HOLY FAMILY CATHOLIC CHURCH
Structural Analysis of Church Building

316 South Logan Street
Lincoln, Illinois

Final Report
October 17, 2013
WJE No. 2013.3390.1

Prepared for:
Holy Family Catholic Church
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Lincoln, Illinois 62656

Prepared by:
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WJE

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INTRODUCTION

At the request of Holy Family Catholic Church, Wiss, Janney, Elstner Associates, Inc. (WJE) has completed a limited structural analysis of key components of the floor and roof structural framing of the main church building at 316 S. Logan Street in Lincoln, Illinois. This report summarizes the findings of our field and analytical investigation and provides general remedial options regarding the existing structural framing.

WJE was initially involved in the determination of the cause of a partial collapse of the church bell tower in June 2013. We were subsequently retained by the Parish to perform a general condition survey of the remainder of the masonry, building envelope, and primary structural systems in the building. Our corresponding report issued to Holy Family on August 21, 2013 identified the need for fairly widespread repair and maintenance of the masonry walls in order to promote the long-term serviceability of the building. Also noted was a pressing need to address advanced masonry deterioration of the upper portions of the bell tower and the building's chimney. WJE's visual survey of the roof and floor framing did not reveal any obvious signs of serious structural distress atypical for buildings of this type and vintage. However, WJE indicated that a more detailed investigation could be undertaken to more precisely document the existing roof framing and better understand the reliable load carrying capacity of the structure as compared to modern design standards. The Parish elected to have WJE proceed with this effort so that the findings could be used by the Parish to guide its decisions regarding future usage of the building and capital planning.

This report amends and expands upon our assessment of the main church building in the August 21, 2013 report and assumes the reader's familiarity with its content.

STRUCTURAL SYSTEMS

The framing system of Holy Family Catholic Church predominantly consists of wood interior members supported by the exterior masonry walls and piers. Nailed connections between wood members are common, though the roof framing also includes notched bearing connections, steel ties, rods, and bolts.

Mr. Kurt Holloway and Mr. Steve Zimmerman of WJE conducted a field investigation on September 18, 2013 to document the specific geometry of the primary roof framing and observe typical conditions not visible previously. A representative from Otto Baum Construction assisted WJE throughout the field assessment, providing access, equipment, and casual labor to facilitate the investigation.

During the course of the field investigation, WJE found that the geometry of the typical trusses supporting the main roof varied substantially from information provided in a 2006 report by Brown Engineers, Inc., a copy of which is appended to this report for reference. The following sections describe the main framing systems.

First Floor Framing

The framing supporting the first floor typically consists of 1x6 wood deck boards running diagonally over 2x10 sawn lumber joists. The joists are typically spaced at 16 inch centers and span approximately 10 feet 6 inches, from pockets in the masonry bearing walls on the exterior to built-up wood beams at the interior of the church. A typical built-up beam consists of five 2x12 members connected together with a nominal amount of nailing and spans approximately 12 feet between square clay masonry piers (1 foot 10 inches wide). The piers are in turn supported by cast-in-place concrete spread footings. Given the vintage and detailing of the structure, it is likely that these concrete footings are unreinforced. The wood framing connections are made with face and toe nail connections. The 2x12 members in the main beams did not appear to be sufficiently fastened together to behave compositely with each other. It is not clear if composite action was intended in the original design.

Roof Framing

The main gabled roof, which slopes 45 degrees, is supported by a series of heavy timber trusses incorporating steel tension rods, bolts, and ties. 2x8 diagonal bracing spans between the trusses and, in conjunction with the roof sheathing and purlins, provides lateral stability for the trusses. Three types of trusses were observed and documented within the attic space above the church.

Typical Main Truss

The typical main trusses are located above the sanctuary of the church. Five trusses of this type support the roof and barrel vault ceiling between the choir loft and the altar. The main trusses, initially believed to be scissors-type trusses with supplemental members, are more accurately described as a hybrid system that relies on a combination of arch, bending, and limited truss action to resist gravity and lateral loads. The main trusses are spaced roughly 13 feet 6 inches apart. They span the full width of the sanctuary (about 41 feet 6 inches) and bear on the north and south masonry walls coincident with the exterior masonry buttresses. Figure 1 is a sketch showing the general arrangement of a typical main truss.

The typical main trusses are generally configured with sloping top chords consisting of 6x8 timbers and a laminated bottom-chord arched member carrying the ceiling rafters, lath, and finishes of the interior sanctuary barrel vault. The arch member in turn bears on top of a pair of “hammer” trusses extending inward from the masonry buttresses (Figure 2), while the top chords bear directly on the exterior masonry walls. A series of web members connect the top chord and arch member; 7/8 and 3/4-inch diameter steel rods transmit tension forces from the arch back to the top chords while timber struts provide opportunities for compressive force transfer from the top chord back into the arch and hammer trusses. A main 6x6 horizontal cross tie between the top chords at the top of the barrel vault also provides a means of redistributing load across the structure.

Compression connections in the main trusses typically incorporate nominal toe nailing and notched bearings for improved engagement. Steel ties, lag bolts, and rods are used for tension connections between main members of the trusses.

The exact size and construction of the main arch and hammer truss members could not be determined without inspection openings into the finishes. In lieu of damaging the finishes, reasonable assumptions were made for elements that could not be definitively confirmed based on the observed portions of these members and external dimensions of the finishes.

Choir Loft Truss

The first truss from the west wall above the choir loft is different from the main truss described above. This truss has no steel tension rods linking its top and bottom chords. Instead, this truss has extensions of the 6x8 bottom chord members toe-nailed into the 6x8 top chords and relies on a main 2x8 horizontal cross tie connecting the bottom chords and extending to the top chords to achieve a measure of truss action. The choir loft truss, which spans approximately 25 feet, bears directly on two interior masonry walls.

Compression connections in the choir loft truss typically incorporate nominal toe nailing and notched bearings for improved engagement. Tension connections are achieved with nails into the faces of the joined members. Arch, strut, and bending actions also contribute significantly to the structural resistance of this system. Bolts are used on a limited basis to comprise the balance of the connections in this truss.

While the barrel vault wood framing is nominally connected with toe-nails into the choir loft truss bottom chords and cross tie at several locations, observed prior movements in this truss suggest that the barrel vault may be largely self-supported at this location. The weight of the barrel vault may be independently carried by the built-up main arch member, which also bears on the masonry walls further below the truss bearing.

Altar Truss

The truss supporting the roof framing above the altar is a more traditional example of a scissor truss. It spans approximately 20 feet 6 inches and is supported by timber columns. The truss is spaced roughly 9 feet 6 inches from the east wall of the church sanctuary, and is centered between this wall and the junction of the lower (altar) roof and main roof.

The top chord of the altar truss is composed of 6x6 timbers, while the bottom chord consists of 2x8 members. A vertical 4x6 timber connects the peak of the truss with center of the bottom chords, acting as the main tension tie. The 2x8 bottom chords extend beyond the junction with the vertical tie to the top chord to permit them to act as struts to redistribute load away from the top chords under lateral loading. It is notable that all of the tension connections in this truss are made with nails into the side faces of the members (i.e. no structural steel connections are present).

There is no substantial connection between the barrel vault ceiling and the altar truss indicating that the vault is self-supporting at this location. Since the two systems are structurally independent, the altar truss is only subjected to roof dead (i.e. self-weight), snow, and wind loads.

Secondary Framing

The remainder of the roof framing consists of 1x6 tongue and groove decking boards running over 2x6 rafters aligned parallel to the main trusses. The rafters are spaced between 17 and 22 inches on center as measured at select locations and are supported by 6x6 purlins which span between the main trusses at a spacing of approximately 7 feet 6 inches. The rafters frame into a 2x6 ridge board at the peak of the roof. The rafters are typically connected to the purlins and ridge board with a pair of toe-nails; the purlins are fastened to the top chord of the trusses with a 3/8 inch diameter bolt with a 1½ inch diameter washer. The framing for the smaller dormer gables adjacent to the exterior wall was not readily visible inside the main attic, and it is likely simply “stick-framed” onto the main roof rafters and decking.

LABORATORY INVESTIGATION

During the course of our initial condition assessment and our current field investigation of the church framing, WJE collected wood samples from representative structural members. The wood samples were sent to Mr. Terry Highley, a wood pathologist, to determine the species of lumber used in the construction. Table 1 summarizes the wood species identifications for the various members sampled during the current and previous field investigations. The species of wood used in a structural member plays a significant role in determining its mechanical properties (e.g. bending strength, compressive strength, stiffness).

Table 1. Species Identification of Structural Members

Sample Location / Member	Wood Species or Species Group
Typical Truss 6x8 Top Chord	Red Pine (Pinus Resinosa)
Typical Truss Arched Bottom Chord	White Pine (Pinus Strobus)
Typical Truss 6x6 Main Cross Tie	Southern Pine (Pinus Spp.)
Roof 2x6 Rafter	Southern Pine (Pinus Spp.)
First Floor 2x10 Joist	Southern Pine (Pinus Spp.)
First Floor Built-Up Main Beam	Southern Pine (Pinus Spp.)

FRAMING CONDITION OBSERVATIONS

WJE noted the following conditions present in the roof framing during our recent site visit. These observations supplement those contained in WJE's August 21, 2013 report.

- Much of the original plaster of the barrel vault ceiling was concealed, but appears to have remained in place behind the plywood finish installed during a recent renovation project. (Figure 3).
- Moisture staining was observed at a purlin bearing location along the east sanctuary wall with some minor insect damage (Figure 4 and 4a). This damage was not deemed structurally significant.
- Moisture and staining were observed at the south bearing of the choir loft truss (Figure 5), but no structurally significant deterioration of the wood was found.
- The bearing areas of the typical main truss top chords and hammer truss top chords on the masonry wall were largely encased in masonry (Figure 6). Accordingly, WJE could not assess whether deterioration was occurring at these locations. Visible portions of the bearing locations did not exhibit any obvious signs of distress.
- WJE observed a partial fracture on the top face of one of the main barrel vault built-up arch members at a typical truss (Figure 7). This was observed near the top of the arch, just south of the center of the truss, north of the first compression web strut.
- Numerous compression struts exhibited partial pull-out of their toe-nailed connections, especially near the center of the typical truss (Figures 8 and 9).

STRUCTURAL ANALYSES

Using the information gleaned from the field and laboratory studies, as well as observations from the condition survey, WJE analyzed the primary structural systems of the church using computer models and hand calculations. Typical secondary roof framing members (i.e. typical rafters and purlins) and selected nailed and bolted connections were also reviewed. The purpose of the analysis was to determine the ability of the main structural framing members to resist characteristic loadings required by contemporary model building codes. Although older structures are typically not required to comply with current code provisions for new structures due to "grandfathering," evaluation of such structures for current code-prescribed loadings can be instructive when making long-term capital decisions.

For the church building, our analysis applied characteristic dead, live (i.e. occupancy), wind, and snow loads in accordance with current structural code provisions (ASCE/SEI Minimum Design Loads for Buildings and Other Structures, 2005 Edition). Seismic (earthquake) loadings were neglected. Computer modeling of each of the main roof truss types was performed using finite-element analysis software. The models captured the effects of key field observations and the sequential application of loads. Figures 10, 11, and 12 illustrate the general arrangement and key features of the main typical truss, choir loft truss, and altar truss models respectively.

The following key assumptions were made in our analysis:

- The wood species assumed for members that were not sampled were assigned based on the laboratory results for samples of members of comparable cross-section (e.g. hammer truss solid lumber 6x6 assigned Southern Pine based on 6x6 typical purlin lab results).
- WJE typically assumed a Select Structural grade of the lumber. Although this is a best-case scenario for determination of allowable stresses, WJE field observations of readily accessible portions of the exposed framing generally confirmed that this grading was not unreasonable.
- The masonry buttresses and walls provide considerable resistance to the outward thrust of the trusses under gravity loads and restraint against lateral wind forces. This assumption permits arch action to develop in the systems in addition to bending and truss actions. These properties were modeled using horizontal springs in addition to the vertical bearing supports at the base of the truss models. The assumed resistance was calibrated based on measured lean of the buttresses and substantiated by a limited capacity analysis of the unreinforced masonry for the forces imparted to the walls.
- Connections where nails had pulled out or members had separated were presumed to have no tensile capacity. Similarly, slender steel rods were assumed to provide only tension resistance (i.e. they would buckle under a minimal compressive load).
- Preliminary calculations revealed that roof live loading would not govern compared to snow loads on the steep roof and was excluded from the analysis.
- Loads coming into the trusses from the roof (i.e. roof dead, snow, and wind) were assumed to act through the purlins at their bearing locations on the top chord members and purlin bracing members, where appropriate.

Loads were applied to the computer model, and the model was used to determine internal stresses in key elements under the various load combinations. The member demands in the typical truss under dead load are illustrated in Figures 13, 14, and 15.

DISCUSSION AND FINDINGS

First Floor Framing

WJE revised our preliminary analysis of the first floor framing described in our August 21, 2013 report to account for the new lumber species information for these members (Southern Pine) and to more precisely consider the typical distribution of live loads in the church sanctuary (i.e. code prescribed live loads for new structures are 100 psf for aisles and walkways and 60 psf for areas with pews). WJE conservatively assumed a No. 1 lumber grading for the floor joists, but assumed a grading of Select Structural for the main beams due to the built-up, redundant nature of the section. Further, WJE conservatively discounted any capacity benefit provided by the fifth 2x12 in the section, which frequently had inadequate or absent bearing on the masonry pier; however, the face-nailing used in the building of the section likely results in a some sharing of bending loads in the fifth 2x12 despite the lack of sufficient end bearing.

Our analysis indicates that the typical floor joists and main beams are capable of supporting the full code-prescribed live loads.

WJE reiterates our recommendations from our August 21, 2013 report for the floor framing. Specifically, the single fractured floor joist warrants repair as a matter of good practice. In addition, metal joist hangers are recommended to replace the less reliable end grain nailed connections, which are frequently found near framing box-outs for mechanical penetrations and where headers are used to pick up joists framing in a different direction (e.g. at the basement stair).

Roof Framing

Roof Trusses

Table 2 summarizes the calculated demand-to-capacity ratio (D/C) of the primary members of each of the main trusses for selected critical loading cases. A member whose load demand exceeds its allowable capacity (i.e. D/C is greater than 1.0) is considered overstressed relative to current code provisions. Some members have calculated overstress under dead (self-weight) loads alone, while other members have calculated overstress when subjected to design-level snow and wind loadings. It is important to note that wood allowable strengths are significantly influenced by the duration of the load as well as its intensity; this is due to wood's inherent tendency to accumulate damage and lose resistance under sustained high loadings. Accordingly, for some members the D/C is greater under dead load alone than under snow or wind loadings, which are considered to act for relatively brief periods of time.

Table 2. Selected Calculated Demand-to-Capacity Ratios of Typical Truss

Member	Load Case	Demand/Capacity*
Center Steel Tie Rod (7/8 inch diameter)	Dead	0.3
	Dead + 0.75 (Snow + Wind)	0.3
Top Chord (6x8)	Dead	1.3
	Dead + Snow	1.6
	Dead + Wind	>2.0
Arched Laminated Bottom Chord	Dead	1.1
	Dead + Snow	1.2
	Dead + Wind	1.0
Main Cross Tie (6x6)	Dead	1.0
	Dead + Snow	1.1
Hammer Truss Top Chord (6x6)	Dead	1.0
	Dead + Snow	0.9
Hammer Truss Bottom Chord (6x6)	Dead	0.6
	Dead + Snow	0.7
Hammer Truss Strut (6x6)	Dead	0.3
	Dead + Snow	0.3
Lower Arch Steel Tie Rod (3/4 inch Diameter)	0.6 Dead + Wind	1.7
Lower Arch Strut (6x6)	0.6 Dead + Wind	0.2

* Demand/capacity ratios based upon member stresses; some connections also overstressed

Table 3. Selected Calculated Demand-to-Capacity Ratios of Choir Loft Truss

Member	Load Case	Demand/Capacity*
Top Chord (6x8)	Dead	0.9
	Dead + Snow	1.3
	Dead + Wind	0.6
Bottom Chord (6x8)	Dead	1.1
	Dead + Snow	1.6
	Dead + Wind	0.9
	Dead + 0.75 (Snow + Wind)	1.2
Main Cross Tie (2x8)*	Dead	0.1
	Dead + Snow	0.1
	Dead + Wind	0.1

* Demand/capacity ratios based upon member stresses; some connections also overstressed

Table 4. Selected Calculated Demand-to-Capacity Ratios of Altar Truss

Member	Load Case	Demand/Capacity*
Top Chord (6x6)	Dead	0.2
	Dead + Snow	0.5
Bottom Chord (2x8)	Dead	0.2
	Dead + Snow	0.3
	Dead + Wind	0.1
Vertical Tie (4x6)	Dead	0.1
	Dead + Snow	0.2

* Demand/capacity ratios based upon member stresses; some connections also overstressed

Based on limited analysis of other nailed and bolted connections in the trusses, significant overstress may exist at many interfaces based on current code provisions. For example, under some loading conditions, the nailed connections in the Altar Truss are overstressed, (Figure 16). More in-depth analysis of other key connections would be warranted as part of a complete capacity determination of the structure.

Rafters and Purlins

Based on the assumed Select Structural grading, our analysis indicates the typical rafters and toe-nailed connections are adequate for code-prescribed demands. The typical purlins were also found to be adequate for code-prescribed snow and wind loadings. The bolted connections between the purlins and the main trusses also perform acceptably under full wind demands.

Summary

The results of the analysis indicate that many of the key components of the roof framing have calculated overstress when subjected to current code-prescribed loadings. It is not uncommon for older structures—especially those with elements such as heavy timber trusses that were typically constructed based upon convention rather than design provisions—to be deficient when evaluated for current requirements applicable to new buildings. Further, such structures are not required to meet current code provisions for new structures unless they are modified or have a change in occupancy. Our field observations and structural analyses suggest that unquantifiable redundancies (e.g. lath and plaster shell action, rafter and sheathing inclined wall action) and material safety factors are being partially relied on throughout much of the structure. While these redundancies are not typically relied upon in current design practice, they can provide significant capacity beyond that calculated using conventional analytical approaches. The observed deflections in the hammer trusses and separations at the interfaces of some truss connections indicate that at least some of the load originally carried by the main structural framing has since transferred to other non-structural components such as the interior finishes.

The sustained presence of significant dead load forces, coupled with exposure to future snow and wind load events, leads to a heightened potential for the roof structure to develop further damage. In particular, members of the structure significantly overstressed under dead load (e.g. the typical truss top chords) have the potential to accumulate damage in the material over time. This cumulative damage phenomenon, which uniquely affects wood structures, results in a gradual reduction in the reliable resistance of the material in zones of sustained high overstress in the wood. At the microscopic level, the interwoven wood fibers continue to strain and deform under the constant stress, often referred to as “creep”. This effect in the micro-structure of the wood results in a reduction in the strength of the structural member. Taken collectively, the results of the analysis indicate that the reliability of the church roof structure is less than that required by current standards for new facilities.

CONCEPTUAL REPAIR OPTIONS

As indicated above, there is no explicit requirement that the church structure comply with the current building code provisions for new structures. However, based upon the level of calculated overstress in the roof framing members, the Parish may wish to consider implementing strengthening repairs as part of the upcoming exterior construction program. Several conceptual repair approaches of varying extent and cost that could be considered by the Parish are described below and summarized in Table 5.

Option 1 - Monitor Existing Structure

Periodic monitoring the existing roof framing is a permissible approach since the church structure is not required to comply with current code provisions for new structures. In this approach, the need for strengthening repairs would be further evaluated if/when additional distress in the roof framing (e.g. further or newer separations at joints) or adjacent interior finishes (e.g. cracking in plaster or drywall) is detected.

Although evidence of displacement and deflections of the roof structure is widespread, our observations indicate that much of this distress has occurred over a long period of time. Though the calculated overstress in the structure under dead loads certainly presents the possibility of gradual damage accumulation in some members, it is also possible that the roof framing has reached equilibrium under dead load forces via reliance on unconventional redundant load paths. While analysis indicates that such redistribution of a portion of the roof loads may be occurring, the extent of this action is challenging to determine, and the capacity of the redundant elements is difficult to reliably calculate.

Accordingly, selection of a monitoring option would provide the least amount of additional reliability beyond what currently exists in the building structure. It is also the least costly of the three approaches.

In addition to visual monitoring by Church staff, the Parish could also consider the installation of survey points at key locations to determine if additional deflections of the roof framing or lateral displacements of the masonry buttresses are occurring. Survey data could be obtained at regular intervals (say every 6 months) and after major wind or snow events. Any additional movement or observed distress should be reported to a licensed structural engineer.

Option 2 - Targeted Strengthening for Increased Structural Reliability

The Parish could also elect to implement targeted strengthening repairs to increase the capacity of selected overstressed elements and by extension improve the overall system reliability. Such repairs could include some or all of the following concepts:

- Inspection openings of the interior finishes to verify connections and member dimensions not accessible from above (e.g. in the barrel vault arches and hammer trusses); this may influence the extent and nature of the repairs.
- Reinforcement of key nailed and bolted connections.
- Localized reinforcement of main members at locations of high bending loads (e.g. steel side plates or channels on the top and bottom chords in regions near steel hanger rods).
- Selective addition of supplemental connections and/or wood members to better support the barrel vault and promote system redundancy.
- Supplemental posts to support the hammer trusses and barrel vault, relieving load on the main truss members.

While this repair approach would stop short of bringing the entire structure into code compliance, it would provide more conventional and reliable load paths for the roof framing. Careful consideration should be given to code provisions regarding upgrading of existing structures, such that additional strengthening repairs are not triggered. A licensed structural engineer should be retained to design the repairs, prepare construction drawings, and provide construction period oversight. For budgeting purposes, a repair allowance of $35,000 to $50,000 per truss seems appropriate for this option.

Option 3 - Strengthening for Compliance with Current Codes

Extensive repairs would be required if the Parish desires to bring the existing roof framing system into compliance with current structural codes. In fact, costs to bring the roof framing in compliance may approach the cost of a complete replacement of the roof framing. Obviously, the nature and extent of the repairs would cause a significant disruption of church operations. While the full extent and details of the repairs would be determined with further investigation and analysis by a design professional, WJE expects a code-compliant strengthening program might include:

- Mechanical anchorage of the main trusses to the buttresses and walls for wind uplift forces, possibly including localized steel reinforcement of the masonry.
- Widespread reinforcement of the main structural members (top and bottom chords) to improve bending capacity.
- Reinforcement of key connections and addition of new connections capable of transmitting tension across existing compression-only joints.
- Addition of new web members to promote redundancy.
- Supplemental posts to support the hammer trusses and barrel vault, relieving load on the main truss members.
- Additional strengthening for earthquake demands.

Again, given the number of conditions warranting strengthening, and the difficulty in working in a congested attic space, it may be more feasible to simply replace the roof framing if the Parish elects to go with Option 3. It is difficult to accurately quantify the costs associated with this option since the actual extent of repairs would have to be determined by a more thorough analysis. It seems likely that repair costs would easily surpass $500,000 for this repair approach.

Table 5 summarizes the three options generally described above.

Table 5. Summary of Conceptual Repair Options

Option	Description	Added Structural Reliability	Order-of-Magnitude Costs
1	Monitor framing and finishes	Minimal	$10,000 annually
2	Targeted strengthening repairs	In between	$200,000 - $300,000
3	Strengthening to current code requirements	Same as new structure	>$500,000

CLOSING

WJE has completed a limited structural assessment of key components of the building framing at Holy Family Church. This investigation has revealed calculated overstress in many of the primary roof framing members when subjected to characteristic loadings prescribed by current structural codes. Localized distress and deflections in the roof framing members generally corroborates the findings of WJE's analytical modeling.

Although the church structure is not required to comply with current structural requirements, the widespread nature of the calculated overstress may warrant consideration of at least some level of strengthening repairs in the near term. If repairs are elected by the Parish, a licensed structural engineer should be retained to design the repairs, prepare construction documents, and provide construction period oversight.

LIMITATION

Because of the limitations in detecting concealed internal distress in many components, this investigation may not find unsafe and imminently hazardous conditions that are not readily visible. WJE shall not be responsible for latent or hidden defects that may exist, nor shall it be inferred that all defects have been either observed or recorded. However, we have performed this inspection and prepared this report in accordance with the applicable standard of care for engineers performing such services.

FIGURES

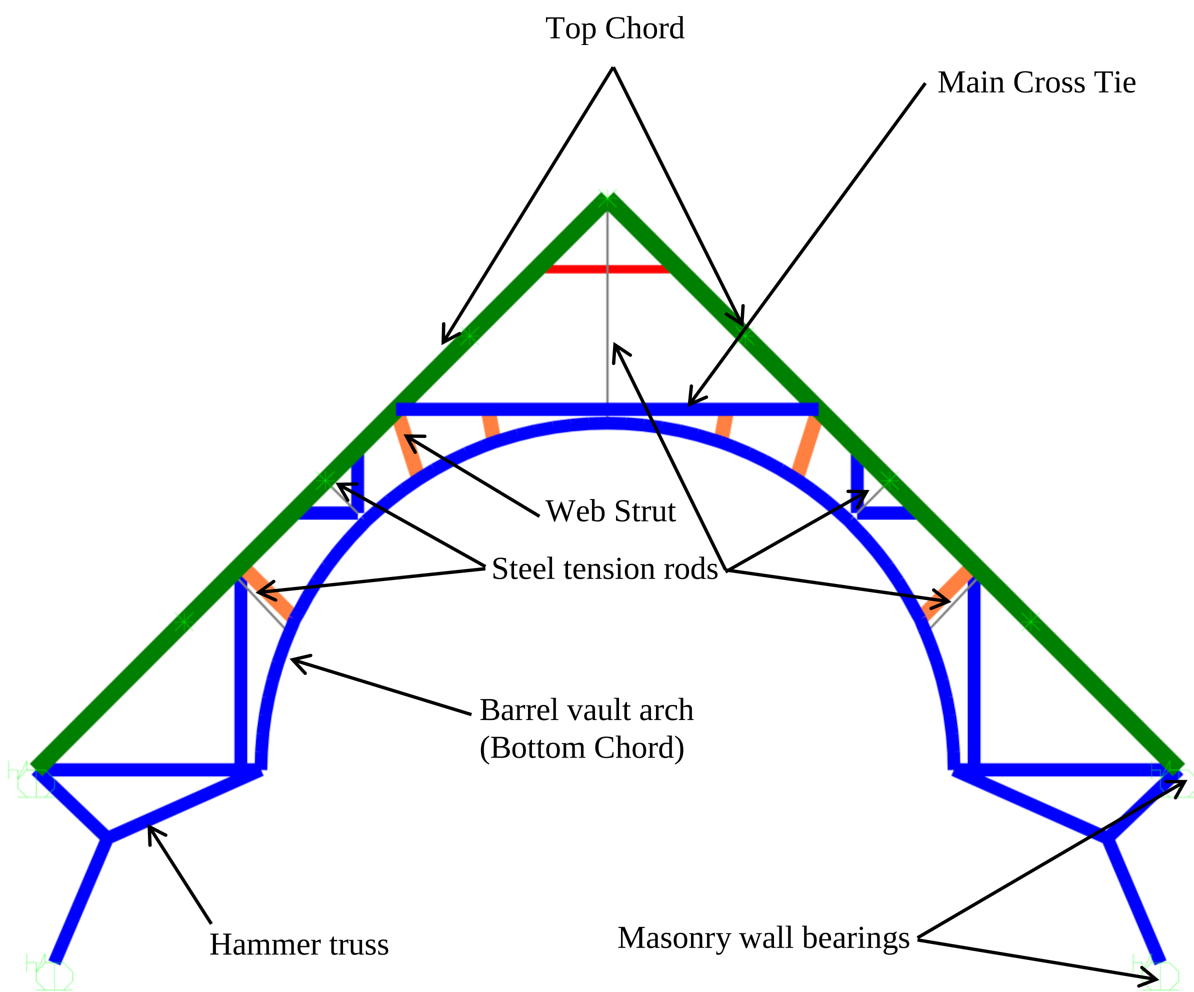

Figure 1. Rendering of typical truss illustrating arrangement of primary members and features of construction.

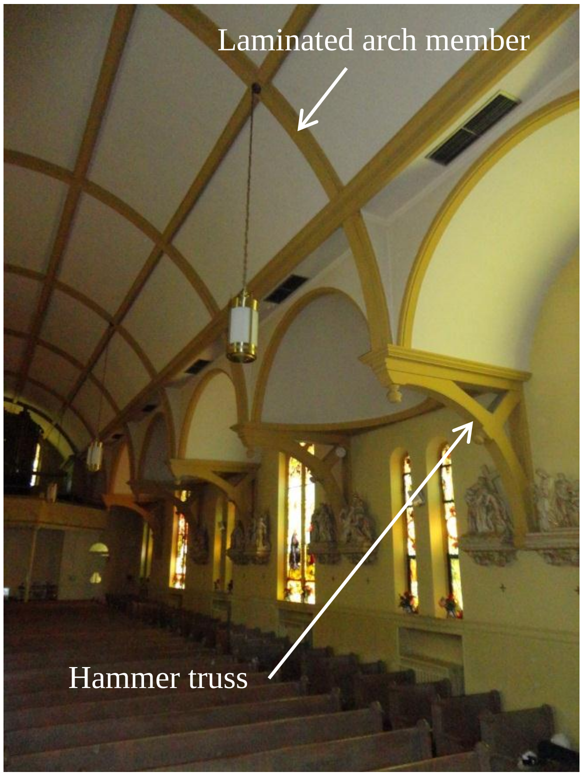

Figure 2. Interior of sanctuary showing main barrel vault arch members bearing on hammer truss.

Figure 3. Original lath and plaster largely intact beneath insulation; plywood infill appears to have been installed over top of existing plaster.

Figure 4. Insect damage visible on corner of purlin near bearing at east wall of sanctuary; not structurally significant.

Figure 4a. Another view of the purlin bearing at east wall of sanctuary showing extent of moisture related staining. No significant wood decay was observed.

Figure 5. South bearing of choir loft truss on interior masonry wall. Note moisture staining; no deterioration observed.

Figure 6. Typical truss bearing at exterior masonry wall (at buttress). Actual bearing condition concealed by masonry beyond steel connection tie in foreground.

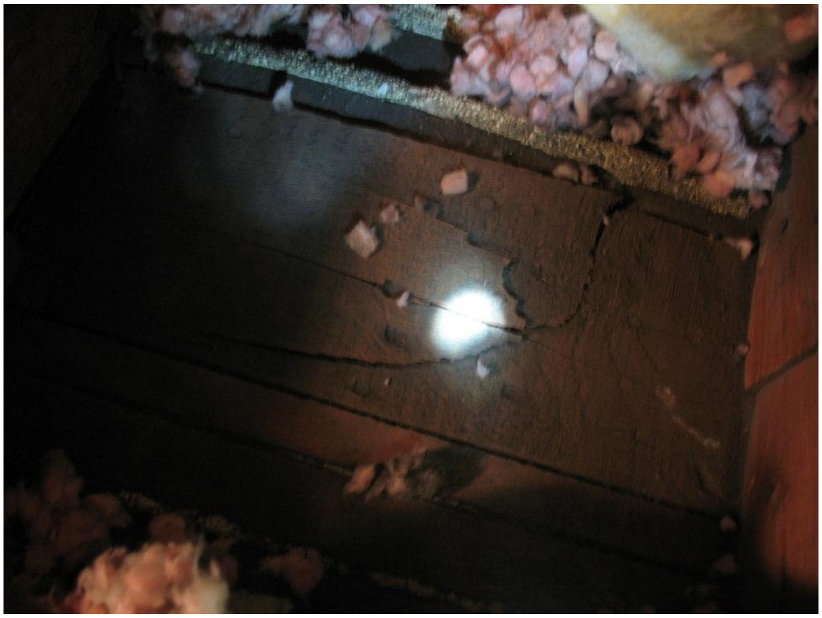

Figure 7. Partial fracture of built-up main arch member of barrel vault in typical truss.

Figure 8. Example of disengaged compression web member connection (double bird's mouth) on typical main truss consistent with downward displacement of the arch and spreading of the top chords.

Figure 9. Another example of a typical truss compression strut with toe-nailed connection partially pulled-out.

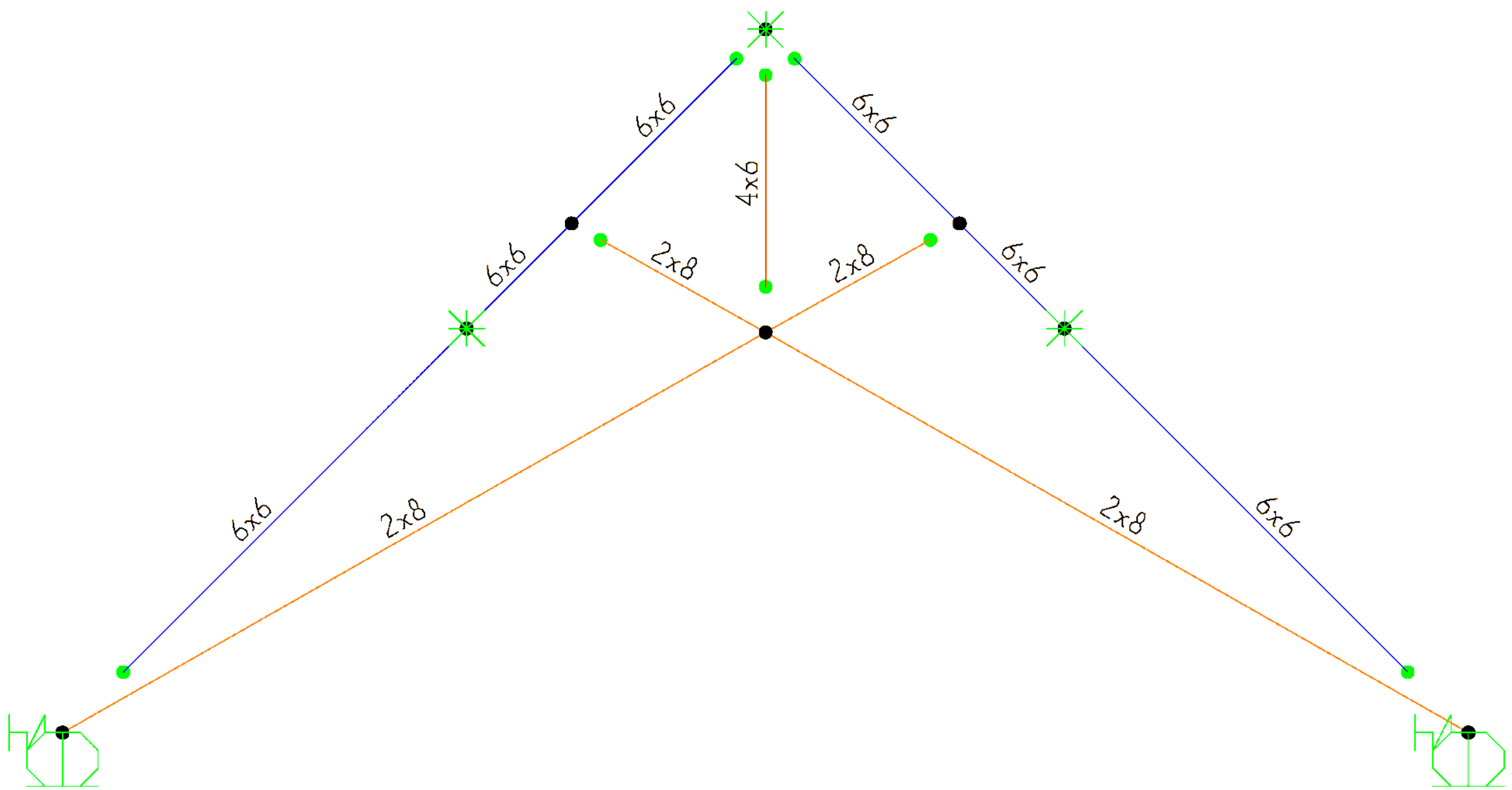

Figure 10. Finite element model of altar truss indicating member sizes.

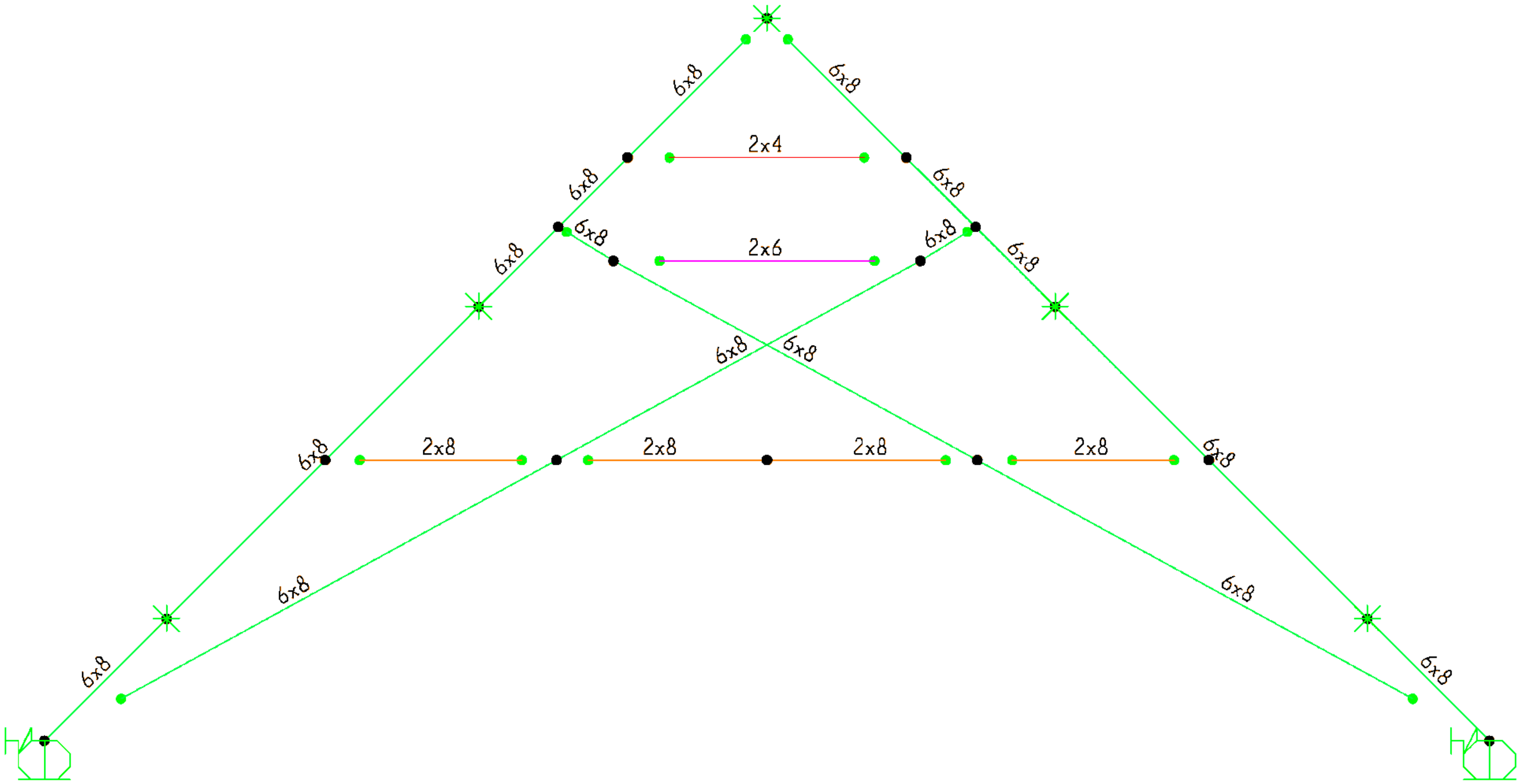

Figure 11. Finite element model of choir loft truss indicating member sizes.

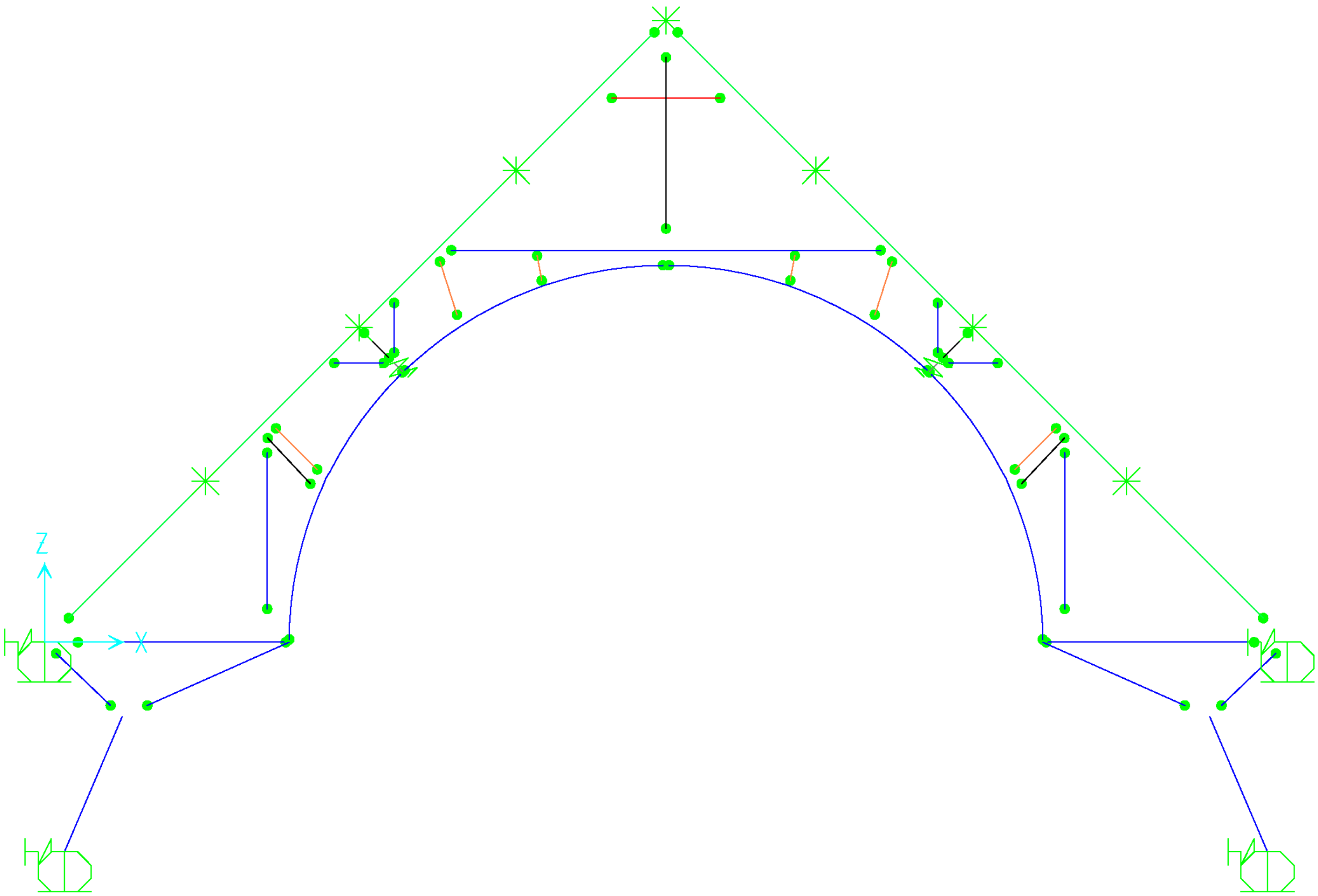

Figure 12. Finite element model of typical truss. Note: Blue members are 6x6 timbers, green members are 6x8 timbers, orange members are 4x6 lumber, red top cross tie is a 2x4, and black members are steel tie rods.

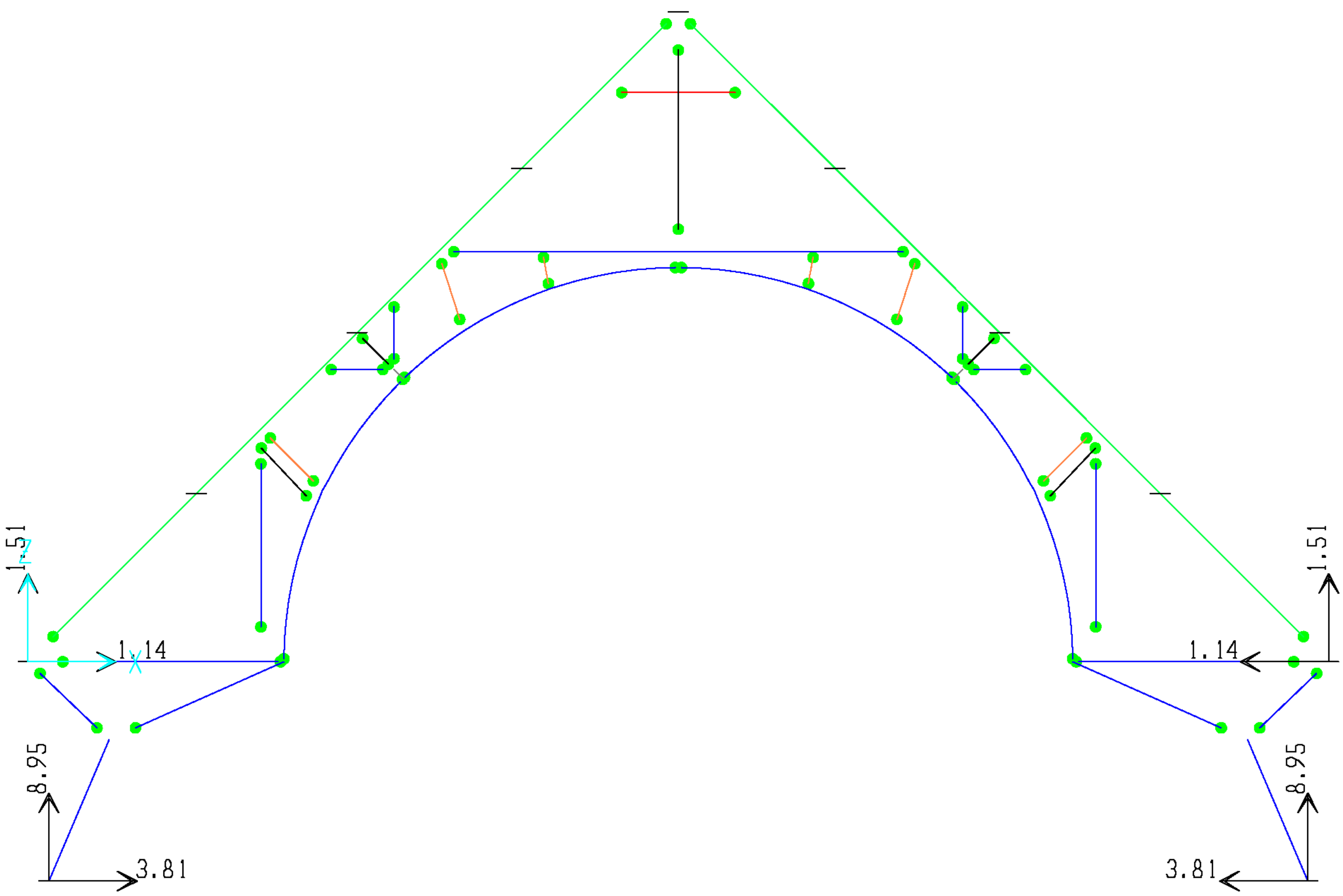

Figure 13. Typical truss force reactions on masonry walls under dead load (self-weight), in kips. Note: 1 kip = 1000 pounds. Indicative of the level of arch action and resistive thrust provided by the masonry buttresses.

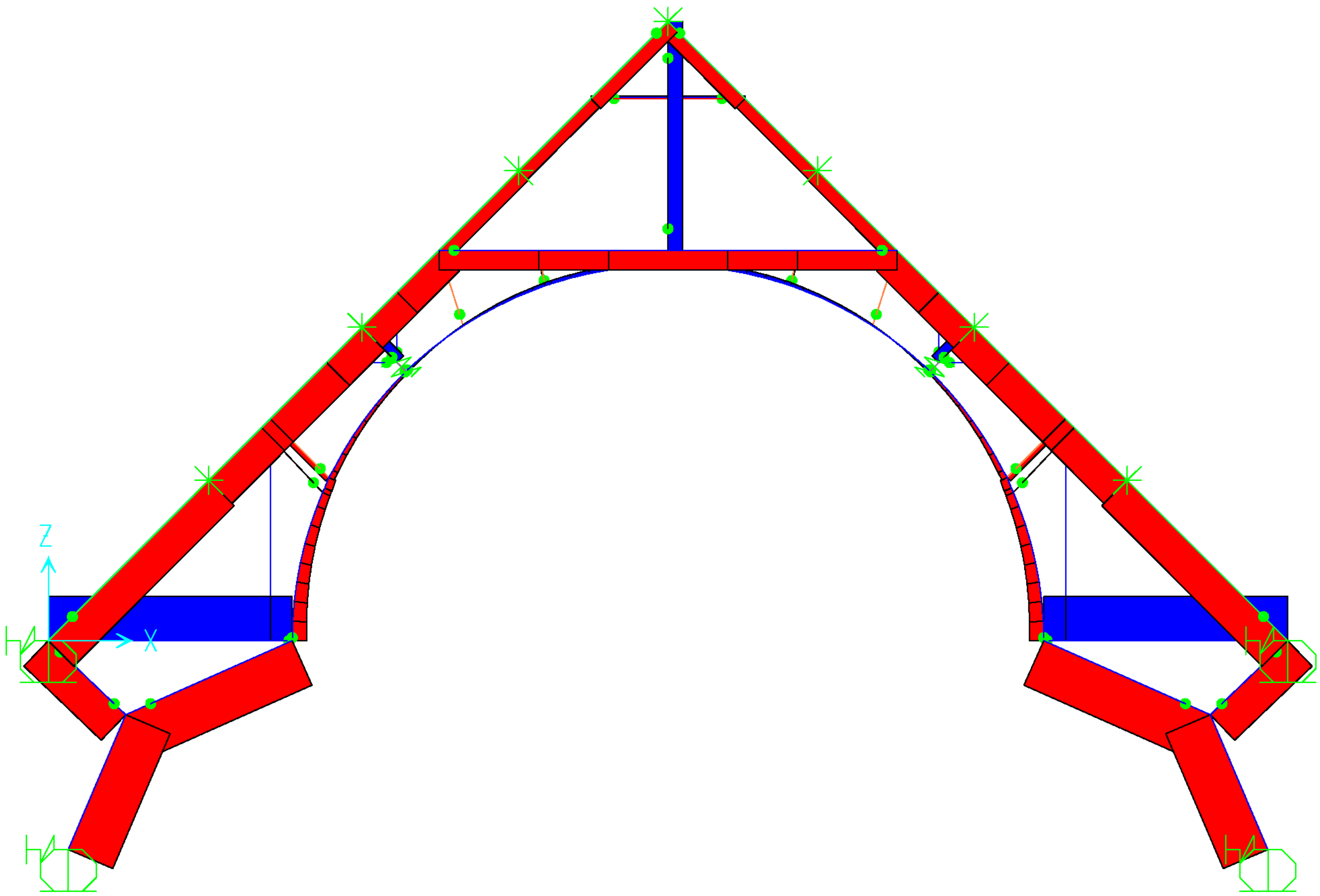

Figure 14. Typical truss diagram showing member axial force results under dead load; force results are to scale. Blue indicates tension, red indicates compressive force. Note: Maximum force shown is in hammer truss bottom chord = 10,000 pounds. Axial forces are indicative of truss and arch structural behavior.

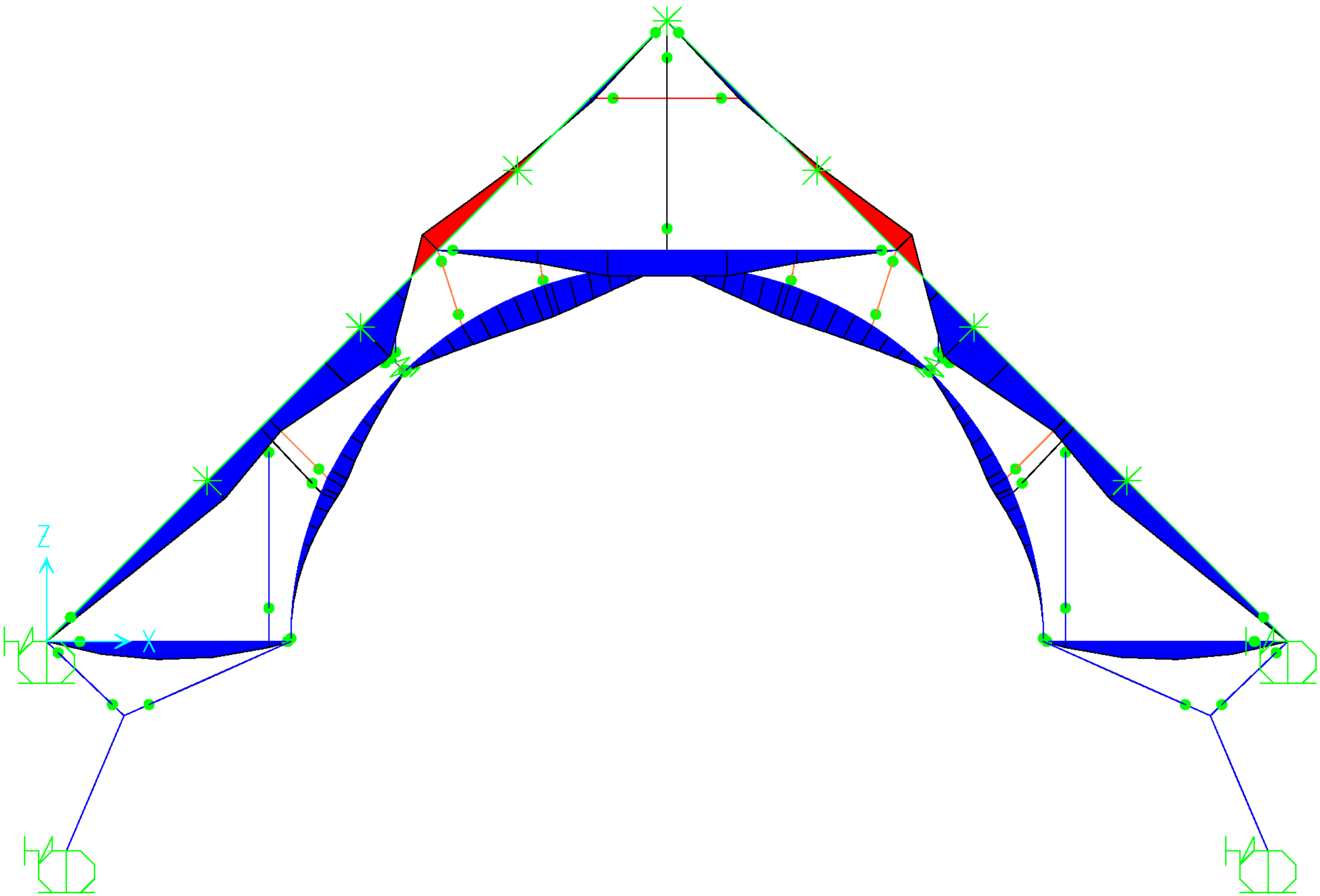

Figure 15. Typical truss diagram showing member bending load (beam action) results under dead load; results are to scale. Blue indicates downward bending, red indicates upward bending relative to the member orientation. Note: Maximum bending in top chord is about 4,500 ft-lbs.

Figure 16. View of altar truss showing face nailed connections throughout.

Appendix A

2006 Report by Brown Engineering

COPY

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RECEIVED MAR 13 2006

March 12, 2006

Holy Family Catholic Church
c/o Joe Fitzpatrick
Fitzpatrick Electric
PO Box 242
Lincoln, Illinois 62656

Dear Mr. Fitzpatrick:

At your request, I met with you at the Holy Family Catholic Church at 320 South Logan Street in Lincoln, Illinois on March 10th. The Church is considering remodeling and improvements and it was desired to have an inspection to determine its structural condition. We spent two hours examining the interior and exterior of the church, the wood floor framing in the crawl space and the roof structure.

The church was built in 1903. It is a tall, one story structure with a partial basement and partial crawl space. There is a masonry bell tower at the southwest corner. The walls supporting the roof structure appear to be wood framing. Alternately, they could be solid masonry with furred out plaster walls. The various structural areas are discussed below. A cross section thru the church is enclosed with the report.

FIRST FLOOR FRAMING

The crawl space framing was seen from the basement area. It was found to consist of 4 spans of 2x10 wood joists at 16 inch centers across the 41'-5" width of the church. The joists are supported on built-up wood beams(5-2x12's) supported on masonry piers. The built-up beams are touching or very close to the earthen crawl space surface. This is violation of codes which requires a minimum clearance of 12 inches between wood framing and earth. Evidence of termite tracks exist on the face of joists and extend up into the wood deck but no termite damage or signs of active termites was seen. The church reportedly has a termite inspection annually.

No calculations of floor capacity were made but the joist and beams appeared to be level and the floor feels stiff as one walks on it. It is likely that the framing would meet the building code load requirement of 60 psf for fixed seating plus pew weight plus floor dead load. Calculations would

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have to be made to verify this.

The floor slopes approximately 12 inches toward the altar. The cast iron radiators placed in wall niches on each side of the Church slope downward. It is not known if these were placed level and have since sunk for some reason or if they were installed with a slope.

BEARING WALLS AND ROOF STRUCTURE

The ceiling of the church is a circular barrel vault in its center area meeting " hammer" trusses projecting out from the walls and hung from the roof structure and the walls. The ceiling between the "hammer" trusses has barrel vaults perpendicular to the center vault.

We went up into the attic space and examined the roof structure. The roof is pitched at 45 degrees, a 12:12 slope. The rafters running up the slope are 1 5/8" x 5 1/4" (2x6 nominal) at 20 inch centers. The rafters are supported on three 6x6 beams which bear on the top chord of five heavy timber trusses spaced at 13'-5". These five trusses line up with and help support the wood "hammer" trusses attached to the walls. There are masonry pier projections on the outside of the building also lining up with the trusses.

The upper potions of the truss, which could be seen from the walk boards, were measured and are shown on the enclosed sketch. Some lower solid timber members of the truss were seen attached to the top chord but their alignment can't be seen without further inspection and possible removal of plaster ceilings.

The top element of the "hammer" truss, which was originally horizontal, now slopes downward on both sides of the sanctuary. This is very noticeable as one enters the Church. You measured a downward slope of 2-3 inches in 8 feet. I placed a 4 foot level on the inside surface of the exterior bearing walls and found that both walls lean outward along their full length.

The leaning of the walls and the sloping of the hammer trusses is due to a basic structural behavior which we have seen before in this kind of roof structure. The roof trusses have a sloping bottom chord, forming trusses called scissor trusses. Such a truss behaves like an arch and exerts a horizontal force on the top of the wall. The walls lean out and the truss deflects downward. Based on the 2 to 3" slope of the "hammer" truss attached to the walls, the roof trusses may be deflecting 6 inches or more.

It is not known if the roof structure and walls have reached a point of stability or if slow, additional outward movement of the walls is occurring or could occur. One solution commonly used to prevent any further wall movement is to run steel tie rods across the interior at the trusses, thru the walls and connected to a horizontal steel member on the outside face of the building. The appearance of the rods will have an architectural appearance which may be objectionable.

If it is desired to determine in detail the structural adequacy of the roof structure and walls, it would be necessary to inspect the trusses and determine all member sizes. A computer analysis would need to be accomplished to verify adequacy. The total out-of-plumbness of walls need to be determined, the details of the wall construction measured and the effect of eccentric load on the leaning walls analyzed.

EXTERIOR WALL SURFACE

The exterior face brick appears to have been completely tuckpointed at some time in the past. Numerous pieces of mortar 2 to 6 inches long or longer have fallen out of the joints. The depth of the tuckpointing appears to be very shallow. Tuckpointing is normally done to a depth of approximately 3/4 inches to bond it adequately to the brick. In my judgement, the mortar joints can be expected to continue to fall out. An experienced, commercial masonry contractor should be engaged to inspect the wall and provide advice and cost estimates for possible tuckponting of the entire exterior walls.

SUMMARY

1. The floor framing appears to be adequate but a more detailed inspection and calculations are needed to verify that. Consideration should be given to lowering the earth level in the crawl space to provide clearance between the wood and the earth and lessen the chances of termite damage.

2. The roof trusses and bearing walls have deflected considerably and leaned outward, respectively, and their structural adequacy should be examined in detail if the intent is to continue to inhabit the Church for a significant period. If a major remodeling takes place, the "hammer" trusses would presumably be rebuilt to a level position. This would entail removing and rebuilding the vaulted, plastered ceiling at the side aisles. This would be very costly. It would be prudent to tierod the building to prevent additional wall movement and truss deflection from taking place.

3. The tuckpointed mortar joints of the exterior walls should be inspected by an experienced masonry contractor. If full tuckpointing of the exterior wall is required, costs may be expected to be high.

If you have any questions regarding this report or if we can be of any further help, please contact us.

Sincerely yours,

BROWN ENGINEERING, INC.

Norman K Brown

Norman K. Brown

LICENSED STRUCTURAL ENGINEER
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ILLINOIS
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BY NB DATE 3/11/06

CHKD. BY ______ DATE ______

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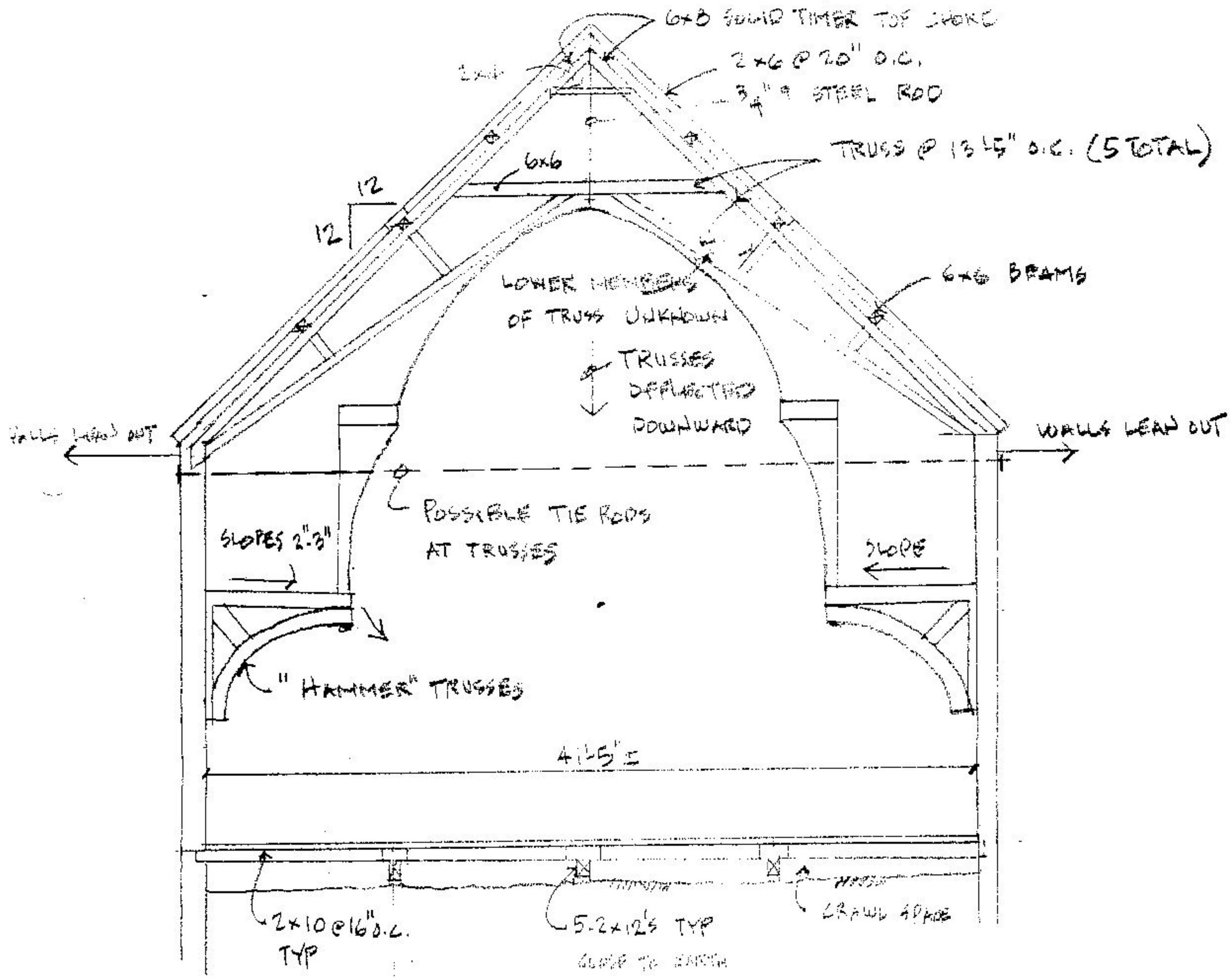
SHEET NO. 1 OF 1

JOB NO. 2632

HOLY FAMILY CHATHOLIC CHURCH



CROSS SECTION

3" = 1'-0"

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