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Structural Report

Trinity Episcopal Church
120 Broad Street
Claremont, NH

22 October 2025

The Rev. Canon Kelly Sundberg Seaman and Senior Warden Gale Delaney have retained Annette Dey Engineering LLC, upon the suggestion in the architect Beth Miller's 'Historic Building Assessment' report of the Trinity Episcopal Church, to 'undertake a structural analysis of the (nave) roof structure'. In particular Annette Dey Engineering LLC was asked to investigate the structural questions regarding the roof of the church structure below:

1. *The cause of the various splits and cracks in the timbers and trim pieces throughout the (nave and) sanctuary, and any potentially required reinforcement measures. In particular Beth Miller was concerned about the torsional split in one of the shed roof purlins at the south side of the nave (see Historic Building Assessment, Part III: Sanctuary Roof Structure - Observations by Beth Miller RA LEED AP)*

2. *The cause of, and potentially required stabilization measures for, the perceived outward leaning nature of the eave wall(s) at the nave*

3. *The potential effect of adding insulation to the nave and sanctuary roof framing, with regard to the roof snow load and the load bearing capacity of the roof.*

Annette Dey Engineering LLC has visited the church three times over the course of eight months in 2024/25, to observe the sanctuary roof framing from the interior and exterior. On-site measurements were taken to conduct a reasonably accurate structural analysis of the roof structure, and joint details were observed as closely as possible, without dismantling the extensive existing trim work . Any available drawings and other documents pertaining to the sanctuary structure have been reviewed as part of this investigative process.

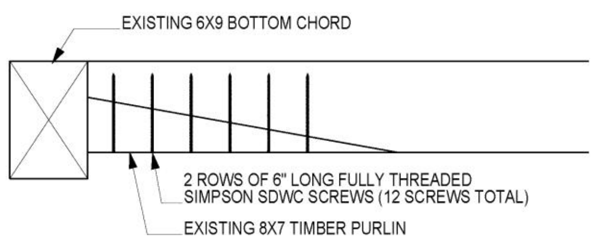
The archived drawings did not provide much meaningful new information (e.g. exact column sizes, timber connection details, etc.) regarding the structure of the nave and sanctuary. However, the structural report provided by Carl Goldknopf of GV Engineering, in August of 2010, provided a good baseline regarding the ongoing effort to establish the structural capacity of the roof structure of the nave. A scanned copy of the report in which pertinent observations regarding the sanctuary roof structure (pages 8 and 9) and recommendations (pages 11 and 12) were highlighted, is attached to this report for reference.

A noteworthy observation on site was that the easternmost freestanding timber bent in the nave does not have a steel rod tying the aisle posts above the shed roof, as the other three free standing timber bents do. Presumably when the other three rods were installed it was assumed that the adjacent building would act as a sort of buttress for this bent. However, this assumption would not have taken into consideration the fact that the bents' performance relies on symmetrical force distribution.

Based on the structural analysis results and the information and gathered from the sources listed above (also attached to this report) Annette Dey Engineering LLC would like to offer the following conclusions and recommendations:

1. The splits and checks observed in the various timbers and trim pieces at the nave and sanctuary are for the most part pretty typical for a timber structure. They are shrinkage cracks which develop in a lot of timbers and other wood framed members upon being exposed to lower humidity levels in enclosed spaces. Generally speaking, shrinkage cracks that run parallel to the grain and do not extend through the entire length of the timber cause relatively little loss of stiffness or strength in the framing member.

   That being said, the torsional crack in the timber purlin that Beth Miller pointed out on page 39 of her report has a significant 'slope' to its grain (runs at a significant angle to the long axis of the purlin). This means that a fair number of wood fibers are discontinuous which reduces that beam's strength capacity. It has been recommended to the church board to have this beam reinforced with 12 fully threaded screws (with minimal head sizes) from below, in order to make the split wood sections function as a unit again. A drawing detailing this reinforcement design is shown below. Since it seemed wise to reinforce this purlin sooner rather than later, the board has already acted on the proposed reinforcement scheme and had the screws installed. A picture of the reinforced beam is also featured below.



PROPOSED REINFORCEMENT OF SHED ROOF PURLIN

SCALE: 1 1/2"=1'-0" DATE: 23MAY2025

Picture 1: Shed roof purlin with fully threaded screws reinforcing cracked section

2. Annette Dey Engineering LLC conducted a structural analysis of the nave roof, as suggested in Beth Miller's report (see analysis results attached to this report). The bent analysis confirmed that the steep (upper) principal rafters and the center aisle posts were likely overstressed, and that the eave joints of the upper roof may have been deflecting outward as much as 2", before the steel rods were installed between the center aisle posts (about 1' above the shed roof). Also, the tension force in the bottom chord of the shed roof trusses appears to have decreased by about a quarter when the rods were installed.

   The adequacy of the existing tension connection of those shed roof truss bottom chords to the center aisle posts has been subject to much speculation over the years (see also the highlighted section on page 8 of Goldknopf's structural report attached to this report). The lack of stain at some of the bottom chords and/or open gaps where they bear on the corbel block of the center aisle posts seem to indicate that at least some of the bottom chords have pulled away from the center aisle posts by as much as 3/4" (see pictures 2 and 3 below). This would also explain why the southern eave wall appears to be leaning outward at the top when sighted along the inside.

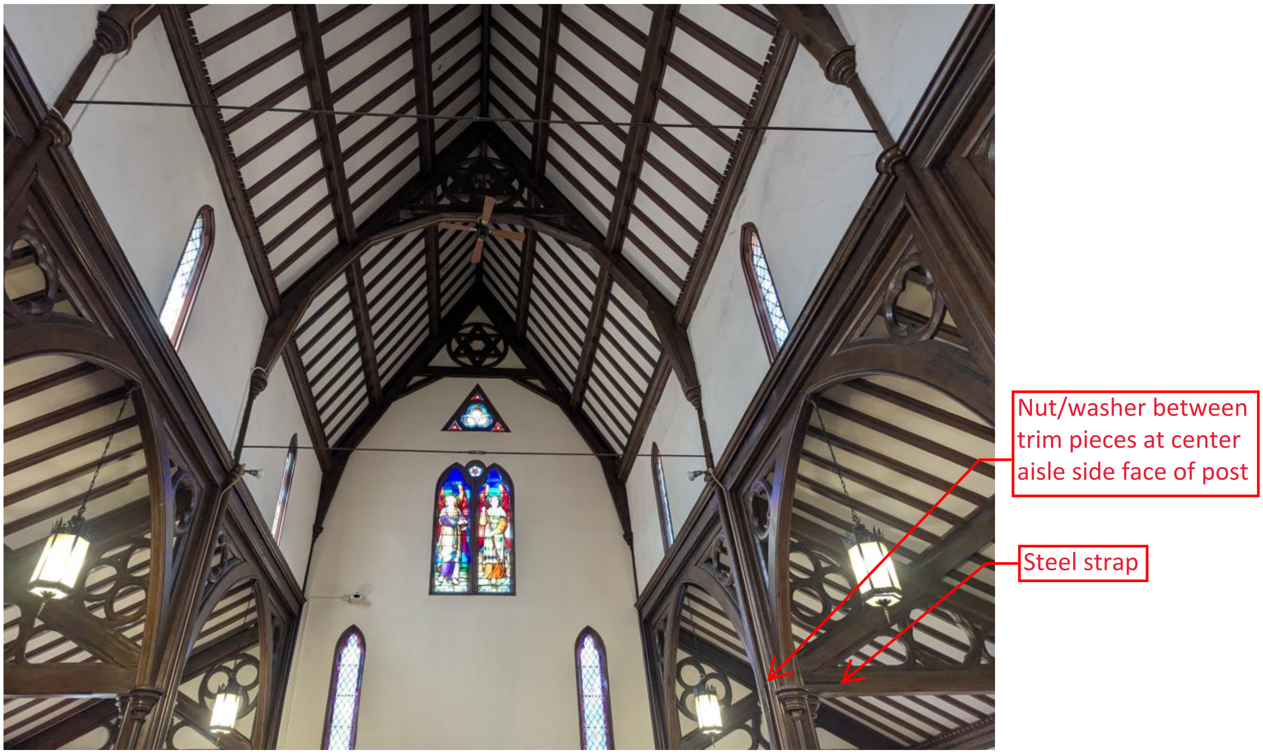
Picture 2: Bottom chord of shed roof truss has pulled away from center aisle post

Picture 3: Diagonal strut and bottom chord pulled away from center aisle post

Based on our recent non-destructive observation of those joints, with a scope camera and a light directed into the existing gap between the end of the bottom chord and the (ornately trimmed out) center aisle posts, there is some evidence that the original joint consisted of a mortise and tenon joint. However, based on the amount of lateral movement of the bottom chord (as evidenced by the extent of the non-stained portion of the bottom chord out of the joint) and the slightly outward tilting eave walls, it seems likely that either the end grain of the mortise has sheared off, or that the pegs have snapped in the mortise and tenon joint.

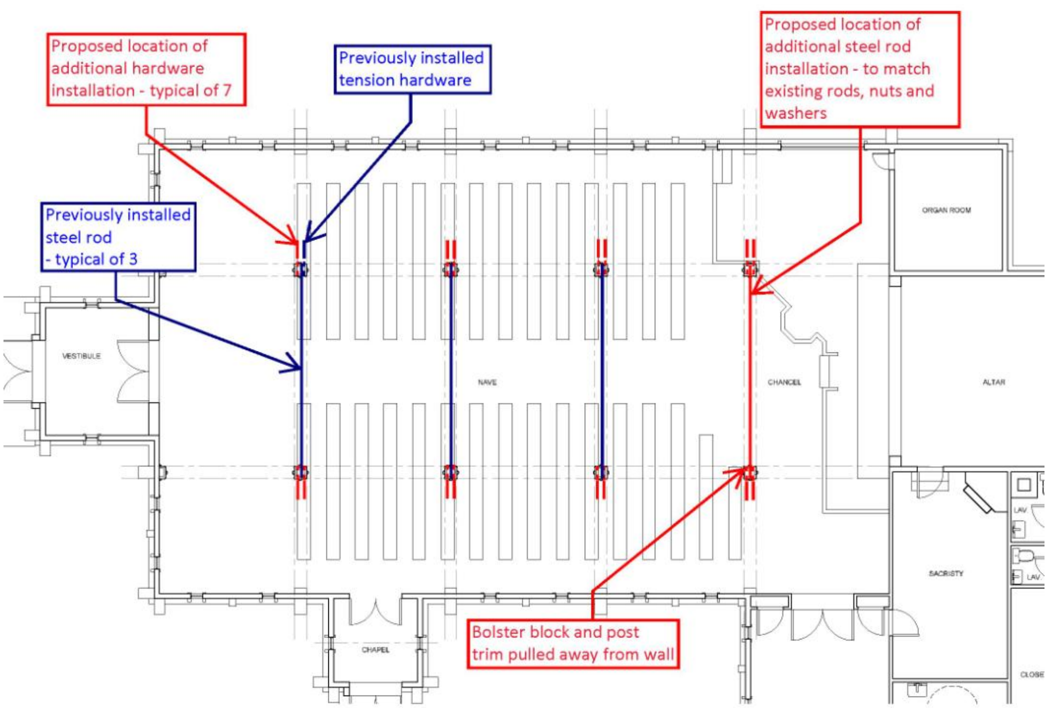
Since it appears that at least some of the various trim pieces surrounding the tension joints in question might break if they were temporarily removed, Annette Dey Engineering LLC recommends a semi-visible reinforcement method that would not require the removal of the existing column trim. It has basically been designed along the lines of the one existing joint that has been reinforced in the past (at the inside corner of the former tower location). As can be seen in the two pictures below, a strap that was welded to a threaded rod was fastened to the face of the bottom chord, extending through the column and is pulling against a washer and nut at the inside face of the center aisle column.

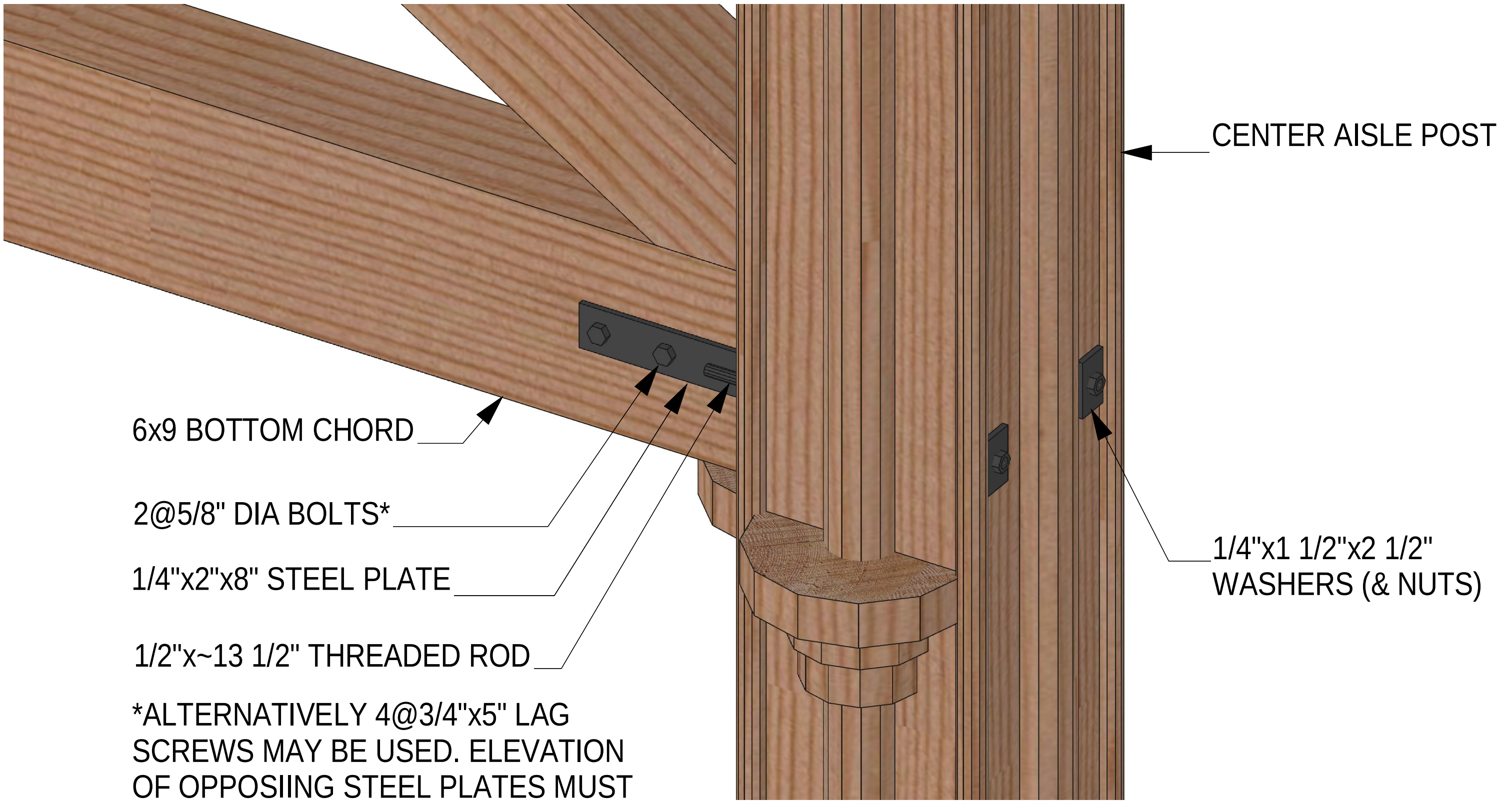
Picture 4: Existing reinforcement of bottom chord tension joint at northwestern-most center aisle post



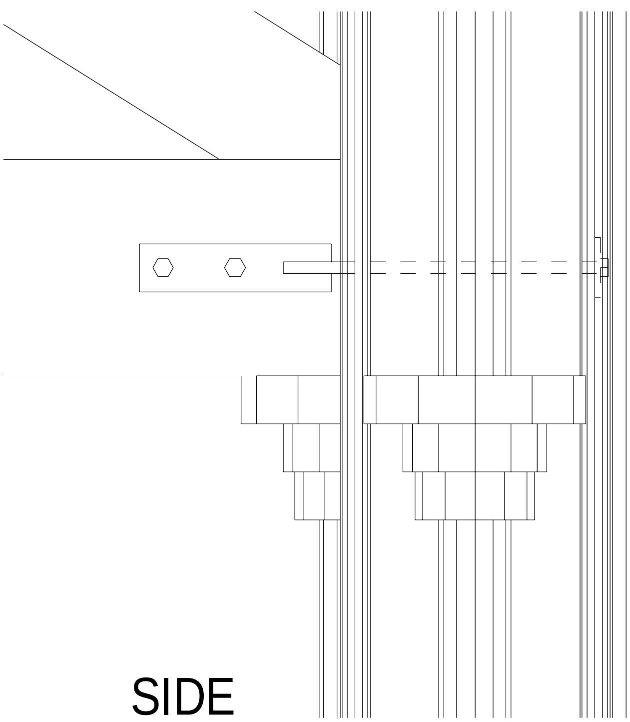
Picture 5: Installed strap, rod and washer/nut at northwestern-most center aisle post

The proposed reinforcement detail for the bottom chord to center aisle post connections is shown on the next page. Below is a floor plan of the nave with the existing and proposed new hardware layout. As featured in the layout plan, ***it is recommended to install a steel rod in the eastern-most timber bent of the nave as well.***

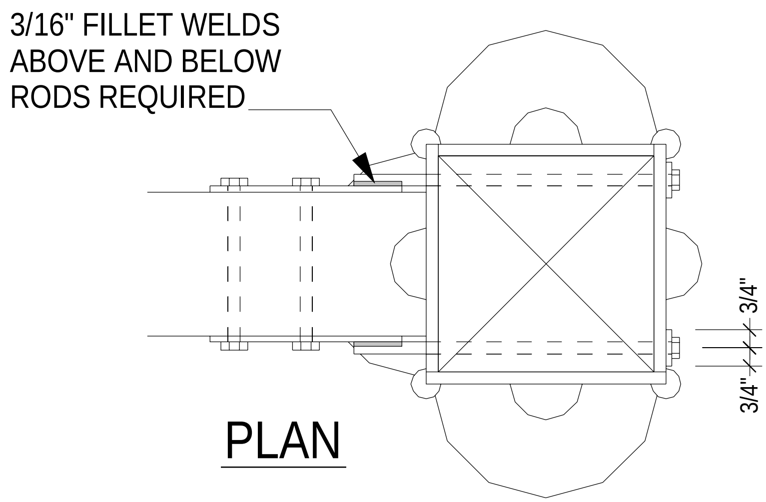




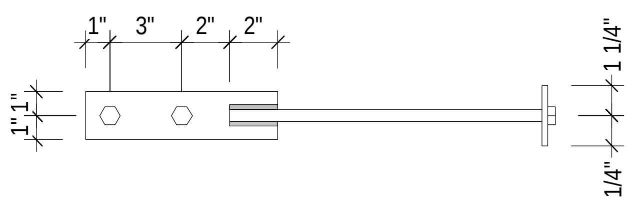
*ALTERNATIVELY 4@3/4"x5" LAG SCREWS MAY BE USED. ELEVATION OF OPPOSIING STEEL PLATES MUST BE OFFSET BY 3" IN THAT CASE.



SIDE



PLAN



SIDE (STEEL ONLY)

PROPOSED JOINT REINFORCEMENT AT CENTER AISLE POSTS

SCALE: 1 1/2"=1'-0"

REQUIRED AT 8 POST LOCATIONS

3. Annette Dey Engineering LLC has reviewed the effect of adding insulation to the roof framing with regard to the roof snow load, according to the current building code. It appears that the snow load accumulation would not be significantly higher due to roof insulation, neither on the steep portion of the nave and sanctuary roofs, nor on the lower shed roof planes of the nave. If a layer of 1/2" Plywood should be added as part of the roof built-up, that would add about 2 psf of dead load. This should be more than offset by the removal of the existing three layers of asphalt and wood shingles (mentioned on pages 29 and 31 of Beth Miller's report, see attachment) during the re-roofing process. Foam insulation itself has no significant weight. Based on those considerations, insulating the roof should be fine from a structural point of view.

   It is recommended that careful consideration be given to how to best insulate the roof, as the choice of insulation material and the installation method, as well as the attention given to detailing the edge conditions, etc. could make a big difference in the overall thermal performance of the church. Consulting a building performance consultant in this matter would be well worthwhile.

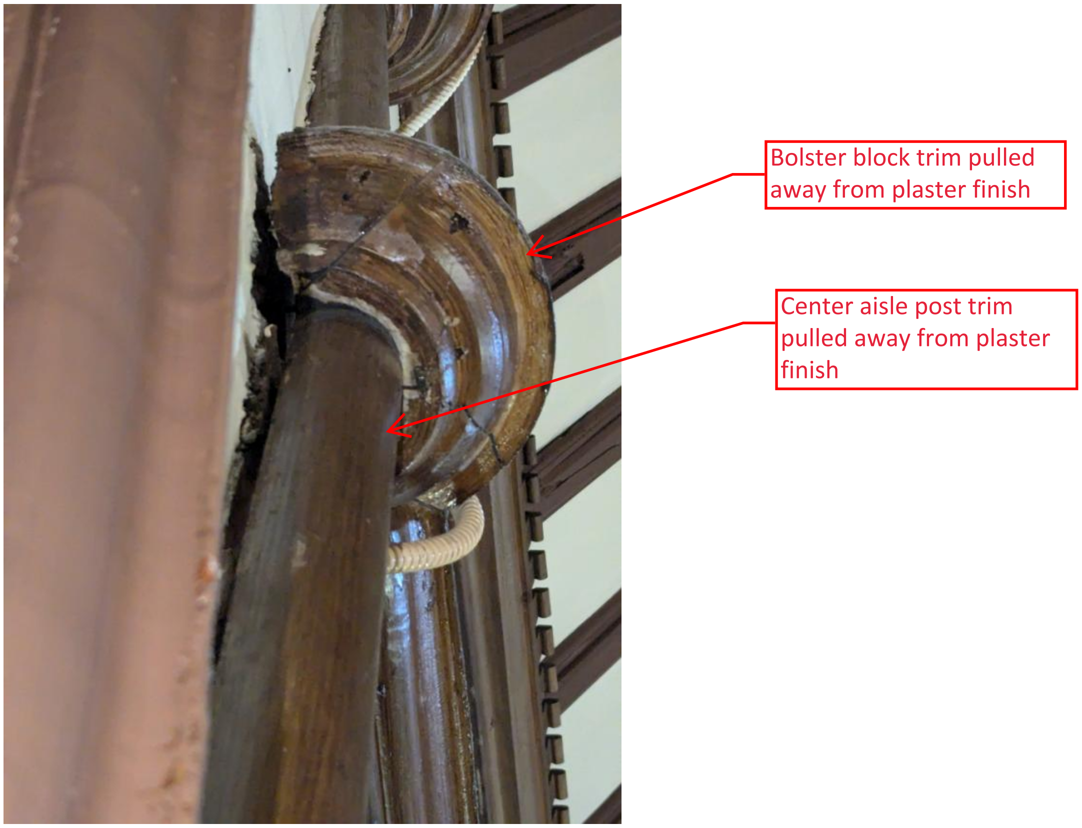
In addition to the splits and cracks observed in the timbers of the nave and sanctuary structure (described and explained in section 1 above), there were a number of splits and displacements observed in the trim pieces of various corbel blocks, particularly at the south side of the sanctuary and the south east corner of the nave. Since it appears that the bottom chords of the nave shed roof planes bear not only on the bolster blocks, but also extend into and bear on the wall framing (see picture below), this should not be cause for too much concern with regard to the bolster blocks along the shed roof eave walls.

Picture 6: Bottom chord of shed roof truss extending beyond bolster block into eave wall

As far as the corbel blocks in the sanctuary go, they appear to be mostly ornamental, since the rafters themselves appear to bear directly in the wall, and only the rafter trim comes down to sit on the corbel blocks (see picture below). Thus the gaps opening up between some of the corbel trim pieces and the wall plaster is not so much of a structural concern. However, they should be observed for ongoing movement and their fastening to the wall framing increased, as needed.

Picture 7: Corbel block and lower section of rafter trim at southeast corner of sanctuary

Of the various displaced corbel block trim and center aisle post trim (above the shed roof planes) Annette Dey Engineering is most concerned about the center aisle post at the southeast corner of the nave (see location indicated on hardware layout plan, above). The picture below shows how the portion of the center aisle post trim above the shed roof plane has been significantly displaced relative to the upper portion of that post trim. It is possible that we are seeing the effect of the center aisle post getting pushed outward significantly at the (upper) eave joint before the adjacent building was constructed (for reference see the lateral displacement potential in the original timber bents indicated in the analysis files attached to this report), in which case the installation of the proposed additional steel rod it this bent should serve to ensure stability going forward.



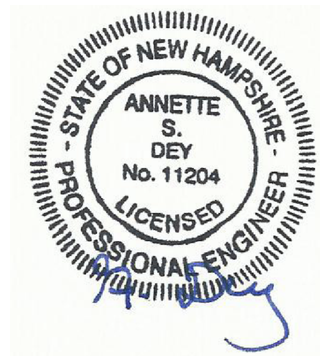
Picture 8: Post trim and bolster block pulling away from wall plaster at southeastern center aisle post of nave

If the corbel block and the lower trim portion of that post section were to be temporarily removed, then the questions below could be investigated:

1. Confirm that the center aisle post in the wall beyond is *continuous* at the shed roof level
2. Find out how the corbel block is fastened to the center aisle post (in the wall). The existing fastening method should be increased/improved, as needed.
3. Confirm that the upper portion of the center aisle wall is leaning outward at the top, and not the post trim leaning inward at the shed roof level.

Such temporary trim removal would perhaps best conducted at the time when (or better yet: right before) the additional steel rod in that timber bent gets installed. At the time of the trim removal Annette Dey Engineering LLC should be called in to observe the exposed framing members and existing attachment methods on site.

In conclusion, as Beth Miller and Carl Goldknopf before me, ***I cannot stress enough the importance of diligent maintenance with regard to the upkeep of painting, flashing and caulking the exterior skeleton structure when it comes to the longevity of this building.*** It is a testament to the dedication of all the caretakers before us, who have ensured that no significant water damage has been inflicted on this timberframe structure, which has been exposed to the elements for 173 years, in a climate that is infamous for promoting wood decay.



Respectfully submitted,

Annette Dey, P.E.

Attachments:

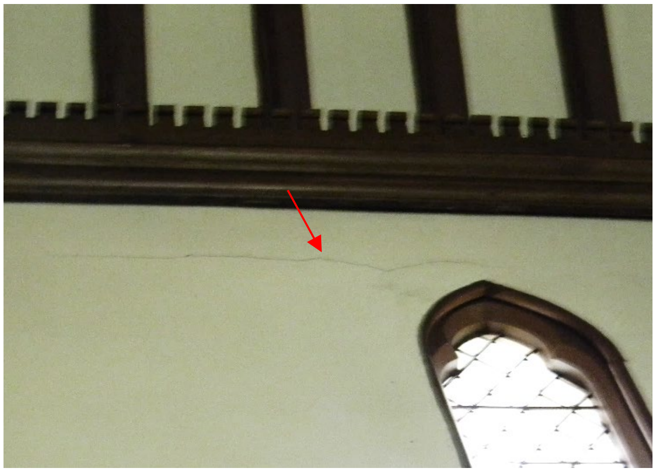
- Part III (Interiors - Observations) of the 'Historic Building Assessment' provided by Beth Miller in May 2024

- Engineering Observations & Report, provided by Carl Goldknopf of GV Engineering, in August of 2010

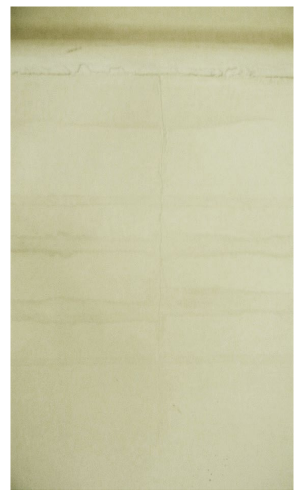
- Results of the Nave Timber Bent Analysis

PART III. – EXISTING CONDITIONS ASSESSMENT

INTERIORS - OBSERVATIONS







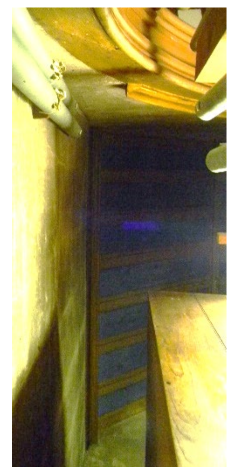
Cracked, stained, bubbled plaster was observed in selective areas throughout the Sanctuary and Parish Hall. Some areas of damage appeared to have resulted from leaks that are no longer active. Most areas of damage correspond to complicated roof junctures / areas of improper roof drainage on the exterior. Hairline plaster cracks at upper level windows in the Sanctuary are likely due to roof movement when stressed by wind or load.

PART III. – EXISTING CONDITIONS ASSESSMENT

Water-stained ceiling tiles throughout the Basement are reported to have resulted from an interior plumbing issue that has since been repaired.

Staining and peeling paint was observed along the Chapel basement south wall.

19th-century graffiti on the Organ Room wall should be preserved



Ceiling of the Organ room is painted blue, perhaps indicative of an earlier color scheme

View of Basement beneath Sanctuary

INTERIOR - RECOMMENDATIONS

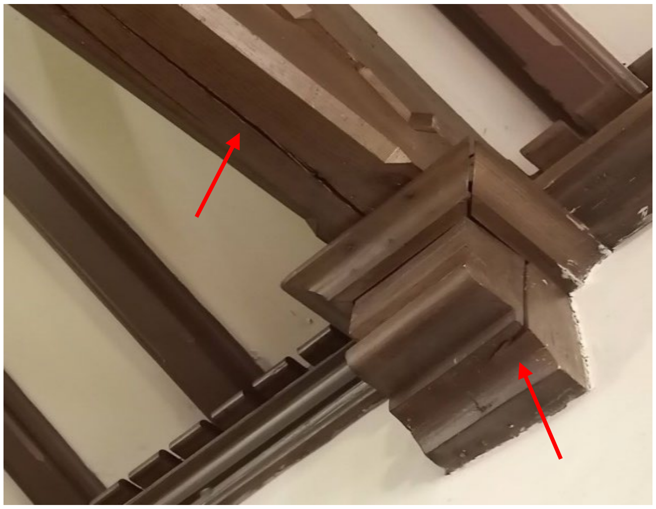
Damaged interior finishes should be replaced. This is included as a Low Priority item, as repairing the conditions that cause the damage are High priority. Paint sampling & analysis may be undertaken during Sanctuary repairs, in order to determine earlier paint color schemes.

The Basement walls were inspected but, despite the severe deterioration of the buildings' perimeter drainage, no sign of moisture ingress at the basement walls was observed.

PART III. – EXISTING CONDITIONS ASSESSMENT

SANCTUARY ROOF STRUCTURE - OBSERVATIONS

Cracking throughout the Sanctuary roof timber structural elements was observed throughout.



PART III. – EXISTING CONDITIONS ASSESSMENT

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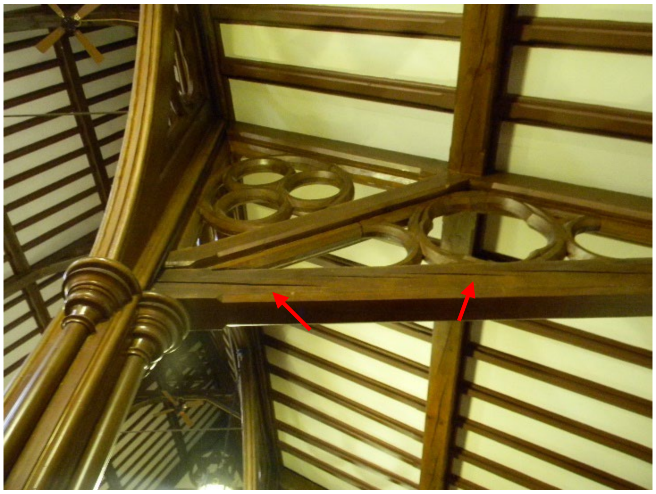
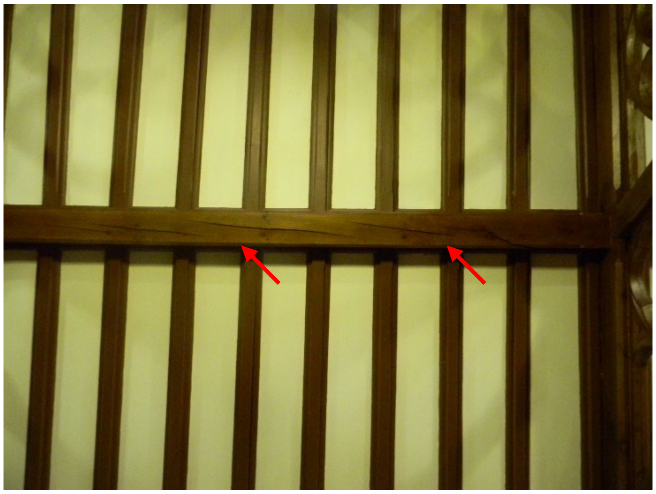
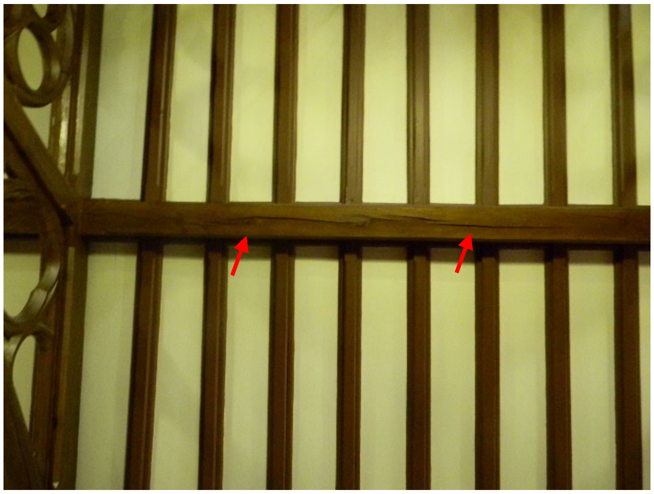
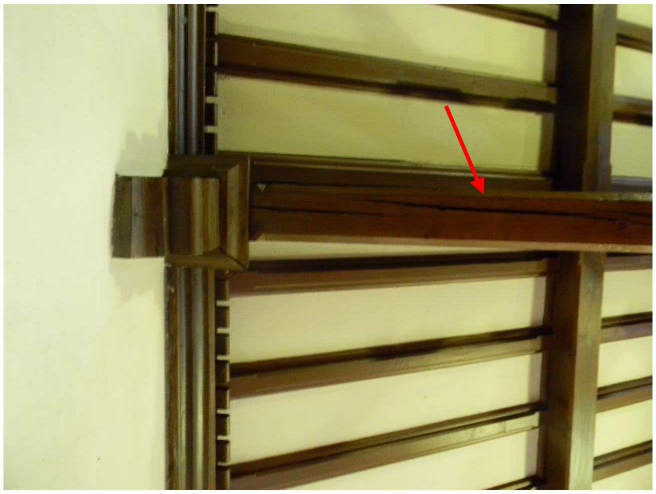
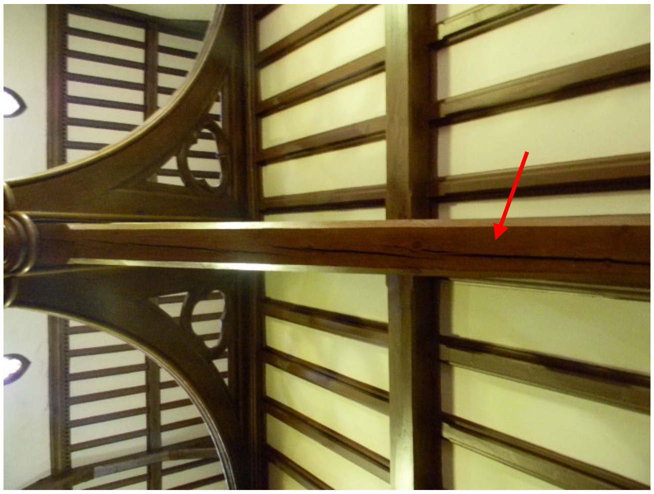
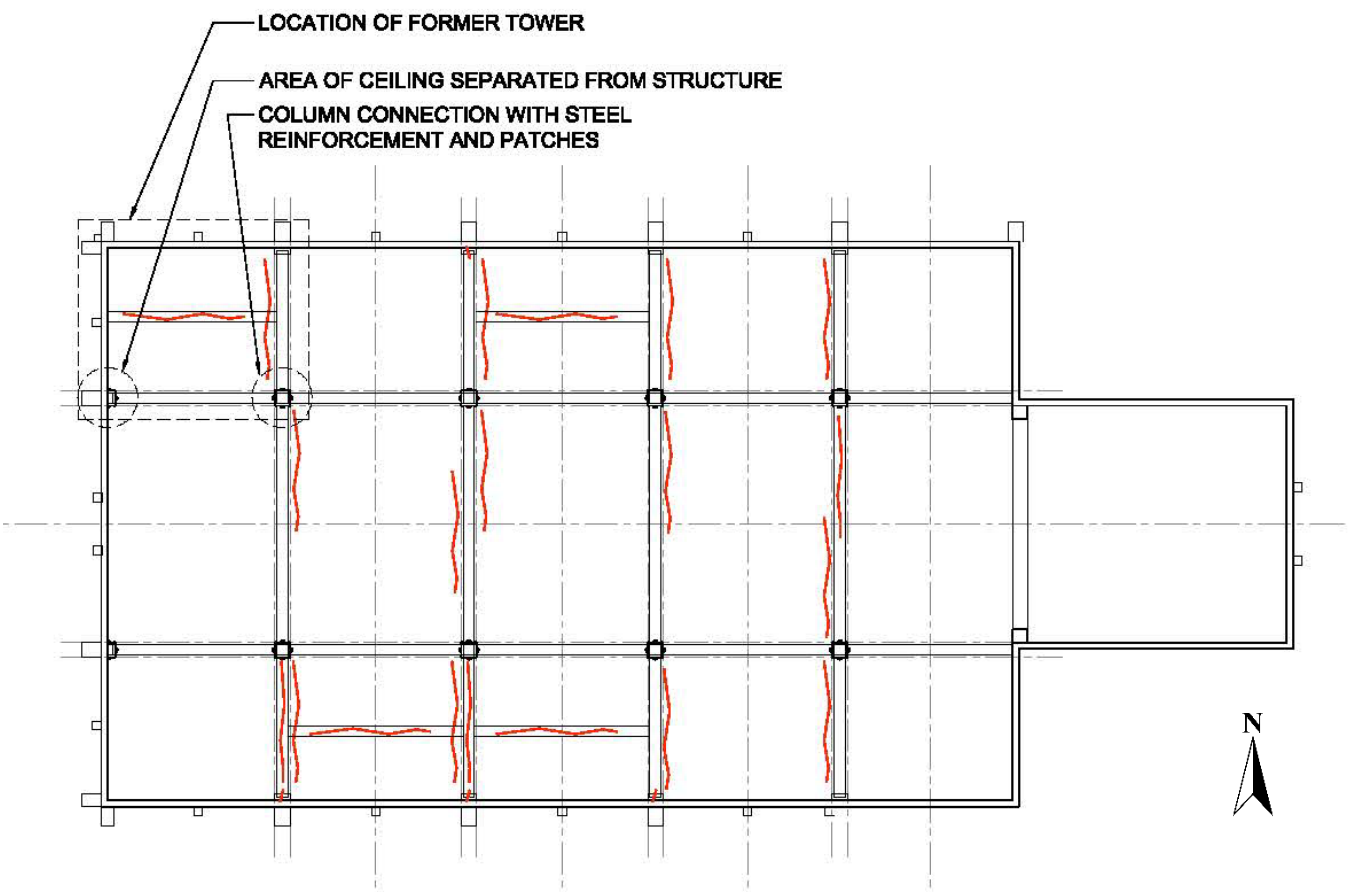






PART III. – EXISTING CONDITIONS ASSESSMENT

Cracking throughout the Sanctuary roof timber structural elements; Torsional cracking, such as that depicted in the image to the right, is of particular concern.

Ceiling structure has separated from wall to expose approximately 2" of lathe or roof strapping behind

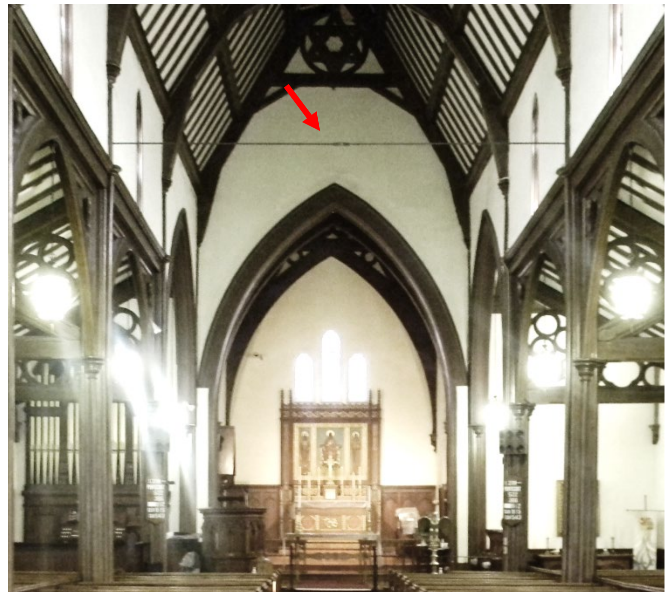


Diagrammatic Structural Plan documenting cracked timber structural elements observed in 2023

PART III. – EXISTING CONDITIONS ASSESSMENT

Below photos suggest that the steel tension rods in the Sanctuary were installed in the 1963 repair campaign, when the corner that was previously occupied by the tower was reconstructed for the second time. The building likely incurred structural damage throughout in the 1938 hurricane, but particularly in the exposed, vulnerable Sanctuary.

Sanctuary 1943, no tie rod, column leaning outwards



Sanctuary 2023

Above left is a view of the tie rod piercing the center of a timber column and right is a view of that same tie rod fastened on the exterior with a flat square plate and bolt.

Upper right and middle shows different views of the same column, with steel reinforcing installed. This column is located in the corner of the original tower, which was reconstructed in 1963. Below right, evidence of Dutchmen patch repair, also located in the corner occupied by the tower.

PART III. – EXISTING CONDITIONS ASSESSMENT

SANCTUARY ROOF STRUCTURE – COMMENTARY & RECOMMENDATIONS

It is evident from the type of cracking observed throughout the Sanctuary roof framing that the structure is, or was at some time, experiencing stress beyond its capacity. This could result from the structure being under-designed for the loads and/or a major stressful event like the collapse of the tower in the 1938 hurricane. It is possible that the tie-rods installed, likely in 1963, were successful at counter-acting the excessive outward stresses. There is no way to know if the cracks observed in 2023 existed in 1963 and were the impetus for the installation of the tie-rods, or if any have worsened or appeared since. Also, because many of the structural members are concealed by cladding on both the exterior and interior, and connections are concealed by ornament on the interior, it is not possible to easily determine the framing member sizes and types of connections in order to run a simple analysis. On a positive note, the cracking is isolated to the framing members, only minor cracks in the plaster and minimally out-of-plane plaster walls were observed.

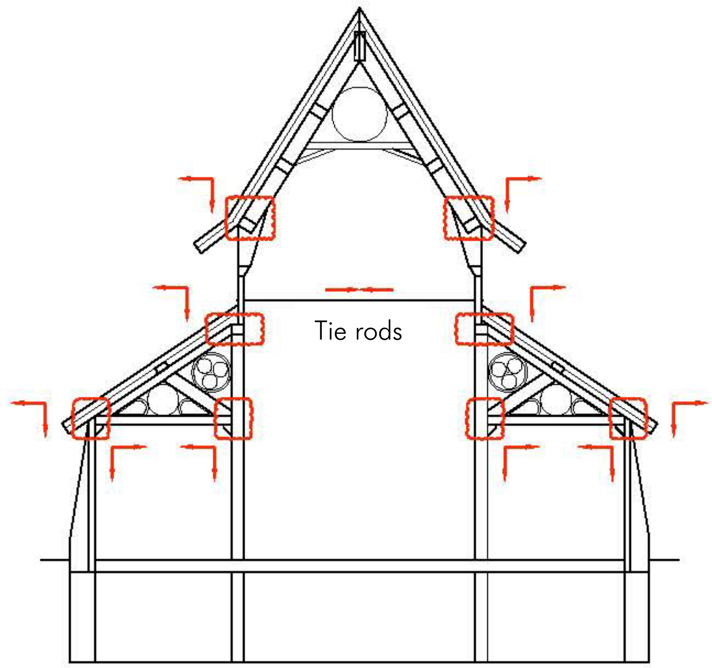


Diagram of stresses acting on roof structure and joints to be examined further.

Interestingly, in his treatise on Gothic churches, Wills states of open timber roofs, "*in many cases a tie beam was inserted, stretching from wall to wall: in others the tie beam was cut in the middle, its two ends was called hammer beams, and were supported by arched braces with the spandrils full of tracery.*" It is curious that no such lateral bracing element was included at Trinity Church, perhaps because Wills felt the roof pitch was steep enough as not to require lateral bracing. Also interesting to note, all but one other of Wills & Dudley's churches is constructed of stone with a wood framed roof. It is possible, being accustomed to buttressing their timber roofs with more substantial stone, they may have underestimated the different material and structural characteristics of stone and timber.

It is recommended that a thorough structural analysis be undertaken by an engineer experienced with timber framing or a timber framing specialist. Some minimal destructive probes will likely be required, one at each type of connection. If roof replacement is undertaken, this would be a good opportunity to use scaffolding access and opened roof areas to perform such an analysis. In the meantime, a system of monitoring should be put in place, with each truss given a number and its cracks photographed and measured every year at minimum.

Engineering Observations & Report

for

Trinity Episcopal Church
120 Broad Street, Claremont, NH

Prepared By:

GV Engineering, LLC
372 West Street, Suite 100
Keene, NH 03431

STATE OF NEW HAMPSHIRE
CARL
L.
GOLDKNOPF
No. 9860
LICENSED
PROFESSIONAL ENGINEER
8/6/10

August, 2010

Background

Trinity Church, on Broad Street in Claremont, NH, is a wood framed church on masonry foundation. Built circa 1852, the Church is just over 150 years old. The roof structure is timber truss and clad with a combination of standing seam metal and asphalt shingles. The walls are timber framed and the floor is rough joists and timber. The foundation is a combination of brick, stone, and poured concrete.

The Church is an active house of worship and host to civic events. In light of the building's age and public use, Trinity Church has commissioned GV Engineering (GV) to provide a structural engineering assessment of the Church facility.

Methods

GV Engineering's evaluation of the existing Trinity Church structure consisted of observations made over two site visits: Friday, April 16 and Friday, April 23, 2010. GV also made preliminary observations on Friday, January 15, 2010. The timing of these three visits provided the opportunity to observe structural elements over the changing seasons.

GV Engineering only observed Structural elements where they were openly visible. No selective demolition or invasive measures were employed to observe conditions where cladding concealed the structure.

Findings

<u>Friday, April 16, 2010</u>

Friday, April 16th the weather was damp and the ground was largely wet. GV's observations on the 16th focused on the building exterior, the roof, and attic. The recent precipitation proved helpful in highlighting areas that held moisture and dried more slowly. Sanctuary roof and wall framing was observed inside and out.

Figure 1: Flashing and siding maintenance have staved off deterioration in this wet area where paint won't hold but siding and sill are sound.

Building Exterior –Siding & Sill

The Building is surrounded by a combination of paved, concrete and stone splash pads that serve to transfer water from the roof drip line, away from the foundation. The surrounding grounds are fairly level and the grade gently slopes away from the structure. The exterior cladding has been well maintained. The bottom cladding has been wrapped in flashing to protect the wood. Where the bottom courses of siding receive considerable splash, such as under a roof valley, the siding has been replaced with more durable fiber composite clapboard (Figure 1). The paint, and flashing are sound and the exterior has been well maintained. Such diligence has served the church well as the exterior reveals no pronounced rot at the sill, or elsewhere.

Figure 2 Northwest corner splash pad slopes toward building directing roof runoff to foundation.

Splash Pads

While the building is wrapped with splash pads, along the north wall, a concrete splash pad slopes the wrong way. Here roof runoff is directed toward the building instead of away (Figure 2). Despite this condition, and a heavy coat of moss, the poured concrete wall below appeared sound and the sill free of rot. Subsequent observation from the interior revealed no sign of moisture, rot, or weak masonry.

Along the same north wall, an accumulation of dirt and moss impeede runoff and hold it close to the foundation (Figure 3). It is likely that this debris was left by winter snow plowing. Cleaning this residue would help keep the north wall dry.

Figure 3 Moss and debris impede runoff

Foundation

The foundation exterior revealed three areas of concern or deterioration. On the north wall, in the third column bay, the brick foundation wall is loose and the masonry can be moved with pressure. Subsequent observations from the interior showed the inner course of bricks to be firm to pressure and unyielding.

Building Exterior – Foundation - Continued

The entry alcove outside the sactuary facing Broad Street revealed a similar loose masony foundation. Here, the south wall is loose and can be moved on the order of ¼". Here the bricks have settled leaving a gap of ¼" to ½" between the sill and brick foundation (Figure 4). Subsequent observations from the crawlspace inside showed the interior brick to be firm and unyielding to pressure. The sill was sound. Though light was visible through the gap between the foundation and sill.

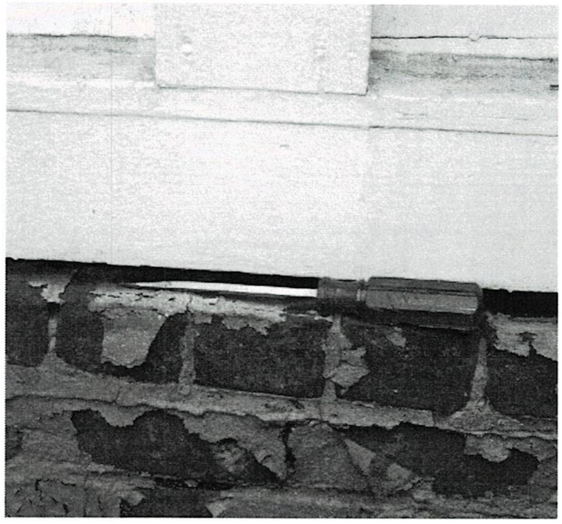


Figure 4 Loose brick on exterior and larrge separation between sill and foundation

The third area of deteriorated foundation exists on the west wall, south of the sactuary. The walls form an interior corner here and a roof valley funnels runoff into the corner. Though a paved splash pad exists here, the walls receive splash as water from the roof hits the paved apron. This area has received past attention. The exterior masonry could not be inspected because the foundation wall has been fitted with plywood and a modified bitumen membrain (Figure 5). The membrain was clearly intalled as a means of protecting the masonry from the elements and further deterioration. It is unclear if the masonry was reparied prior to the installation of the protective membrane. Cladded as it was with plywood and membrane, the condition of the wall could not be observed or asertained.

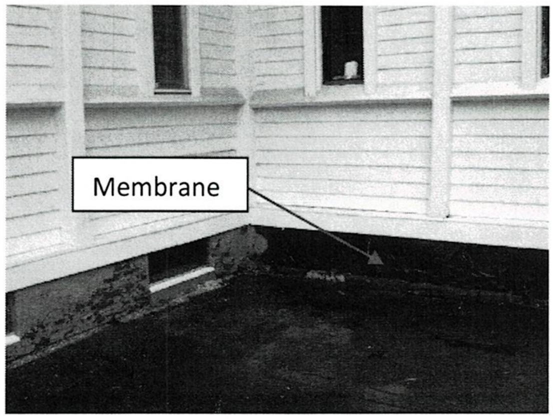


Figure 5 Brickwork has been fitted with a membrane under this roof valley.

Cladding

The exterior cladding and paint finish has been well maintained and reveals no rot. It is important to note however, that cladding on a column in the northwest corner of the structure has separated(Figures 6 & 7). The wood column inside is exposed to the elements and should be protected before rot ensues.

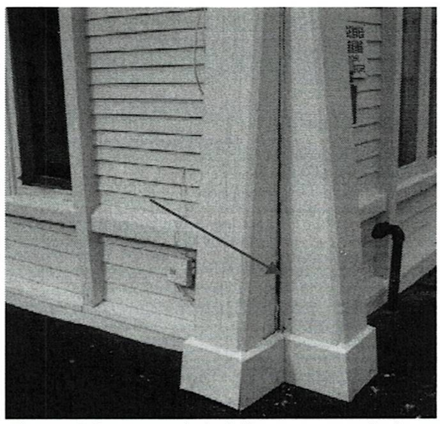


Figure 6 Separated cladding exposes timber column inside

Attic

The building has two attic spaces; one over the kitchen and one south and east of the sanctuary. The church bell is located in the attic space south and east of the sanctuary. Both attics were found to be dry. Neither attic revealed sings of moisture. The wood framing observed was sound with not obvious deformation.

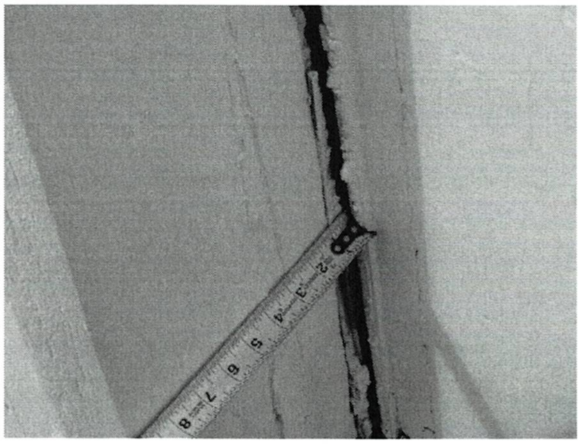


Figure 7

Friday, April 23, 2010

Friday, April 23rd the weather was clearing with a heavy fog lifting. GV Engineering's observations on the 23rd focused on the foundation interiors and crawl spaces, visible interior sills and framing. Exterior observations from the 16th were also confirmed and the Sanctuary roof and wall framing was observed inside and out.

Interior Foundation and Sills

It is important to note that in all three site visits: January 15, April 16th, and April 23, the basement was warm and dry with no signs of moisture intrusion or dampness. The foundation interiors revealed no loose brick or rotted sills.

The foundation interior has been covered in spray cellulose insulation (Figure 8) for several feet below grade. In most areas the wood sill and masonry could not be visually inspected because of the cellulose insulation coating. GV performed a tactile investigation at regular intervals and found brick, stone and poured concrete walls to resist penetration and pressure. The foundation interior revealed no loose brick, soft, or crumbling masonry.

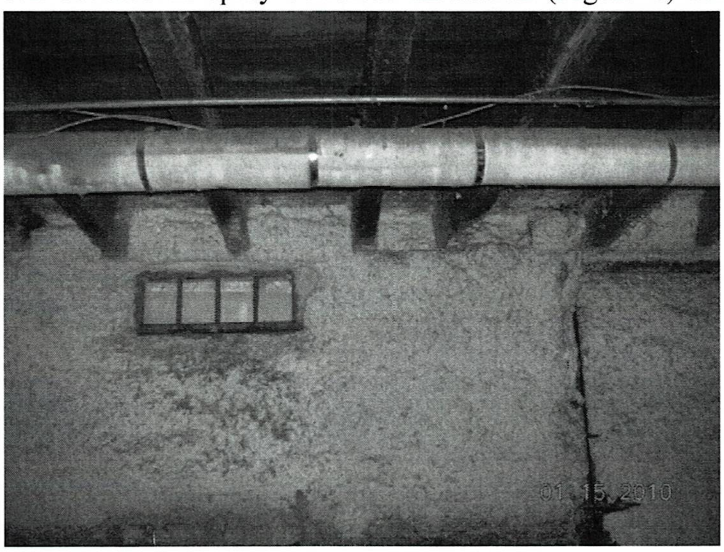


Figure 8 Insulation covers sill and wall but sill, joist and masonry resisted penetration wherever checked.

In the two areas where loose brick was found on the exterior (north wall and south side of entry alcove), the interior walls appeared rigid and resisted penetration and pressure. The entire north wall was dry and, to the degree that insulation impeded visual inspection, revealed no loose brick, mortar or stone.

The south wall of the entry alcove facing Broad Street is accessed by a crawl space with low head room. Light is clearly visible from the outside through the gap between the sill and brick. Still, the interior brick appears rigid and resists pressure from the inside despite the exterior weakness.

The third area of concern, the brick on the west wall, covered in membrane could not be closely observed due to obstacles and low head room in the crawl space. At one time the joists perpendicular the wall had been shored and the shoring remains below the joists. Some joists have been notched for their previous bearing but no cracks are evident. While the existing connection or bearing of these joists was not visible and could not be observed, the joists were rigid, as was the floor above. The floor above revealed little bounce or deflection.

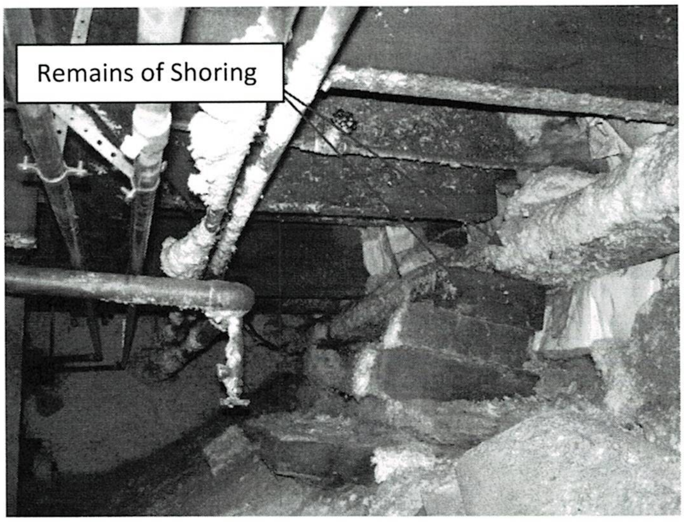


Figure 9 Plumbing, rubble, and insulation prevented closer observation of this north crawl space wall. Once shored these joist connections could not be observed

The rest of the crawl spaces proceeding, south and east, revealed dry, sound joists and timber posts.

Two beams in the crawl space appear to have problematic supports. The more serious of the two bears in a pocket on the outside corner of a brick wall. The wall has deflected and a significant crack appears at a diagonal to the load (Figure 10). Since the bearing masonry wall has failed, the beam should be temporarily supported until a permanent bearing can be fitted.

The second troublesome bearing in the crawl space has been shored (Figure 11). The beam rests on a brick column with stone cap. A timber support has been fitted inside the brick column. The timber support is not vertical and blocking has been fitted atop the timber column to make up the several inches between the top of column and the bottom of the beam. The beam/column connection likely offers no lateral stability or positive means of fixity.

Figure 10 Bearing wall in failure.

Figure 11 Bearing & Support lack fixity and lateral stability.

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Sanctuary Roof System:

The Sanctuary Roof was observed during all three visits. The Center roof is vaulted and flanked by two lower roofs on each side (Figure 12). The outward horizontal thrust of the vaulted center roof is resisted by metal ties with turnbuckles. The metal ties connect to columns, well below the bottom of the upper roof. The position of these ties, well down the column, puts the columns in bending. That is to say, as the tension in the tie is transferred to the column, a moment develops (tension in the tie becomes moment in the column). The lower roofs flanking the vaulted center are supported by a timber truss. Here, tension in the bottom chord of the truss resists outward thrust (Sketch 1).

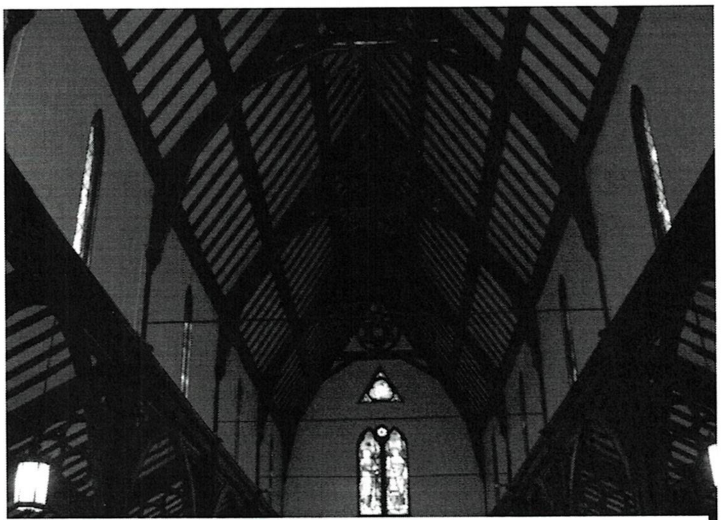
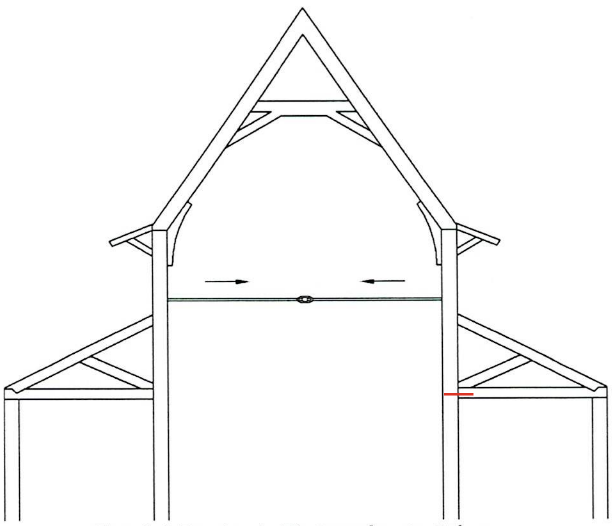


Figure 12 Vaulted Center Roof with flanking Lower Roofs. Horizontal Ties connecting to columns.

GV Engineering was asked to look at the Roof-Wall interface for evidence of wall movement resulting from horizontal thrust. Church officials, expressed concerned over possible signs of outward movement at the base of the roof.

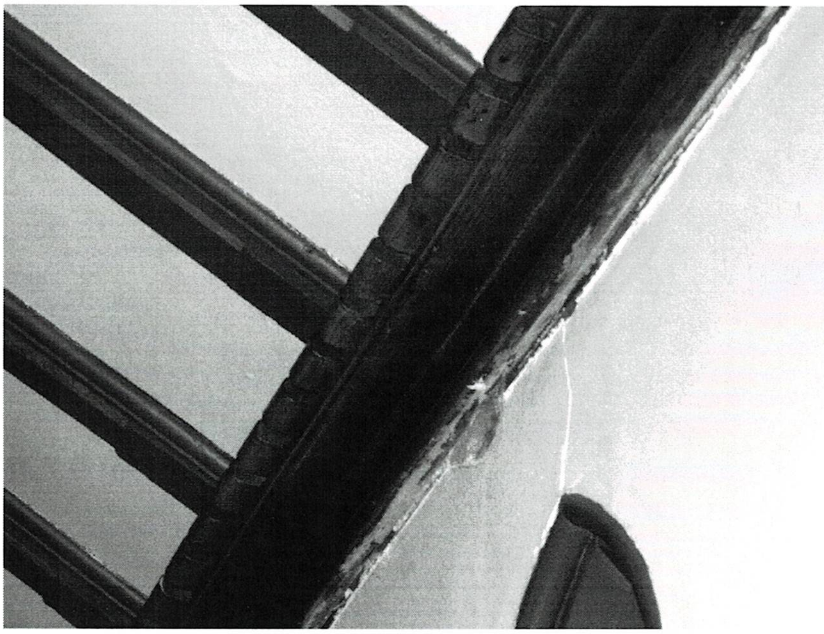
Observations from both inside and outside the sanctuary show no obvious failure or profound outward movement of the walls. Evidence suggests however there may be small movement in the outer wall. Specifically, in the north wall, the plaster has separated from the cornice molding (Figure 13). Also, lack of stain (color variation) at the inner connection of one bottom truss chord suggests movement on the order of 1/2"–3/4" (Figure 14). The plaster has also cracked over the stained glass windows; but this is not surprising, given the size and shape of the windows. If the plaster is going to crack, it will tend to crack over the window where the wall section is smallest.



Sketch 1 Tension in Tie transfers to Column. (Not to Scale)

Given the age of the structure, it is reasonable to assume that the metal upper tie is wrought iron. The building predates the regular use of steel.

By inspection, the bottom chord timbers appear to be adequately sized to carry the tension to resist outward thrust *provided the connections on both ends adequately transfer the thrust into tension* in the bottom chord. The connection at the outer wall is a visible timber connection designed to do just that. The top chord is notched into the bottom chord (Figure 15). Because the inner end connection of the bottom chord is concealed, it is not clear how this bottom chord is connected to the column however. Because this connection cannot be inspected, it is impossible to know if the bottom chord is developing enough tension to resist the roof thrust.



Over three visits, dating from January to April, the sanctuary roof revealed no obvious or profound failure. There is evidence however of some movement (up to ¾"±) of the bottom truss chord from the inner column. This was particularly visible on the north side. The bottom chord joints at the outer wall are tight. Without more exhaustive study little more can be known.

Figure 13 Loose Plaster at cornice, owing to the window size and shape, the plaster crack is no surprise.

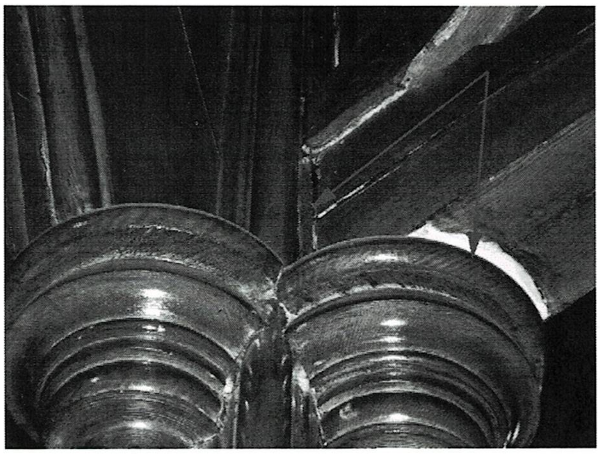


Figure 14 Lack of Stain suggests Bottom Chord has moved 1/2" to 3/4". It is not clear how the chord connects to the Column.

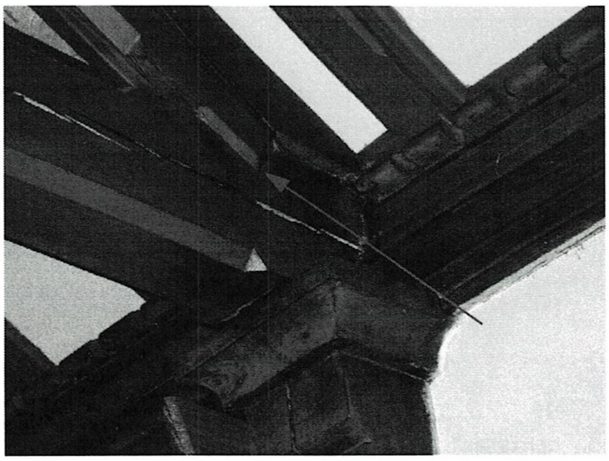


Figure 15 Exterior Timber connection of Bottom Chord is designed to transfers thrust into Tension in Bottom Chord.

Recommendations:

Recommendation have been broken down into Short, Medium and Long-Term.

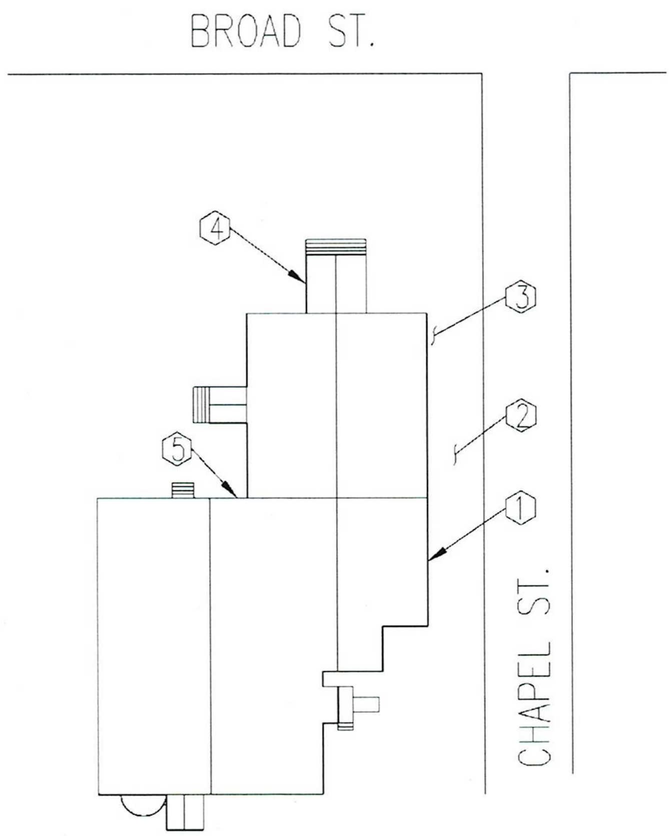
Short-Term Recommendations:

Install Temporary Shoring Under Beam with Failed Bearing:

Figure 10 showed a failed masonry wall under a beam. The Beam in question should be temporarily supported until a permanent bearing can be designed and put into service.

Other Short-Term Recommendations:

Several things can be done to the building exterior in the short-term which would help reduce runoff and increase the longevity of the foundation. The approximate locations of these recommendations are shown on the sketch below. Many of the recommendations involve easy fixes which should be relatively inexpensive. Owing to the relative ease and anticipated minimal cost of these repairs, they are included here as short-term recommendations



1 & 4: Repair Brickwork

1 and 4 show the approximate locations of the two areas where loose foundation brickwork was discovered. A masonry contractor should be employed to repair these walls to the limits of their degradation.

Once repaired, it may be possible to better seal water from the brick and mortar. The brickwork has been painted in the past, but in some areas, the coating has deteriorated (Figures 4 & 5 previously).

2: Clean Concrete Splash Pad

2 shows the approximate area along Chapel St. where sediment and moss create puddles and block runoff (Figure 3 previously). Cleaning these splash pads along Chapel St. would require no special tools or skills.

Since it is likely that much of the debris comes from road sand deposited by snow plows, it would be a good practice to sweep the splash pads free every spring.

3: Rebuild or Reset Splash Pad

3 shows the location of the westernmost splash pad on the northern wall. This splash pad slopes the wrong way in that it directs water toward the foundation. The splash pad should be either reset to slope away from the building, or demolished and replaced. The problem should be remediated before water and ice degrade the foundation wall.

5: Verify Permanent Repairs.

Location 5 is the inside corner under the roof valley where a membrane has been placed to protect the brick foundation (Figure 5 previously). The area appears to have received past attention; it is important to confirm that the remedies were permanent repairs. Because of the membrane, it was not possible to observe the condition of the foundation. The inside crawl space showed evidence of past shoring but joist connections were obscured by debris. It is very possible that the repairs were permanent. Until this can be confirmed, it is equally likely that the repairs were temporary and more extensive work is called for.

Hopefully someone in the church has the institutional memory to speak to what work was done in this area and the permanency of these repairs. If improperly designed and installed, the membrane outside could be trapping moisture and promoting continued deterioration, albeit perhaps at a slower rate. If the repairs were anything but permanent, permanent remedies should be designed and implemented.

Short-Term Recommendations:

Permanent Bearing and Foundation for Crawl Space Beams:

Beams shown in Figures 10 and 11 should have proper end bearing designed and permanently installed. This work would include: footing size, design, and installation, post size, design, and installation; and installation of post base plates and caps for connections.

Sanctuary Roof:

There was considerable discussion above on the roof wall interaction in the main sanctuary. Despite the robust framing, there is evidence of some outward movement. The evidence is limited and it is impossible to know if the movement is localized or more uniform. At this point, the extent of the movement is not clear. Nor is it clear if the structure requires remedial action. Since knowing more definitively how the structure behaves would require more exhaustive study, the following options are presented not for remediation, but for further observation.

1. The church could simply observe the roof/wall interface over the seasons to gain a better feel, and perhaps comfort level, with any movement. The building could be

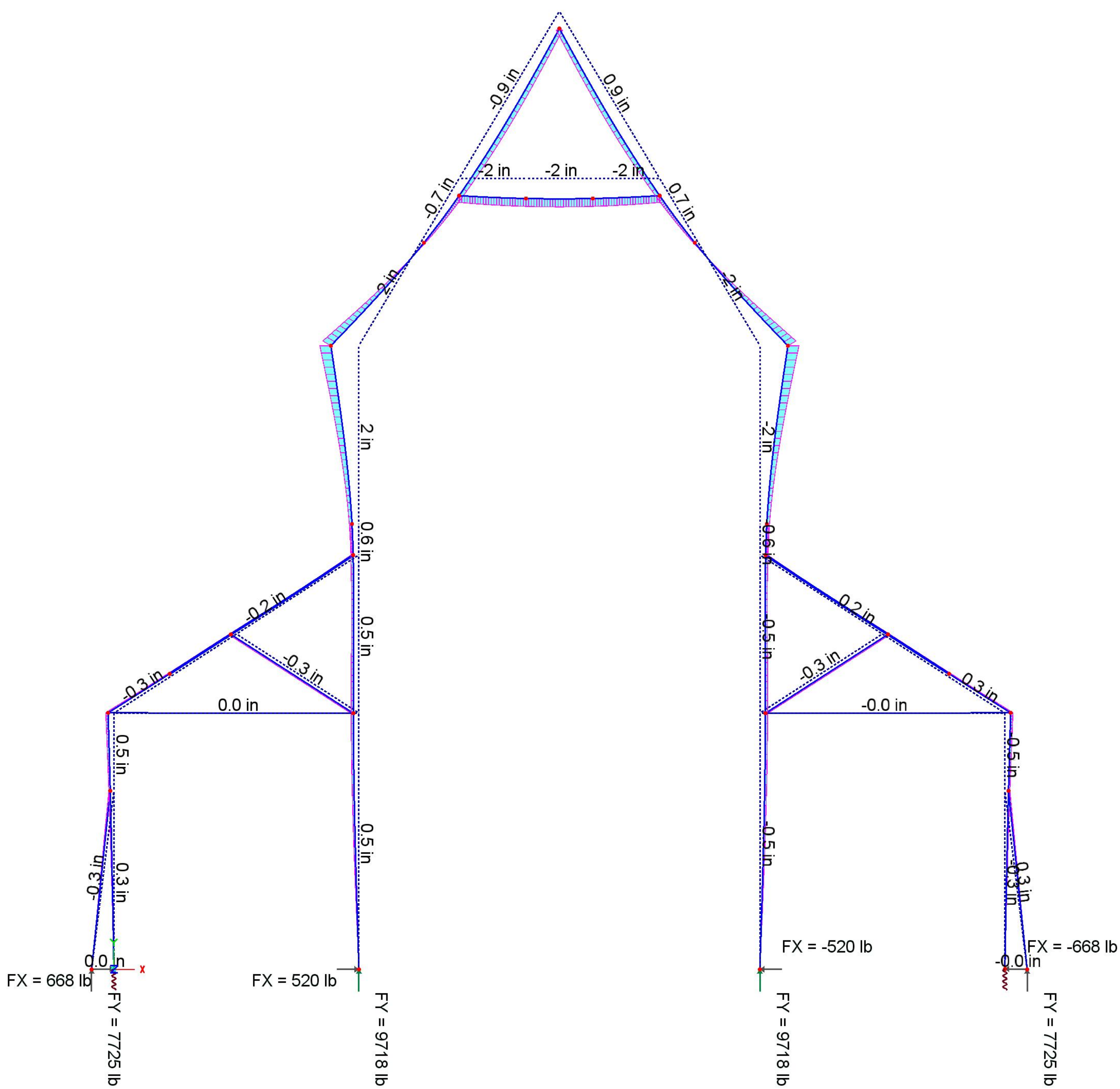
observed over the seasons to see how it behaves over wet and dry, warm and cold, and snow and wind. This option is the least cost solution but provides the least information. Still, longer-term observation of the situation may justify more exhaustive study in the future, or negate the need for such.

2. Additional engineering study could be employed to measure and create a computer model of the structure. While this model would be helpful to ascertain the internal forces within the structural elements, it will not tell us how the outer bottom truss chord is attached to the inner column. Since this connection is unknown, we cannot check its adequacy. A computer model will only speak to the adequacy of the known elements. This connection has shown movement (Figure 14 previously).
3. It is possible that a timber framer or someone from the Timber Framers Guild can render an opinion as to the nature of the connection between the outer bottom truss chord and the inner column. Obviously, an opinion is no substitute for knowing the nature of this connection but it would preclude the selective cladding removal required to observe the connection's exact nature.
4. It may be possible to employ craftspeople to perform selective cladding removal and expose the connection between the inner column and the outer truss bottom chord for inspection.

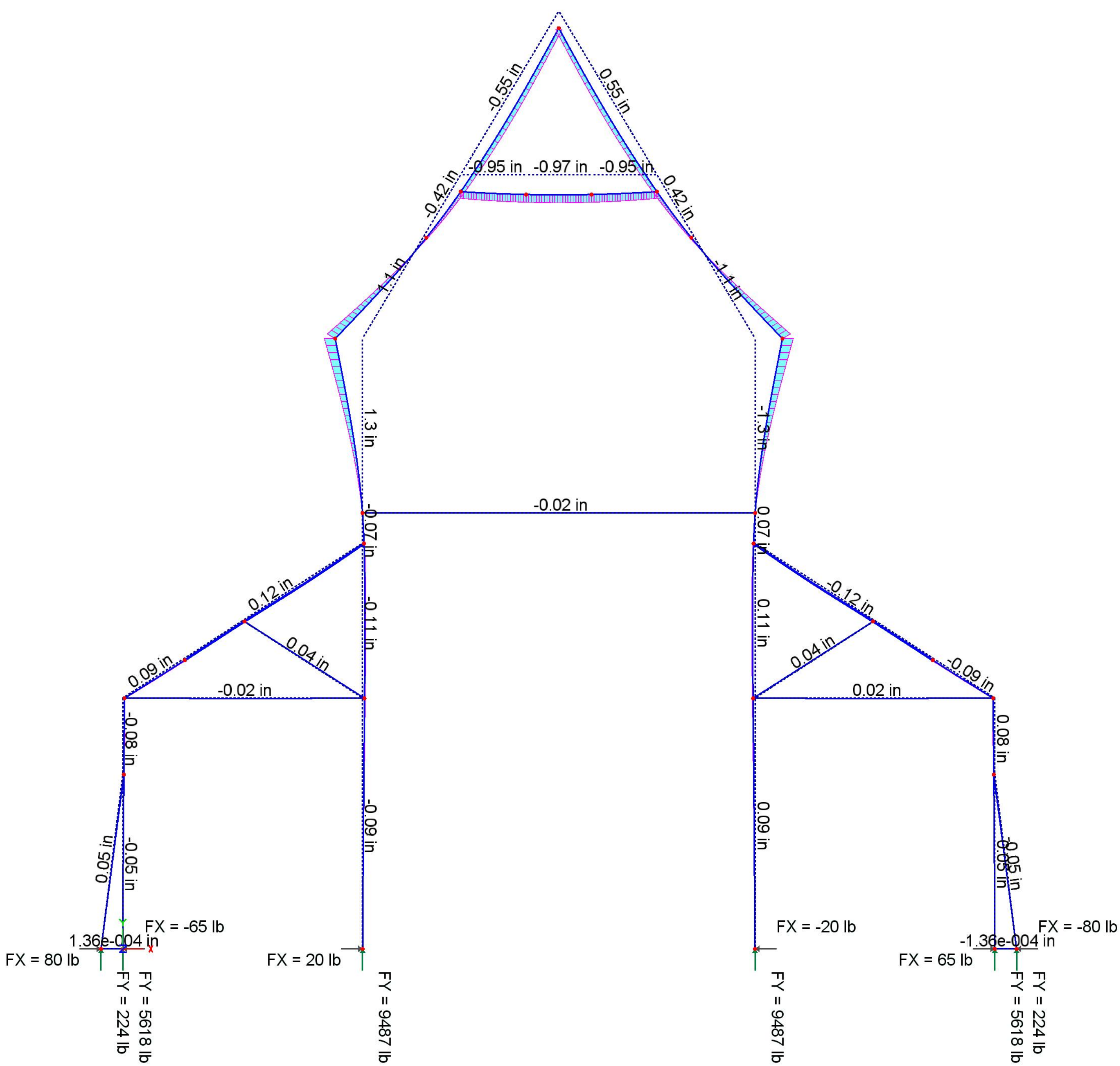
Long-Term Recommendations:

Generally speaking, given the age of the structure, the church is in good shape. There was no profound evidence of rotted wood or moisture inside the structure, its basement or crawl spaces. The structure's health is testament to diligent maintenance and care over the years. Continued diligence and care will preserve this unique structure for many future generations.

Original Timber Bent
ANNETTE DEY ENGINEERING, Annette Dey P.E.
Oct 20, 2025; 12:17 PM
Result Case: D+S
Member Dy, displacement
IES VisualAnalysis 12.00.0020



Timber Bent with tie rod
ANNETTE DEY ENGINEERING, Annette Dey P.E.
Oct 20, 2025; 12:16 PM
Result Case: D+S
Member Dy, displacement
IES VisualAnalysis 12.00.0020



Original Timber Bent
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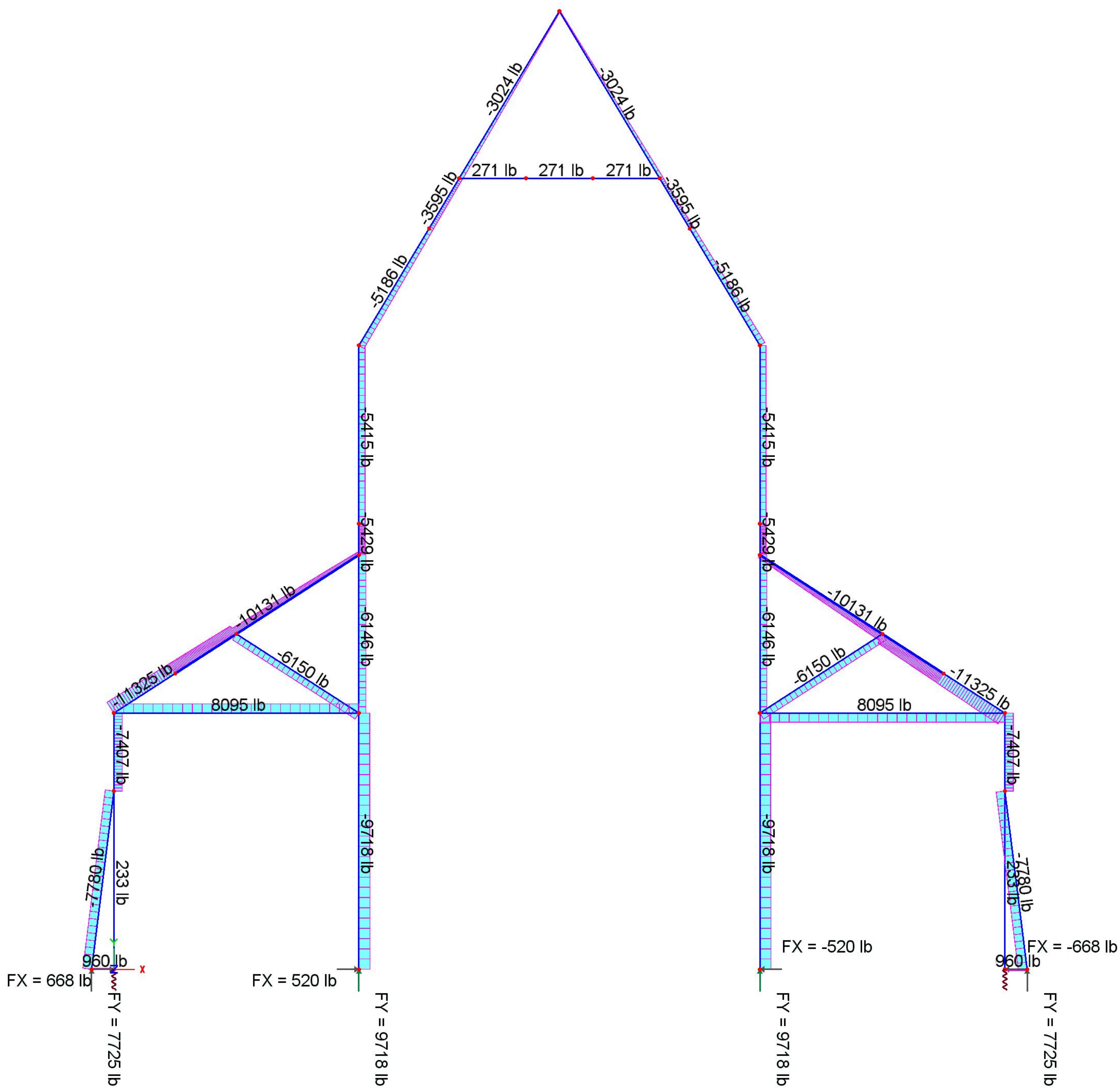
Result Case: D+S
Member Fx, axial force
IES VisualAnalysis 12.00.0020



Timber Bent with tie rod
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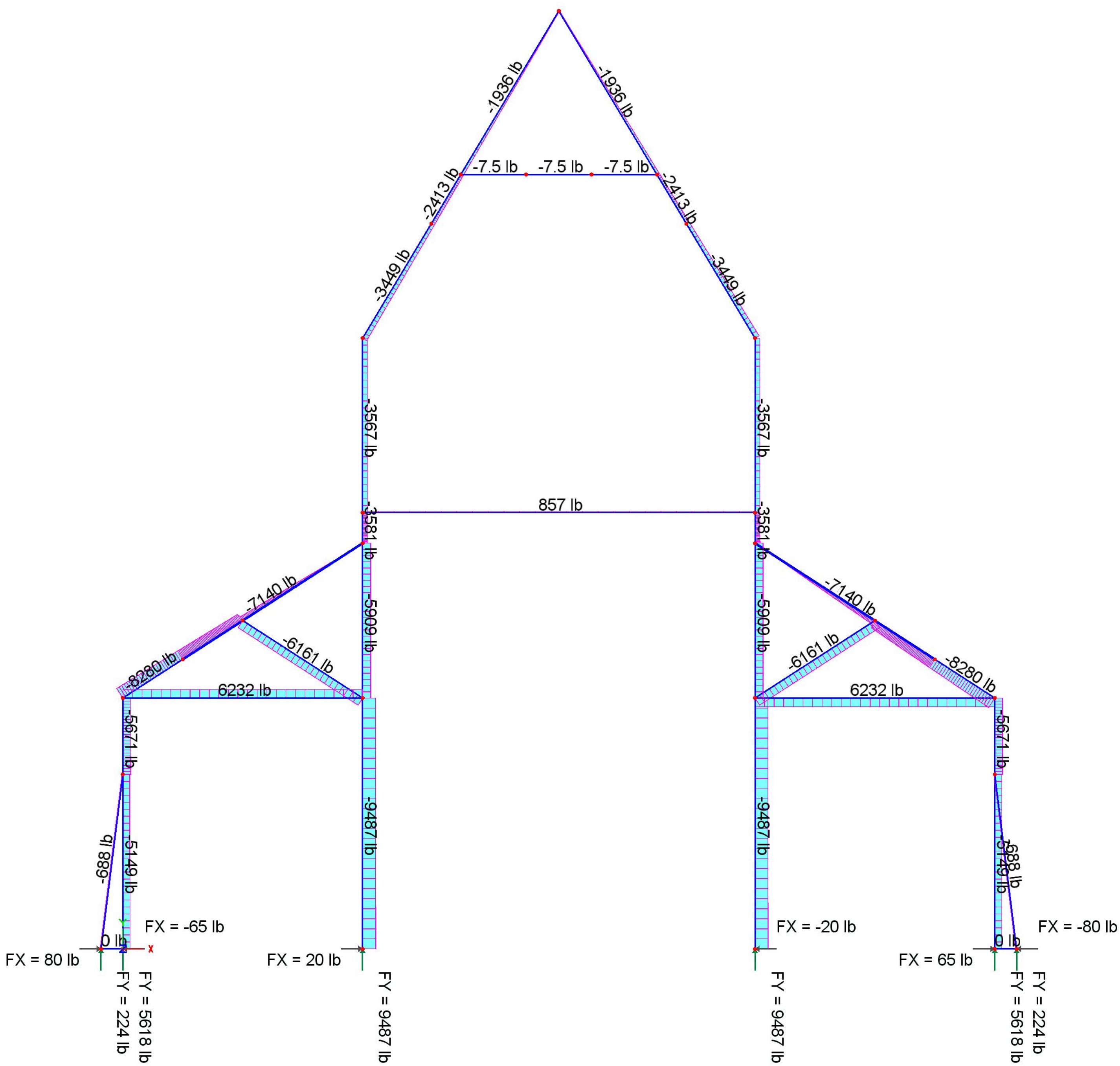
Oct 20, 2025; 12:22 PM
Result Case: D+S
Member Fx, axial force
IES VisualAnalysis 12.00.0020



Original Timber Bent
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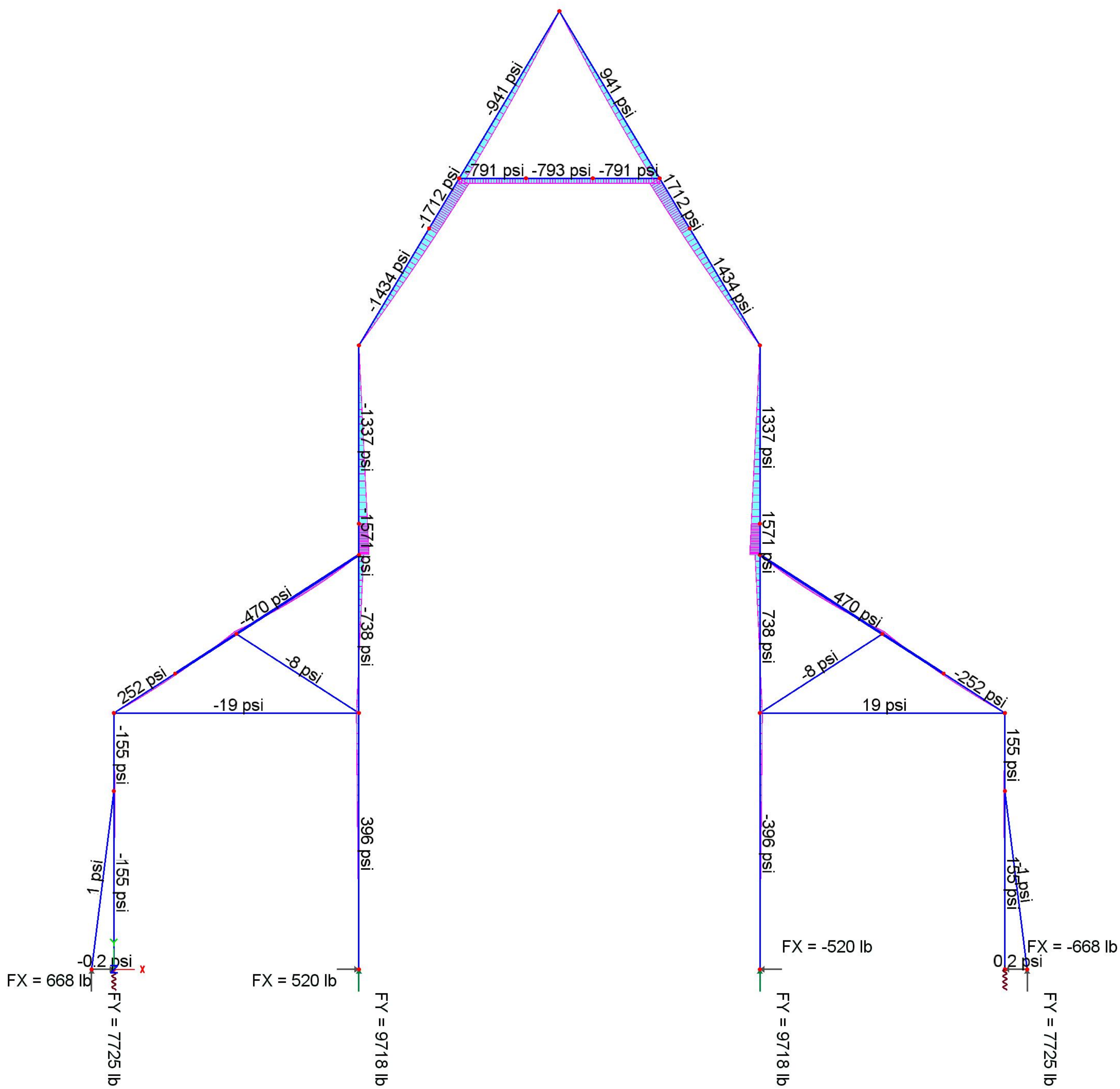
Result Case: D+S
Member fbz(+y), bending
IES VisualAnalysis 12.00.0020



Timber Bent with tie rod
ANNETTE DEY ENGINEERING, Annette Dey P.E.
Oct 20, 2025; 12:19 PM
Result Case: D+S
Member fbz(+y), bending
IES VisualAnalysis 12.00.0020