TECHNICAL NOTE: LIGHT FRAME WOOD TRUSS ROOF COLLAPSE IN MISSISSIPPI, A CASE STUDY

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Abstract. In or around 1972, an experimental building was constructed. One of the intents of the construction project was to demonstrate advancements in wood building construction design. It was value-engineered throughout. That is, its materials and systems were intended to function at or near design capacity. In 2019, part of the roof of the structure collapsed. This case study investigates two potential factors that led to the failure: stress concentration in excess of the allowable stress for 2 × 4 web members and insufficient plywood sheathing to support live loads caused by large rain events.

Keywords: Building failure, roof collapse, wood trusses, southern yellow pine.

INTRODUCTION

Building and or roof collapses are not entirely uncommon. Regardless of building materials and roofing systems, failures can occur. Often the post-failure analyses (postmortem) are protected and become discovery for insurance claims and civil, or sometimes criminal, litigation. In the case of public construction however, related documents from the time of building inception are both preserved and available. In this instance, in or around 1972, a 557 m^2 (6000 square foot), two-story, light-wood frame construction building was conceived and erected. The rectangular building is framed on a slab 8.5 m (28 feet) wide and 29 m (96 feet) long. Its purpose was for use as both office and laboratory space at the Mississippi Forest Products Utilization Laboratory, hereafter MFPUL. At that time, the MFPUL was both a separately funded state agency, with a research and service driven mission, and an academic entity within Mississippi State University. Since construction, this structure has been known as “Building 4” as it was the fourth building to be constructed as part of the MFPUL.

The first written notion of constructing this building is excerpted below by MFPUL Director, Dr. Warren S. Thompson. “As part of a new research project on timber construction, the Mississippi Forest Products Utilization Laboratory plans to erect a building that will incorporate several innovations in construction techniques. Data collected during and subsequent to the construction of this building will be of value in evaluating the applicability of the techniques used to residential and other types of light-frame construction” (Thompson 1971). This manuscript is intended in support of the building’s documented intent toward collection and dissemination of data subsequent to construction. As a public building, its conception, construction, service life, and repair history are well recorded. Of particular interest and

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focus herein is the roof structure. It was framed with dimension lumber-based flat roof trusses.

LIGHT FRAME BUILDING CONSTRUCTION

Roof framing, sheathing, and surface: The roof's framing system is light frame dimension lumber flat trusses. The trusses are placed 1.2-m (4-foot) on center. The live load design for the trusses is 0.96 kPa (20 pounds per square foot (psf)) (Thompson 1972). The trusses have a depth of 1.8 m (six feet) and a length of 8.53 m (28 feet). The exterior stud walls are 14-cm (5.5-inch) thick resulting in a clear span of 8.25 m (27 feet 1-inch). Atop the trusses is 1.9-cm (3/4-inch) thick plywood sheathing. Atop the plywood sheathing is layered rigid cellulosic insulation, approximately 10-cm (4-inches) thick at mid span. Part of the intent of this insulation is to give the roof surface a small amount of pitch, on the order of 2.5-cm (1-inch) rise across 3 m (10 feet) of length. The roofing material is built up of heavy asphalt impregnated felt paper, with tar and gravel—approximately 1-cm (3/8-inch) washed aggregate—on the surface. Dead loads are estimated in Table 1. A rendition of the truss design is provided in Fig 1. The top chord is 2×6 southern pine dimension lumber, the bottom chord and webs are 2×4 southern pine dimension lumber (Clearspan Components, Inc. 1972). The web material is specified as Number 2 grade, kiln dried.

MATERIALS AND METHODS

Intermediate Repair

In or around 2005, after approximately 33 yr in service, some truss web members failed. These failures were audibly noted by building occupants who reported periodically hearing "wood snapping" and "wood cracking" above their heads, inside and above the suspended ceiling. Inspection of the roof framing revealed that, the web members who failed were the ones who ran upward, diagonally, from the exterior southeast wall toward top chord, toward the interior of the building. The failure was due to columnar buckling in nonbraced compression web members. Figure 1 illustrates the web critical web member on the original truss blueprint. Neither decay nor overloading seemed to be factors in the failure.

Exterior inspection revealed water ponding, on the roof, over approximately half of the span of three of the trusses. That is, the ponding extended across several trusses. The trusses were shored up by reducing the span and locating supporting beams underneath the top chords of both the failing trusses as well as the trusses immediately adjacent thereto. This repair transferred a portion of the loading through an interior wall, the floor system, and down to the building's reinforced concrete slab. This repair effectively lifted up the roof to the point of being level for approximately 6.1 m (20 feet) of length near the middle of the building's 29.3-m (96-foot) long dimension, eliminated the ponding, and increased the service life of the roof system. At that time, no additional analyses were conducted and no building-wide roof repairs were considered.

Final Collapse

During the winter of 2019, occupants again noted audible wood failure inside the building's ceiling, this time toward the building's southwest end. The area of the building at issue was vacant at the time. The area was cordoned off, locked, and then

Table 1. Estimated roof structure dead loads, kilopascal (kPa) (pounds per square foot (psf)).

Material	Estimated dead load, kPa (psf)	Source
Wood trusses, 4-feet on center	0.07 (1.5)	Midwest Plan Service 1978.
3/4-inch thick plywood sheathing	0.09 (1.9)	Estimate based on approximate 61 pounds per 32-square foot sheet
Roofing, Felt, and Gravel	0.43 (9)	Hansen 1948
Combined weight of fasteners, suspended ceiling, cellulosic insulation, sprinkler system, HVAC, and utilities	0.08 (1.6)	Author's estimation
Sum	0.67 (14)	—

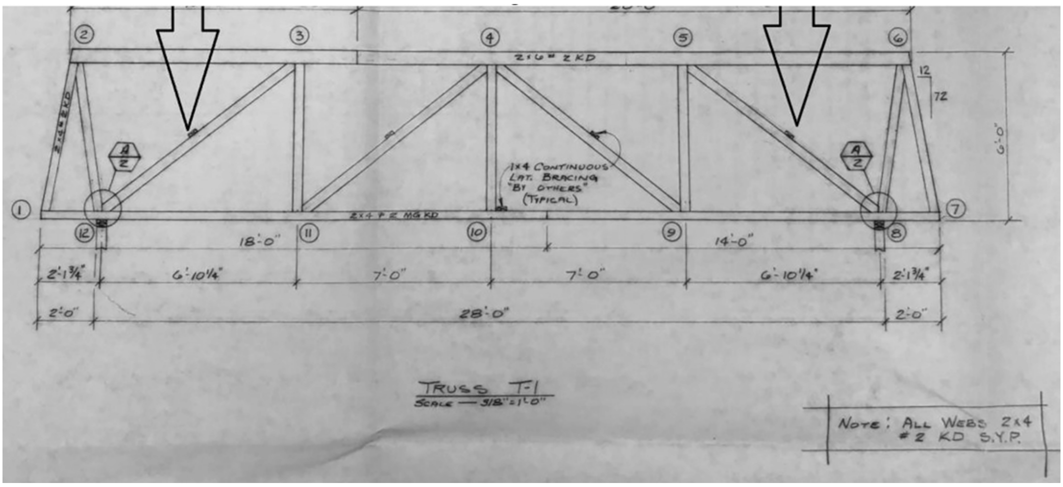

Figure 1. Original truss blueprint with the critical web member(s) denoted.

immediately inspected. The suspended ceiling tile system had come down as much as 12.25 cm (6 inches) under some of the trusses in the area of failure. Inspection of the roof truss system, by ladder from inside the building, indicated that several trusses were failing. Ultimately, these failures should likely be classified as deflection induced, that is, long and unbraced columns deflecting, which led to ponding, which exacerbated the deflection, and so on up until catastrophic failure. Common to these failures were the unbraced compression web members immediately atop the southeast wall (similar to the web members who broke and were repaired, in another area of the building, approximately 15 yr prior). Inspection of the roof showed ponding, that is, additional dead weight on the system. The pond's maximum depth was estimated at 7.5-10 cm (3-4 inches) and its area was estimated at approximately 6.7 × 6.7 m (22 × 22 feet) (Fig 2). At that stage, it appeared that catastrophic roof failure was likely imminent and there would not be time to propose and/or enact a repair. As such, the area at issue under the roof was blocked off to avoid any potential injury. That evening, during a heavy rain and wind event, approximately 8 h after the initial reports of "wood breaking" were received, the roof collapsed (Fig 3).

RESULTS AND DISCUSSION

Analysis

Subsequent to the failure, the truss plans and roof system were analyzed. Two findings are of particular interest. One finding relates to the roof's plywood sheathing and the other to the compression stress in one of the repetitive truss's web members.

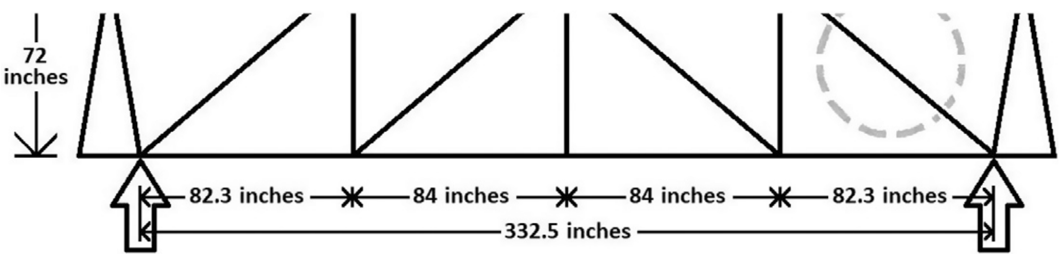

Figure 2. Truss diagram. The support reactions are shown at the centerline of the exterior support walls. The dotted circle indicates the web member who failed on multiple trusses and appears to have initiated the collapse.

Figure 3. Ponding on roof, over an estimated 6.7 × 6.7 m (22 × 22 foot) area, hours before a heavy rain and windstorm event during which the roof collapsed.

It is estimated that the dead load of the roof system is at least 0.67 kPa (14 psf). Combined with a 0.96 kPa (20 psf) live load (by design), the total design load for the structure is 1.63 kPa (34 psf). It is noted that a 7.6-cm (3-inch) deep pond equates to approximately 0.77 kPa (16 psf) live load—in perpetuity. As such, it seems reasonable to assume that, at least in some portion of the roof's area, it experienced this loading more or less in perpetuity. In contemporary allowable stress design, it is anticipated that members and or structures will only experience full design load for 10 yr, cumulatively, throughout the life of the structure. Permanent or perpetual loading requires an adjustment factor of 0.9 to bending strength.

Though stretched to its potential maximum, the 122-cm (48-inch) on-center roof spacing under 1.9-cm (3/4-inch) thick plywood sheathing is acceptable. However, to achieve 122-cm (48-inch) on-center spacing, it is recommended that the plywood be edge-blocked (Hoyle 1978). That is, 48/24-span rated plywood needs H-clips, tongue and groove, lumber/block supports, or some other manner of sharing the load between adjacent sheets to achieve the 122-cm (48-inch) rating. The time of the construction may predate H-clips and tongue and groove sheathing but the need for load sharing among panels, as noted by lumber blocking or edge blocking was noted. Absent this edge support, the maximum allowable on-center spacing for roof framing beneath the plywood is 91.5 cm (36 inches). In this case, no edge supports were noted. As such, based on the guidance from Hoyle (1978), the plywood supports were likely too far apart. That factor would seemingly create or facilitate ponding in a scalloped fashion, between adjacent trusses. This situation may have contributed to between-truss ponding and initial sagging of the sheathing and or trusses.

Postcollapse inspection of the remainder of the building's noncollapsed roof framing, revealed significant compression-induced deflection in the web member of interest. Figure 4 illustrates a series of trusses with their unbraced web members as they sit atop the exterior wall. An exemplar web member (Fig 5) was photographed in an effort to measure the deflection against a stringline. At the time of the photograph in Fig 4, there was no standing water on the roof. Thus the only loading on that truss at that time is the approximate 0.67 kPa (14 psf) dead load. At 1.63 kPa (34 psf) combined load, 9.22 m (30.25 foot) top chord length, and 1.22-m (4-foot) spacing, each truss is intended to carry 1866 kg (4114 pounds). That level equates

Figure 4. Roof section, approximately 7.3 × 8.5 m (24 × 28 feet), (7 trusses each 28-feet long) immediately following collapse.

to 202 kg per lineal meter (136 pounds per lineal foot), uniformly distributed along the symmetric truss length. As such, the estimated reaction at each exterior wall is 933 kg (2057 pounds). The load from the outermost half of the outermost 243.8-cm (96-inch) section of top chord is subtracted from this value to determine the reaction force that is pushing on the web member of interest. The top chord span above the exterior wall is approximately 243.8 cm (96 inches) (Fig 6). The loading associated with outer most 1.22 m (4 feet) (half of the chord) is transferred to the reaction wall through the shorter web member that extends from the exterior wall, slightly outward, to the top corner of the roof. That vertical reaction load value associated with that web member is estimated as 246.75 kg (544 pounds) (1.22 lineal meters [4 lineal feet] times 202.4 kg per lineal meter [136 pounds per lineal foot]). The balance of the reaction force (933 kg − 247 kg = 686 kg [2057 pounds − 544 pounds = 1513 pounds]) is transferred vertically through the web member of interest. Figure 7 illustrates this post-collapse analysis.

As measured, the unsupported length of the web member of interest is 256.5 cm (101 inches). Its vertical rise is 160 cm (63 inches), that is, 182.9-cm (72-inch) total truss height less the 8.9-cm (3.5-inch) deep bottom chord and 14-cm (5.5-inch) deep top chord. As such, with respect to the horizontal, the web member's angle theta is 38.6 degrees (sin 38.6 degrees = 63/101). At that angle, the 686 kg (1513 pound) vertical reaction force

Figure 5. Series of repetitive flat roof trusses sitting on the exterior roof wall. Of particular interest is the unbraced 2 × 4 compression web member running diagonally up from the exterior wall.

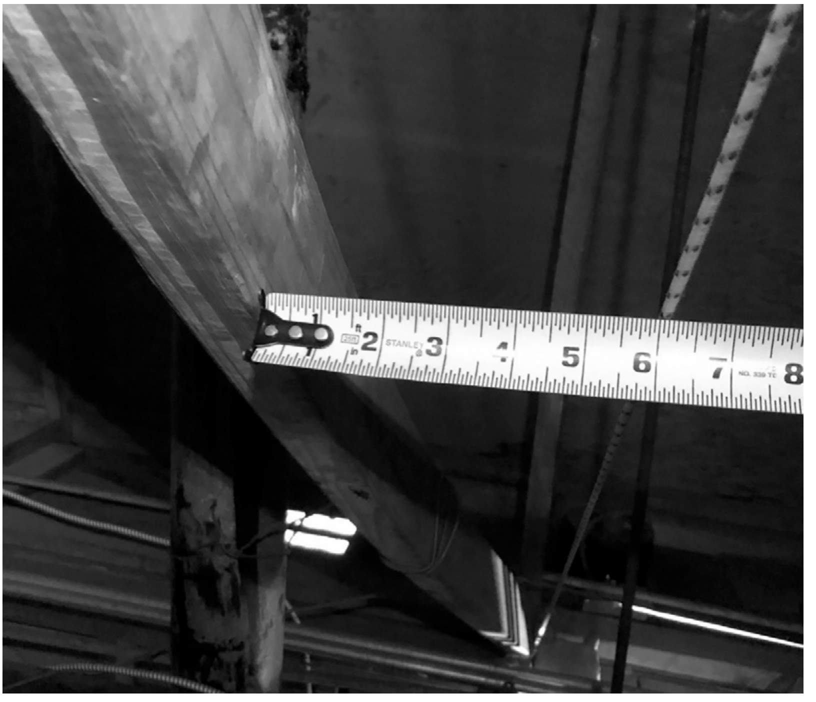

Figure 6. Exemplar web member showing 15.5 cm (6-1/8 inches) of deflection. The member is 2 × 4, #2 as called for in the truss diagram. Its unbraced length is 256.5 cm (101 inches). The truss had little or no live loading at the time of the photograph.

equates to 1100 kg (2426 pounds) of compressive force to the web member of interest. With a cross-sectional area of 33.9 cm^2 (3.8 × 8.9 cm) (5.25 inches^2 [1.5 × 3.5 inches]), the web member of interest is estimated to be routinely experiencing 3185 kPa (1100 kg/33.87 cm^2) (462 psi [2426 pounds/5.25 in^2]) of columnar compression due to total loading.

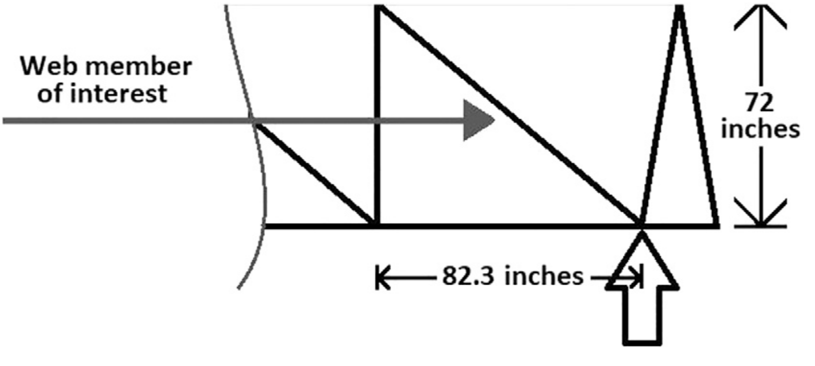

Figure 7. Determination of reaction force transferred to the web member of interest. The total vertical reaction is estimated as 933 kg (2057 pounds). The load from the outer 1.22 lineal meters (4 lineal feet) (246.75 kg or 544 pounds) of the top chord is transferred to the support wall by the outer compression web member. The balance, 686.3 kg (1513 pounds) is the vertical reaction force that the web member of interest resists.

With a 256.5-cm (101-inch) unsupported length and a least cross-sectional dimension of 3.8 cm (1.5 inches), the slenderness ratio of this web member of interest is 67.3 (ie 101/1.5). With that relatively large slenderness ratio, this web member can be approximated as a long column; that is, its allowable strength is a function of its stiffness, not tabulated parallel to grain compressive strength. Per the design values of that time period, the allowable bending stress (single member) and modulus of elasticity for this web material are 9308 and 1.03×10^7 kPa (1350 and 1.5×10^6 psi), respectively (Southern Pine Manual of Standard Wood Construction 1975). The Southern Pine Manual of Standard Wood Construction (1975) also provided guidance regarding column design. In the solid column section, it provides a formula for calculating allowable column design stress. The column design formula is

$$Fc' = \frac{(0.30E)}{\left(\frac{l}{d}\right)^2} \tag{1}$$

$$Fc' = \frac{(0.30 \times 1,500,000\ psi)}{\left(\frac{256.5}{3.81}\right)} = 99.3\ psi$$

$$= 685\ kPa$$

A further reduction of this allowable compressive stress should be taken as the loading is more or less permanent or perpetual. The correction factor for permanent loading in the mid- to early 1970s was 0.9 (Southern Pine Manual of Standard Wood Construction 1975). The same factor remains in effect today. If applied, this reduces the Fc' value to 616 kPa (89.4 psi). The Southern Pine Manual's section on columns also states "for a simple solid column, the 'l/d' ratio should not exceed 50." As such it appears that as-constructed the slenderness ratio of this web member significantly exceeds the maximum allowable slenderness ratio and that its loading is excessive.

Another way of visualizing or estimating the bending stress in the compression web members is by calculating the stress required to achieve the observed level of deflection. The deflection in the web member of interest stems from both loading and creep however the potential creep deflection is unknown. Creep aside, the analogous uniform load (pounds per inch) on the member can be calculated by the following equation (Douglas Fir Use Book 1958):

$$w = \frac{384\delta_m EI}{5l^4} = \left(\frac{384\delta_m E}{5l^4}\right)\left(\frac{bh^3}{12}\right) \tag{2}$$

where

w = the uniform load on a member
$^{\delta}$m = the maximum deflection of 15.6 cm (6.125 inches)
E = modulus of elasticity at 1.03×10^7 kPa (1500,000 psi)
b = the width of the member at 8.9 cm (3.5 inches)
h = the height of the member at 3.8 cm (1.5 inches)
I = the moment of inertia at 40.96 cm^4 (0.984 in^4) 384 and 5 are constants based on load configurations and assume a simply supported uniformly loaded beam.
l^4 = the length to the fourth power, which is 4.33 $\times 10^9$ cm (1.04×10^8 inches)

Thus, the analogous uniform load on the member required to create that level of midspan deflection is 1.19 kg per cm (6.67 pounds per inch). The maximum moment associated with this load configuration is calculated as

$$M_{\max} = \frac{wl^2}{8} \tag{3}$$

where

Mmax = maximum moment at mid span
w = the unit load per length = 1.19 kg per cm (6.67 pounds per inch)
l2 = unsupported span length squared = $101^2 = 6{,}581^2$ cm^2 (1,0201 in^2)

Thus, M_{max} = 961 N meters (8505-pound inches). By the flexure formula, one can then calculate the bending stress associated with this bending moment.

$$s = \frac{Mc}{I} \tag{4}$$

where

s = observed bending stress (kPa (psi)) = $4{,}469^2$ kPa (6482 psi)
M = Maximum moment = 961 N meters (8505-pound inches)
c = ½ the beam's depth = 0.5 * 1.5 inches = 1.9 cm (0.75 inches).
I = Moment of inertia = 40.96 cm^4 (0.984 in^4)

Thus, the estimated bending stress (s), absent any creep, to create the level deflection as observed is 4,4692 kPa (6482 psi) in a series of compression web members whose Fb values is 9308 kPa (1350 psi) (Southern Pine Manual of Standard Wood Construction 1975).

CONCLUSIONS

As constructed, the plywood sheathing appears to have inadequate framing underneath it. Without edge supports (H clips, tongue and groove, lumber blocking, etc.) as a means of sharing the load, the maximum recommended able on-center spacing for roof framing is 91 cm (36 inches). As built, the on-center spacing of the framing is 122 cm (48 inches), with not edge blocking, and is therefore beyond that recommended. Given that deflection is a cubic function of length, one can expect that the deflection associated with 122-cm (48-inch) spacing (118-cm [46.5-inch] actual span) is 2.4 times more than that associated with 91-cm (36-inch) on-center spacing (34.5-inch actual span). This situation likely led to initial water ponding on the roof between adjacent trusses.

As designed, and as constructed, the web member of interest behaves as a long column. In this type of situation, its allowable columnar stress is limited by its modulus of elasticity. The truss design calls for a (nonstructural) "1 × 4 continuous lateral bracing by others (typical)" such that each column (compression web member) would be braced to the analogous members in the trusses immediately adjacent thereto. If that bracing had been installed, and if it had acted in some type of structural capacity, in the best-case scenario, the web member of interest's slenderness ratio could have been reduced to approximately 33.7. In that case, the bracing may have raised the allowable design capacity, corrected for duration of load, to approximately 2468 kPa (358 psi). In all likelihood, this action would have prevented and mitigated the long-term deflection and avoided the premature catastrophic failure.

As built however the columns (compression web members of these trusses) are entirely unbraced. Thus, their slenderness ratio (67.3) exceeds that allowable per guidance of that period (50), per the Southern Pine Manual of Standard Wood Construction (1975). By calculation, the Fc' for these web members in service approximately 620 kPa (90 psi). As the actual columnar stress on each of these web members appears to be on the order of 3185 kPa (462 psi). As such, in service, these web members appear to be overloaded by a factor of approximately 5.

The excessive deflection in this web member of interest facilitates significant sagging in the top chord and thereby permits excessive ponding. This ponding, and its associated perpetual loading likely contributed to the collapse. At the time of the inspection, many of the roof trusses that remain in service in the building appear significantly overloaded. Based on this finding, it will be recommended that the roof system be completely redesigned and rebuilt prior to reoccupancy or that the structure be razed and rebuilt depending on the associated costs of each alternative.

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