

# **Repair and Reinforcement of Glulam Beams for Tinora High School**

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## **1. Introduction**

A timber roof beam experienced a structural failure in the gymnasium at Tinora High School in Defiance, Ohio USA in January of 2004. The beam in question was manufactured of structural glued-laminated timber (glulam). An onsite structural investigation was performed and an analysis of the failed beam was completed. It was determined from this investigation and analysis that the failed beam required structural repair and that the rest of the beams required strengthening. The analysis indicated that the undamaged beams did not meet the current code requirements for strength. A repair and strengthening program was chosen to make the repairs to the failed member and upgrade the remainder of the beams. This repair method involved the installation of reinforcing steel shear dowels to resist shear forces and high strength steel post-tensioning cables to resist the bending forces.

## **2. Project Overview**

The owner contacted Gray Engineering and Design, LLC of London, Kentucky and requested a structural investigation and analysis of the failed beam and recommendations for the repair of the structure. Western Wood Structures, Inc. of Tualatin, Oregon was hired to design the repair scheme and perform the beam repair and upgrades.

### **2.1 Roof Layout**

The gymnasium had been constructed in the early 1960's and consisted of the 27.4m long main glulam girder beams that had a reverse camber (or sag) of 914mm at center span. These beams 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 1% of positive drainage perpendicular to the length of the beams. The beams were spaced at approximately 3.81m on center and were 222mm wide by 1219mm deep.

### **2.2 Original Beam manufacturing methods**

The laminated wood beams were manufactured in 1963 by Timber Structures, Inc. of Portland, Oregon. The manufacturer has subsequently gone out of business, however, some general information about their beam production during the time these beams were produced was obtained from a former company engineer. The beams were manufactured using west coast Douglas-fir lumber with 51mm net thickness laminations. End joints in the individual laminations were made using a single sloped "scarf" joint. Casein adhesive was the standard dry-use adhesive used during this period for face bonding the individual laminations into the beam assembly.

### 3. Observations of Beam Failure

The owner reported that the beams directly above the center court line cracked near the mid-span. The onsite investigation confirmed that the glulam beam had experienced a structural “bending failure” and was no longer adequate to safely support the roof and was in fact in danger of a structural collapse.



Figure 1. Failed beam due to overstress in bending.

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. (See Figure 1) The crack had propagated up from the bottom face of the beam approximately 300mm at the time of the structural investigation. The beam had deflected approximately 76mm lower than the adjacent beams and the open crack at the failure zone was approximately 20mm wide and completely through the full width of the beam. The undamaged portion of the beam above the damage maintained enough structural integrity to prevent a catastrophic collapse of the beam. Inspection of the masonry walls at the beam bearing locations revealed no signs of distress 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 recommended that the beam be adequately shored up using two 150x150x6 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 2.74m 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 was gathered which could be used to perform a structural design analysis and determine possible repair/reinforcement methods. The curve of the beam was determined by vertical offset measurements from the floor. Each side of the beam was mapped showing all significant cracks, openings, surface checks, etc., along with recorded internal moisture content readings. Similar detailed field data was obtained for all of 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. The owner informed Gray Engineering and Design that the original built-up roofing was replaced with a heavier ballasted roof membrane approximately 20 years ago.

#### **4. Design Analysis of Failed Beam**

The information collected was used to perform a structural engineering design review. The 2002 Ohio Building Code was the governing building code in effect at the time of the analysis. This code required a basic ground snow load of 0.96 KPa for this geographic location. The original design dead load was estimated to be 0.57 KPa plus beam weights and miscellaneous concentrated loads from ball goals, HVAC units, etc.

The design review indicated that all of the beams in this roof beams were significantly overstressed. The addition of the ballast weight nearly 20 years earlier had resulted in an approximate 40% increase in dead load only bending stresses.

Another factor that affected the design review was that glulam beams manufactured prior to 1970 did not utilize specific graded tension laminations. These tension laminations are now required by current laminating standards. The American Institute of Timber Construction and other agencies indicate that an allowable stress reduction of 25% is required for glulam beams manufactured without these specific graded tension laminations. These beams were discovered to be overstressed approximately 24% under dead load only and approximately 62% under full design load requirements (DL + SL). These beams were originally designed assuming an extreme fiber bending stress of 16.5 MPa however the allowable bending stress used in the investigation analysis included the recommended 25% reduction or 12.4 MPa.

#### **5. Selection of Repair Method**

Due to the significant loss of capacity of the failed beam and the calculated overstress condition of the remaining beams, Gray Engineering and Design recommended a repair and reinforcement method which included shear dowel reinforcing and the addition of a post-tensioning system to be utilized for both the failed beam and the remaining beams.

These methods were selected for two reasons. First, the basketball court clearance to the bottom of the beams was only 6.1m ±, which was significantly less than the recommended clearance of 7.6m by today's standards. The post-tensioning system reduced the clearance by only 200mm. Secondly, since the failed beam was already severely damaged, a repair technique was needed which would restore the bending moment capacity of the beam to the current code level forces. The post-tensioning technique provided a method with a proven track record (this system has been used effectively since the early 1980's) 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 1.5MPa to protect against any future unusually heavy snow events and because of the low positive roof slope drainage (1%) that was available.

The owner was also very concerned about the finished appearance of the beams. These beams were exposed to view and had a translucent stain 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 were painted to blend with the stained finish of the wood beams.

## **6. Description of repair methods**

Due to the nature and extent of the damage to failed beam 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 a specialty contractor with extensive experience in the repair of large timber beams should be employed to perform these types repairs.

### **6.1 Shear Dowel Reinforcing**

The shear capacity of a member can be restored or increased by installing shear dowels. Shear dowels consist of steel reinforcing bars in epoxy filled vertical holes. The holes are typically 10mm larger than the reinforcing bar. For example a 25mm rebar will use a 35mm 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. Secondly, 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.

### **6.2 Post-Tensioning Reinforcing**

With a beam post-tensioning system, cables are installed on the beam soffit. A post-tensioning force is applied, which induces an eccentrically located compressive force to the beam. The eccentricity produces a bending moment in the member. Figure 2 shows the combination of the post tensioning stresses and the applied stresses. The  $P/A$  compressive stresses and  $P_e/S$  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.

The cables are anchored to the beam using steel anchorage assemblies. These anchorage assemblies have reinforcing bars welded to them that are inserted into the holes drilled into the beam soffit at the ends of the beam. After the anchorage assemblies are installed, the holes are pumped full of 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 12.7mm $\phi$  ASTM A416 Grade 270 seven wire, low relaxation, strand. This type of cable is commonly used in prestressed and post tensioned concrete construction.

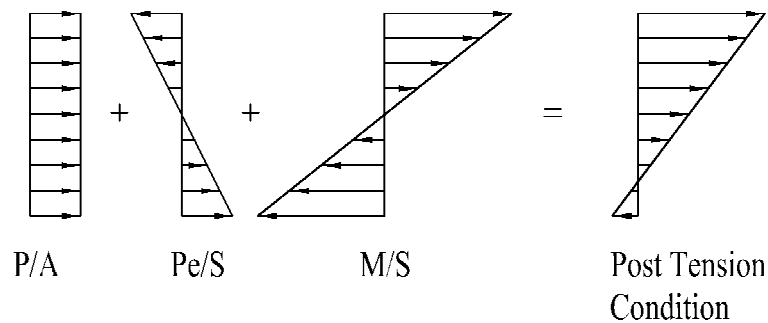


Figure 2. Combination of stress with post-tensioned system.

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

1. 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%.
2. 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 210mm.
3. 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.

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 in the field 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 dead loads. At the beam over mid-court 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 the beam deflection under live loading. 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.

### 6.3 Repair design results

The design of the repairs for this was performed by Western Wood Structures, Inc. The bottom 152mm inches or three laminations of beam #5 was 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 305mm inches above the soffit. Shear dowels were designed 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 15.25m of the beam. The beam was jacked back into place using the HSS column and a hydraulic jack. The withdrawal strength of the shear dowels was used to stitch the beam back together. The vertical shear at 7.63m each side of the beam centerline was calculated to be 8216kg. The total shear force in the damaged area was 46,718.49 kg. Twelve 25mmf shear dowels were required to resist this force. The dowels were spaced at 300 mm on center.

The original beam had a design capacity of 57,138 kg-m when the adjustments for volume, duration of load and the absence of tension laminations were taken into account. The new design criteria with the increase dead loads and live loads required a moment capacity of 103,062 kg-m. The total tensile force required to provide this capacity (using an effective beam depth 152mm shallower than the original beam ) was 95,475 kg. The maximum allowable tensile force for these cables is 10,886 kg. This repair therefore required (10) 12mm $\phi$  cables. Each anchor assembly was able to hold two cables so five anchor assemblies were needed at each end.

### 7. Field repair methods

The structural repair procedure is a eight-step process as follows.



Figure 3. 152 x 152 HSS member used to jack beam to unloaded position during beam repair.

1. The beams were jacked up to approximately an unloaded position. A 152 x 152 x 6 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. See Figure 3.
2. 35mm holes were drilled vertically in the beam from the soffit to a point 152mm above the highest crack. See Figure 4.
3. Small port holes are drilled near the bottom of the beam and just below the top of the vertical hole
4. 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



Figure 4. Holes drilled to receive Shear Dowels.



Figure 5. Drilling the injection and exhaust ports. Epoxy paste is used to seal the sides of the beam.



Figure 6. Cables are post-tensioned with hydraulic jack.

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.

5. Rebar is installed in the 1 3/8" hole at the soffit and the hole is covered and sealed. The anchor assemblies are installed at the ends of the beams.
6. Epoxy is pumped into the lower port at both the shear dowels and anchor assemblies. 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.
7. 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. See Figure 6.
8. With the structural repairs completed, the owner hired a finish painter to stain the beams and blend the new color into the existing color. Neither the epoxy paste or the plugs are noticeable from the floor.

## 8. Summary

One of the 27.4m 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 300mm 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 12mm $\phi$  high strength cables. The repairs were accomplished and the capacity of the beam was increased to meet current load requirements.



Figure 7. Completed repair/reinforcement of glulam girder beams.