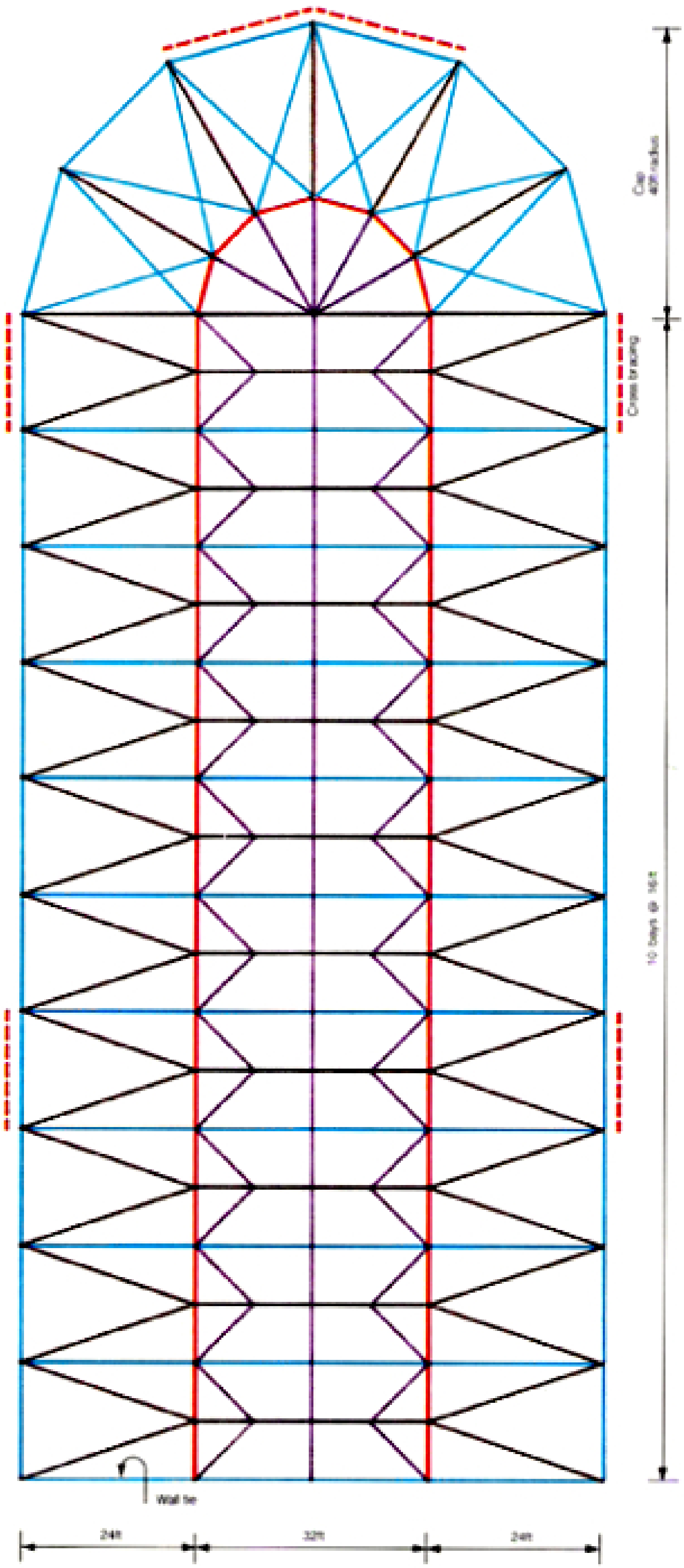



1. Plan of roof structure.

# Weatherstone Riding Ring

**Architect:**
**Cooper Robertson & Partners**

Guy Nordenson
Liam O'Hanlon

## Introduction

Weatherstone Riding Ring is an indoor facility at a private client's country estate in Northern Connecticut. The building is 80ft × 200ft[1] in overall area with the roof rising from 16ft at the eaves to 40ft at the ridge. The ring itself forms an 80ft × 160ft rectangle plus a 40ft radius semi-circle at one end.

Adjacent are stables for 12 horses and living facilities for the manager and groom.

The architect, Jaquelin Robertson of Cooper Robertson & Partners, sought a structure for the roof that was of a uniform density, rather than consisting of parallel trusses, arches, or portals. His preferred material was wood. In addition it had to accommodate a 4ft high clerestory all around, thus creating a 32ft wide gable about the ridge separated by the clerestory from a surrounding 24ft sloping apron.

The design of the structure was undertaken by the New York office of Ove Arup & Partners; the outcome of our deliberations with the architect consisted of a series of wood and steel trusses spanning the rectangular portion of the plan and a pair of half cones (apron and gable) at the end 'cap' (Fig. 1), organized on an 8ft module in all directions. Columns are double this height and the same distance, 16ft, apart, with the truss rising a further 16ft over the 24ft apron and 4ft to the ridge from the top of the clerestory (Figs. 2 & 3).

## Trusses

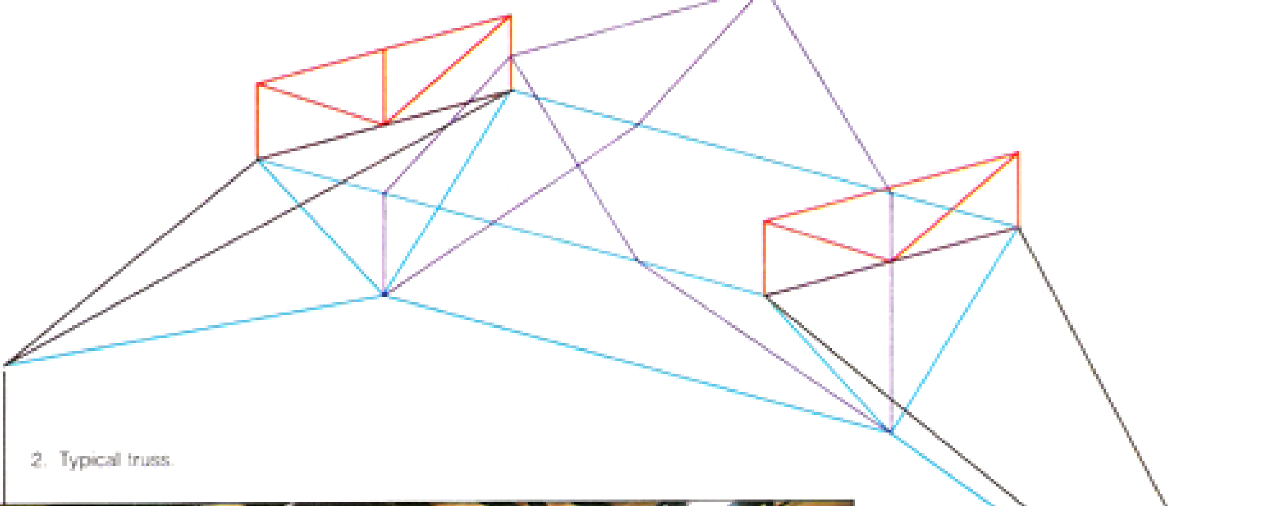
The trusses are of a single two-directional geometry that has been, in effect, unfolded to obtain a partly three-dimensional system. The main structural action is nevertheless primarily in one direction, across the span. As shown in Fig. 2, the compression elements, of generally paired, glue-laminated timber, rise from the columns at an angle to meet the horizontal cross-members midway between the columns, directly beneath the clerestory. Pairs of tension rods extend directly between the columns. They are pulled up by single rods that come down 8ft from the compression member joint. These form a pattern of Vs along the length of the ring, under the clerestory. The ridge is then supported by saw-horse/A-frames over the horizontal cross-members that split at these members to rest on the tension rod joints. This last joint was nicknamed the 'stirrup' (Figs. 4 & 5). Finally, a series of light wires was extended from the joint of the A-frame and cross-member to hold up the double tension rods at mid-span.

The connections are steel plates worked into the rather complex geometry by being hidden between the twin glulam members, their presence, extent and function revealed only by the bolt pattern (Fig. 6). Six joints are included in each typical truss, of which there are 10 in all.

Within the clerestory a truss of glue laminated timber and 3.5in diameter steel tubes serves as 'bridging' if the live loading were unbalanced, or if one of the trusses suffered damage (Fig. 7).

(1) As this project was designed in US units, these have been retained throughout.

Colour key for all diagrams
- Glulam
- Wire, steel rod or pipe members
- Clerestory or stiffening truss
- Cross bracing (rod)
- Ridge framing and diagonals (glulam)

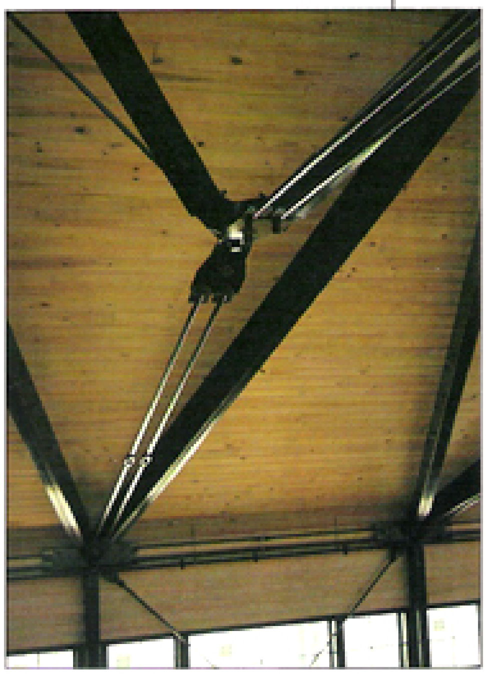


2. Typical truss.

3△

4▽

5▷



3. Interior view looking toward front entry.

4. Tension chord and diagonal 'stirrup' connection.

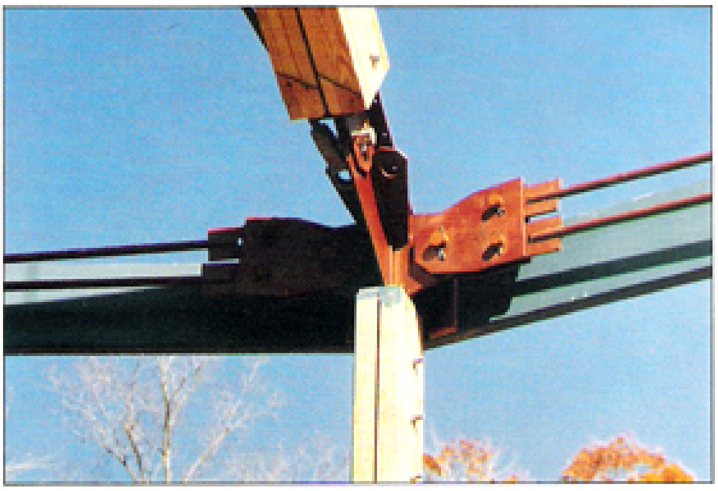
5. Close-up of 'stirrup' connection.

6. Typical connection at top of columns.

7. Exterior view during construction with main and clerestory trusses in place.

7▽

6▽



### The cap

To maintain the structure's uniformity, it was important that the scale and density of that part of the roof around the cap be constant. To achieve this the cap was treated as a framed half-dome relying on a tension ring of double tie-rods along the eave, and a compression ring consisting of the clerestory truss and an 8ft deep, lightly braced, glulam stiffening truss (Fig. 8). The latter was required to allow for unbalanced live loading that would cause bending in the compression ring. The top portion of the cap, above the clerestory, is a simple arrangement of ribs radiating from the end of the ridge towards the top of the clerestory (Fig. 9).

The effect of this approach was to allow the use of ribs similar in size to the compression members of the main trusses, and eliminate the need for a large truss across the end of the cap. The results were large tension and compression loads delivered to the eave and clerestory. On the eaves these are resisted by the tie rod bracing between the columns. The compression load to the clerestory truss also finds its way to the rod bracing via the connection to the compression glulam members beneath the apron. The tendency of the two apron portions to splay apart (in plan) as a result of these force couples is resisted by a tie between the columns embedded in the wall at the end opposite the cap.

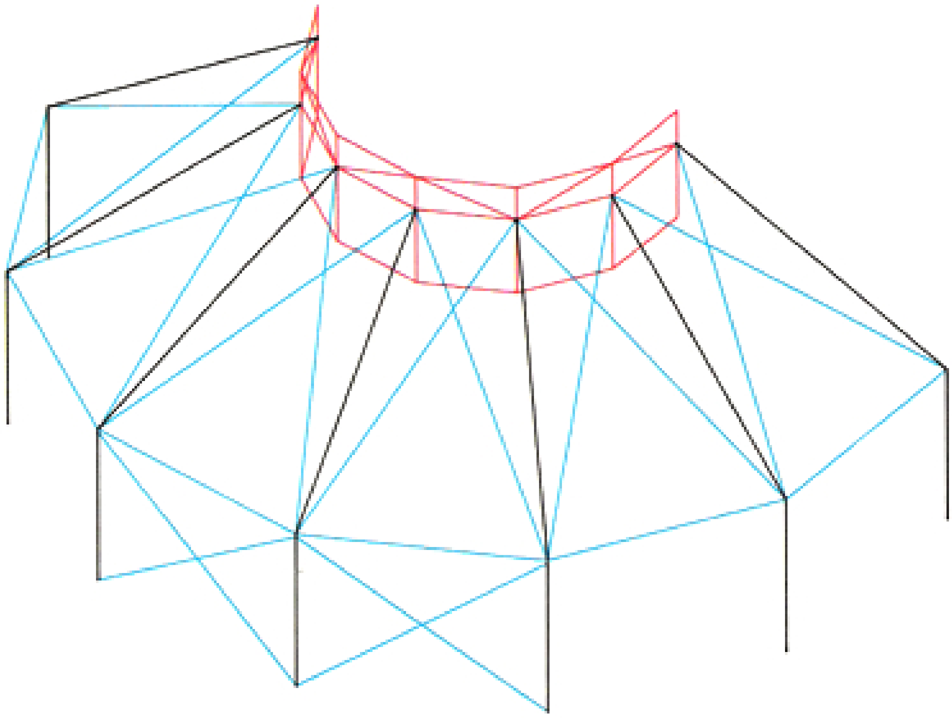
### Stability

Careful evaluation of all extreme load conditions, including 100 year wind and severe unbalanced snow load cases, revealed that the tensions in the rods would not be overcome. Overall stability is provided by six bays of rod cross-bracing, two on each side and two at the cap (Fig. 1). The timber decking overlaid with ⅝in plywood and the compression members (forming a truss in plan) together act as a diaphragm to distribute the lateral loads to the wind bracing. The clerestory truss ties the gable roof to the apron diaphragm.

Under the effect of unbalanced snow loads or cross winds the main trusses will deflect, causing bending in the horizontal cross compression member (since the truss is not fully triangulated the light wires will relax). To resist this bending, a horizontal 'flitch' plate is inserted as an extension of the A frame/cross member joint.

### Decking

The roof surface consists of tongue and groove laminate decking 3½in thick, overlaid with plywood, exposed on the underside. The spans are up to 16ft on the apron. The decking served a useful role, acting as a deep 'horizontal' beam along the aprons. This contributed to the resistance of the structure to vertical loads by distributing these to the end wall and cap bracing, and thereby serving as a kind of abutment for the trusses to arch.



8. Computer plot of the cap, and below 9. Interior view of end cap.

### Conclusion

All the members and connections were fabricated off-site to the required dimensions. Tolerances were quite small. The structure was erected off a full scaffolding and was completed in 10 weeks. All the bolt holes were pre-drilled.

A sequenced pretensioning schedule was prepared in conjunction with the glulam fabricator and the contractor to obtain the final geometry after the shoring was removed. The geometry was achieved to within ¼in tolerance all round.

10. Weatherstone Riding Ring, stables and living facilities.

**Credits**

*Client:*
Private client

*Architect:*
Cooper Robertson & Partners

*Structural engineer:*
Ove Arup & Partners

*Services consultant:*
John L. Altieri Consulting Engineers, Norwalk, Connecticut

*Main contractor:*
Herbert Construction, New York

*Glulam fabricator:*
Unadilla Laminated Products, Unadilla, New York

*Photos:*
Courtesy the architects