

Structural Design of the Arch and Roof of Wembley Stadium

Kourosh Kayvani, PhD, FIEAust, CPEng
Aurecon, Sydney, Australia.
KayvaniK@ap.aurecongroup.com.

ROOF DESCRIPTION

The aim of the new Wembley Stadium was to design and build a state-of-the-art national stadium, unlike any other in the world. The new stadium, with its elegant exposed steel structure arch, is an international icon as was the old stadium with its twin towers which was built in 1923.

The design brief required the roof not to cover the playing field which lead to one unique aspect of the roof in that it partially retracts over the seats to allow the daylight to reach all points of the pitch and thus a shadow-free playing field.

The retractable roof is formed by seven separate independently driven roof panels totalling 15,000 sqm that move in a parallel motion to the south as they "open" and stack on the top of one another when in a fully "open" position.

With the retracting roof panels all moving to the south, the roof design exploits the opportunity to have a tall, structurally efficient structure on the north side to support the north and south roofs. The solution was to have an elegant and structurally efficient arch which spans the entire width of the stadium's seating bowl (Figure 1).

The large central roof panel cantilevers over the south fixed roof. The three panels at the eastern and western ends of the stadium include two lower panels which slide under a large upper panel so that they do not protrude outside the perimeter of the stadium bowl.

As the moving roof retracts, first the east and west panels and finally the central panel, they revert to their parked position on top of the south roof. The panels are framed with steel truss and plate girder supported on bogies. Driving the panels along the fixed roof is done using a "rack-and-pinion" system, with bogies travelling on rails supported along the runway trusses spanning north-south.

The general arrangement of the structural elements of the fixed roof is presented in Figure 2.

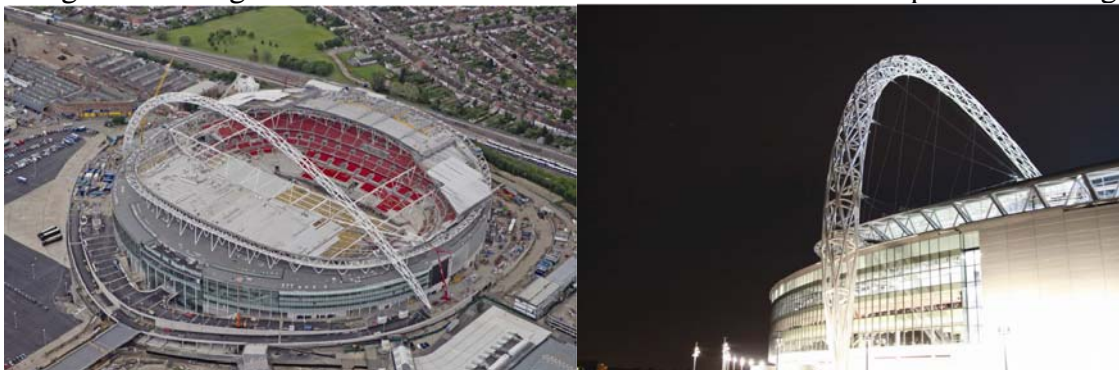


FIGURE 1: ARIAL VIEW DURING CONSTRUCTION AND ARCH DETAILS

The roof cladding is primarily metallic with a continuous "standing seam" configuration. This metal cladding is supported by flexurally continuous rectangular hollow section purlins, which span typically 15.5m in the east-west direction between parallel north-south running roof rafters and runway trusses. An area of translucent cladding up to 25m wide is included at the southern edge of the north roof to achieve a level architectural symmetry between the north and south roof in the roof open position.

The roof rafters and runway trusses share a common structural form, as underslung beam systems. The top chord is a continuous fabricated beam and the bottom chord ranges from a single rod to multiple spiral strand cables. The V-shaped "web" elements allow the underslung system to function as a semi-Vierendeel truss (Figure 3).

The design of these semi-Vierendeel trusses is governed by the lateral buckling of the top chord and the overall torsional buckling of the truss. The flexural continuity of the top chord and the "push-pull" action in the two "legs" of the vee provide the Vierendeel mode of resistance to non-uniform loads. The top chord of the runway trusses also acts as the runway beam for the moving panel bogie systems.

The roof rafters and runway trusses are supported around the perimeter of the roof by a Prismatic Perimeter Truss (PPT) that is, itself, propped from the main stadium bowl. The PPT provides the structural transition between the parallel (north-south) grid of the roof rafters/trusses and the radial grid of the raking beams of the seating bowl. The PPT also acts as the lateral stability system in the form of a "ring truss" resisting forces created within the plane of the roof. The lateral loads are transferred from the PPT into the reinforced concrete cores of the stadium building and hence to their piled foundations.

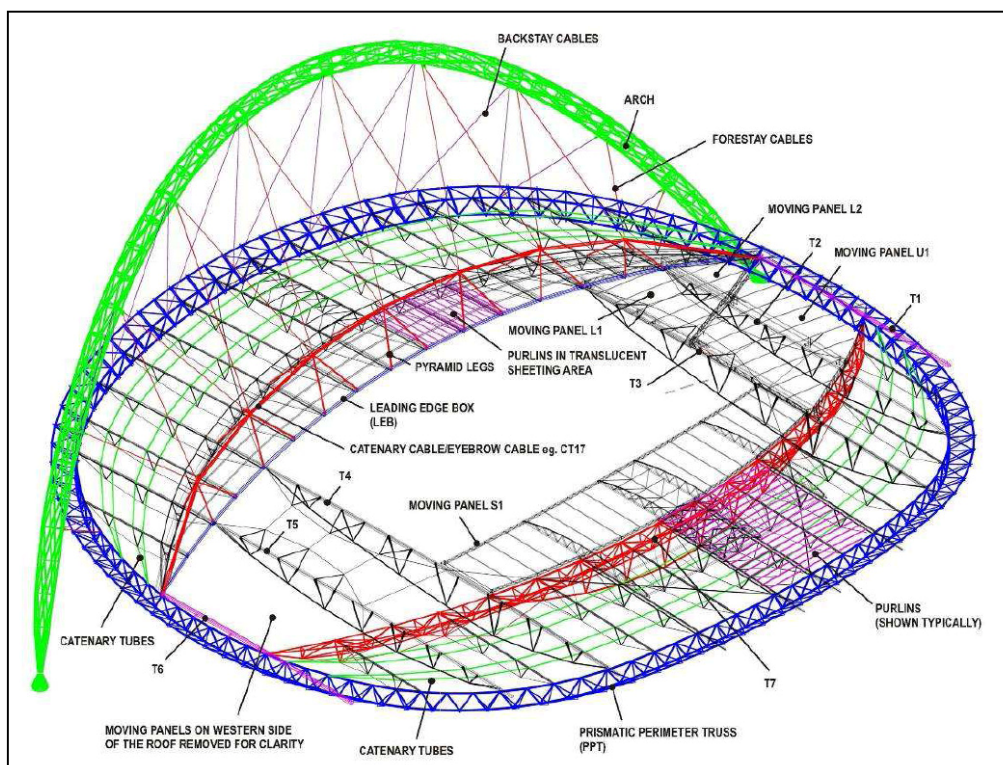


FIGURE 2: STRUCTURAL ELEMENTS OF THE FIXED ROOF (Arch is on the Northern Side)

The north end of the southern roof rafters and runway trusses is supported by an east-west running bowstring truss. This five span, statically indeterminate truss is curved in plan and elevation. The internal supports are provided by four primary north-south trusses and the external supports by the PPT at the east and west ends. The central span of this cable truss is 140m long and 15m deep and is detailed to accommodate the services gantry. To prevent the truss from rolling over in torsion due to its overall plan curvature, the bottom chord of the truss is tied back to the PPT by restraint rods.

The four primary north-south trusses span up to 180m between the PPT on the south and the leading edge of the north roof, where they are supported by pyramid struts attached to the arch via primary forestay cables. Apart from supporting the east-west bowstring truss, the top chord of these four primary trusses act as runway beams for the bogie system of the retractable panels.

These trusses are restrained laterally by a slender, tensioned cable net system. The elements of the trusses and cable net are deliberately slender to minimise shadows.

The south end of the northern roof rafters is picked up by pyramid struts attached to the arch via forestay cables. Therefore, the arch ultimately carries nearly 70% of the roof. Backstay cables connected to the arch are tied back to the PPT and are post-tensioned to provide lateral stability to the arch under all loading conditions.

Wind uplift restraint of the rafters is provided by a series of tubular elements spanning the length of the north and south roofs. These tubular elements act as a catenary during wind uplift conditions. In combination with the purlins, these tubular elements also provide lateral-torsional restraint to the rafter top chords under downward loading conditions.



FIGURE 3: SEMI-VIERENDEEL RUNWAY TRUSS

THE ARCH STRUCTURE

Weighing in at 1,500 tonnes, the arch is a 7m diameter lattice shell structure with a span of 315m and a height of 135m. The lattice shell is formed by 12 No. 457mm diameter Circular Hollow Section chords laced together with 300mm square hollow section diaphragm rings at 11m centres. The lattice form of the arch allows the variable global axial and shear actions to be resisted in an efficient manner while maintaining an elegant uniform architectural form. This is achieved by varying the thickness of the arch chords from 10 to 60mm depending on loads.

The forestay cables to the front of the north roof and the backstay cables to the PPT attach to the arch at stiffened diaphragms. These cables are arranged in a diagonal pattern to provide a flexible triangulation in the form of a "Warren" cable truss. This assists the arch with spreading the loads and controlling its in- and out-of-plane bending and buckling. The arch is inclined to the vertical plane by 22 degrees. The stability of the arch is provided by twin 300m long, 120mm diameter catenary cables located above the southern edge of the north roof and the backstays tied back to the PPT. Leaning the arch back allows forestay cables to receive "natural" pretension as they form a cable net with the catenary cable preventing the arch from rolling backward under its self weight (refer to Figure 4).

Overall the geometry of the arch has been optimised to provide efficient support to the roof, maintain sufficient tension in the cable systems and to clear the bowl whilst keeping its abutments within the stadium precinct.

At its ends, the arch tapers to a "pencil end" and sits on a bearing that permits rotational movement in three orthogonal directions.

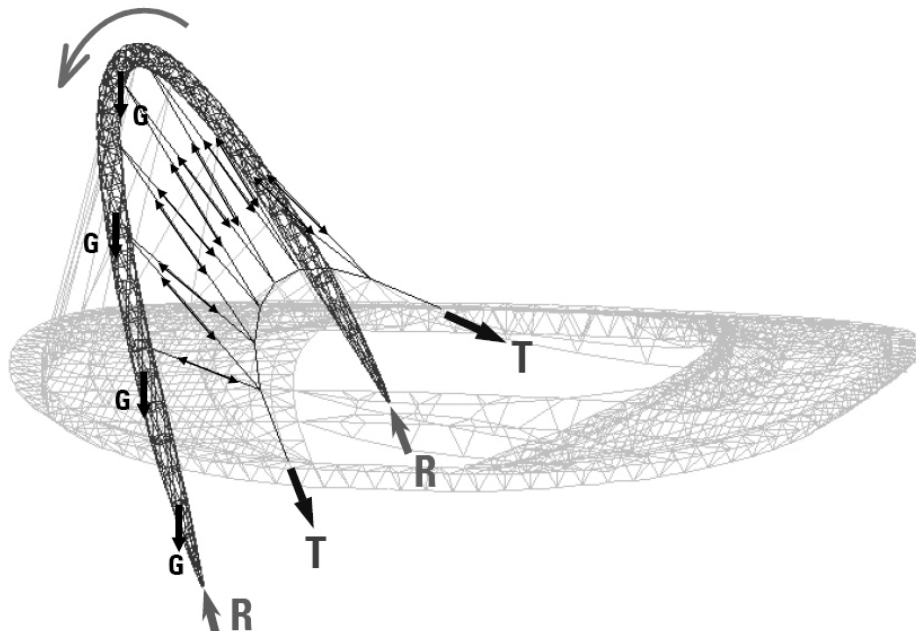


FIGURE 4: ARCH AND CABLE NET PRETENSION SYSTEM

The arch was constructed segment by segment on trestles in the horizontal plane. It was then erected in a 3 week period by a "roll up" operation where the arch was incrementally rotated from zero to 112 degrees. The arch was rolled up using five lines of strand jacks and five rotating struts up to 100m in length. During the lift the arch geometry was continuously surveyed, at five degree increments, and checked against the pre-calculated conditions before proceeding to the next lift increment.

ANALYTICAL CHALLENGES

The long span nature and architectural intent of the arch required it to be designed to be as efficient, slender and lightweight as possible. The structural implication of this slenderness is a unique level of interaction between the overall buckling of the arch and the local buckling of the individual chords. The global buckling of the arch, depends on the loading in the arch and the degree of global restraint offered by the forestay and backstay cables.

Extensive nonlinear buckling analyses were conducted considering a range of expected geometrical imperfections in the arch in its erected form. These included developing novel incremental analysis techniques that accounted for tension stiffening and softening of the cable net formed by the forestays and backstays, in restraining the arch at the onset of buckling.

The large number (500) of arch nodal connections warranted detailed finite element (FE) based analysis and design optimisation to determine the final thicknesses of the connection plates.

An automated design procedure was developed which took advantage of the similarity in the geometry of the arch nodes and effectively dealt with variability of loading conditions. In this procedure 3D models of the arch nodes were generated on a CAD platform. These models were then imported into the FE analysis software as a combination of shell and solid objects.

The objects were then connected and auto-meshed with an optimum level of refinement, striking a balance between computer run time and the precision of the results. Design criteria developed based on first principles and calibrated against safety indices of BS5950 were then applied to assess the analysed stresses and strains, in order to ascertain the adequacy and efficiency of the connection design.

The design of the roof structure is based on global models incorporating over 40,000 elements of the roof (purlins, rafters), trusses and the arch. Using the Prosteel CAD package, the geometry of this model was developed through an interactive process and was then imported into a number of analytical packages wherein extensive nonlinear analyses were performed.

Detailed optimisation studies were also conducted to ascertain the required magnitude and distribution of prestressing forces in the cable structure. Full non-linear buckling analyses of the semi-Vierendeel trusses were conducted accounting for the flexibility of the bracing and the unusual lateral-torsional buckling behaviour of the truss.

All analyses included for the nonlinear effects of the cable sag, and nodal and member level large displacements (P-Delta effects). Separate moving roof panel models were superimposed

onto the global fixed roof model to assess the behaviour of the panels due to the movement of the fixed roof structure and the warping stiffness of the panels was also assessed.

ACKNOWLEDGEMENT

Aurecon provided structural engineering services for the design of Wembley Stadium's arch and roof to Mott MacDonald.

The roles of the following parties in this project are acknowledged:

- Project owner - Wembley National Stadium
- Architects - World Stadium Team (Foster & Partners, HOK)
- Engineers - Mott MacDonald, Aurecon, Sinclair Knight Merz
- Main Contractor - Brookfield Multiplex