

Failure is not always what it seems to be Timber Roof Trusses Behaving Badly

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Introduction

In the assessment of existing structures it is axiomatic that distortions of the fabric are *prima facie* evidence of failure, or incipient failure. This is particularly true when elements of the structural system, such as trusses, exhibit changes in geometry. While one cannot ignore such evidence of distress, changes in geometry do not always indicate failure. The following three case studies illustrate lessons we have learned in our practice regarding the interpretation of distress and distortions in timber roof trusses.

Peace Church, Camp Hill, PA.

Peace Church, built in 1798, is a two-story stone masonry building, approximately 41' by 49' in plan, surmounted by a gable roof supported by three timber scissor trusses spaced at approximately 12' on center, spanning the 41' dimension.

The building is at the intersection of two state highways, one of which had been widened by the Pennsylvania DOT to within a few feet of one corner of the building. So when the user group noted new cracks in the masonry walls and barrel vault plaster ceiling over the sanctuary cracks, we were engaged by the owner, the Pennsylvania Historic and Museum Commission, to assess the problem.

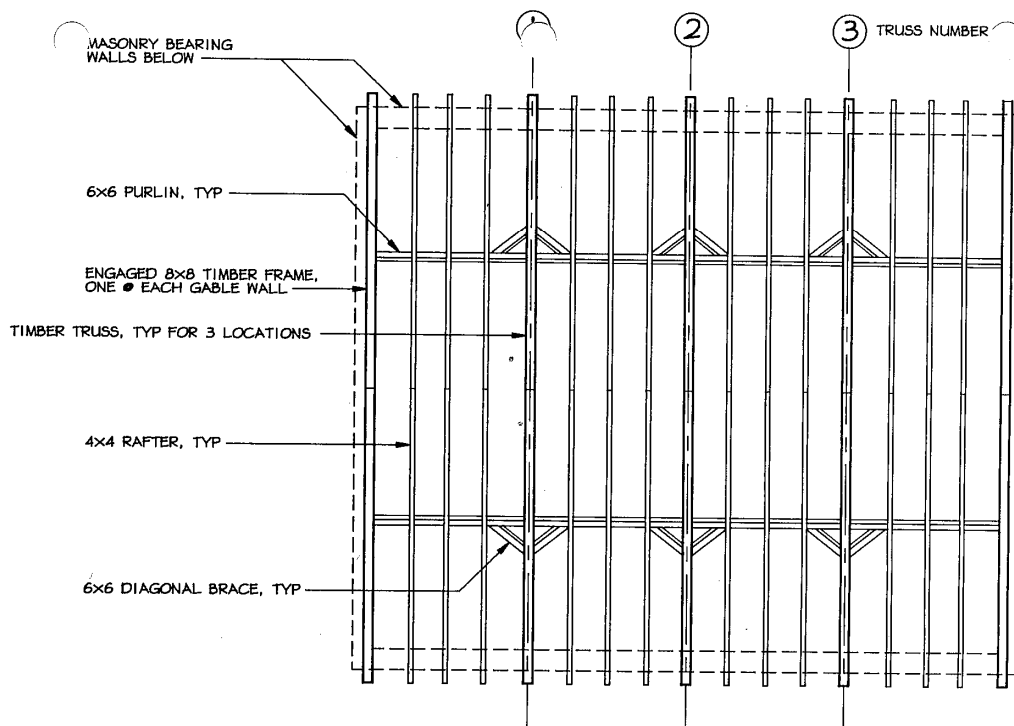
The owner's initial suspicion was that the cracks were associated with vibrations from heavy traffic on the widened highway, but a careful monitoring program proved that theory to be unfounded and we initiated a careful survey of the roof trusses.

The trusses are comprised of two top chords, two diagonal scissor chords, and a horizontal tie (see Drawing A). All of the truss members are nominal 8" x 8" timbers except for the top chords which are 8" wide but vary in depth between 10-1/2" at the truss apex to 14" at the truss bearings. The wood appears to be white oak. The center truss has an additional vertical member at the apex which, judging from the iron spikes used to attach it, is assumed to have been an early modification done after the building was built. All connections to the top chords are mortise and tenon joints. Several of the joints have been reinforced with wrought iron cramps. Where the scissor chords and horizontal tie intersect, lap joints are used.

The top chords of the truss bear on 7" by 8" timber corbel blocks, which in turn bear on a thick continuous 18-1/2" wide wood sill plate set on top of the masonry bearing walls. Each corbel block below the bearing ends of the middle truss has been anchored to the sill plate below it with a vertical 2" diameter wood dowel. This dowel may also extend into the masonry wall to anchor the sill plate to the wall.

Nominal 6" by 6" roof purlins frame into the midpoint of the top chords to support the roof rafters, and 6" by 6" ceiling purlins frame into the scissor chords and horizontal tie at the quarter points along the span of the truss to support the ceiling joists of the interior barrel vault plaster ceiling.

Initial analyses of the trusses used the geometry described above with the bearing ends modeled as fixed in both the vertical and horizontal directions. Under these assumptions, the analyses had all of the truss members were more than amply sized to support the existing, and code-mandated service loads, even when using the lower end of the allowable stress range for white oak.



B	ORTEGA CONSULTING <small>Post Office Box 428 Media, PA 19063 (215) 585-1131</small>	DRAWING TITLE: ROOF FRAMING PLAN	SCALE: 1/8" = 1'-0"
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At variance with the analyses, our survey of the trusses found numerous failures:

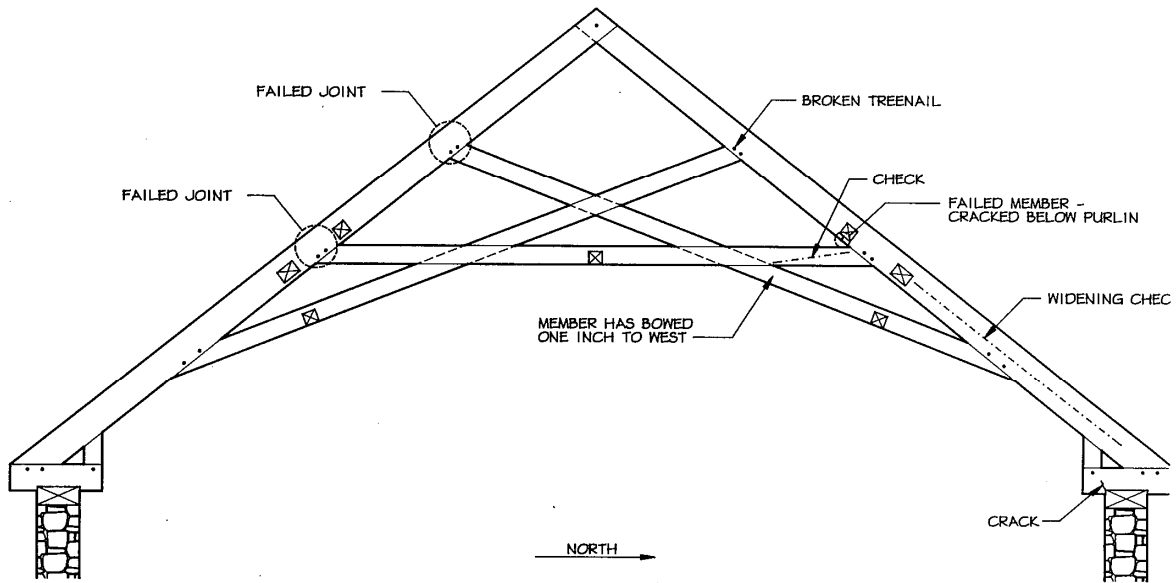
- All the trusses exhibited numerous failed pegged joints. In addition to the failed pegs, a few of the joints where the pegs were intact exhibited failure of the tenons.
- All the lapped joints of the trusses had rotated to some extent producing gaps at the far ends of the laps and localized crushing at the near ends of the joints.
- All of the top chords had failed.
- Several of the joints connecting the horizontal tie to the top chord had failed in tension.

Using string lines we also found that the member of the trusses exhibited significant deflections:

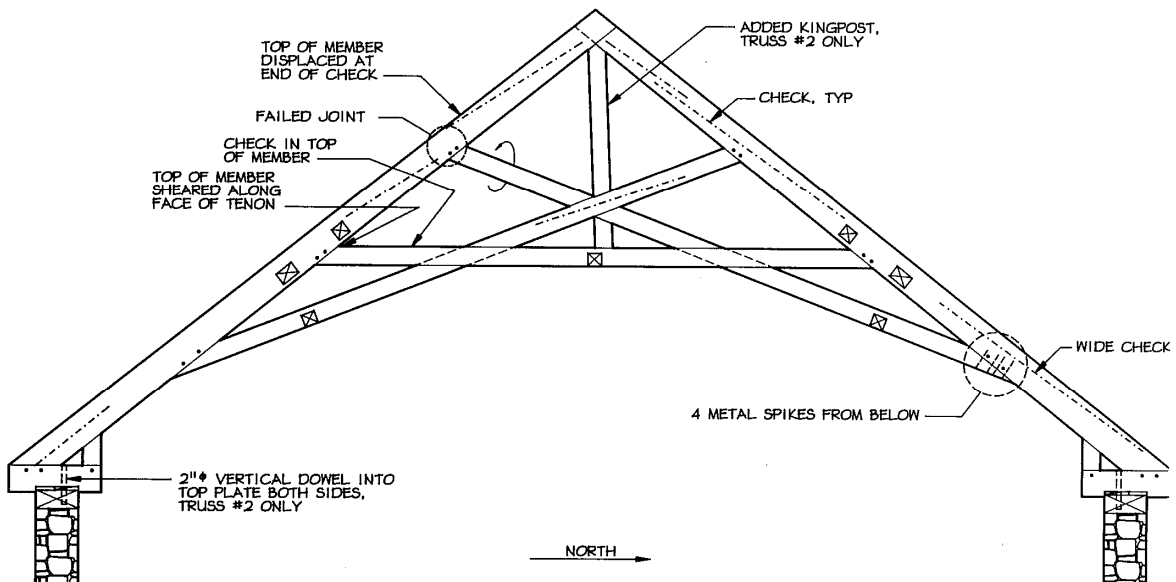
- The top chords had deflected approximately 2 inches.
- The diagonal scissor chord had deflected 2-1/2 to 3 inches.
- The horizontal tie had deflected about 3 inches.

We also closely examined the 20" thick masonry walls supporting the trusses and found that the tops of both walls had bowed outward as much as 6". This observation meant that our initial assumption that the masonry walls provided unyielding support for the trusses was not realistic. Thus, the same truss geometry was re-analyzed with one bearing end modeled as a roller so that spreading of the truss was theoretically permitted. The results of this analysis were consistent with the observed failures in the truss: the diagonal scissor chords go into tension to resist the spreading of the truss, and many of the truss members become overstressed.

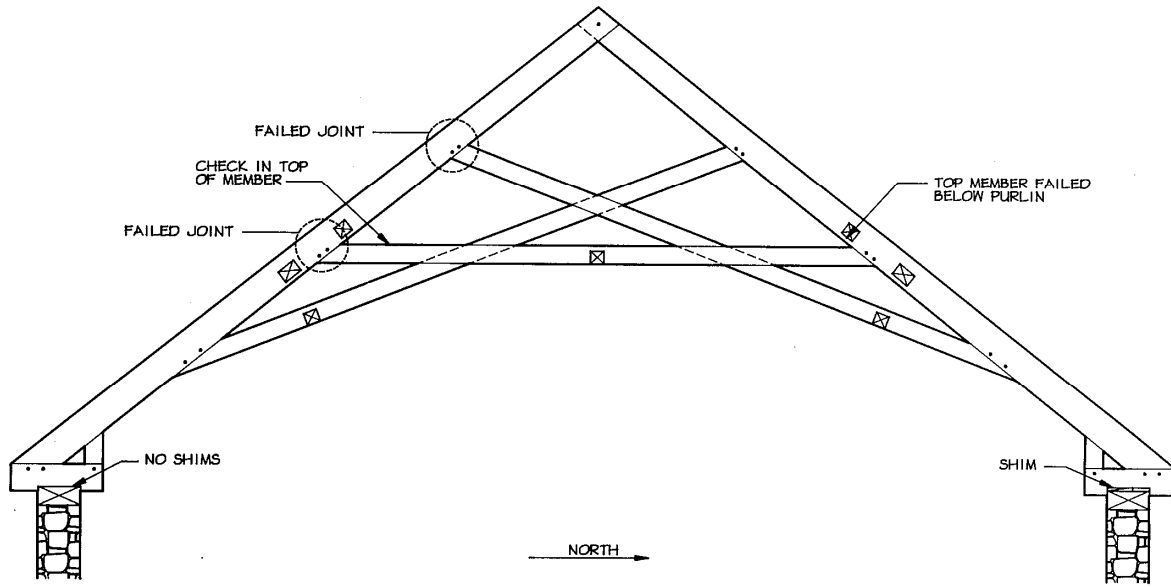
The lesson learned at the Peace Church was that even a relatively small movement in a truss assembly and its support conditions can produce failure inducing stresses in the truss.



E	ORTEGA CONSULTING	DRAWING TITLE: TRUSS #1 EAST ELEVATION	SCALE: 1/4" = 1'-0"
	Port Office Box 428 (215) 585-1131 Media, PA 19063	DATE: 3/12/92 DRAWN BY: RDN APPROVED BY: PROJECT #: 92-03B	NOTE: UNLESS OTHERWISE NOTED, JOINTS NOTED AS "FAILED" ARE DUE TO BROKEN TREENAILS &/OR TENON.

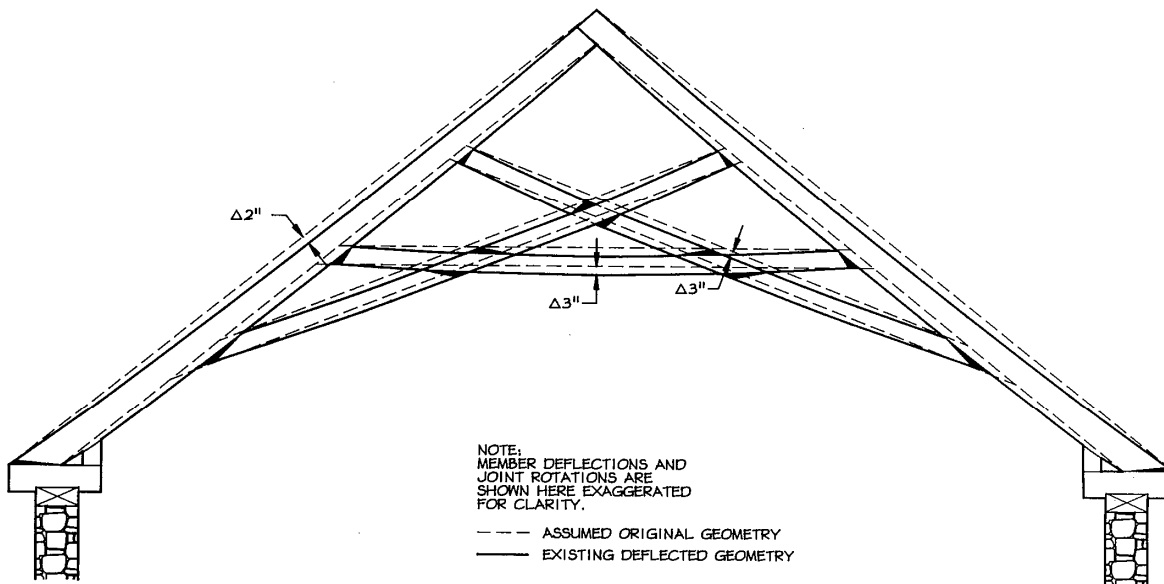


G	ORTEGA CONSULTING	DRAWING TITLE: TRUSS #2 EAST ELEVATION	SCALE: 1/4" = 1'-0"
	Port Office Box 428 (215) 585-1131 Media, PA 19063	DATE: 3/12/92 DRAWN BY: RDN APPROVED BY: PROJECT #: 92-03B	NOTE: UNLESS OTHERWISE NOTED, JOINTS NOTED AS "FAILED" ARE DUE TO BROKEN TREENAILS &/OR TENON.



DRAWING No: I	ORTEGA CONSULTING Post Office Box 428 Media, PA 19063 (215) 565-1131	DRAWING TITLE: TRUSS #3 EAST ELEVATION	SCALE: 1/4" = 1'-0" DATE: 3/12/92 DRAWN BY: RDN APPROVED BY: PROJECT #: 92-03B
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NOTE:
UNLESS OTHERWISE NOTED, JOINTS NOTED AS "FAILED" ARE DUE TO BROKEN TREENAILS &/OR TENON.



DRAWING No: J	ORTEGA CONSULTING Post Office Box 428 Media, PA 19063 (215) 565-1131	DRAWING TITLE: DEFLECTIONS & JOINT ROTATIONS	SCALE: 1/4" = 1'-0" DATE: 2/14/92 DRAWN BY: RDN APPROVED BY: PROJECT #: 92-03B
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The Athenaeum of Philadelphia

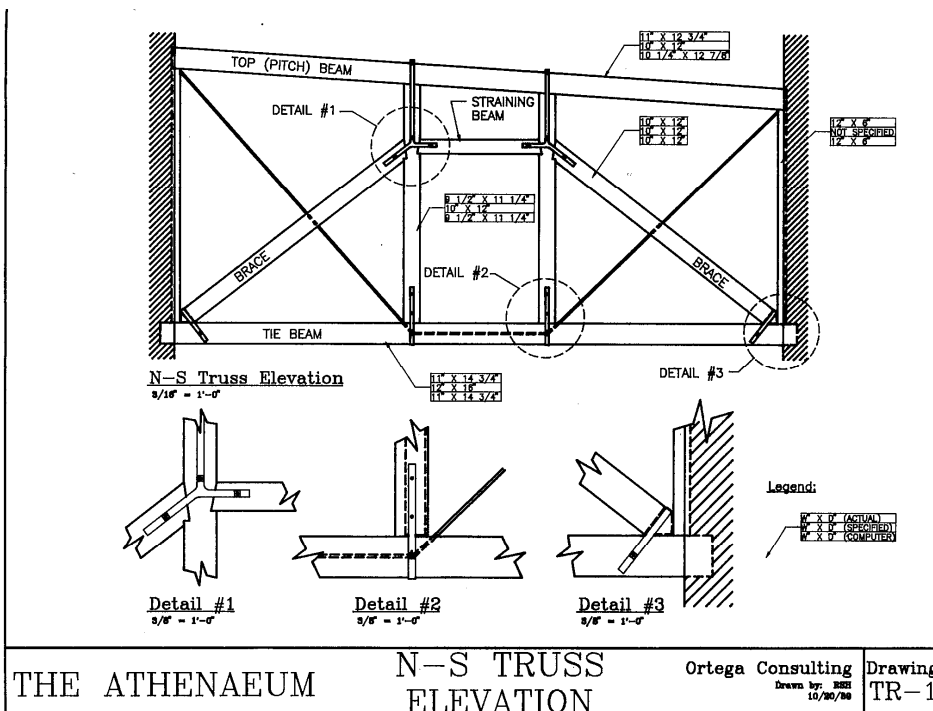
The Athenaeum of Philadelphia, built 1845-1847, was a library and museum, designed by noted Philadelphia Architect John Notman. It had served that purpose for a century and a half when we were engaged to assess the existing condition of the structure, both with respect to the current loads on the building and proposed renovations.

As part of our work we reviewed existing historic documents (which were plentiful, as the Athenaeum's mission is to be a repository of historic documents related to architecture), surveyed existing conditions, and performed engineering analyses to assess the existing load capacities.

Of particular interest was the third floor, which housed much of the institution's library and exhibited noticeable distortion of the floors, causing concern regarding the stability of both the freestanding shelving and the wall racks. Furthermore, the pattern of cracks in the plaster walls and ceilings clearly indicated excessive deflections of the trusses and joists.

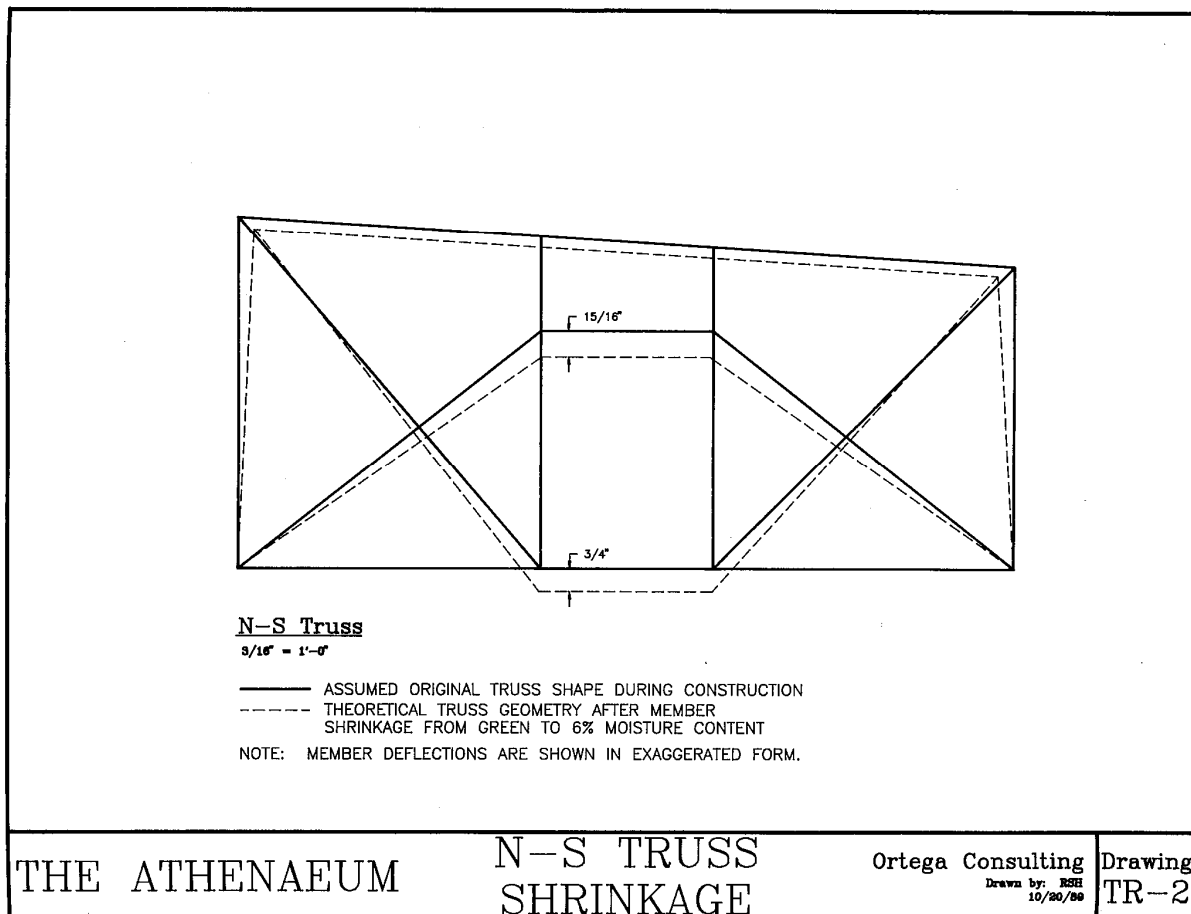
In order to provide the large uninterrupted spaces of the second-floor reading rooms, the third floor and roof framing are carried on large timber trusses. This is described in the original Notman specifications:

"The joists of the third story will be supported by trusses formed as shown in transverse section; tie beams of scantling, 12 by 16 inches; in three pieces of 4 inches thickness each; queen posts of oak, 10 by 12 inches; straining beam, 10 by 10 inches; braces 10 by 12; and top or pitch beams, 10 by 12 inches; the iron stirrups, collars and straps to be of iron, 2-1/2 by 5/8 inches; the suspension iron rod will be of 1-1/4 inch round iron, and will pass from each end of the pitch beam down to the lower end of the queen post, and pass over a 2 inch bolt let through the stirrup straps, thence to the other queen post, in like manner secured, and to the other end of the pitch beam, where at each end it will be secured through a cast iron collar over the end of the beam, and have a nut and screw to tighten it to the required bearing. All the scantling for trusses and girders to be of white pine, except what white oak is specified."

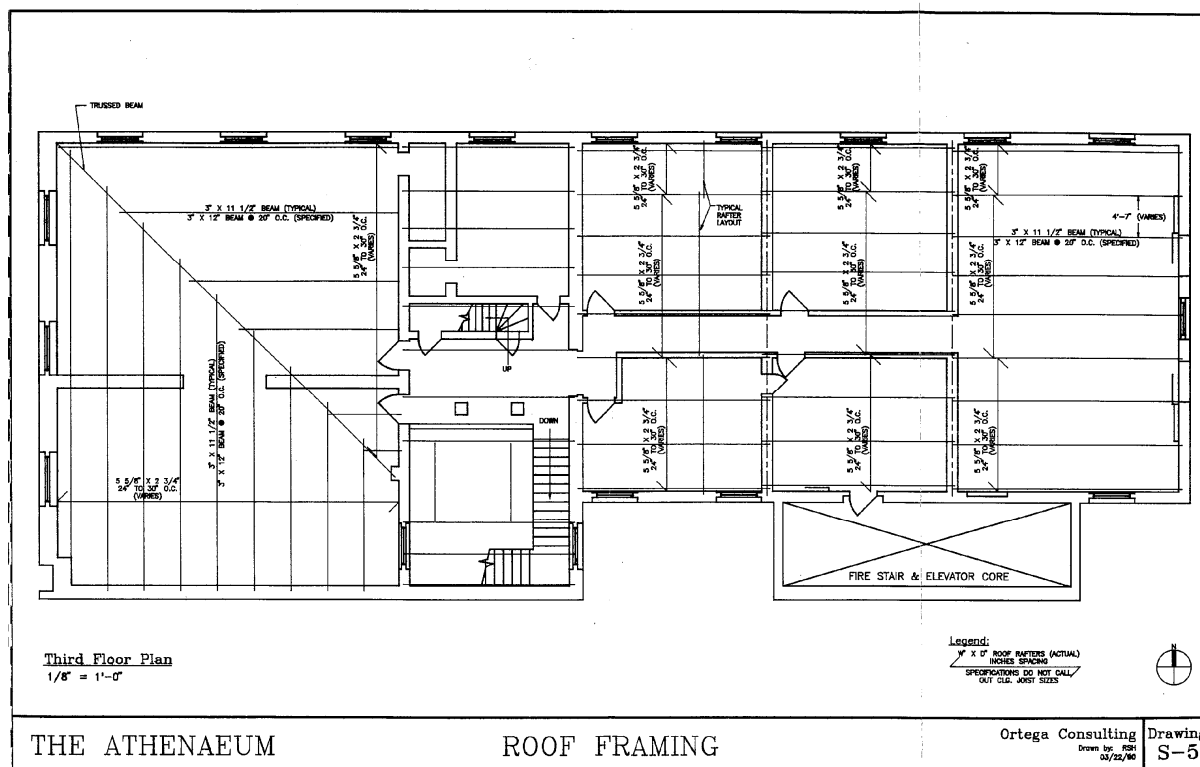


A survey of the framing found it to be generally as specified. Aside from some minor dimensional variations, the queen posts, straining beams, braces, pitch beams, iron straps, and rod conform to the specifications (see Drawing TR-1). The bottom chord, or tie beams, is smaller than specified, using two 5-1/2 by 14-3/4 inch timbers with a gap in between for the suspension rod. Likewise, the bolt below the queen posts, under which the suspension rod passes, appears to be only 7/8 inch in diameter instead of the 2 inches specified. The end posts, which are not mentioned at all in the specifications, are partially concealed in chases in the exterior masonry walls, but appear to be 6 by 12 inch pine timbers. We were not able to confirm the existence of the cast iron collars at the ends of the pitch beams and the rod connections to them as they were inaccessible.

As part of our assessment we surveyed the floor levels throughout the building. For the most part the deflection patterns conformed closely to those anticipated for the deflection of joists. And except for the third floor, the magnitudes of the measured deflections were neither excessive nor substantially different from the calculated deflections. The glaring exception, however, was the deflection of the third-floor trusses. While deflection calculations of spanning wood members are notoriously approximate, the measured deflection of the trusses, at slightly more than one inch was well in excess of the calculated theoretical deflection of no more than 0.28 inches.



This sent us back to re-examine the trusses more carefully for damage which could have caused the excessive deflections. The joints appeared tight and there were no apparent failures of the connections. We had limited access to the heel joints, but saw no evidence of movement or shear block failure. The rods seemed reasonably tight. We found checking in most of the members, but no signs of failure. There seemed to be no mechanism to account for the excessive deflections.



While mapping the checking of the timbers we realized that they must have gone through a considerable change in moisture content since the time of original construction. Considering the sizes of the timbers, and assuming that it was unlikely that the timber was aged for several years before use, we concluded that they were probably green when the trusses were built – i.e. the wood had a moisture content approaching 30%. From readings taken during our survey we knew that the moisture content was about 6%.

With that much drying, we decided to explore the effects of drying-induced shrinkage on the trusses, especially whether it could account for the extra deflections. Using the lumber species cited in the specifications and shrinkage coefficients listed in the *Wood Handbook*, published by the Forest Products Laboratory, we calculated longitudinal shrinkage for all the members based on a 24% decrease in moisture content. We ignored the transverse shrinkage as being inconsequential for the truss geometry, even though we recognized it as the principal source of the checking.

When we graphically reconstructed the truss using just the timber centerlines and with the shortened member lengths, we found that the iron rod, which would not shrink, would become slack unless the truss deflected between $\frac{3}{4}$ and 1 inch to re-engage it. (see drawing TR-2).

Interestingly, our analyses of the trusses had indicated that removal of the iron rods had little effect on the load capacity of the trusses. The rods in combination with the queenpost braces were redundant elements and render the trusses statically indeterminate, which makes it difficult to determine how much of the loads are apportioned to the braces in compression or the rods in tension. Our initial inclination was to conclude that the rods were not required and not loaded, yet our inspection indicated that the rods were, indeed, in tension. The graphic analysis of the affects of shrinkage suggested both the source of the excess deflection and the true function of the rods.

When the trusses were built it is apparent that great care and attention was given to fitting the members into tight joints and reinforcing the joints with iron straps. In

addition, the specifications call for the rod to “have a nut and screw to tighten it to the required bearing.” At completion the trusses were probably tight and straight.

As time passed the wood began to dry and shrink. The rate of shrinkage is unknown, but it probably would have occurred over a number of years. During the initial shrinkage, as the truss geometry changed the tightened rods were probably relieved of load as the queenpost braces did the work, but eventually the continued shrinkage-induced deflection, plus some load-induced deflection, and probably some creep, or time-dependent deformation, caused the truss to deflect enough to re-engage the rod and transfer load to it.

Clearly Notman understood the characteristics of timber structures, particularly the affects of shrinkage on geometry. His use of the term “suspension rod”, which we initially thought curious, suggests its function as a deflection limiter. The lesson learned here was that not all changes in truss geometry confirm structural inadequacy.

The Feast Hall at Old Economy Village

Built in 1826, the Feast Hall is one building in a complex of structures built by the Harmonists to serve the Utopian community's needs: it is believed to be the first building of its type to be used as a museum, offices, community meeting hall and dining facility. The Harmonists died out in the early 20th-century, and their property, including the Feast Hall, was acquired by the Commonwealth of Pennsylvania in the 1930's. It is currently operated by the Pennsylvania Historical and Museum Commission.

As part of the scope of work for the restoration of the Feast Hall in the 1990's, we were engaged to determine the feasibility of removing any, or all, of the nine tie-rods spanning across the ceiling of the Hall. Seven of the tie-rods were located at the spring-point of the elliptical vaulted ceiling and two were located a few feet below.

In plan the building is approximately 54' wide by 119' long and about 50' high. It is a two-story wood frame structure with brick masonry exterior walls and a gambrel, so-called German mansard, roof. The first floor has a central corridor running the length of the building with flanking rooms on both sides. The second floor is devoted almost entirely to the Hall, which is a remarkable, vaulted space, free of interior columns. At its highest point the elliptical-shaped, tongue-and-groove board ceiling is 23' above the floor.

The exterior brick masonry walls were in generally good condition with no visible cracks, the north and south walls (the walls upon which the roof trusses bear) were visibly out of plumb. At the cornice, about 25' above grade, the walls leaned outward 8", but 6" of that occurred in the upper half of the wall, between the cornice and the second floor.

Between the out-of-plumb bearing walls and the plethora of tie-rods, there was every reason to believe that there was a significant defect, or failure, in the roof structure. Thus, we decided that we needed to do a detailed inspection and assessment of the very complex roof framing system used in the Feast Hall.

The roof structure above the Hall consists of seven timber “trusses” spanning the width of the building spaced at 12' on center bearing on timber columns partially engaged in the masonry walls and morticed into the timber floor beams of the Hall. Although they will be referred to as trusses in this paper, they are really a form of braced frame which forms the lower part of the gambrel roof and supports the framing of the upper part of the gambrel. Each consists of bearing members at each end framed at a 60° slope, an upper and a lower horizontal tie across the top, and two knee braces between the lower tie and the bearing member, framed at about a 30° slope. The lower horizontal tie frames into the side of the bearing members, the upper horizontal tie passes over and projects beyond the tops of the bearing members. The bearing members, the lower ties, and the knee braces are all joined with pegged mortise-and-tenon connections. Spanning between the trusses at the elevation of the horizontal ties are three purlins – one at each end at the tops of the bearing members, and one at midspan. The purlin at each end is custom shaped and mortised into the top of the

bearing member; it also projects above the top of the bearing member to fit into a notch on the bottom of the upper tie. The purlin at midspan is positioned between the two horizontal ties; the bottom of the purlin is notched to fit over the bottom tie and secure it in place, while the bottom of the upper tie is notched to secure the purlin in place. Flanking the midspan purlin are two vertical bolts through both ties.

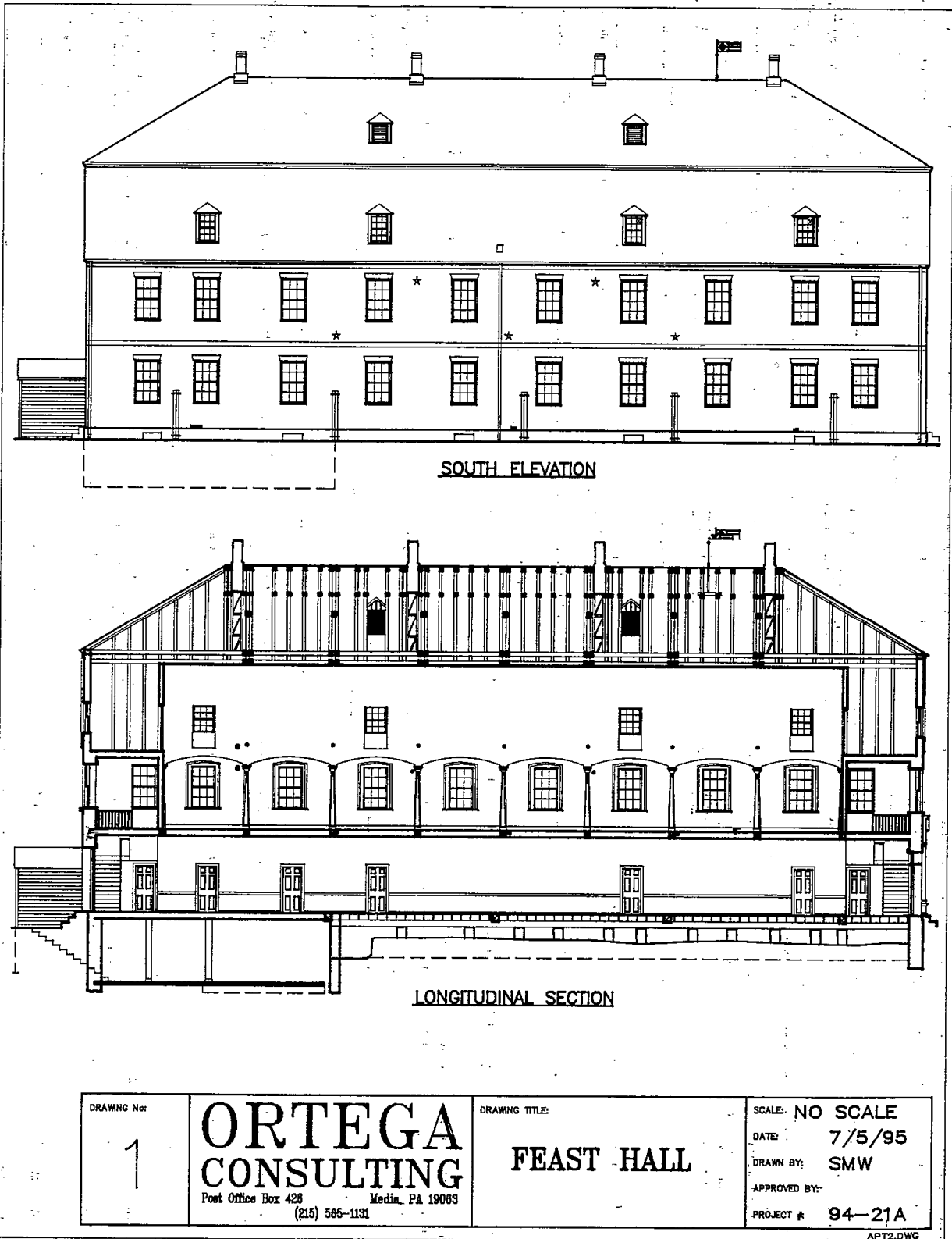
Parallel to the knee braces but offset below them are the top chords, or diagonal members, of the king rod trusses. Each chord is framed into the bearing members of the braced frames above where they bear; it then passes between the four ties of the flanking braced frames and frames to the opposing top chord at the apex of the king-rod assembly, but well below the top of the roof. All the connections associated with the king-rod members are bolted assemblies. Where each king-rod chord passes between the horizontal ties the chord and all four ties are rabbeted and lapped and then through bolted at the center of each lapped connection. At the apex of the king-rod assembly a horizontal bolt secures the two chords together and a vertical rod between the chords is the king-rod which goes through and is secured to the midspan purlin with a nut and washer.

The upper framing of the roof consists of simple gable framing with collar ties. To support the upper framing and transfer loads to the trusses, each rafter is framed onto an outlooker which bears on the trapped purlin described above and framed into a sweep purlin that frames between the upper horizontal ties of the trusses. Each outlooker is mortised through the sweep purlin and secured with a treenail. The sweep purlin is so called because at one end it is framed into the upper horizontal tie with a conventional mortise-and-tenon connection, but at the other end the purlin fits tight into the end of a long curved rabbet in the side of the upper tie. This "sweep" was necessary because the purlins were installed after the trusses were erected and had to be swung into place.

Finally, four more purlins frame below the trusses to support the ceiling joists. The two exterior purlins frame into the sides of the king-rod chords while the two interior purlins are hung from the lower horizontal ties with bolts.

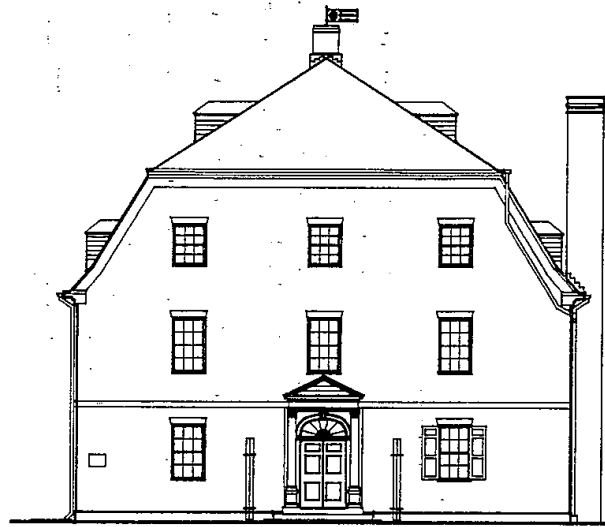
In general we found that the trusses appeared to be in sound condition with no evidence of member or joint failures. Nevertheless, we found certain conditions worth noting.

- The king-rod top chords were generally checked their entire length; for the most part the checking was more pronounced near the joints, such as at the apex of the 30° chords and at the intersection with the upper and lower horizontal ties.
- The joints between the king-rod chord and the horizontal ties have rotated. Where the rabbeted members intersect there are gaps between the chord and upper tie at the exterior shoulder and between the chord and the lower tie at the interior shoulder. At the opposite shoulder of each joint the intersecting members are tight against each other and in most cases exhibit localized crushing of the wood.
- Similar rotation was noted at the joint between the king-rod chord and the bearing member of the frame. In this joint the chord is shouldered to fit between the two frame members. At the upper face of the connection there are gaps between the members whereas the joint was tight at the bottom edge; but not generally to tight as to cause crushing of the fibers.
- At the connection between the knee brace and the bearing member there is a gap at the bottom of the mortised joint. At the opposite end of the brace, where it is morticed into the lower horizontal tie there is a similar gap. In both cases the opposing edge of the joint was tight. This situation also indicates rotation of the joints.
- There is a gap between the faces of the lower tie and the bearing member at the upper face of the joint where the former is morticed into the latter; the lower edge was tight, though not crushing.
- The maximum deflection of the upper tie, as measured at the mid-point of the top face relative to a string line between the two ends of the tie, was 2-5/8".

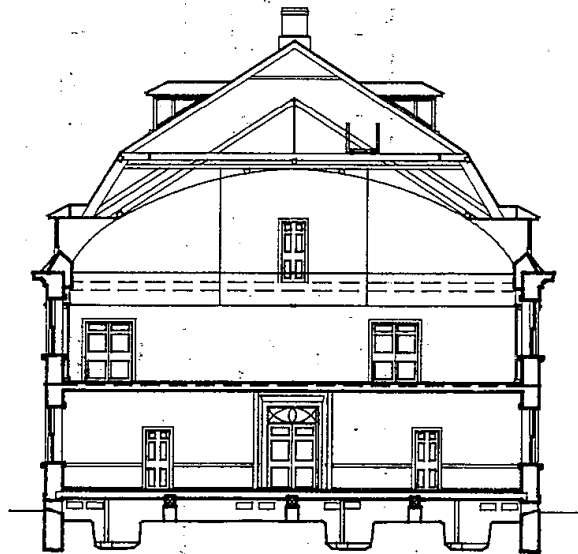


Inspection and sounding of the ties-rods yielded the surprising finding that they were slack – i.e. were not loaded.

These observations were at variance with the results of our analyses and modeling of the trusses, which, among other findings, could not replicate the 6" displacement of the truss bearings (the calculated amount was 1-1/2"), and resulted in a 12 Kip tensile load on the tie-rod.



EAST ELEVATION



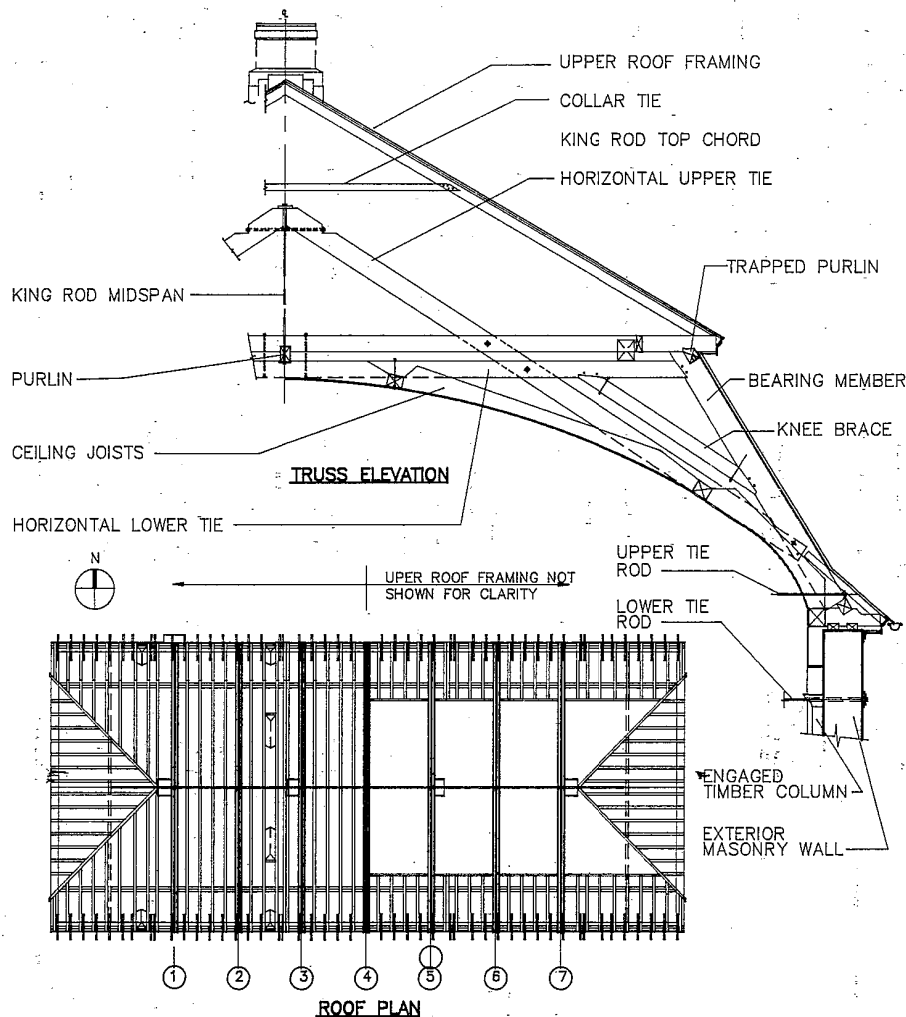
TRANSVERSE SECTION

DRAWING No: 2	ORTEGA CONSULTING Post Office Box 428 Media, PA 19063 (215) 586-1131	DRAWING TITLE: FEAST HALL	SCALE: NO SCALE DATE: 7/5/95 DRAWN BY: SMW APPROVED BY: PROJECT #: 94-21A
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In light of these discrepancies we returned to re-examine the building and research the history of the tie-rods. This revisit and new research yielded interesting information and observations.

- Documentary research determined that the seven upper tie-rods were installed Victorianization of the building in the late 1880's, the lower tie-rods around the turn of the century – long after the builders of the Feast Hall died.
- Strain gauge monitoring of the tie-rods confirmed that they were loaded only by their own dead weight.
- Re-inspection of the brick masonry walls revealed a slight humoring of the brick coursing in the upper portion of the walls -- i.e. each successive course projected, or was slightly corbelled, beyond the one below.
- At the top of the walls there was no evidence of relative movement between the walls and the engaged columns supporting the trusses.
- There was no evidence of movement of the wood plate at the top of the walls that receive the rafters relative to the brick masonry.
- At least half the scribe marks on the braced frames are on the concealed faces of the pairs of frames – i.e on the surface facing the other braced frame in pair rather than on the face toward the next truss.
- All the treenails of the mortise-and-tenon joints were driven from the concealed face.



DRAWING No. 3	ORTEGA CONSULTING Post Office Box 428 Media, PA 19063 (215) 585-1131	DRAWING TITLE: FEAST HALL	SCALE: NO SCALE DATE: 7/5/95 DRAWN BY: SMW APPROVED BY: PROJECT #: 94-21A
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These new observations led us to reach some startling conclusions regarding the sequence of the timber framing and the source of the truss and wall distortions.

- The building is not a two-story masonry building surmounted by a timber frame gambrel roof; it is essentially a timber frame building (like a barn) built atop a one-story masonry base, then clad in brick masonry.
- Each braced frame part of the trusses was custom fabricated and fitted, the pieces disassembled, then reassembled and erected into place. The chords of the King-rod portion of the trusses would have been placed between the frames when they were raised into place.
- The roof trusses were erected in sequence from each end toward the middle.
- The sweep purlins were the last primary members installed and locked the upper horizontal ties in place.
- None of the upper horizontal ties, which lock the trapped purlins in place, were not installed until all seven trusses were in place, which means that the integration and full connection of the king-rod trusses and braced frames could not be effected any sooner than after partial erection of all the trusses.
- The upper horizontal ties were slid in place laterally, using the purlins as rails. Thus, the shouldered connection between the upper ties and the king-rod chord could not be tightly fitted; there had to be allowances for construction irregularities. The absence of framing scribe marks on these members indicate that those joints were not pre-assembled, as is customary in harmonist frame construction, and custom fitted in place.
- Finally, the mix of mortise-and-tenon and bolted connections further suggests the sequence of construction. The former were prefitted and erected as described above. The latter were drilled after the truss pieces were assembled, and those final connections were used to tighten and finalize the truss geometries.

The design of the trusses at the Feast Hall cleverly provides a long-span structural form that conforms to the gambrel roof shape and allows for a vaulted ceiling over the Hall. After carefully examining and analyzing the trusses, there was ample evidence of high stress in several members, but no evidence of failure – past, present, or imminent.

The design dictated the construction sequence and details, which, in turn, imposed limitations on the joinery methods, provides an explanation for the extra two to four inches of lateral displacement at the truss bearings as well as the basis for the argument that the upper wall of the Feast Hall were never plumb, indeed, may only be cladding and not structurally relevant. The strength and stability of these trusses is provided by the moment resistance of the king-rod chord and horizontal tie assemblies, especially their lapped joints. But, as noted above, the construction sequence meant that it was virtually impossible to truss with those joints tight. Thus, a certain amount of rotation of the joint, with concomitant lateral displacement of the end bearings must occur before the strength of the joint could be mobilized and the entire truss assume a stable attitude.

The lesson learned here was that context and history are essential to distinguish unintended changes in truss geometry from functional distortions.

Closing

We are generally conditioned to assume that floor planes were built flat, walls built plumb, and corners true; so when we find distortions in the building geometry, we naturally assume that such changes are due to something broken, or that there is some form of structural failure. Certainly, as engineers and architects assessing existing buildings we cannot ignore any such physical evidence, but as we have attempted to show with the case histories presented above, timber trusses can exhibit significant changes in geometry without necessarily be in failure.

Wood is perhaps unique among the construction materials we use. Not only does it exhibit different mechanical properties depending on the direction of stress, its

mechanical properties and dimensions change over time and with changes in environmental conditions.

These characteristics have been known to builders for centuries and those with greater skills have been adept at contriving ingenious ways to both exploit wood's strengths and mitigate its vexing drawbacks. Yet too often today's design professionals ignorant of archaic timber framing techniques install extensive steel "enhancements" in historic timber frame buildings.

If we are to fairly assess the structural competence of historic timber structures and not misconstrue the "bad behavior" of the wood as indicative of failure we must not only be more imaginative in our analyses, but also delve into the design world of the original builder. If we can better understand how he regarded the materials he had at his disposal, his fabrication methods, and construction sequencing, perhaps we will begin to give archaic timber structures more credit for performing as expected and be less inclined to consider them failures.