Wood Serviceability and the Unexpected

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Wood assemblies have survived for centuries when properly designed, detailed, and maintained. Examples are found throughout the world, such as temples and shrines in Japan and China dating as early as the 8th century, as well as numerous structures within the United States, including churches, barns, and bridges. Wood is a fibrous anisotropic material with strength and behavior properties that vary based on orientation to the wood grain, and which need to be accommodated in design. Although wood is susceptible to decay or fire, design approaches are available to address these hazards. In general, if kept dry (below 20% moisture content) or completely saturated (submerged in water without access to oxygen) wood may last indefinitely. Fire resistance can be achieved in heavy timber assemblies. As exposed timber chars during fire events, an insulating layer is formed that protects the underlying wood from damage. This has been demonstrated in construction techniques for over 100 years, such as in mill buildings. At present, wood systems are gaining greater code approvals with cross-laminated timber (CLT) construction for acknowledged fire resistance. As anticipated, mass-timber building height allowances of up to 18 stories tall with CLT is forecasted for the 2021 International Building Code.

Figure 1. Parson Capen house from Topsfield, Massachusetts built in 1683.

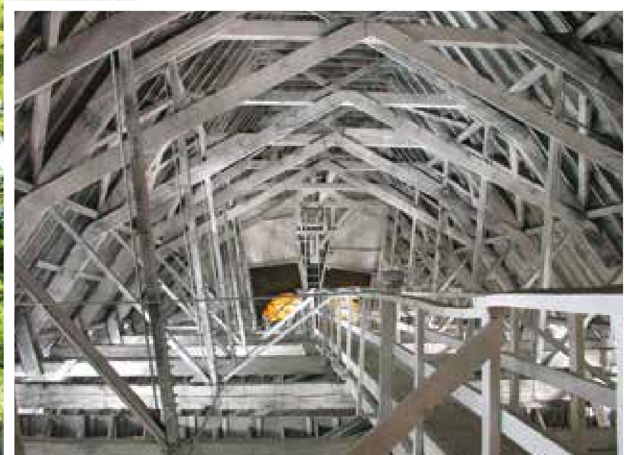

Figure 2. Attic framing from 1860s church in Chicago, Illinois.

Previous *Second Chances* articles have discussed investigation and repair methods to address wood and timber structures. In this article, we are going to review dubious treatments of wood structures as a cautionary tale and reminder that maintaining a critical perspective is essential when examining existing structures. Though unintentional, one would believe, remediation efforts may occasionally adversely affect wood serviceability and even weaken the structures that they are intending to repair.

Community Center

In the first example, we have a community center located in central Indiana. This 1960s building has load-bearing masonry walls, a wood-trussed roof, and was finished in the interior with plaster and lath ceilings and walls. The roof trusses – fabricated with dimension lumber and steel split-ring connectors – were spaced 2 feet on center, and spanned the 60-foot building width. The owners expressed concerns regarding ceiling deflections of several inches in the social hall that they attributed to snow loads. Although this was not an unreasonable hypothesis, a closer look identified other contributing factors.

Figure 3. Steel cable reinforcing introduced at roof trusses from Indiana community center.

Figure 4. Fractured roof truss members at split-ring connectors attributed to high perpendicular-to-grain loading of members.

Our investigation discovered modifications to the community center that incorporated moveable partitions within the social hall suspended from the roof structure. There was no significant cracking present in the plaster ceiling, however deflections as much as 5 inches existed, indicating that some deflections likely existed when the ceiling had last been refinished. The suspended partitions added loads to the roof trusses for which they were not designed. Strengthening efforts employed the addition of steel cables that engaged the top chords and bottom chords of adjacent trusses. This reinforcing approach to enhance the load-carrying ability of the truss bottom chords applied perpendicular-to-grain tensile stresses on the top chord members at the split-ring connectors. Wood does not provide any appreciable strength perpendicular to grain. The National Design Specification for Wood Construction (NDS) specifically states that designs that induce tension stresses perpendicular to grain shall be avoided, unless mechanical reinforcement to members is provided to resist all such stresses. In this case, no mechanical reinforcement was provided and, consequently, the cable provided to reinforce this roof structure actually pulled the top chord member connections apart over time. This resulted in multiple connection failures of the roof trusses leading to the corresponding deflections.

Administration Building

The second example brings us to an administration building located in a Chicago suburb. This building,

Figure 5. Partial view of lattice truss within the roof structure above the administration building.

Figure 6. Partial view of lattice truss within the roof structure above the administration building.

constructed circa 1910, is a single story brick masonry structure that is rectangular in plan, measuring approximately 50 feet by 150 feet. The roof structure has lattice trusses spaced 12 to 18 feet apart that span the width of the building, bearing on the exterior masonry walls.

The building, formerly a retail occupancy, was remodeled and code upgrades necessitated by the change of occupancy affected the structure and building enclosure. The remodel design team recognized that strengthening the roof structure was needed to address the potential of drifting snow loads, as significantly taller structures had been erected along the side of the building since its original construction. Building codes around 1910 did not generally recognize the need to design for drifting snow. In addition to the snow loads, dead load associated with added insulation needed to be considered if International Energy Conservation Code requirements were to be addressed. Though the weight of insulation is usually not that significant, on buildings such as this, which were typically uninsulated through time, it is particularly worth examining more closely. Added insulation in colder climates prevents the melting of snow, and can increase the likelihood of snow accumulation that may not have previously been experienced at the building.

Excessive truss deflections and localized crushing at the tops of interior partitions were observed by the owner within a year after the remodeling work had been performed. It was apparent that the crushing at the top of the interior partitions was attributed to inadequate accommodations of differential movement of the roof structure, as the partitions were built tight to the bottom of the trusses. Roof structure deflections will fluctuate over time with changes of loads and temperatures. With wooden trusses, deflections can further fluctuate with changes in relative humidity resulting in moisture content changes in the chord members. Generally, framing practices accommodate this differential movement. Our investigation revealed that another significant factor related to these deflections had to do with the truss modifications implemented during the remodeling project. In particular, supplemental 2x10s were added to the lattice truss bottom chords in an apparent attempt to strengthen them. However, this reinforcing was not continuous along the truss length and the specified bolted attachment of the reinforcement actually created holes, reducing the net cross section of the original bottom chord. The

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Figure 8. Partial view of framing above corner entrance added to the 1890s church in central Illinois. Note that the wall studs and corner framing was cut away without provision of any supplemental support.

Figure 7. Partial view of corner entrance added to the 1890s church in central Illinois.

reduced section resulted in increased tensile stresses in the bottom chords under gravity loads. Under sustained loads, these stresses can result in an increase in deflection. Other connections of this bottom chord reinforcing included welding of new steel components to existing bolts at the ends of these trusses, raising additional questions regarding the appropriateness of these repairs. Thus, this reinforcing approach actually weakened the roof structure.

Historic Church

The third example is a historic church located in central Illinois that was constructed circa 1890. This modest, balloon-framed structure is set on a masonry foundation, is rectangular in plan, and measures approximately 31 feet by 50 feet. In the early 1900s, a new building entrance was created at the corner of the original church. Building damage at the entrance attributed to a more recent motor vehicle impact initiated an assessment of the structure.

This assessment revealed that framing modifications performed to accommodate the corner entrance simply cut out the wall stud framing at the building corner without providing any support for the framing above the opening. As peculiar as this sounds, this framing seemed to perform without incident, as the horizontal plank sheathing and interior plaster finishes on the balloon-framed assembly provided the support of the building construction above it. Interior and exterior finishes had also been added to these walls, masking any distress and movement within the framing such that minor settlement of these unsupported elements would have gone unnoticed. Longer-term repair recommendations included incorporating a resolved support and load paths for the framing around the repaired entrance. But these conditions again demonstrate the unpredictable nature of prior remediation work on these structures. You don't always know what you are working with until you perform a proper assessment.

It is necessary to recognize that not all prior repairs and previous building structure interventions are properly implemented; they should not be taken for granted. Rather, it is essential to retain qualified architects and/or engineers familiar with the materials and assemblies to critically examine the structures to ensure that the repairs selected and implemented are appropriate and compatible with the structure, and provide the best and most enduring serviceable life of the structure into the future.

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