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2008097

IAP World Services, Inc.
Building 510 Dugan Street
Moffett Field, CA 94035

Attention: Mr. Edgar Martinez

Subject: Moffett Airfield
Hangars 2 & 3 Door Support Beam Study
Moffett Field, CA

Dear Edgar:

Biggs Cardosa Associates is pleased to present the results of our structural engineering review for the above noted hangars. The south sliding doors of each hangar are unable to close, prompting the investigation. It appears that the existing door support beams have deflected enough over time to interfere with the operation of the doors. This report describes the existing hangar door support beam's structural system, ascertains its present condition, points out deficiencies in relation to current design standards, makes recommendations regarding structural upgrades, and provides an engineer's estimate of repair costs. Our work included field observation of the existing conditions on May 6, 2008. We also reviewed the original structural drawings supplied to our office. Original calculations and specifications were not available. On May 7, 2008 a survey crew determined ground and support beam elevations along the opening widths to help identify the extent of the beam deflection.

HANGAR 2

A. Box Beam Description

The timber box beam system attaches to concrete towers at each end of the hangar door opening. Steel tee sections mount to the underside of each beam and act as guides for the rolling hangar doors. The box beam spans approximately 220'-9". Shorter cantilever segments, which span approximately 29'-8", attach to the outside of the towers. According to original drawings from 1943, the beam bottom elevation measures approximately 121'-0" above the ground floor slab. See Appendix A, Figures 1 through 3.

The box beam cross section is approximately 21'-9" square. All four sides of the beam are sheathed with sawn lumber planks. The sheathing consists of two 3x12 diagonal timber layers, at 45 degrees in each direction, attached to transverse timber trusses at approximately 7'-1" on center. Refer to Figures 4 and 5 of Appendix A. The trusses act as ribs that stiffen the box beam for gravity and wind loads, and support the roof and floor planks.

The top, bottom and sides of the box beam consist of the two plank layers sandwiched between steel angles at the four horizontal corners. The angles at these corners are 48'-2" in maximum length and spliced using bolted steel plates. The planks are connected to the angles using combinations of machine bolts, split rings, and bearing plates.

The planks to corner angle connections consist of two different types of timber to steel connections based on their location along the length of the timber box beam system. The outer connection type extends approximately 60'-0" from each end of the box beam toward the middle of the box beam. The middle connection type spans approximately 100'-9" at the middle section of the box beam between the outer connection zones.

The outer connection type consists of the two 3x12 plank layers between the steel angles connected with two 7/8" diameter machine bolts per plank. Two 4" diameter split rings are located between planks at each bolt. There are also four 4" diameter shear plates between the planks and angles (two at each bolt). See Appendix A, Figure 7. The mid-span connection type consists of one 7/8" diameter machine bolt per plank. Due to a smaller shear flow, there are no split rings between planks and two 4" diameter shear plates at each bolt. Figure 6 of Appendix A shows this condition, along with the steel angle splice detail.

The vertical ends of the box beam bolt to the concrete tower walls with steel tee sections. The timber layers attach to the tee sections with two 7/8" diameter bolts per plank. There are split rings between planks at each bolt connection. See Appendix A, Figure 10.

B. Box Beam Deficiencies

Site observations revealed splits at the 3x plank to steel angle bolted connections along the bottom edge of the exterior (south) header face. See Appendix A, Figures 7 through 9, and Figure 11. Only inside plank layers were visible but it is probable that similar conditions exist at the outside plank layer. There were also signs of dry-rot at some of these same connections. At some locations, the steel angle has slipped downward approximately 1 3/8" likely due to possible failure of the bolted connections resulting from these two conditions. The upper south edge of the box beam shows no obvious significant damage. Additionally, the box beam north face (against the hangar) shows no obvious distress at either the top or bottom connections.

Site weather may also have contributed to the distress in the box beams. Temperature fluctuations result in movement of the beams as wood members expand and shrink. This movement, along with the drying and shrinkage of the beams over time, can contribute to the creation and enlargement of cracks in the wood planks. It is likely that the south side of the box beams experience this condition more than the north side, as the north side is more protected from weather by the adjacent hanger. The hangar roof slopes southward, which drains water onto the south box beam face. Photos of previous field observations show that the existing beam roof membrane has deteriorated over time. During our recent site observation, evidence of past moisture was clearly noticeable at the underside of the beam roof structure (See Appendix A, Figure 11). The combination of connection overstress, shown below in Code Analysis, temperature fluctuations, and moisture infiltration are the most likely causes of the connection failure along the box beams bottom south horizontal corner. The connection failure and the associated downward slippage of the corner steel angle, contributed to the deflection of the bottom of the box beam thus obstructing the free movement of the hanger doors.

C. Code Analysis

Analysis of the bolted wood to steel angle connections was based on the 2007 California Building Code and compared values derived from an older building code used in the 1940's. The double bolted outer edge connections are adequate using split ring connectors in addition to the 7/8" diameter bolts. The split ring connections yield a demand to capacity ratio of 0.85 (15% under stressed), while the bolts have a demand to capacity ratio of 0.99. The mid-span connections relying only on single 7/8" diameter bolts are overstressed by 330%. The bolts at the beam end structural tee connections are 290% overstressed in shear assuming that only the side connections of the box beams are used to transfer the gravity loads to the support towers. There are splits in some of the planks at the tee connections which show signs of distress at these connections also (See Appendix A, Figure 10).

Our office compared bolt strength values from the 2007 California Building Code to those from the 1949 Uniform Building Code which should have approximated the values used during the original design. The allowable stress values from the 1949 code were 8 to 20 percent larger than the current code values; however this difference is not enough to fully account for calculated overstresses discussed above. Current 2007 code allowable material stresses for Douglas Fir lumber grades are similar to values from the 1949 code.

D. Survey Results

Based on the approximate box beam section properties, the expected dead plus live load deflection at the centerline of the beam is approximately 1 3/4". Survey measurements indicate that the beam has deflected in excess of expectations. The end portions of the beam are close to their intended elevations, while the center portion has deflected between approximately 2.7" at the interior edge and 4.02" at the damaged exterior edge. The original construction details provide the hangar doors with only 3" of clearance to allow for movement and beam deflection. The survey also shows that the average mid-span clear height distance from the ground slab to the bottom of the beam is approximately 120'-9". The existing plans of the door repair dated 8-5-83 shows the actual door height as 120'-9 1/8" which confirms that the distressed condition has caused the doors to bind.

E. Summary/Recommendations

As discussed in the body of this document, the box beams are wood strutures with minimal steel components. It appears that, at some locations, the critical connections between the steel angles and the wood sheathing are underdesigned. These same connections also appear to have dryrot. It is likely that these conditions along with the movement in the box beams due to temperature effects, drying/shrinkage of the wood members and minimal bolt to wood end distance at the connections caused splitting in the wood at the bolt locations, resulting in downward movement of the bolts and an overally excessive deflection of the beams.

At this time it is not clear, due to limited access at the connections, how much additional damage to the beams may exist, but given what was discovered our office recommends that, at minimum, the doors remain in their current locations and access in the area below the beams is restricted. Assuming a decision to repair/replace the beams will be made in the near future; phase 1 of the work should be to issue a design/build contract to shore the beams. This could be done while contract documents are being prepared for the repair/replacement of the beams.

Given the overstress in the beam connections and the damage to the beam, our office recommends that the box beams be either retrofitted or replaced. We will describe two retrofit options and one replacement option, however additional options may be viable. It is our opinion that the first phase of actual design should focus on determining which option is best suited for the client given their functional needs, aesthetic desires and possible historic issues. Assessing these design criteria are beyond the scope of this report.

Retrofit options proposed in this report include an internal retrofit and an external retrofit. With either retrofit option, the end connections from the beam to the tower will require strengthening. Also, the existing beam needs to be adequately shored and jacked to the original profile prior to any retrofit work.

Under both retrofit options, our office recommends installing a new truss system to support the existing box beam. If historic appearances are a concern, a truss can be erected inside the box beam. A single truss can be fabricated along the existing centerline, or two separate trusses could be installed, one on each side of the beam. Alternatively, trusses could be placed outside the existing beam and left exposed. Another option is to completely remove the existing box beam and replace it with a new truss system that can be finished to match the look of the existing beam, if desired.

F. Engineer's Estimate of Construction Costs

The engineer's estimate of construction costs is intended to be a rough order of magnitude type approximation of the structural costs associated with the proposed option. Actual costs may be above or below those shown.

Option 1- Internal Truss Retrofit

The beams should be shored and jacked back to a level conditon. A new steel truss system would be installed inside each existing beam. The advantages of this option are that the work would be hidden and the finished product would look the same as the original. Also, the original historic materials are retained. The disadvantage is that the work may be more expensive than Option 2 because of having to work with smaller pieces and in a confined space.

Description	**Quantity**		**Cost**
Shoring	-		$ 75,000
Mobilization and Soft Demo	-		$ 160,000
Construct Trusses	-		$ 950,000
Re-roof	10,000 sq. ft.		$ 100,000
Replace Ext. Finish	10,000 sq. ft.		$ 150,000
Contingency	20%		$ 287,000
Overhead & Profit	15%		$ 258,000
		Total	**$1,980,000**
		Say	**$2,000,000**

Option 2-External Truss Retrofit

The beams should be shored and jacked back to a level conditon. A new steel truss system would be installed on the outside faces of each existing beam. The advantage of this option is that the work is done on the outside of the beam and should be less expensive than Option 1 or 3. Also the historic material will remain. The disadvantage is that the finished product will be visible and will not look like the original beam.

Description	**Quantity**		**Cost**
Shoring	-		$ 75,000
Mobilization and Soft Demo	-		$ 160,000
Fabricate Trusses	-		$ 100,000
Erect Trusses	-		$ 500,000
Re-roof	10,000 sq. ft.		$ 100,000
Replace Ext. Finish	10,000 sq. ft.		$ 150,000
Contingency	20%		$ 217,000
Overhead & Profit	15%		$ 195,000
		Total	**$1,497,000**
		Say	**$1,500,000**

Option 3-New Truss Box Beam

The beams should be shored and then deconstructed. A new steel truss beam system can be erected and finished to match the look of the original product. The advantages are that the final product is all new and looks like the original product. The disadvantages are that the cost may be more expensive that Option 2 and the original historic material has been removed.

Description	**Quantity**		**Cost**
Mobilization and Demo	-		$ 360,000
Fabricate Trusses	-		$ 100,000
Erect Trusses	-		$ 400,000
Install Roofing	10,000 sq. ft.		$ 100,000
Install Ext. Finish	40,000 sq. ft.		$ 400,000
Contingency	20%		$ 272,000
Overhead & Profit	15%		$ 245,000
		Total	**$1,877,000**
		Say	**$1,900,000**

*Construction costs include structural retrofit costs only. Costs do not include soft costs, hazardous materials removal costs, or costs for other trades.

HANGAR 3

A. Box Beam Description

See Hangar 2 for box beam description.

B. Box Beam Deficiencies

See Hangar 2 for a discussion of box beam deficiencies. Hanger 3 analytical deficiencies are similar to Hanger 2 and damage issues are similar but not as extensive.

C. Code Analysis

See Hanger 2 for code analysis results.

D. Survey Results

Based on survey data, the entire box beam is lower than originally designed. Available plans show approximately 121 feet of clear distance from top of ground-level slab to bottom of the beam. Field measurements locate the beam ends between 1 3/8" and 2 1/8" below this value, while the deflections at the center span range from 2 3/8" to 3 7/8" below this value. Construction details provided hangar doors with only 3" clearance for movement and beam deflection.

E. Summary/Recommendations

See Hangar 2 for recommendations.

F. Engineer's Estimate of Construction Costs

See Hangar 2 for cost estimate.

LIMITATIONS OF THIS REVIEW

1. The information given in this report is based on a walk through of the site and a review of the drawings which were supplied to our office. The site walk through was brief and was not intended to be a comprehensive site investigation of the structures. In most locations, architectural finishes were not removed to allow our office to view hidden structural elements.

2. Biggs Cardosa Associates make no warranty either expressed or implied, as to the findings, recommendations, or professional opinions stated in this report.

3. Biggs Cardosa Associates take no responsibility for the conformance of the as-constructed structure with the intent of the original design documents.

4. No reliance of this report shall be made by anyone other than the client whose name appears above.

5. Biggs Cardosa Associates has made reasonable efforts to assure that this report is accurate; however, we cannot assume liability for damages, which may result from its use or any conditions, which this report might fail to disclose.

Please call if you have any questions or comments regarding this report. We look forward to working with you in the future.

Sincerely,

BIGGS CARDOSA
ASSOCIATES, INC.

Mark A Cardosa

Mark A. Cardosa
Vice President
MAC/mc

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