Timber Repair saves College Gymnasium from the Wrecking Ball

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Introduction

After several attempts to repair the glued-laminated (glulam) arch-frames at the Dana Hall Gymnasium failed, the Canton College of Technology in upstate New York had decided to demolish the building. The college received a bid of $2.35 million to perform the work. But before they pulled the trigger on the demolition, consultants for the college decided to try one more search in hopes of finding a viable solution to the problem. The search led to Western Wood Structures, Inc. of Tualatin, Oregon who had previously repaired several glulam beams for the Tinora High School in Defiance, Ohio.

The gymnasium framing consists of 18 glulam arch frames spaced 15’-0” on center and span 108’-5¾”. Two Tudor arch halves are separated by and support a center beam. The connection between the Tudor arches and the beam is a moment resisting connection which forms a two-hinged frame. The center beam is a pitched and cambered beam. The legs of the Tudor arches are sloped inward 7.4 degrees. (See Figure 1)

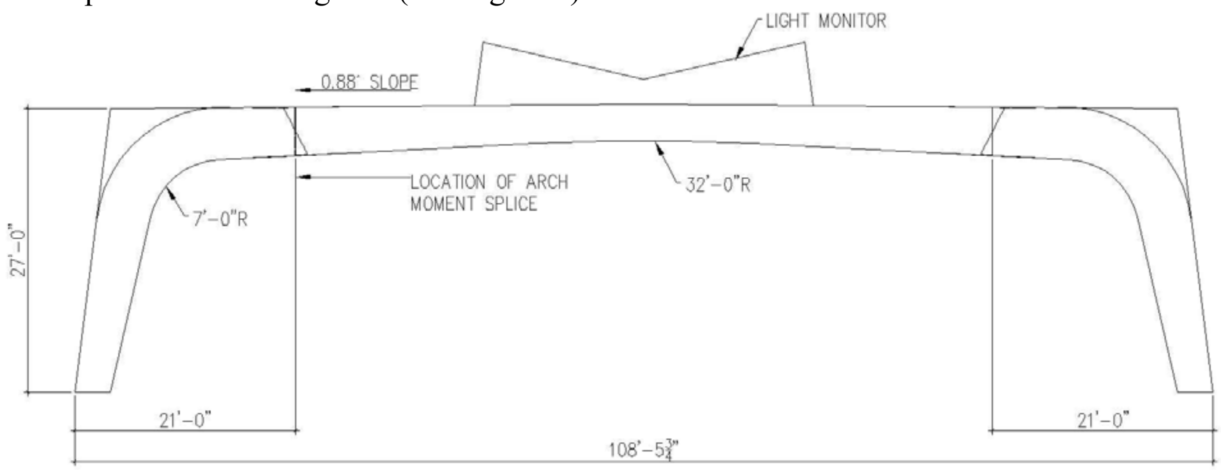

Figure 1. Configuration of Arch Frame.

Historical Background

The building was completed in 1969 but cracks were noticed at the moment splice shortly after the building was opened. A series of inspections were performed and it was determined that the cracks were not structurally significant as long as they didn’t propagate. Each of the 18 arches had developed the splits in the same location which confirmed that there was an inherent issue with the design. The cause of the splits was identified by Robert Kaseguma of Unadilla Laminators in 1975. These arch-frames utilized a hidden moment connection consisting of top and bottom plates lagged into the members and a “Z” hanger that acted like a hinge connector. The top and bottom plates transferred the axial loads associated with the bending moments and the “Z” hanger transferred the shear loads from the center beam to the arch section. (See Figure 2) The downfall of this connection was that it did not allow the wood to shrink and swell with changes in the moisture content. To understand this, consider the center beam portion. It is supported on its base by the bottom plate of the “Z” hanger. When the wood shrinks, the member will shrink toward this bottom plate. However, the lag bolts in the top splice plate and the stiffness of this top plate prevent this movement from occurring inducing tension perpendicular-to-grain stresses in the member. These perpendicular-to-grain stresses exceeded the strength of the wood and the split developed. The arches are approximately 56 inches deep at this point so even a small change in moisture content will result in a significant amount of shrinkage.

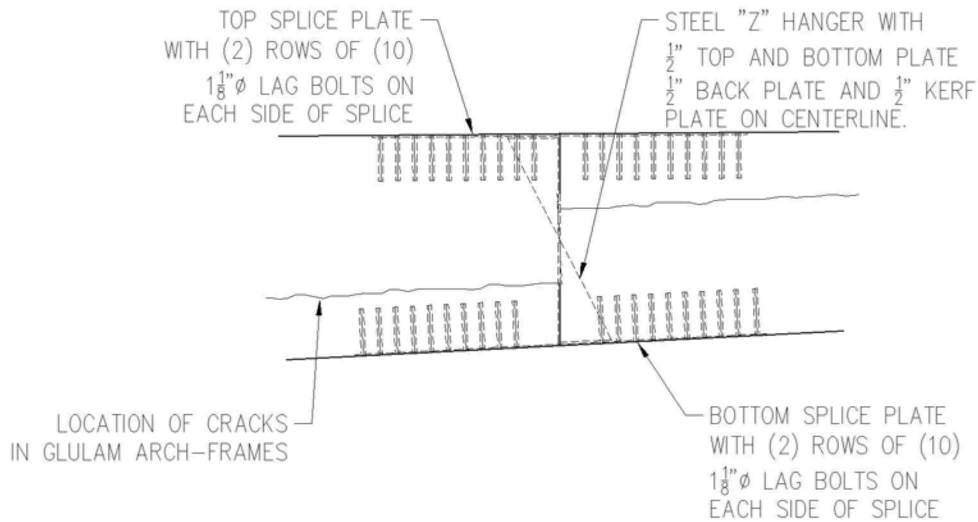

Figure 2. Moment Splice Layout.

The cracks in the arch members were first noticed in 1971. After his inspection in 1975, Kaseguma suggested that the cracks were not structurally significant since the remaining sections should be adequate to carry the shear forces. However, he recommended that the splits should be monitored to make sure the splits did not progress to the point where they would be structurally damaging. The building experienced heavy snow load in 1998. In 2006 college maintenance personnel noticed that the cracks had propagated and opened significantly. An inspection by a local engineer concluded that the arches were "substantially compromised" and that a repair was needed.

An attempt was made to repair the most heavily split arch frame using 1"ϕ x 48" lag screws from the bottom of the arch. The intent was to close the split and pump epoxy into the damaged area to restore the shear capacity of the member. Shortly after these lags were installed, a new split occurred at the top of the new lag bolts. A second attempt was made to close the split using two steel angles attached to the arch near the ceiling and an HSS section placed perpendicular to the arch at the soffit. Threaded rods were installed vertically through the horizontal leg of the angle and through the HSS to clamp the arch back together.

After the lack of success of these trials, the university considered a scheme of installing steel frames adjacent to the arches to support the roof loads. The estimated cost of this scheme was found to be excessive so the college began to explore demolishing the building. Initial demolition estimates were $2.35 million.

An internet search led the college to Western Wood Structures, Inc. (WWSI) of Tualatin, Oregon. WWSI has been specializing in the design, installation and repair of timber structures for most of their 42-year history. WWSI Chief Engineer, Paul C. Gilham, P.E., S.E., inspected the building in March of 2009. College facilities personnel made the previous reports and repair designs available to Mr. Gilham during this inspection. It became evident that previous repair methods addressed the existing splits but did not remove the cause of the splits, i.e. the moment splice plates and the “Z” hanger. As long as these splice plates and “Z” hanger were intact, the arch member would experience tension perpendicular to grain stresses. It was determined that the previous repair attempts were able to close the splits but the internal stresses found the next weak link and the splits re-appeared.

The repair scheme needed to address these perpendicular-to-grain stresses while still transferring the shear forces from the center beam to the arch member. To do this the “Z” hanger had to be cut in two while in place and a new shear connection had to be installed on the outside face of the member.

Upgrade to Current Code Requirements
Additionally, the college requested that the repair scheme upgrade the capacity of the arches to meet the 2007 Building Code of New York State (BCNYS) snow load requirements. Prior to the 2007 BCNYS, the design snow load for Canton was 40 psf. The 2007 BCNYS specifies a ground snow load of 60 psf. Using an importance factor, I_s, of 1.1, an Exposure factor, Ce, and a thermal factor, Ct, equal to 1.0, results in a roof snow load of 46.2 psf. Additionally, the effects of drifting snow adjacent to the light monitors and a buildup of snow in the well of the light monitor was to be considered. A uniform dead load 21 psf was used. This loading is shown in Figure 3.

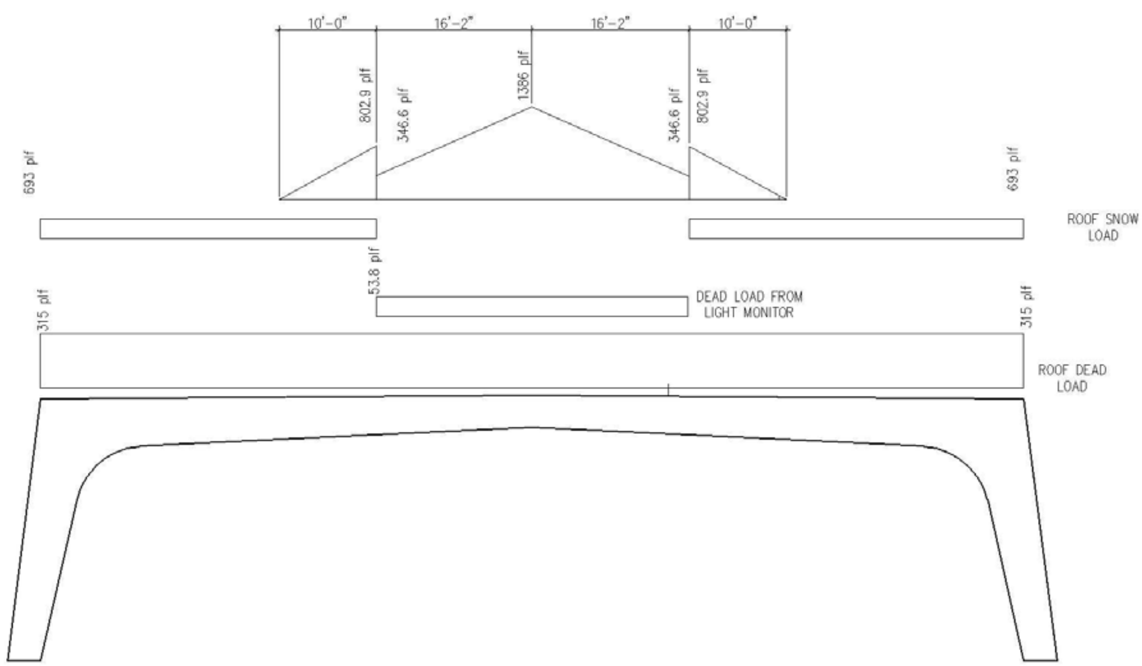

Figure 3. Loading diagram at arch.

A structural analysis was completed with the new loading criteria to determine where the arch-frames needed to be upgraded. One of the first challenges of this analysis was to determine the proper allowable bending stress for the arches. The governing glulam standard at the time of fabrication was the “Standards for STRUCTURAL GLUED LAMINATED MEMBERS Assembled with WWPA GRADES of Douglas Fir and Larch Lumber.” This document included layups for members with allowable bending stresses equal to 2400 psi and 2600 psi. The arches were manufactured by Timber Structures, Inc. of Portland, Oregon in 1969. Timber Structures commonly used the 26f grade in its manufacturing of these large members. However, a comparison of the layups used to produce the 26F grade with today's 24F-V4, indicates that the 2600 psi bending stress was unwarranted. To further compound the problem, glulam bending members manufactured prior to 1970 did not utilize tension laminations in the tension zones. AITC Technical note 26 – "Design Values for Structural Glued Laminated Timber in Existing Structures" recommends using a 25 percent reduction in bending stresses to account for the lack of these tension laminations. It has been the policy of Western Wood Structures, Inc. to use a bending stress of 2400 psi with the 25 percent reduction to obtain an allowable bending stress of 1800 psi when analyzing these existing structures.

Analysis of Center Beam

The center beam is a pitched and tapered curved beam. The top slope of the roof is 0.88 degrees. The tapered section and change of direction of these members at the crown causes an increase in the bending stress. The design method for determining the actual bending stresses for these types of members is specified in the AITC Timber Construction Manual (TCM). However, this procedure was developed for members with a minimum top slope of 2.5 degrees. This procedure does not include members with combined axial and bending stresses. Finally, the research used to develop these methods only considered members that were deeper at the crown than at the supports. To account for these deviations from the method specified in the TCM, the arch frame was analyzed using the finite element method (FEM). The bending stresses given in this analysis were compared to those derived from a straight frame analysis. The bending stress factor, K_θ, for this member was determined by dividing the bending stress found in the FEM analysis by the bending stress found in the frame analysis. K_θ was determined to be equal to 1.14 from this analysis. Using this value, the beam was found to be overstressed 125.4 percent in combined bending plus compression on the bottom or tension side of the beam. (See Figure 4)

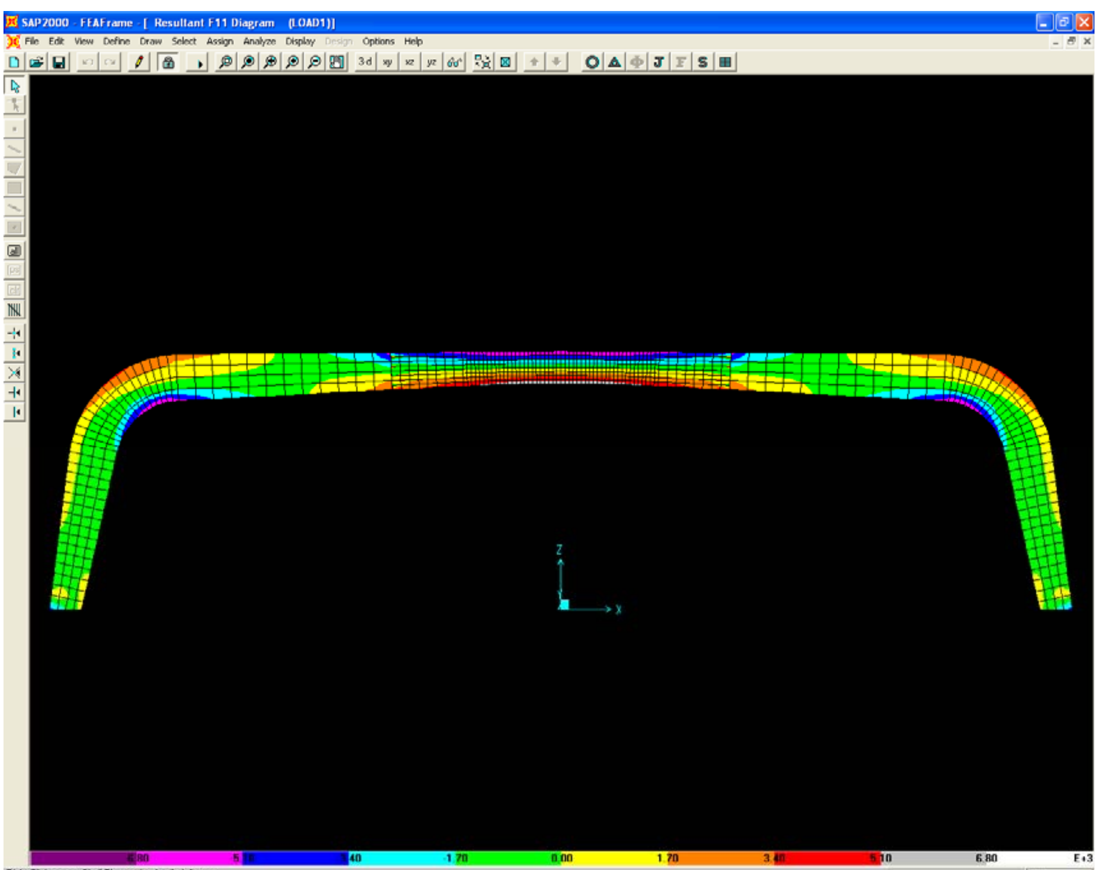

Figure 4. Results of FEA of Arch-Frame.

A separate FEM analysis was prepared to confirm the choice of material properties for the finite elements. This analysis used a pitched and tapered beam that fit well within the parameters of the TCM. The K_θ value determined from this analysis agreed closely with the value in the TCM confirming the choice of material properties use in the analysis.

The results of these analyses indicated that the arches were deficient in bending at the haunch and at midspan and that the top plate and lag bolts in the moment splice were insufficient to transfer the tension due to bending at the splice.

The repair/upgrade design needed to address these deficiencies as well as transfer the reaction from the center beam to the arch at the splice. Additionally, the repair scheme needed to restore the shear capacity of the members at the splits.

Contracting Methods

Because this project was publicly funded, the college was required to advertise this project to the general public. The college chose to advertise a design/build type of contract. Each bidder was required to design a repair scheme that addressed all of the deficiencies found during the inspections and provide increased load carry capacity capable of meeting the current code requirements. The specifications listed minimum qualifications for bidders. To qualify, a contractor needed a minimum of five years experience in the inspection, evaluation and repair of timber structural systems, members and connections. The inspections were to be performed by personnel qualified through training and experience under the supervision of a professional engineer licensed in the state of New York. The job was awarded to Western Wood Structures, Inc. who submitted a bid of $1,102,010.00

Upgrade and Repair Methods

Four types of repair/upgrade methods were used to make this repair:

1. Post-tensioning members overstressed in bending
2. Adding additional laminations to members overstressed in bending,
3. Installing shear dowels to restore the shear capacity of the members and,
4. Adding side plates to transfer the reaction of the beam to the arch section.

Upgrade of Arch Haunch

At the arch haunch, the members were overstressed by 8.5 percent in bending. The bending stress at the haunch results in compression on the inside face and tension at the outside face of the arch. Due to the curvature of the member, it is not practical to relieve the tensile forces due to bending with a post-tensioning system. Therefore the upgrade required adding four additional 5/8” laminations to the soffit of the arch. A profile of the arch curvature was prepared and sent to a laminator who made full-length billets. (See Figure 5) These billets were then drawn up tight to the arch with clamps and hydraulic jacks and attached with a structural adhesive and screws. (See Figure 6)

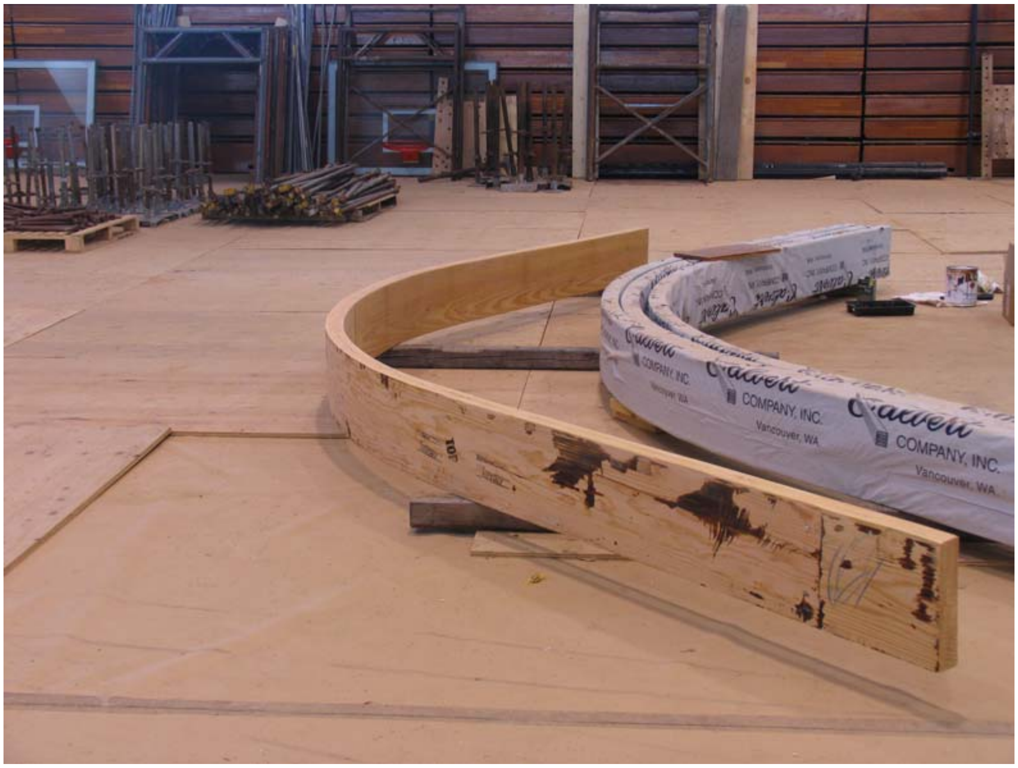

Figure 5. Glulam billets for reinforcing the arch haunch.

Figure 6. Glulam billets attached to the arch haunch.

Upgrade of Center Beam

As mentioned previously, the center beam was overstressed 125.4 percent in combined bending and compression at midspan. A cable post-tensioning system was used to reduce the tensile stresses in this location. Two cables were installed on the beam soffit and post-tensioned to

12,000 lbs. each. The cables are located with an eccentricity from the beam centerline, and induce compression and negative bending into the member. The compression and bending stresses due to the post-tensioning force are summed with the bending and compression in the member due to applied loads to determine the net bending stress in the member. This bending stress must be less than the allowable bending stress on the tension side of the beam. Figure 7 shows the summation of these stresses graphically.

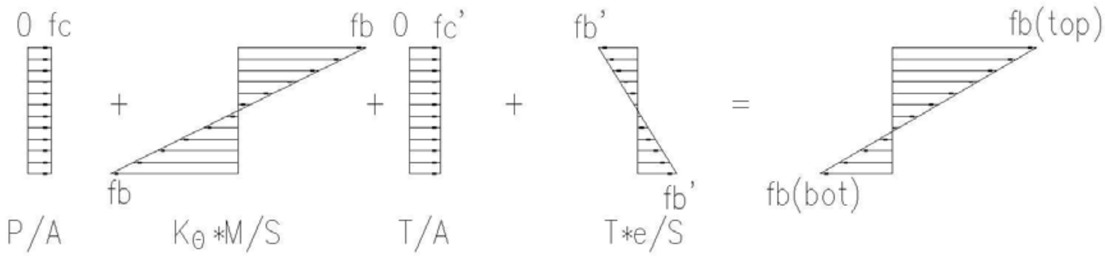

Figure 7. Summation of applied stresses and post-tensioning stresses

The allowable bending stress on the bottom side of the beam is equal to the tabulated bending stress, F_b, multiplied by the volume factor, C_V, and the load duration factor, C_D. The allowable bending stress on the top of the beam is simply the tabulated bending stress multiplied by the load duration factor since the lateral stability factor, $C_L = 1.0$ for a fully supported member. Figure 8 shows the post-tensioning cables with their anchors on the bottom of the beam.

Figure 8. Post tensioning cables attached to beam soffit.

Cable post-tensioning was also used to reduce the tension force on the top moment splice plate. The capacity of the splice plate and lag bolts was 48,688 lbs. With the new loading requirements, the applied tension force was 55,600 lbs. Cables were installed on each side and post-tensioned to 5000 lbs. each to reduce the force in the splice plate.

Center Beam Shear Transfer to Arches

The repair procedure called for splitting the "Z" hanger at the middepth of the splice. This eliminated the ability of the splice to transfer the shear due to the reaction of the center beam to the arches. Steel side plates were designed to replace the "Z" hanger. These plates used three rows of four 1"ϕ machine bolts loaded perpendicular to grain to transfer the 43,432 lb. reaction from the center beam to the arch. These bolt groups were centered 8" on either side of the splice location. The rotation of the side plate caused by the eccentricity of the bolt groups is resisted by one row of five 1" ϕ bolts at the top and bottom of the plate. These bolts are loaded parallel to grain. The holes for these "rotation" bolts are slotted vertically to eliminate splitting of the member due to shrinkage between these outer rows of bolts and to limit the resistance of these bolts to parallel to grain loads only. The bending moment at the connection is resisted by the existing top and bottom plates and lag bolts. (See Figure 9)

Figure 9. Steel side plates used to transfer reaction from center beam to arch section. The paste type of epoxy used to seal the member at the original split does not receive the stain and can be seen as a light line at the bottom of the plate. The post tensioning cable for the top plate is seen next to the timber decking.

Repair of Splits

Finally the split members were repaired by installing epoxy shear dowels through the depth of the member. These dowels consist of reinforcing bars inserted into vertically drilled holes which are subsequently pumped full of epoxy. These dowels serve to restore the shear capacity of the member and reinforce the member against future splitting. Two 54" long shear dowels were installed immediately adjacent to the splice location on each side and two rows of dowels were installed along the length of the member 9” on center for the length of the split plus six inches. The sides of the member are sealed with a paste type epoxy and the dowels are then pumped full of a low viscosity epoxy. An injection port is drilled into the side of each vertical hole near the bottom of the member and an exhaust port is drilled into the top of the member. The epoxy is pumped into the injection port until it fills the hole and exits through the exhaust port. The epoxy fills all of the voids in the member as it rises in the vertical holes. Figure 10 shows one of the drill bits used to drill a 54” hole in the arch member and Figure 11 shows two workmen pumping epoxy into the arch.

Figure 10. WWSI jobsite superintendent, Ray Kennedy with 54 inch drill bit used to drill for epoxy dowels.

Figure 11. Workmen pump epoxy into arch member. The member is completely sealed to prevent the epoxy from leaking while allowing the epoxy to fill all of the voids in the member.

Rendering the "Z" Plate ineffective

Once the side plates are installed, the "Z" hanger is cut in half using a rebar cutting bit. This procedure removes the restraint in the member that was causing the splitting. 1½"ϕ rebar cutting bits were used to core out a 1½" hole at the mid-depth of the member which effectively separated the top of the Z hanger from the bottom. (See Figures 12 and 13) Additional holes were drilled through the kerf plates and the kerf plates were completely separated using a reciprocating saw.

Figure 12. Cutting the "Z" hanger in half with rebar cutting bit. The workman on the left is using two framing squares to align the drill bit. It took just over an hour to core through the ½" x 10" back plate of the "Z" hanger.

Figure 13. Core removed from arch at "Z" hanger. The center kerf plate is at the center of the member.

Figure 14. Frames with structural repairs complete, sanded and stained.

Once the structural repairs were completed, the arches were sanded smooth and stained. The repair procedure required four months to repair 13 arch-frames. This repair procedure saved the college more than $1 million, when compared to the price of demolition. This repair scheme also preserved building space desperately needed by the college.

Summary

Several repair and upgrade methods were employed to repair and upgrade a series of glulam arch-frames at the SUNY College of Technology in Canton, NY. Post-tensioning systems or additional laminations were installed where the members were deficient due to bending stresses. Shear capacity was restored by the installation of rebar shear dowels in epoxy-filled holes. Finally, the cause of the original splitting was removed by separating the “Z” hanger at the mid-depth of the moment splice and providing side plates to transfer the reaction of the center beam to the arch members.

References:

“Standards for STRUCTURAL GLUED LAMINATED MEMBERS Assembled with WWPA GRADES of Douglas Fir and Larch Lumber.” 1966 Edition. Western Wood Products Association. Portland, OR

“AITC Technical note 26 – Design Values for Structural Glued Laminated Timber in Existing Structures” 2007. American Institute of Timber Construction, Centennial, CO

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