GLULAM BEAM REPAIR/REINFORCEMENT
TINORA HIGH SCHOOL
DEFIANCE, OHIO

By: Gary W. Gray, PE and Paul C. Gilham, PE

Introduction

In January of 2004, a structural roof beam bending failure occurred in the gymnasium of the Tinora High School located in Defiance, Ohio. This beam was only one of a series of main roof beams which were manufactured of structural glued laminated timber (glulam).
Gray Engineering and Design, LLC of London, Kentucky was contacted by the owner and requested to perform a structural investigation and analysis of the failed beam and to make subsequent recommendations of possible repair scenarios.

The gymnasium had been constructed in the early 1960s and consisted of the main glulam girder beams which were curved downward by approximately 3 ft at center span. These beams spanned the 90 ft width of the gymnasium and supported the overlaying tectum roof deck panels and a ballasted roof membrane covering. Each glulam beam had the same vertical profile but each one was set at a varying elevation which provided a drainage slope of approximately ⅛" per foot of positive drainage. The beams were spaced at approximately 12'-6" on center and were 8 ¾" wide by 48" deep.

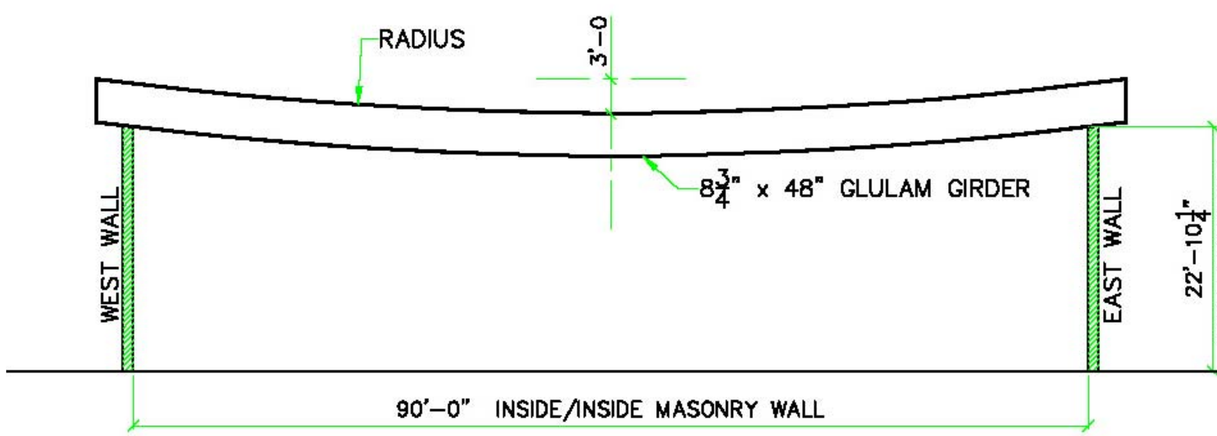

Figure 1. Profile of glulam beam girder. (Beam #5)

Based on information obtained from the original contractor, the laminated wood beams were delivered to the project site in November 1963. Upon further research, it was learned that the beams were manufactured by Timber Structures, Inc. of Portland, Oregon. The manufacturer was no longer in business, however one of the company's staff engineers was tracked down to obtain some general information about their beam production during the time these beams were produced. The beams were manufactured using west coast Douglas-fir lumber with two-inch net thickness laminations and sloped "scarf" joints for individual lumber end joints. Casein adhesive was the standard dry-use adhesive generally used during this period to bond the individual laminations into the beam assembly.

Observations of Beam Failure

One of the beams located directly above the center court line (Beam #5 – see Figure 1) had been reported to have cracked near the mid-span. Upon investigation of this beam, it was determined that the glulam beam had undergone a structural "bending failure" and was no longer adequate to safely support the roof above. The failure had initiated at a knot near mid-span at the bottom lamination and the crack had propagated upward and outward in a flat "V" pattern which is a classic type bending failure in wood beams. The crack had propagated up from the bottom face of the beam approximately 12" at the time of the structural investigation.

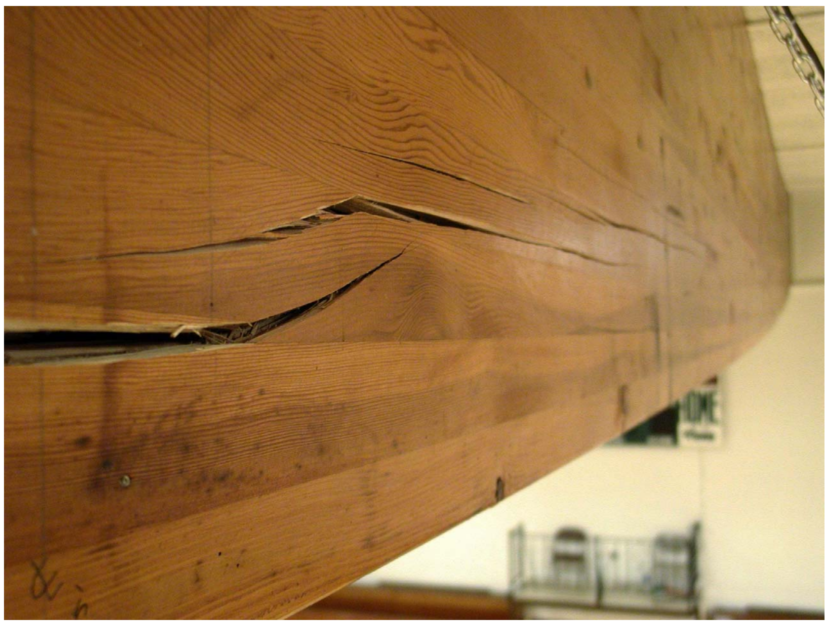

Figure 2. Failed beam due to overstress in bending.

Beam #5 had deflected approximately 3" lower than the adjacent beams and the open crack at the failure zone was approximately ¾" wide and completely through the full width of the beam. The section of beam above the crack (although overstressed) maintained enough structural integrity for a period of time to prevent a catastrophic collapse of the beam. There were no visible signs of distress at the masonry walls supporting the beam ends due to horizontal displacements resulting from the beam failure.

Prior to the structural investigation, the owner had placed a temporary column support at the center span of the beam, but it was determined that the support was not adequate to safely shore up the beam. It was further recommended that the beam be adequately shored up using two 6 x 6 x ¼" HSS steel columns to prevent any further damage to the beam and to safe guard against a beam collapse. These two temporary supports were located symmetrically 9 feet from the center of the beam to allow for direct access to the bottom of the beam at the failure zone for possible repair applications.

Pertinent field data related to the failed Beam #5 was gathered which could be used to perform a structural design analysis and determine possible repair/reinforcement methods. The beam profile was established by vertical offset measurements from the floor and each side of the beam was mapped showing all significant cracks, openings, surface checks, etc., along with recorded internal moisture content readings. An effort was also made to obtain similar detailed field data of all the other glulam girder beams so that an assessment could be made of their existing load carrying capacity. In addition, annual snowfall records (1963-2004) and climatology data were obtained from certified sources so that the history of precipitation data near the school location could be evaluated in applying live loads and load duration effects in the beam analysis. It was also made known by the owner that the original built-up roofing was replaced with a heavier ballasted roof membrane approximately 20 years ago.

Design Analysis of Failed Beam

A structural engineering design review and analysis was then performed based on all the information obtained. The building code in effect at the time of the analysis (2002 Ohio Building Code), which was modeled from the 2000 International Building Code, required a basic ground snow load of 20psf for this geographic location. The original design dead load was estimated to be 12psf along with the beam weights and miscellaneous concentrated loads from ball goals, HVAC units, etc.

The design analysis indicated that the addition of the ballast weight nearly 20 years earlier had resulted in an approximate 40% increase in dead load only bending stresses. In addition, glulam beams manufactured prior to 1970 did not utilize specific graded tension laminations, which are now required by current laminating standards. Various full-size glulam beam tests have been performed by the American Institute of Timber Construction and other agencies which have predicted an allowable stress reduction of approximately 25% for glulam beams manufactured without these specific graded tension laminations. This equates to the beams used in the Tinora High School gymnasium being overstressed approximately 24% under permanent dead loads and approximately 62% under full design load requirements (DL + SL). These beams were originally designed assuming an F_b=2400 psi (extreme fiber bending stress), however the allowable bending stress used in the investigation analysis included the recommended 25% reduction or F_b=1800 psi. Many experienced timber engineers use this approach when evaluating older glulam timber structures.

Selection of Repair Method

Based on the extent of damage sustained by Beam # 5 and the calculated overstress condition of the other glulam beams, Gray Engineering and Design recommended a repair and reinforcement method which included shear dowel reinforcing and the addition of post-tensioning cables to be utilized for both the "failed" beam and the remaining beams. This method was selected for two main reasons, the first giving consideration for the basketball court clearance to the bottom of the beams. The existing clearance was only 20 ft. ± which was only marginal at best when considering a minimum recommended clearance of 25 ft. by today's standards. Secondly, since Beam #5 was already in a "failed" state, a repair technique was needed which would restore the bending moment capacity of the beam to the confidence level needed. The post-tensioning technique provided a method with a proven track record (this system has been used effectively since the early 1980s) and the ability to apply a pre-calculated level of stress to the failed beam to obtain the bending capacity required. In addition, the owner requested that the roof design live load be upgraded to 30 psf in lieu of the code required 20 psf to protect against any future unusually heavy snow events and because of the low positive roof slope drainage (⅛" per foot) which was available.

The finished appearance of the beams also played an important role in the owner's decision. Since these beams were architecturally exposed and consisted of a stained finish, the owner wanted to preserve the beauty of the wood "look" within the roof structure and maintain the visual qualities of the wood during and after the repair process. The shear dowel and post-tensioning system provided for a total concealment of the shear dowels, with the post-tensioning cables and anchor brackets painted to blend with the stained finish of the wood beams.

Description of repair methods

Due to the nature and extent of the damage to Beam #5 and the magnitude of reinforcing to be performed, Western Wood Structures, Inc. of Tualatin, Oregon was selected to perform the design and installation of the repair-reinforcing work. Western Wood Structures has designed and performed numerous repairs to wood framed structures utilizing these techniques. It should be noted that these types of repairs should only be attempted by specialty contractors with extensive experience in the repair of large timber beams.

Shear Dowel Reinforcing

Shear dowels consist of steel reinforcing bars in epoxy filled holes drilled vertically on the member centerline. The holes are typically 3/8" larger than the reinforcing bar. For example a 1" rebar will use a 1 3/8" hole. These shear dowels serve two purposes. In any cracked beam the shear capacity is compromised. Once a beam cracks, it is not possible to glue and clamp the member back together and insure that the original shear capacity is restored. These shear dowels were developed to restore the horizontal shear capacity of a damaged beam in place. In a bending failure, the shear dowels are also used to stitch the beam back together. The allowable loads for these dowels were developed by the Weyerhaeuser Company in the early 1970s. The capacity was based on the strength of an equivalent dowel type connector and then confirmed with full scale testing. Each dowel size is assigned an allowable load. The spacing of the dowels is then determined by dividing actual shear flow by this allowable load. In wider beams, multiple rows of dowels may be used.

Post-Tensioning Reinforcing

Using this system, post-tensioning cables were installed on the beam soffit. When post-tensioned, these cables applied a compressive force to the beam. This force was applied eccentrically to the beam's neutral axis and produced a bending moment to the member. Figure 3 shows the combination of the post tensioning stresses and the applied stresses. The P/A compressive stresses and Pe/S bending stresses are due to the cable tensioning. The M/S bending stress is in the opposite sense of the cable stresses and is due to the applied dead and live loads. The resulting tensile stress at the bottom of the beam in the post tension condition must be less than the allowable tensile stress of the member.

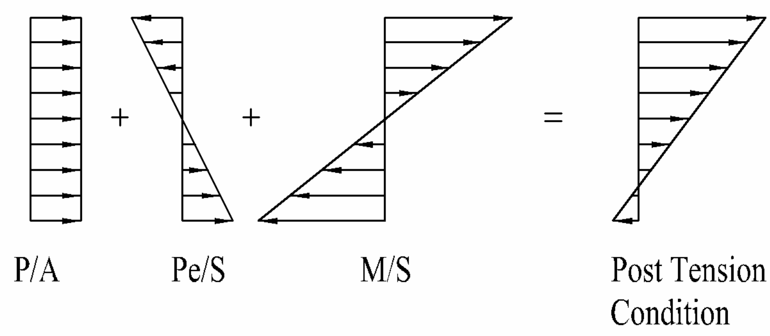

Figure 3. Combination of stress with post-tensioned system.

The cables are anchored to the beam using steel anchorage assemblies. Steel reinforcing bars are inserted into the holes drilled into the ends of the beams and the voids are pumped full of an epoxy. The allowable strength of the anchorages was determined from full scale testing. The cable anchorage is achieved using standard button type wedge anchors used in the prestressed concrete industry.

The cables used are ½"ϕ ASTM A416 Grade 270 seven wire, low relaxation, strand. This type of cable is commonly used in prestressed and post tensioned concrete construction.

There are several advantages to applying a post-tensioning system compared to adding members to the beam.

1. Decreases stress in beam. This system applies a reverse moment to the member as well as a uniform compressive stress. This added stress state reduces the tensile stresses in the tension zone of the beam.
2. The cables, anchors and pivot assemblies add only minimal loads to beam. For example, on this job, the post tensioning materials increased the applied dead loads by less than 1½%.
3. Maintains unobstructed use of floor area. A common method to repair a broken beam or increase the capacity of the roof system is simply adding a column in the span. Obviously this is not a viable repair option for a gymnasium. The addition of the post tensioning system does not impact the use of the floor area, and in this case the clearance from the floor to the bottom of the reinforcing system was only reduced by 8¼".

When the required load needed to upgrade a beam is greater than the capacity of a single cable, multiple cables and heel brackets are used. Using more than one cable increases the complexity of the design as the compressive force of subsequent cables shorten previously installed cables. Therefore the calculations must account for the beam shortening and stress relieving of the cables during installation. Cables are tensioned to a force greater than the final requirement to allow for this stress relieving. As might be expected, the sequence on cable installation and tensioning must be coordinated with the calculations.

The placement of the cable anchorages was set so that the bottom of the beam remained in tension under dead load only loading. The tension stress on the bottom face prevented buckling of the member. Therefore the combination of stresses due to the cable forces could not be greater than the bending stress due to the applied loads. At beam #5 where multiple cables were installed, the location of each anchorage had to be determined to prevent compressive stresses in the bottom of the beam.

The last aspect of the cable design was to account for the changes in cable stress due to the application of subsequent cables and the beam deflection under live loading. When a second cable is tensioned, the applied compressive stress relieves the tension in the first cable. As the beam deflects, the cable is stretched, increasing the load in cable. The added load was calculated and added to the initial load less the decrease due to subsequent cables.

Repair design results

The design of the repairs for this was performed by Terry McKee, P.E. of Western Wood Structures, Inc. McKee has been involved in timber design and repair for nearly 40 years. The bottom 6 inches or three laminations of beam #5 were severely splintered. McKee determined that these three laminations could not be counted on to carry tensile stresses and therefore would be disregarded in all strengthening calculations. This determination significantly decreased the available beam section and increased the requirement for the post tensioning system. A significant crack was also seen at 12 inches above the soffit. McKee designed shear dowels to resist the shear loads at this level. For the damaged beam on this project, the shear dowels were used both to tie the beam back together and to resist all of the shear forces in the middle 50 ft. of the beam. The beam was jacked back into place and the withdrawal strength of the shear dowels was used to stitch the beam back together. The shear at 25' each side of the beam centerline was calculated to be 18,076 lbs. The horizontal shear stress at this point is

$$(3)(18076)/(2)(346) = 78.36 \text{ psi.}$$

The shear flow was

$$78.36*8.75 = 680.8 \text{ pli.}$$

The total shear force in the damaged area was

$$680.8\text{pli}*25.05'/(2)(12) = 102{,}323 \text{ lbs.}$$

The allowable load for one 1"ϕ grade 60 bar in 1 3/8"ϕ epoxy filled hole (increased for duration of load) is

$$7767 \text{ lbs.} * 1.15 = 8932 \text{ lbs.}$$

The number of dowels required in 25 ft. each side of center line is

$$102{,}323/8932 = 11.46 \text{ – say 12 minimum}$$

The spacing used was determined by dividing the allowable load of the dowel by the maximum shear flow.

8932/680.8 = 13.11 in. o.c. – use 12" o.c.

The original beam had a design capacity of 413,280 ft. lbs when the adjustments for volume, duration of load and the absence of tension laminations were taken into account. The new design criteria with the increased dead loads and live loads required a moment capacity of 745,450 ft. lbs. The total tensile force required to provide this capacity (using an effective beam depth six inches shallower than the original beam) was 210,486 lbs. The maximum allowable tensile force for these cables is 24,000 lbs. This repair therefore required (10) ½"ϕ cables. Each anchor assembly was able to hold two cables, so five assemblies were needed at each end.

Field repair methods

The structural repair procedure is a six-step process.

1. The beams were jacked up to approximately an unloaded position. A 6 x 6 x ¼" HSS was installed on timber cribbing with a 20 ton bottle jack at the base. The jack was used to close the cracks as much as possible.

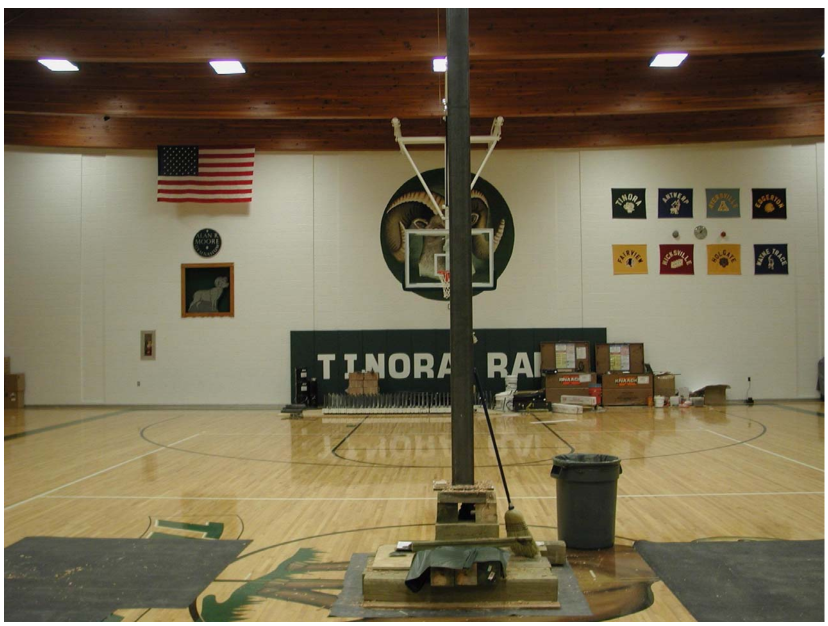

Figure 4. 6 x 6 HSS member used to jack beam to unloaded position during beam repair.

2. 1 3/8" holes were drilled vertically in the beam from the soffit to a point 6" above the highest crack. Small port holes are drilled near the bottom of the beam and just below the top of the vertical hole.

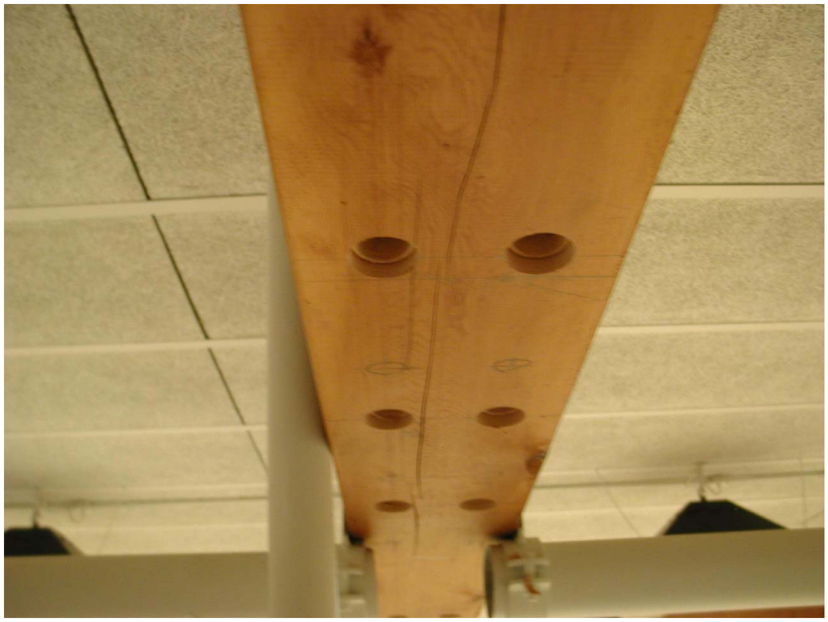

Figure 5. Holes drilled to receive Post-Tensioning Anchor Brackets.

Figure 6. Drilling the injection and exhaust ports.

3. The sides of the beam were sealed with an epoxy paste. The epoxy used in the shear dowels has a very low viscosity and if possible will travel along a void in a beam to the closest opening where it will run out. Sealing the sides of the beam prevented the liquid epoxy from leaking out of the beam.
4. Rebar is installed in the 1 3/8" hole at the soffit and the hole is covered and sealed.
5. Epoxy is pumped into the lower port and pumping continues until epoxy spills out of the upper port. A third party inspector continuously inspects this process. In this case the

owner hired an engineer from Gray Engineering to act as the inspector. To insure proper mixing and hardening of the epoxy, test samples are prepared consisting of shear dowels epoxied into GLULAM test blocks. These samples are sent to a testing laboratory where the dowels are tested in withdrawal to failure.

6. The cables are installed into the jacking assemblies and tensioned. This process begins with the innermost cables and continues outward. Five pairs of cables are required to supply the required negative moment and compression. Calibrated hydraulic jacks are used to apply the tension to the cables.

Figure 7. Calibrated Hydraulic jack.

Figure 8. Head of hydraulic jack pulling on cable.

Conclusions

One of the 90'-0" roof beams at Tinora High School in Defiance, Ohio failed due to a combination of cumulative effects of snow loads, inadequate roof drainage, and increased dead loads from the original design. The beam failed in bending near mid-span. The damage extended up 12" from the soffit. A review of the original beam lay-up and current dead and snow load requirements indicated that the beams were severely overstressed. A repair design was selected that included installation of epoxy and rebar dowels and post-tensioning using ½"ϕ high strength cables. The repairs were accomplished and the capacity of the beam was increased to meet current load requirements.

Figure 9. Completed repair/reinforcement of glulam girder beams.

About the Authors:

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