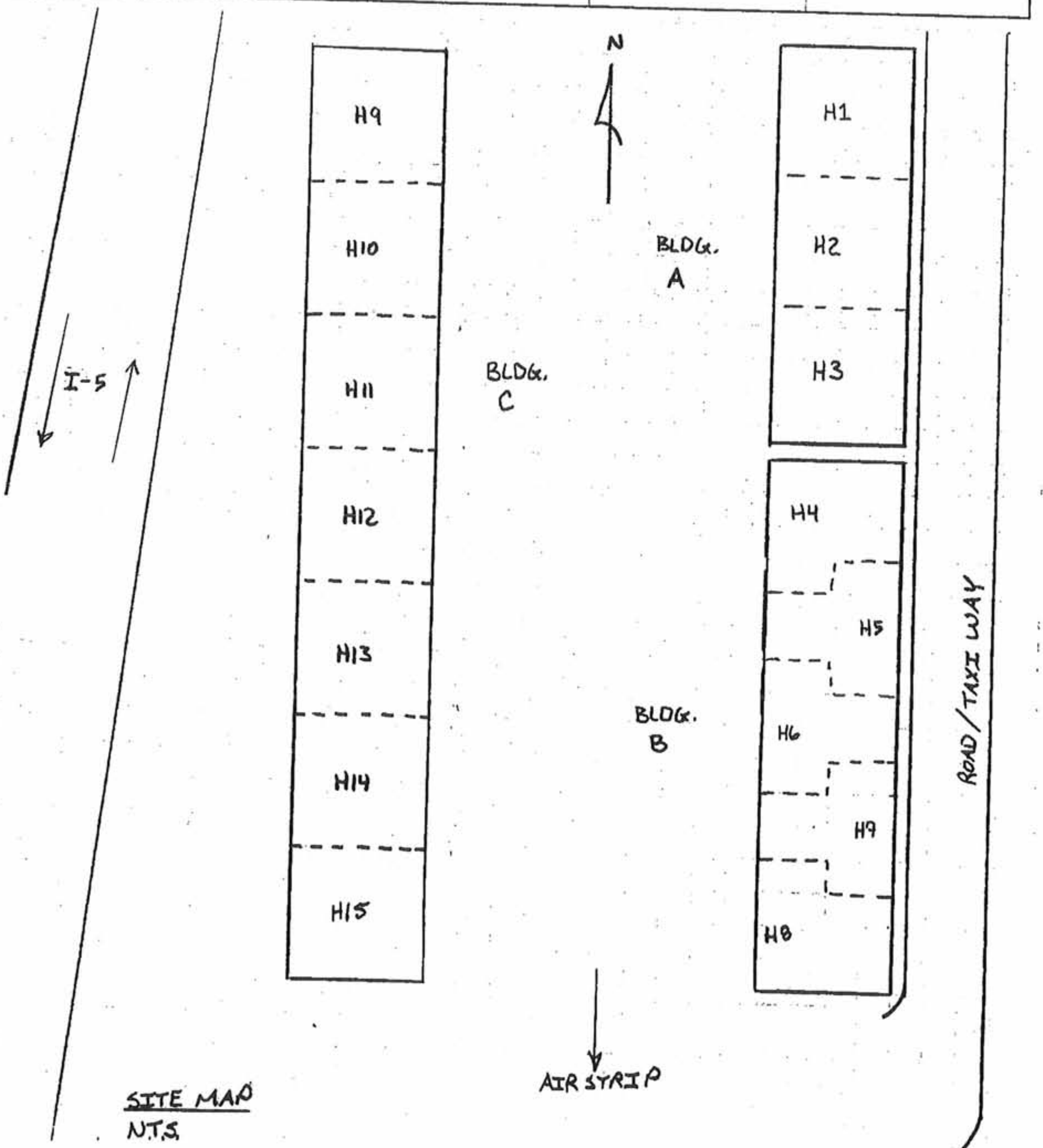

Site Map

Photo #1
Building A, Hangars 1 Thru 3

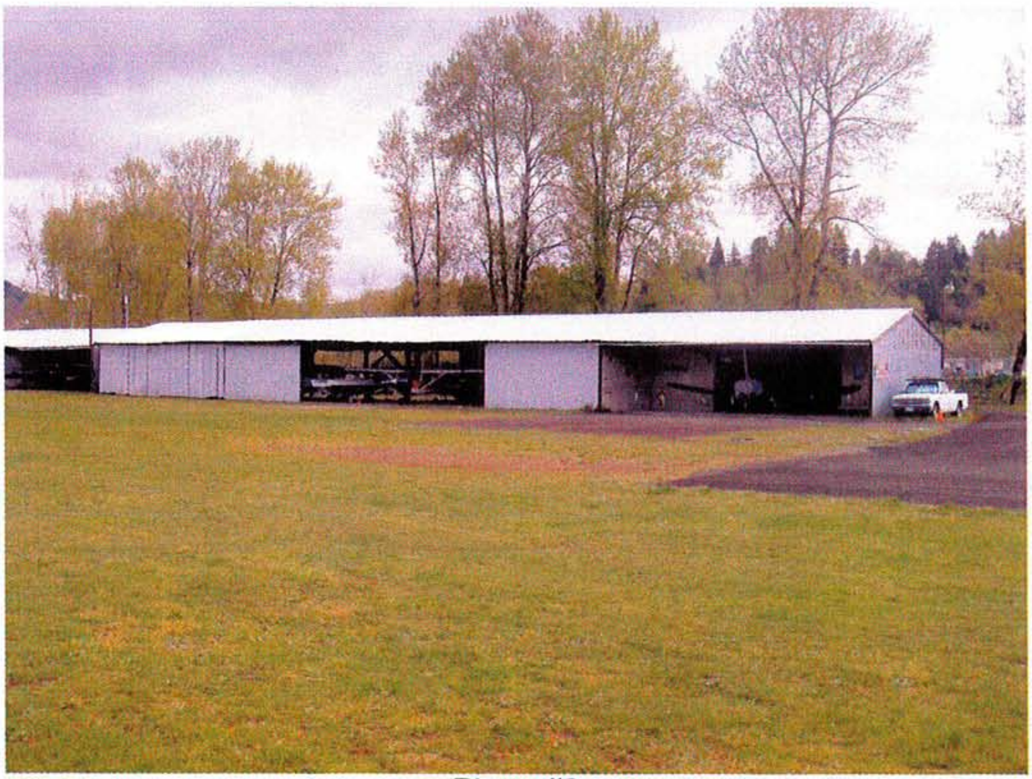

Photo #2
Building B, Hangers 4 Thru 8

Photo #3
Building C, Hangars 9 Thru 15

Photo #4
Cantilevered Framing Typical, Hangar 8

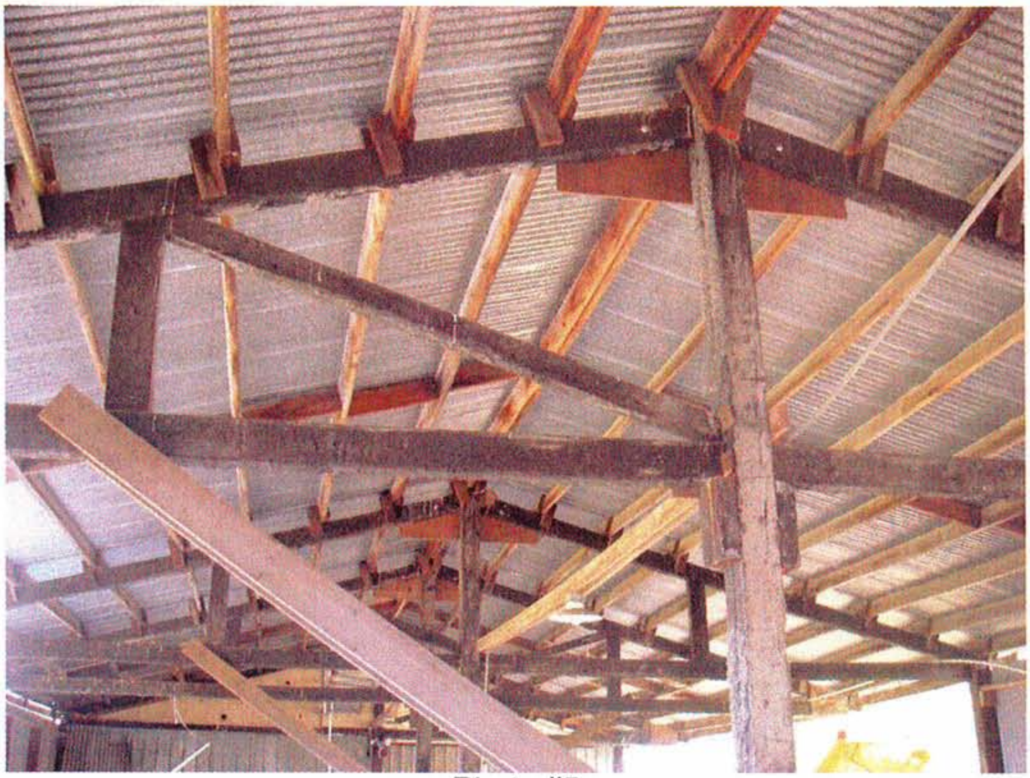

Photo #5
Truss Type Framing Support. Buildings A and B

Photo #6
Multiple Band-Aids

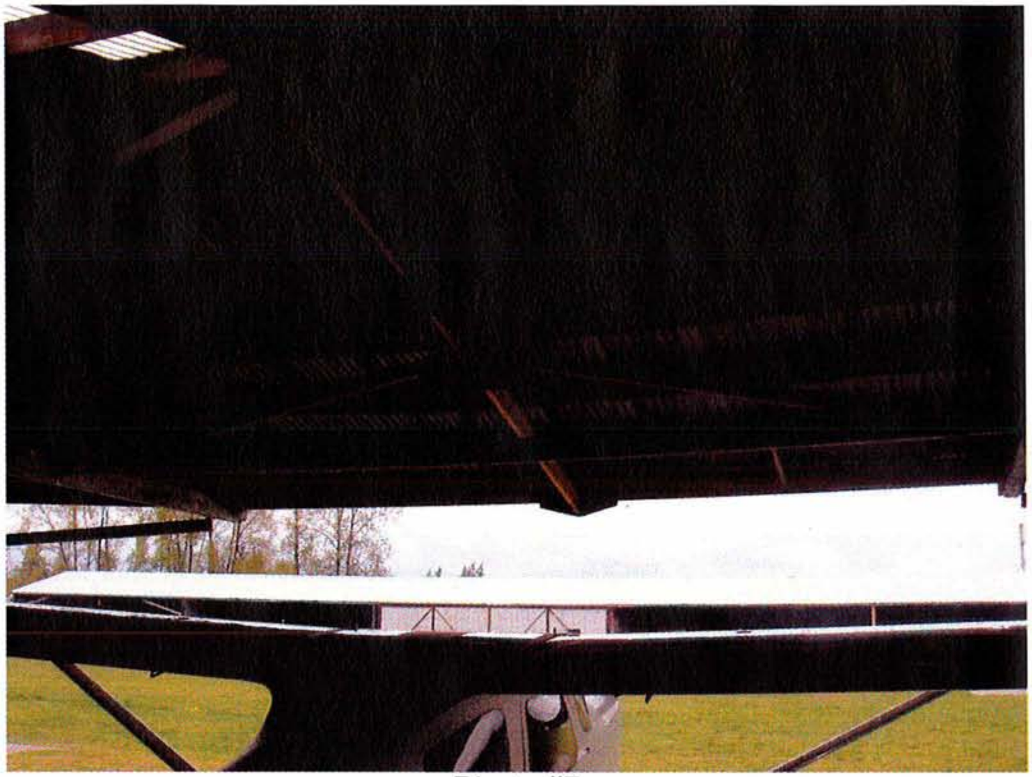

Photo #7
Additional Truss Work At Hanger 2

Photo #8
W4x13 Steel Section Deflected Downward And Buckling, Hanger 4

Photo #9
Double Truss Cantilevered Over Open Hangar Bay, Building C

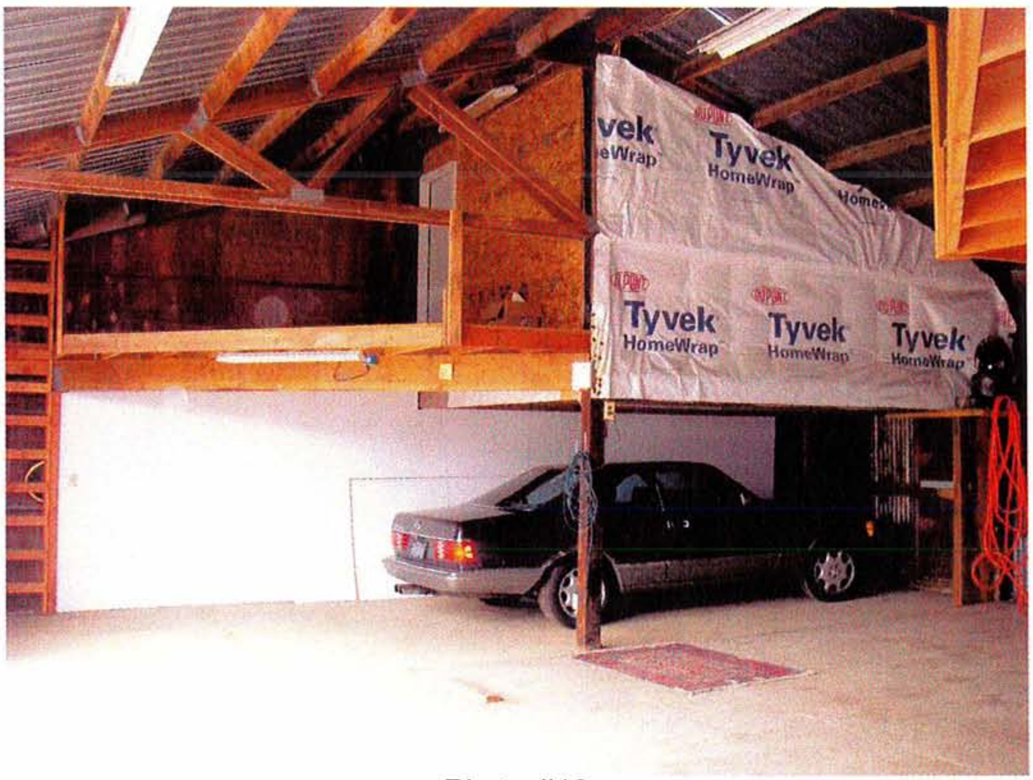

Photo #10
Mezzanine With Enclosed Storage Space, Hangar 13