Remediation of Massachusetts Institute of Technology's W33 Cage Building

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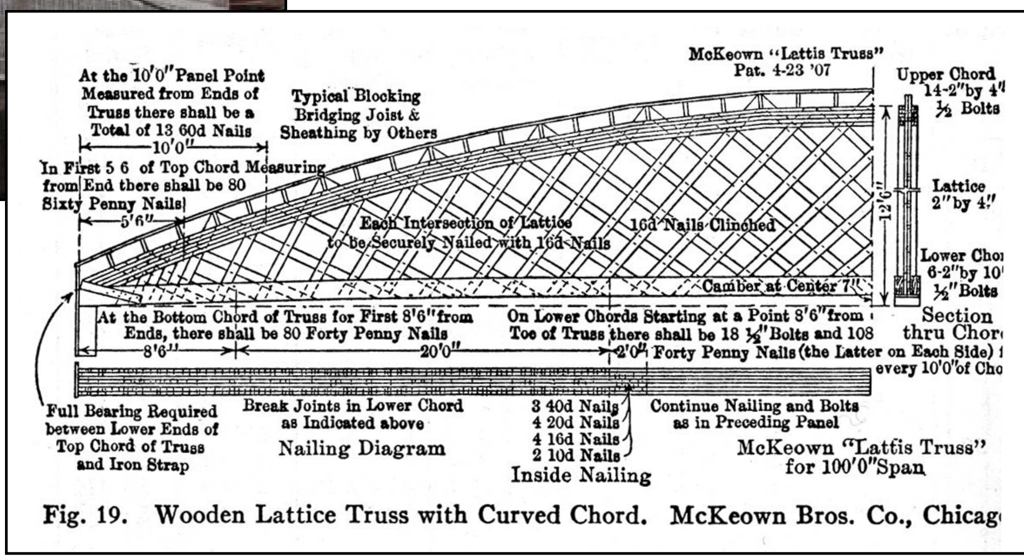
Figure 1: Double-barrel vault roof profile of MIT's W33 Cage Building.

The Massachusetts Institute of Technology (MIT) W33 Cage Building is a 32,000 square foot athletic facility, located on the MIT campus in Cambridge, Massachusetts. The facility is a functional space for many of MIT's athletic programs. The roof structure is supported by twenty-four wooden lattice roof trusses with curved top chords. The trusses, spaced at roughly 12 feet 8 inches, span 100 feet from an interior line of support towards the east and west exterior walls. The support at the exterior walls is provided by steel columns, and the interior support consists of steel transfer trusses spanning 25 feet between steel columns. Rows of curved-chord trusses spanning on both sides of the interior line of supports produce a double-barrel vault roof profile (*Figure 1*). The trusses are believed to have been salvaged from a hangar building constructed during World War II.

Condition Survey

The primary ailment observed was existing repairs at tension fractures along the bottom chord of several trusses. Generally, the existing repairs consisted of through-bolted steel sections sistered to the sides of the bottom chord. Top chords, however, appeared to be in excellent condition, with the exception of one truss, which had sustained damage and was visibly deflected (*Figure 2*).



Wooden Lattice Truss with Curved Chord. McKeown Bros. Co., Chicago, IL.

While no grade stamps were present on any of the lumber, growth characteristics, such as slope-of-grain, size and location of knots, are consistent with No. 2 grade lumber. *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber* (ASTM D 245) was used as a guideline to grade the lumber. By microscopic examination of samples, it was determined that chords were constructed of Douglas-fir and web members were western pine species.

Figure 2: Curved-chord wooden lattice trusses support the roof of MIT's W33 Cage Building.

While the observed tension failures can generally be attributed to the presence of knots and to the lack of splice connections in the bottom chord, it is also likely that these trusses were designed with fictitiously high allowable stress values assigned to the lumber. During the national emergency of World War II, the use of metals was restricted to producing machinery, tanks and ships. Thus, the national lumber supply, required to construct buildings to house manufacturing facilities, was heavily burdened. As the supply of wood dwindled, the armed forces responded by utilizing lower-grade material from limited resources to maximize the building volume. As a result, it was not uncommon for allowable stresses assigned to lumber to be arbitrarily increased 50% or greater.

Figure 3: Lower bracket transfers tension from diagonal tie-rods into horizontal tie-rods.

Analysis

Forces and consequent member stresses imposed on the truss members by snow loading were investigated in accordance with the *Massachusetts Building Code*. Using the multiple roof provisions of the *Code*, a 90 psf snow load was used at drift areas in the valleys and 15 psf at the ridges, with a linear transition between the two points.

Under full dead and snow loading, sections at bottom-chord splices (effectively three 2x12s), were found to be overstressed approximately 140% in tension alone, and nearly 200% in combined tension and bending. While bending was found to be negligible throughout the majority of the bottom-chord span, large axial forces, developed in web members near supports, imparted flexural demand on the bottom chord adjacent to heel joints. It was at these areas of relatively high flexural demand (between 3 and 10 feet from supports) that several of the observed fractures had occurred. This provided confidence that the model was accurately depicting interaction between truss members under heavy snow loading.

Another important conclusion from the analyses was that, provided it remained braced by web members and roof joists, the top chord remained adequate to accommodate large compression forces associated with full snow loading.

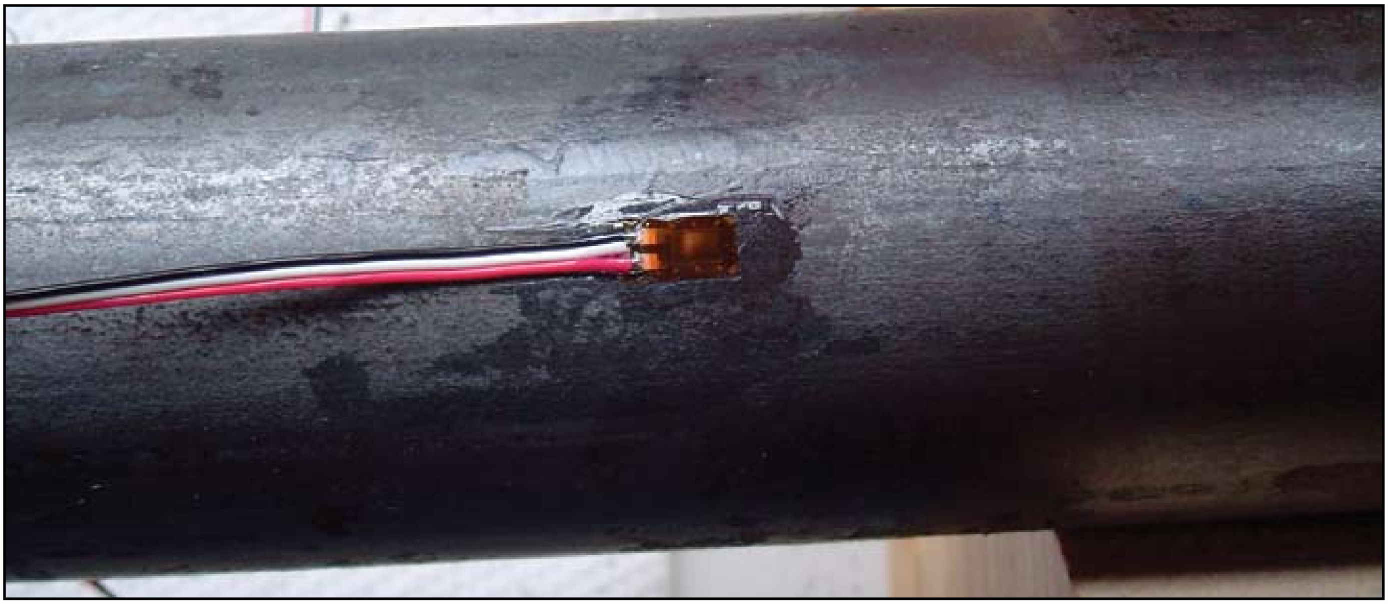


Figure 4: Electronic resistance-type strain gauge used to monitor tensioning of tie-rods.

Strengthening Scheme

As disassembly of existing repairs was deemed too risky, options for chord strengthening were limited. This was the key obstacle in detailing the strengthening scheme and led to recommending the installation of steel tension rods on each side of the bottom chord of the trusses. This an elegant means of avoiding contact with the existing repair; it also allowed for the option to post-tension the rods and remove existing dead load from the bottom chords, such that they were not overstressed during snow loading. Furthermore, since the capacity of existing repairs was unknown, the post-tensioning rods were sized to safely carry the entire lateral thrust of the compression chord under dead and live load combined.

Custom-welded steel brackets were designed to transfer load from the top chord into the new steel tension rods. Three separate assemblies were employed at each support end; 2 upper brackets to transfer load into the steel rods on either side of the truss (oriented diagonally downward), and a single lower bracket, which transferred tension from the diagonal rods into the horizontal rods which run the remaining length of the truss (*Figure 3*).

Post-tensioning

In order to get the bottom chord stresses within the allowable range under snow loading, the design concept required transferring essentially all of the dead load tension from the existing wood bottom chord to the steel rods. Tensioning of the tie-rods was monitored using electronic resistance-type strain gauges. Two gauges were adhered to opposing sides of each tie-rod. During tensioning, gauges were kept as close to mid-depth of the tie-rods as possible to avoid flexural strain (*Figure 4*).

Conclusions

The post-tensioned, steel tie-rod strengthen-ing system proved to be a time and cost effective means for strengthening the roof trusses. The result is a structurally elegant repair that successfully extends the life of the building. Recognizing the sound condition of existing top chords and heel joint assemblies allowed for minimization of labor and eliminated the need for expensive and obstructive shoring during repairs. A total of twenty trusses were repaired over the course of 10 weeks during which only limited portions of the space needed to be closed. The total project cost was around $500,000, a fraction of the cost associated with replacing the roof or constructing a new facility.▪

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