REHABILITATION AND ADAPTIVE REUSE OF THE HISTORIC 1940s HOWARD HUGHES WOODEN AIRCRAFT HANGAR

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Abstract

This project consists of a large historic aircraft hangar (250 ft x 740 ft) with two additional side buildings. The existing structure was designed and built in the early 1940s using large glue-laminated arches, which were a ground-breaking technology at that time. The original seismic design was based on lateral loads equal to 10% of the self-weight of the structure.

In 2018, the large space inside the hangar became the home of a new approximately 450,000 square feet office building for Google. The structural work prior to the retrofit design included seismic evaluation of the structure, the determination of wood properties of the existing structure, and the development of a retrofit approach with building officials. The retrofit design includes the incorporation of a steel buckling restrained braced frame (BRBF) system into the existing structure with minimal visual impact to preserve the historic character of the building as much as possible. Space restraints required the use of 55 feet tall Multi-Tier Braced Frames (MTBF) which were not yet codified at the time. Retrofit measures were chosen so as not to interfere with the historic appearance of the building and reviewed by the US army corps of engineers and historic preservation board.

One hundred feet long tension rods were used to increase the stiffness of the glulam hangar and reduce deformation-induced stresses in the glulam elements under seismic loads. The glulam structure was assessed, and each delamination, damage or decay of the existing wood elements was documented on structural drawings. Repair of glulam elements using approximately 22,000 fully threaded, self-drilling screws, some as long as 52-inches was required to bring the glulam arches back to the initial strength. Detailed installation guidance was provided to the contractor to simplify procurement and installation of these screws. Inside the hangar, Arup designed a four-story tall office building comprising of steel framing and a BRBF lateral system.

Keywords: Adaptive Reuse; Rehabilitation; Seismic Retrofit; Glulam; Historic Building;

Fig. 1 Spruce Goose Playa Vista (Photo Copyright: Google)

1. Introduction

This project consists of the rehabilitation and seismic upgrade of the Howard Hughes aircraft hangars in Playa Vista, Los Angeles and creation of four new levels of office and film production spaces within the hangars.

1.1. Historical Context

The Cargo Building Complex was built by Howard Hughes Aircraft in 1943 as fabrication and assembly space for the Hercules HK-4, also known as the Spruce Goose, the world's largest wooden airplane. The site was previously used from the 1940s through the 1990s for manufacture, research, and development of machining operations, aircraft, and other electronics. The site comprises of a large hangar (Building 15) and smaller build-outs on the north and south sides of the hangar (Buildings 14 and 16). The three buildings cover approximately 240,000 square feet. The site included the former offices and manufacturing operations of the McDonnel Douglas Helicopter Company and the Hughes Aircraft Company. The Cargo Building Complex is of historical significance because of the early use of glued-laminated timber as the main structural support system.

1.2. Existing Building Descriptions

Building 15 (the Cargo Building) is approx. 740 ft x 250 ft and 55 ft to 75 ft high and comprised of two 125 ft wide hangers with a central office area (also called spine) in the center of building 15. It consists of 38 pairs of three-pinned glulam arches positioned in the short direction of the building, spaced at 20 feet on center and carry both gravity and seismic loading. The arches were innovative I-section glue-laminated (glulam) timber portal frames and supported roof purlins running longitudinally spaced at 5 feet on center and diagonal roof sheathing. Additionally, wood diagonal sheathed shear walls resisted seismic loading in the long direction.

Building 14 (the Hull Pattern Building) is a post-and-beam structure using bowstring trusses to support the roof, spanning between a timber wall and the north wall of Building 15. Building 16 (the Duramold Building) is supported by three-pinned arches, a reduced scale version of Building 15 on the south. Buildings 14 and 16 are both laterally supported by Building 15. Adjacent to Building 16 is a seismically separated concrete boiler room. Voluntary seismic retrofit of the boiler room building was performed but is not the subject of this paper and not discussed any further.

2. Historic building rehabilitation challenges and limitations

2.1. Building Standards and Building officials

ASCE 31-03 - Seismic Evaluation of Existing Buildings [1] and ASCE 41-06 Seismic Rehabilitation of Existing Buildings [2] were the applicable design standards at the time of design. The newer building standard ASCE 41-13 [3], which combines ASCE 31-03 and 41-06 and removes the inconsistencies between both codes was not yet adopted by the Los Angeles Building Code (LABC 2014) [4] yet.

The Los Angeles Department for Building and Safety (LADBS) is the authority having jurisdiction (AHJ). In addition, Building 15 is an individually designated eligible United States historic building subject to the requirements of the Army Corps of Engineers' Programmatic Agreement and Historic Resources Treatment Plan. All building alterations needed to be coordinated and reviewed by the Army Corps.

While design professionals typically use building codes and find solutions within their limits, the design team had to work around requirements to conserve the historic character of the building. In this case, historic requirements limited the removal of any existing façade elements, which reduced the retrofit options from the exterior of the building to a minimum.

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2.2. Retrofit concepts

Following retrofit concepts were discussed as potential solution with client and the LADBS.

1. Follow ASCE 31-03 and ASCE 41-06 to show that the building as is meets the requirements for rehabilitation of existing buildings and avoid any major retrofit.
2. Follow LABC 2014 chapter 34, which allows the use of existing buildings if the building remains mostly un-altered and the increase in demand on the seismic force resisting elements is less than 10%.
3. Upgrade the seismic force resisting system (SFRS) to current building standards using the current design earthquake.

Additional retrofit options were briefly discussed but eventually deemed impractical. For example, base isolation may have potentially led to a higher performance than code design but were much more expensive and difficult to realize given construction challenges and the historic nature and its limitations in the building.

The future use of the building as an office and assembly space with high occupancy number pushed the design parameter from risk category II to risk category III. The change in occupancy class and risk category has a major impact on the seismic and retrofit design as it triggers a mandatory retrofit following the provisions in Chapter 34 of the building code versus voluntary measures. This also meant the use of an importance factor of I_e=1.25 in lieu of I_e=1.0.

With the change of occupancy class, the only feasible solution proved to be concept #3 above, to upgrade the building to current seismic code requirements and using an importance factor of I_e=1.25.

The following sections describe the evaluation and conditions assessment of the existing building and the design of the structural system to strengthen and retrofit the existing structure.

3. Existing Building Performance

3.1. Existing Seismic Force Resisting System

The existing building, designed in early 1940s, was designed for a lateral load equal to 10% of the self-weight of the structure (0.1*g, see Fig. 2).

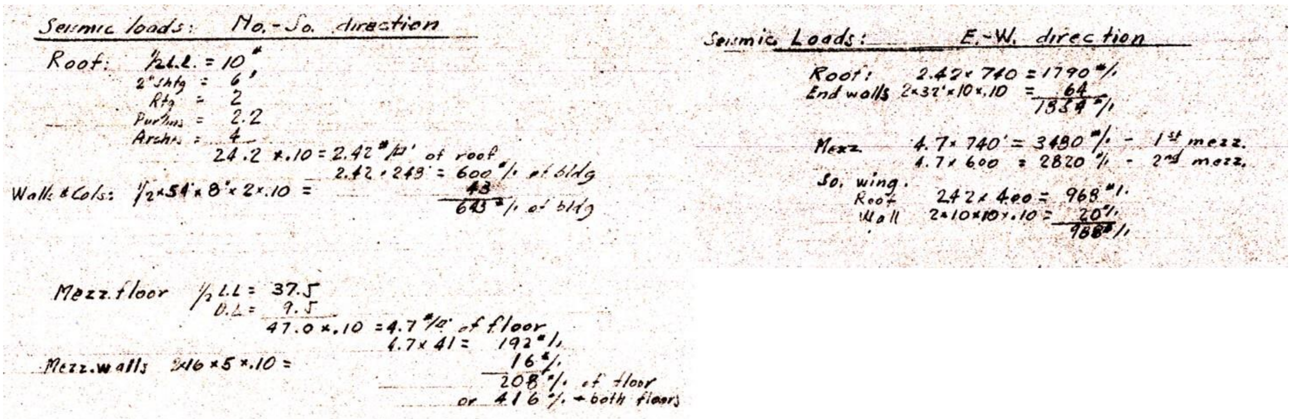

Fig. 2 - Extract from the 1940s calculations, showing the seismic load of 0.1*g

The analysis of the existing building under current seismic loads revealed several deficiencies in the seismic force resisting system, ranging from overstressed elements to components not meeting current code requirements (e.g. detailing requirements, minimum dimensions). The diagonal sheathed shear walls and transfer diaphragm in the center of the building were the main overstressed elements. Fig. 3 shows the

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existing building elements and their performance, where elements in green are acceptable and elements in red were deficient.

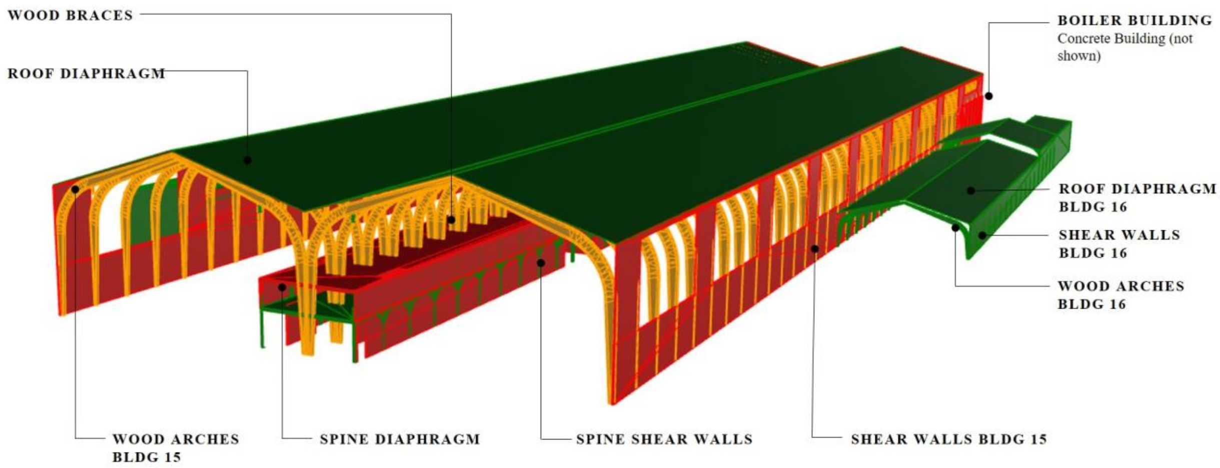

Fig. 3 - Existing Building deficiencies

3.2. Existing Gravity System

The existing structure was not subject to additional significant gravity loads, except for minor mechanical equipment supported on the roof. LABC allows an increase of 5% of the gravity loads for building alterations before a structural retrofit is required. The design was coordinated in such a way that all heavy equipment would be supported by the new structure and any installation on the existing structure was less than 5% of the initial loading, thus triggering only minor local repairs and beam strengthening. Gravity strengthening is not included in this paper.

3.3. Glulam/Wood Assessment and Existing Material Properties

The design team was able to find existing drawings and existing calculations (5 sheets, for a 740ftx250ft building !) in an archive for the Howard Hughes Campus at the construction site.

The glulam grade was called out as “Select Structural” and the sawn lumber as “Grade #1 common” on existing drawings. However, there were no project specifications or indications which design code has been used and to which timber properties these grades refer to. Also found in the archive, a design report from the 1960s for minor building repairs indicated an allowable stress of 2800psi for the glulam beams.

Given the building age of 70 years, the condition of the wood was uncertain and Ron Anthony Associates were contracted to perform the condition assessment of the building.

3.3.1. Existing Material Properties

Laboratory glulam testing was not possible as all glulam arches form a significant part of the structure and could not be removed or tested. In-situ testing was not possible either. It was agreed with LADBS that a conservative approach, comprising of the condition assessment results and a literature review of timber properties of that era, would be acceptable to determine wood properties.

Material property research compared the values of legacy codes such as Uniform Building Code 1943 (UBC 1943), UBC 1952 to UBC 1958 as well as the findings of Powell[5]. While Powell’s strength properties are lower than the UBC values, his findings are based on the reduction for tension grade lamination, which was not considered in many early building codes.

The design team concluded and LADBS agreed that using the lower bound values (e.g. 1800 psi allowable bending stress) from the property research was appropriate for the further development of the retrofit approach.

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3.3.2. Condition Assessment

A team of wood scientists assessed and documented the condition of the building over a period of 5 days. The findings were detailed and documented in a report including AutoCAD drawings.

The report confirmed that the wood species used in the glulam and sawn lumber is Douglas Fir (Pseudotsuga menziesii) contrary to popular belief that it was Redwood or Spruce (given the name Spruce Goose for the Hughes Flying Boat, which was ironically built from birch).

Several measurements were taken to confirm the moisture content of the existing structure to be approximately 10%.

Visual Grading of the sawn lumber and glulam confirmed in general the assumptions from the literature review. Purlins in the adjacent buildings 14 and 16 were graded lower than expected and a reduction in timber grade/capacity was considered for the design checks of these elements.

The most important outcome of the assessment was the condition of the glulam arches itself and the bolted connections. A detailed drawing for each gridline showed every missing bolt to be replaced and the amount of delamination in the glulam arches. Each delamination was documented to scale on the elevation view. These drawings allowed the engineering team to evaluate the reduced capacity due to delamination and determine repairs to restore the initial strength of the glulam arches.

Other findings, such as minor termite infestation, water stains, decay or mechanical defects were repaired or replaced accordingly, and are not further discussed in this paper.

4. Seismic Retrofit and Repair

4.1. Overall Retrofit Strategy

The retrofit strategy was to use a stiff seismic system with a high response modification factor, R, to reduce the seismic demand on the existing building elements as much as possible and thus to limit the required retrofit work. Special reinforced concrete shear walls (R=6) do not allow for flexible floor design and the impact on the architectural design and the MEP systems was too severe to be a viable design solution. Special Steel Moment frames (R=8) did not provide enough stiffness. Buckling Restrained Braced Frames (BRBF) (R=8) provided enough stiffness, were architecturally acceptable and were found to provide the right solution for the new SFRS.

The cross section (Fig. 4) and isometric (Fig. 5) summarizes the retrofit scheme for Hangers B14, B15 and B16 and show the BRBF locations for the new SFRS of the existing structure. The new structure within the building is not shown in figures for clarity.

Four BRBF cores in the center of Building 15 provide the main lateral system for the hanger structure. Multi-tier braced frames (MTBF) at the exterior north and south walls of B15 were added for additional lateral stability and work together with the BRBF cores in the longitudinal direction. As most of the lateral roof forces are transferred through the center BRBF cores, the roof diaphragm is subject to higher demands than the existing roof sheathing could resist. A horizontal truss of square hollow steel sections bolted to the top of the existing wooden diaphragm was used to strengthen the diaphragm and to provide sufficient diaphragm capacity to bring the diaphragm forces to the BRBF cores in the center of the building. New steel collector elements were installed in the longitudinal direction to drag diaphragm forces to the BRBF locations, as the existing wood members and especially their connections were not strong enough to transfer the accumulated collector forces.

Steel tie rods, between 1.25 inches and 2 inches in diameter, at 53 feet 6 inches above ground were added in transverse direction to limit drift and control the deformation compatibility requirements. Instead of using tie rods at each gridline 20 feet on center, we were able to optimize the layout and show that a 60 feet spacing is sufficient for drift control. The tie rods help significantly to reduce bending stresses in the glue laminated arches when the building sways away from the center BRBFs in a seismic event. The reduction in

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bending stress in the glulam was critical to show sufficient strength in the glulam arches, which makes the tie rods a critical part of the overall retrofit strategy.

BRBFs in Building 14 and Building 16 along the perimeter in the longitudinal direction and at intermittent locations in the transverse direction provide new SFRS for the hangars B14 and B16 adjacent to Building 15.

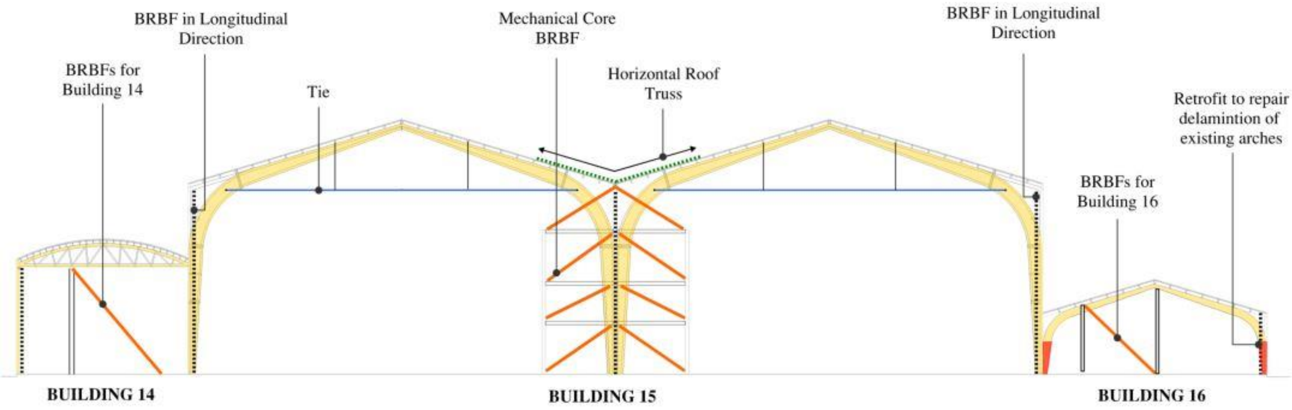

Fig. 4 - Building Cross section with Retrofit scheme

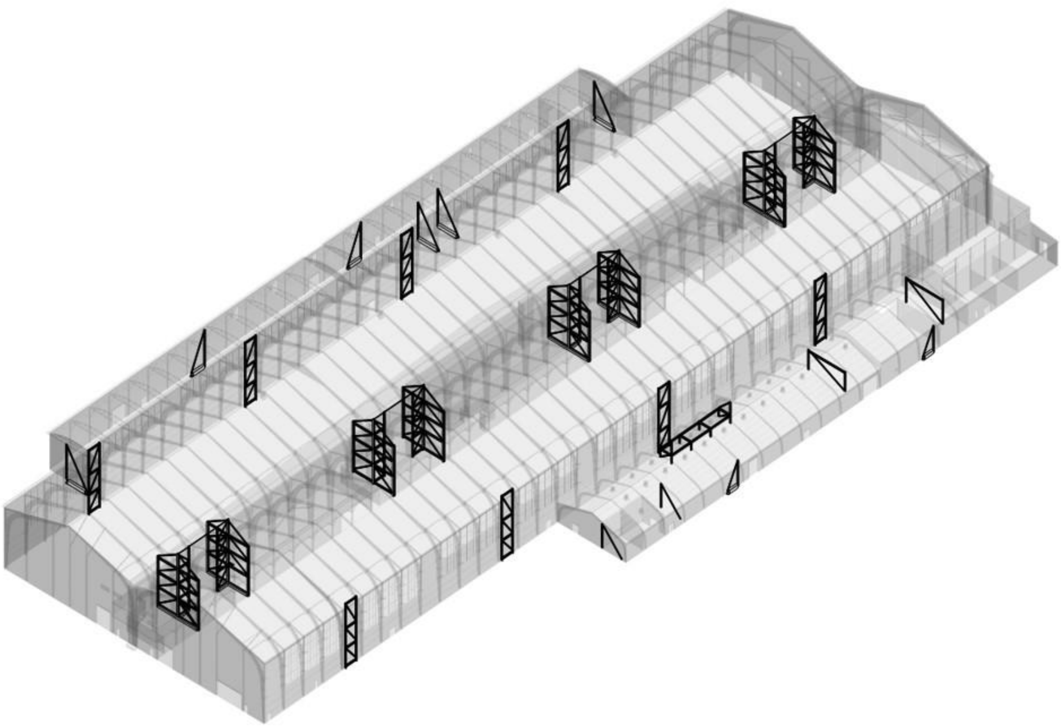

Fig. 5 - Building Isometric with layout of Retrofit BRBF locations

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4.2. Analysis Model

ETABS® from Computer and Structures, Inc. was used to analyze the building. Given the geometry and different materials, the design team used digital tools such as Grasshopper and the ETABS® API to define the geometry, model sloped surfaces and input all single wooden stick elements and their end connection releases.

Visualization templates were used to present the analysis and design results, facilitating and accelerating the design process and permit process. Analysis results from the ETABS® model of the entire existing structure were post-processed in Grasshopper and Rhino to create a stress distribution envelope of the glue laminated arches over the 38 gridlines. The final representation was one cross section (Fig. 6), allowing for simple verification that all stresses were below the capacity limit.

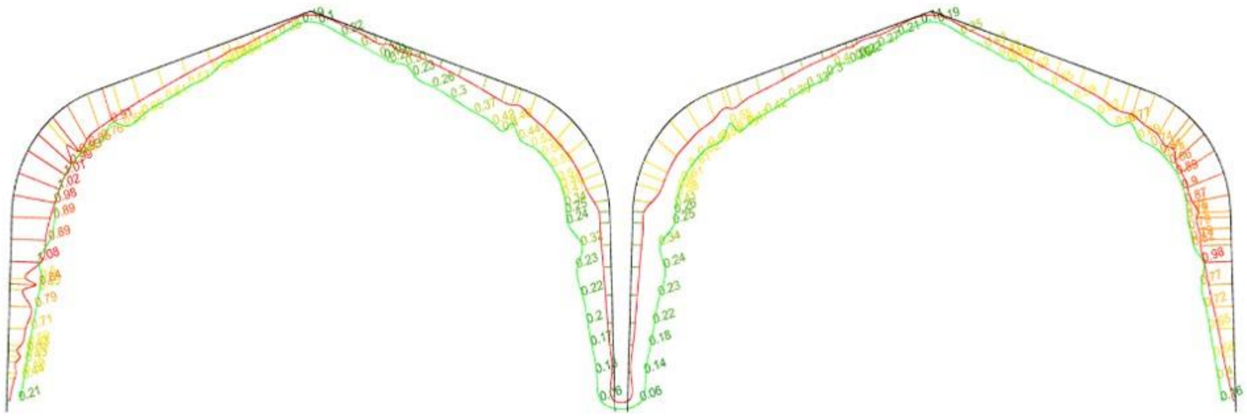

Fig. 6 - DCR envelope of all Glulam arches at all Gridlines

4.3. Multi-Tier Braced Frames (MTBF)

The existing lateral elements along the edge of Building 15 in the longitudinal direction consist of ten existing diagonal sheathed wood shear walls, 20 feet wide and 55 feet high (see Fig. 3). The historic conversation board requested that any seismic retrofit be limited to these bays to minimize visual impact to the building as seen from the outside. Considering the size of the existing frames and foundations, the maximum space available for new BRBFs led to a column center to center spacing of 11 feet, resulting in an undesirable aspect ratio requiring a design outside of the prescriptive code limits.

AISC 341-10 [6] requires all nodes of braced frames to be adequately braced out of plane. A single diagonal brace would allow for bracing of all nodes, but brace angle and aspect ratio would make this a very in efficient and non-workable solution. A BRBF core, like the ones in the center of the building, would interfere with the architectural design and the planned egress strategy.

MTBFs have several braces in the same frame without intermediate nodal bracing. In coordination with the manufacturer and members of the code committee, we were aware that the American Institute for Steel Construction (AISC) was working on provisions for MTBFs to be included in the upcoming AISC 341-16 [7]. However, the committee work was not completed yet or recognized by LADBS. Collaborating with LADBS, we were able to agree that we can use MTBFs for this application provided we follow draft code provisions and provide additional buckling analysis of the MTBFs.

Fig. 7 shows the installed MTBF directly located in front of an existing shear wall, neatly fitting in front of the existing shear wall with minor visual impact to the building appearance. Additional construction challenges, e.g. anchor bolt spacing and design of foundations, were closely coordinated with the contractor and adjusted to allow for erection with limited space avoiding existing footings and structural elements.

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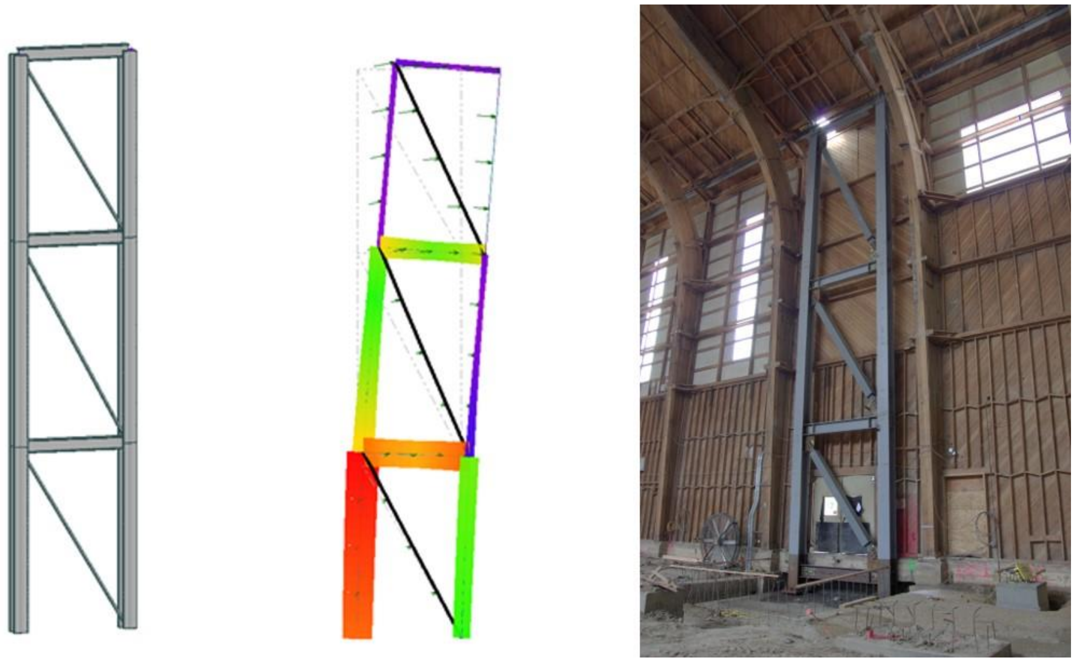

Fig. 7 - MTBF analysis model and installed on site

General Structural Analysis Software by Oasys (GSA) was used to run the buckling calculations (see model in Fig. 7). The columns need to withstand the typical BRBF forces for capacity design as well as provide sufficient stiffness at the nodes themselves. Unlike typical BRBFs, the columns in MTBF are rotated by 90 degrees such that their strong axis is perpendicular to the brace direction to account for the different geometry and stiffness requirements.

4.4. Glulam arch delamination and repair

4.4.1. Repair method and Design

Glue laminated timber was a fairly new technology in 1940s and had not been used at the scale of this project before in the United States. Timber as a building material was chosen over steel due to the high demand and shortage of steel during World War II.

The Duramold Building (B16) was built first as a test building for the main two hangars of Building 15 and served as the wood shop for the glulam production. The large glulam arches were generally in good condition after 70 years but showed signs of minor to severe delamination in several locations. While the reasons of delamination remain unclear and were not further investigated (possible reason could be: glue quality, moisture content, steam inside the building over 70 years, lack of quality control during production), the engineering team had to develop a repair method.

The condition assessment showed delamination of approximately 5800 linear feet across the entire building. The drawings of the condition assessment recorded each single delamination on elevation, provided a good understanding of the task at hand and eventually became the basis of the engineered repair solution. The depth or severity of the delamination was not documented.

While glulam delamination repairs have used different methods in the past, such as glued dowels, plywood or steel plates added to the sides of the beam or nail plates, the required method in this instance needed to meet architectural, historic preservation board and building code requirements.

Nail plates did not provide sufficient capacity for the size of the glulam beams. The method familiar to the contractor was to use plates on the side of the glulam arches. Although this method could have provided

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enough structural strength, it would have also added significant weight to the structure and the seismic mass. Given a goal of keeping added gravity loads under 5% of the initial loading to avoid costly gravity upgrades, the plated beam solution was not feasible.

Fully threaded screws are a relatively new product in wood construction and provided a feasible solution with minimal added weight to the building structure. The screws used in this project range from 3/8” diameter to 9/16” diameter and up to 52” length. Due to the self-tapping tip of the screw, pre-drilling is not required. The design team was familiar with the product and had used these screws before in practical applications in Europe, but the product and application were new to the general contractor.

Several local carpentry contractors were reluctant to bid for this project using this “new screw” and would have preferred to see familiar methods of retrofit. To add further challenges, the screws had not been used for timber strengthening in the City of Los Angeles, nor did they have a product compliance report from the City of Los Angeles. The design team continued to see the advantages of the screw and kept on pursuing this solution over traditional solutions. The manufacturer Wuerth and their North American affiliate MyTiCon were introduced to the project and given the opportunity to collaborate and to provide screws for use in Southern California. They provided additional design input and worked with LADBS to get the screws approved for use in the City of Los Angeles and received an Los Angeles Research Report (local compliance report). Given the innovative and first of its kind application (in California), the client decided to peer review the proposed solution. The peer review by Fire Tower Engineered Timber confirmed that the screw solution presented the best retrofit option.

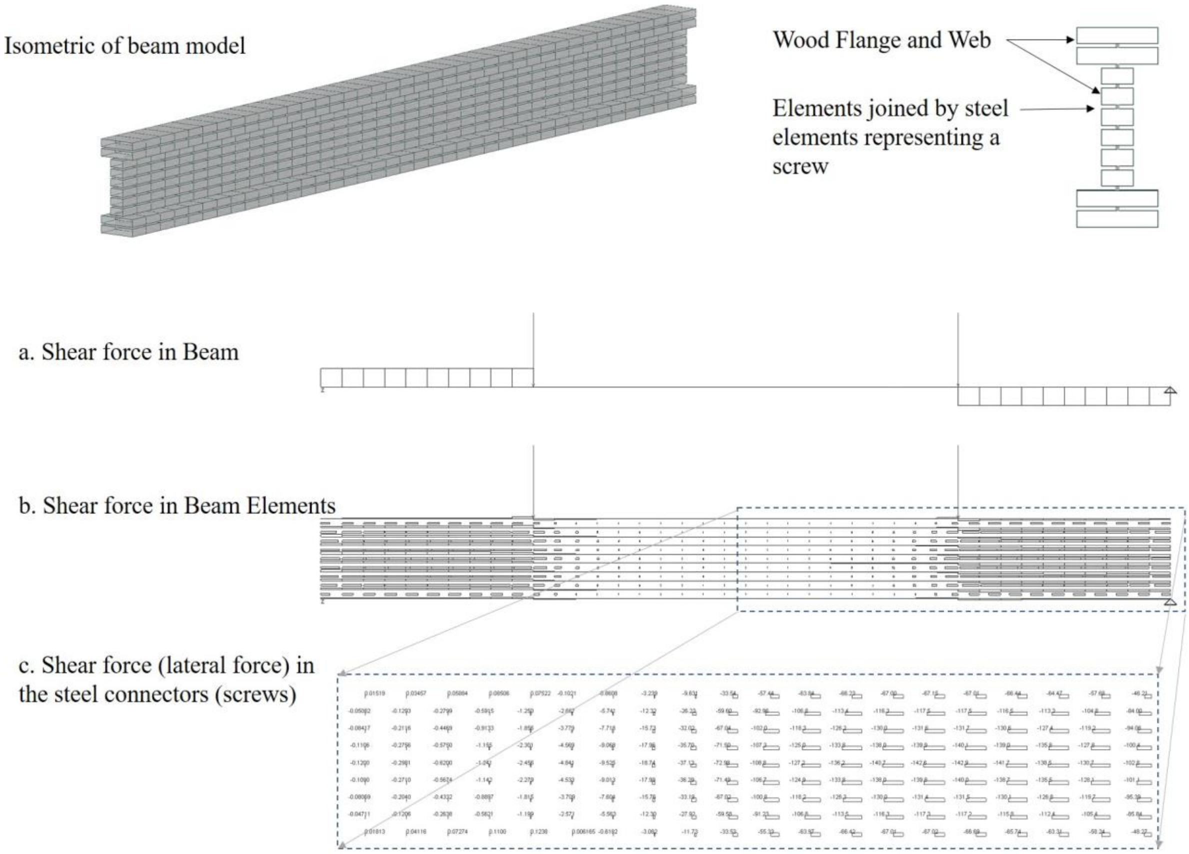

Fig. 8 - Simplified Beam model for group action factor clarification

We discussed the use of the group action factor Cg per NDS 2012 [8] with the peer review team and agreed that there was potential to avoid the reduction due to group effects in this case. The group action factor accounts for the higher shear in the outermost connectors in a tension connection.

Arup prepared detailed study models to show that the group action factor Cg applicable to a typical tension connection does not apply to the repair of glulam delamination. The analysis shows that in a repair situation (bending in the beam) the connectors are subject to very similar forces and there is no amplification of the force in the outermost connectors, which would require the reduction. These findings are in line with the findings about shear keys in mechanically laminated beams by Miller et al [9]. Fig. 8 shows the shear distribution in a beam and the shear connectors of a beam subject to bending, which is also the expected stress distribution for a repair of the glulam beam delamination.

Based on these results and working closely with the LADBS, we were able to prepare a request for modification and agree that the group action reduction in connection strength is not required for this application. Significant cost savings were achieved by reducing the screw count by 19% to approximately 17,000 screws for the project.

4.4.2. Installation of fully threaded screws

The required length of the screws depends on the location of delamination in the glulam arch and whether it was accessible from the top or the bottom chord of the existing glulam arches.

Without an additional and expensive field survey, the general contractor was concerned that it would be difficult to procure the correct screws (length and number of screws). To alleviate this concern, an installation guide (Fig. 9) was developed which defined the following steps for the field work: (1) Measure depth of delamination to determine if repair is required, (2) Measure location of delamination with regards to accessible edge of the beam.

These two steps together with the location of the delamination in the building enabled the contractor to choose the applicable repair detail and screw length from nine different repair details in field. Fig. 10 shows a screw during installation and two already installed screws below.

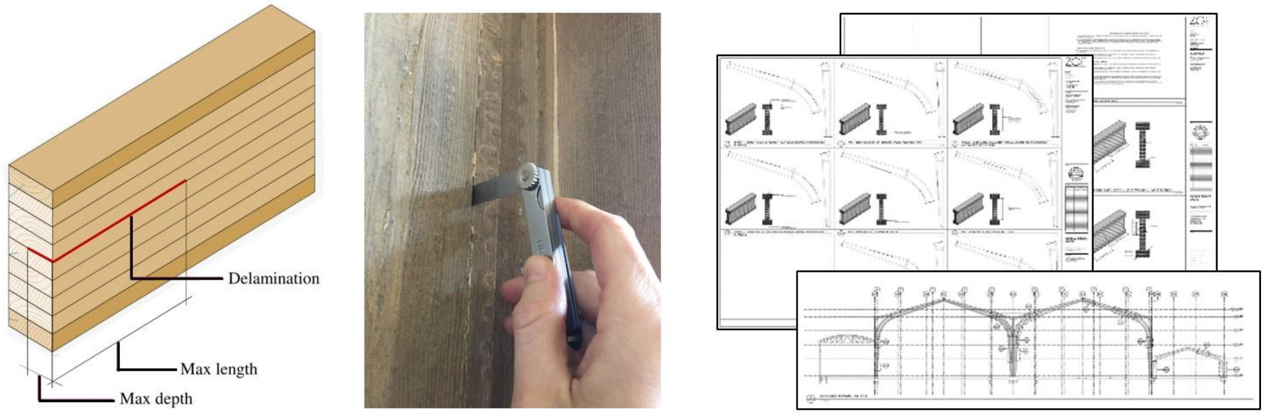

Fig. 9 – Extracts from the fully threaded screw installation guide

Fig. 10 - Fully threaded screw during installation

The ease of installation and detailed installation guide led to a very smooth and faster than expected installation. The work was completed ahead of schedule and the contractor ultimately preferred this method of strengthening.

5. New Building Design

5.1. BRBF Cores

The architectural concept of the new office building housed inside the B15 hangars incorporated floorplates that branched out from the spine structure. The spine was rebuilt as a steel framed building and the original timber cladding was reinstalled and refurbished to comply with historical building requirements. The spine structure housed four BRBF cores that provided lateral stability in the both directions of the 4-story office building as well as tying into the overall retrofit scheme discussed in Chapter 5. The design of the BRBF cores necessitated a combined analysis model of the existing structure and new building structure. Additional BRBFs in the long direction were provided along the exterior gridlines of the new building to limit torsionally irregular behavior of the floor plates as it twisted away from the spine structure under seismic loading.

5.2. Foundations

Based on examination of the original structural drawings, it was determined that the existing structure was supported on 8” diameter x 35-foot-deep treated timber piles. The existing timber piles were not considered adequate to provide capacity for the new building structure or the seismic upgrade of the existing structure. Much of the foundation design was driven by coordination of the existing structures above and below grade. Gridlines and new column locations were located to avoid conflict with existing pile caps approximately spaced 20 feet on center. New underground utilities such as 84 inch diameter mechanical duct banks at each core and existing remediation wells to treat groundwater contamination posed challenges in locating new piles and columns. A 12 inches thick slab on grade (250 feet x 740 feet) was designed to act as a structural diaphragm and transfer lateral load amongst a grid of approximately 1,150 auger cast displacement piles 14 inches in diameter and ranging from 50 feet to 55 feet in length.

Pile rig installation overhead clearance and unforeseen site conditions such as pile refusals required the team to closely monitor field conditions and collaborate on unique field solutions, such as designing eccentrically loaded pile caps.

6. Conclusion and Summary

The success of the adaptive reuse of these aircraft hangars into modern office accommodations is largely due to the collaboration with LADBS in developing an accepted building retrofit and repair strategy. The early involvement of LADBS allowed Arup to focus on innovative retrofit solutions that complemented

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the architectural vision and historic nature of the building. Despite initial constructability concerns raised by the contractor, the glulam repair method using fully threaded screws ultimately proved to be one of the most innovative solutions and key elements on the project.

The historic aircraft hangar is now home to a new, approximately 450,000 square feet cutting edge office building for Google, and the restoration of original building elements including exterior windows, glulam arches and central spine structure is fully on display to the employees and visitors of the Google office.

7. Acknowledgements

Google: Owner
ZGF Architects: Architects
Arup North America Limited: Structural, Civil, and MEP engineers, Fire Life Safety, Acoustics, Energy, and Lighting consultants
MATT Construction: Contractor
Ron Anthony Associates: Wood Scientist
Fire tower Engineered Timber: Glulam Repair Peer review
MyTiCon Timber Connectors: Wood Screw Manufacturer

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