London 2012 Olympic Velodrome
How the structure works

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Expedition
Structural and Civil Engineering

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1.0 Introduction

The Velodrome is located at the North end of the Stratford Olympic Park. The venue provides seating for 6,000 spectators, arranged around a 250m long track. The building was used for the track cycling events for the London 2012 Olympics and Paralympics, and subsequently will be converted to its Legacy format as the centrepiece of a Velopark. This document provides a general description of the primary structural elements, design assumptions and load paths.

Location of London 2012 Velodrome

2.0 Detailed description of structure

2.1 Roof

The primary structure of the roof consists of a doubly curved cable net with a total area of 12,500 m². The maximum horizontal span on plan in the East-West Direction is 131m and 119m in the North-South direction.

The final dip in the completed state (the 'quasi-permanent' case, with no environmental loads) is 7.54m between the centre of the roof and the highest cable termination. The corresponding cable rise of the East-West cables is 4.83m.

The cable roof consists of pairs of 36mm diameter spiral strand cables, separated by 120mm. The pairs of cables are arranged at 3.6m centres in both North-South and East-West directions. At the locations of the roof lights the East-West cables close to 2.0m centres. The cables have swaged end fittings and are to be fabricated to a dead length: no adjustment to cable length is possible once the cables have been fabricated. The total length of cables is approximately 14,900m. The cables have a Galfan coating, which is suitable for external environments, while the nodes are galvanised.

Cables being pulled into place and connected into the ring beam on site.

Steel nodes clamp the four cables together at every location where the pairs of cables cross. The nodes are made up of three forged elements which clamp the cables (top, middle and bottom plates). The clamping forces are typically designed to prevent slip during erection rather than forces induced once the cable net has been installed.

The nodes are used to support the roof cladding system. This consists of 3.6m x 3.6m (nominal) timber cassettes, which are proportioned to be able to carry all imposed and environmental loads. The cassettes are corner fixed to the nodes located underneath. A Kalzip aluminium standing roof system is supported by the top surface of the timber cassettes. Where roof lights are provided, the 3.6m square cassettes are substituted with 3.6m x 2.0m wide cassettes which contain an integral roof light panel. The vertical separation of the East-West and North-South cables is 44mm centre to centre; the lower of the cables is orientated in the North-South direction.

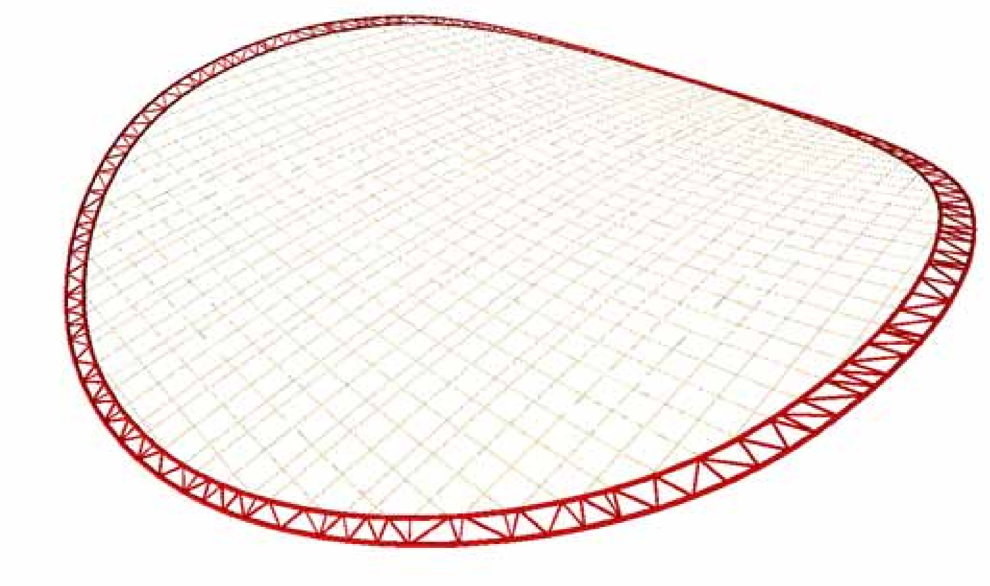
Roof node

2.2 Ring beam

The roof cables are stressed against a steel ring truss which runs around the perimeter of the roof. The ring truss provides several structural purposes:

- Provides a reaction to the cables
- Transfers cable forces to adjacent rib trusses
- Provides a perimeter compression member which carries a proportion of the cable forces at high level
- Directly supports roof panels located above the truss
- Provides support to the perimeter gutter
- Provides temporary support to a perimeter access walkway (construction phase)
- Provides a reaction for stressing jacks during the erection of the cable net (construction phase).

The ring truss consists of a pair of 457CHS chords with smaller CHS web members. The separation of the two chords varies around the structure, and is a function of the geometry of the gutter and orientation of the rib trusses and roof profile. The arrangement of the web members is governed by the need to limit secondary stresses in the chords and to provide local support to the timber cassettes directly above the truss. The width of the truss varies from 3.6m at the northern point to 2.0m at the lowest points.



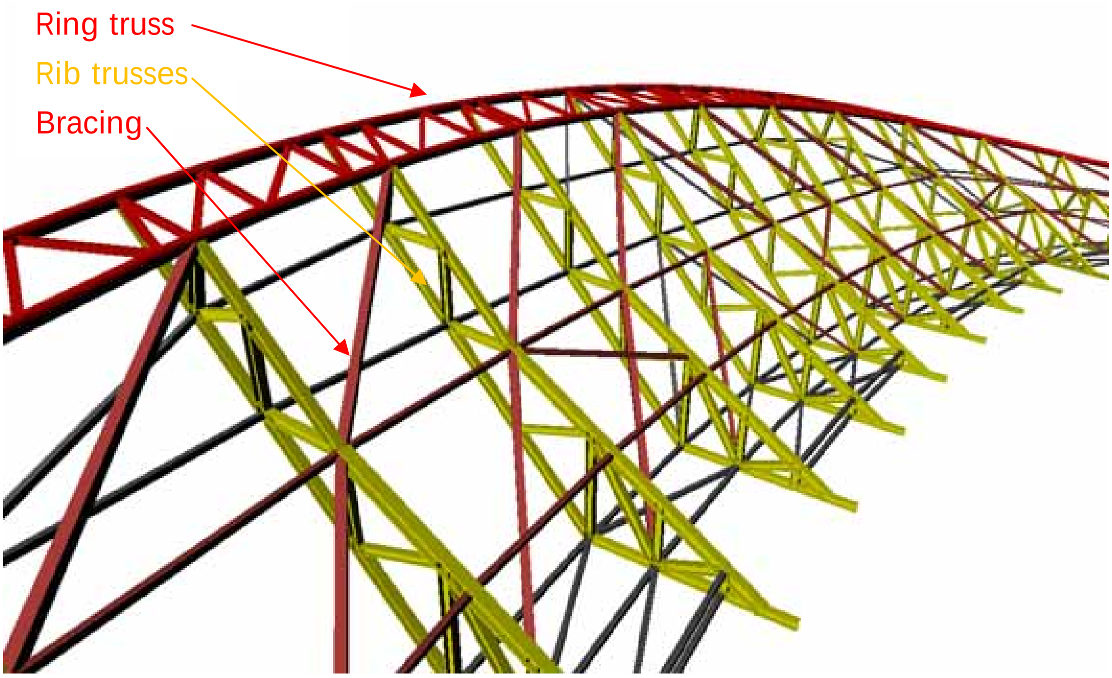
Ring truss and cables

2.3 Rib Trusses

The ring truss is supported by 48 approximately equally spaced rib trusses arranged radially around the building. The steel truss weights vary between approximately 2 tonnes and 20 tonnes.

The trusses serve a number of purposes:

- Support of the ring truss
- Resist a large proportion of the horizontal loads from the cables by transferring the loads to low level
- Support of the perimeter cladding
- Support of the upper tier seating and vomitories
- Support of the upper tier plantroom slab.



Ring truss, rib trusses and bracing

The rib trusses (shown yellow, above) are generally fabricated from UC sections and are fully welded.

The trusses are supported by pairs of stub steel columns that sit on the top of concrete piers, with a top of concrete level set at +19.20m. The lower part the stub columns are visible to the general public.

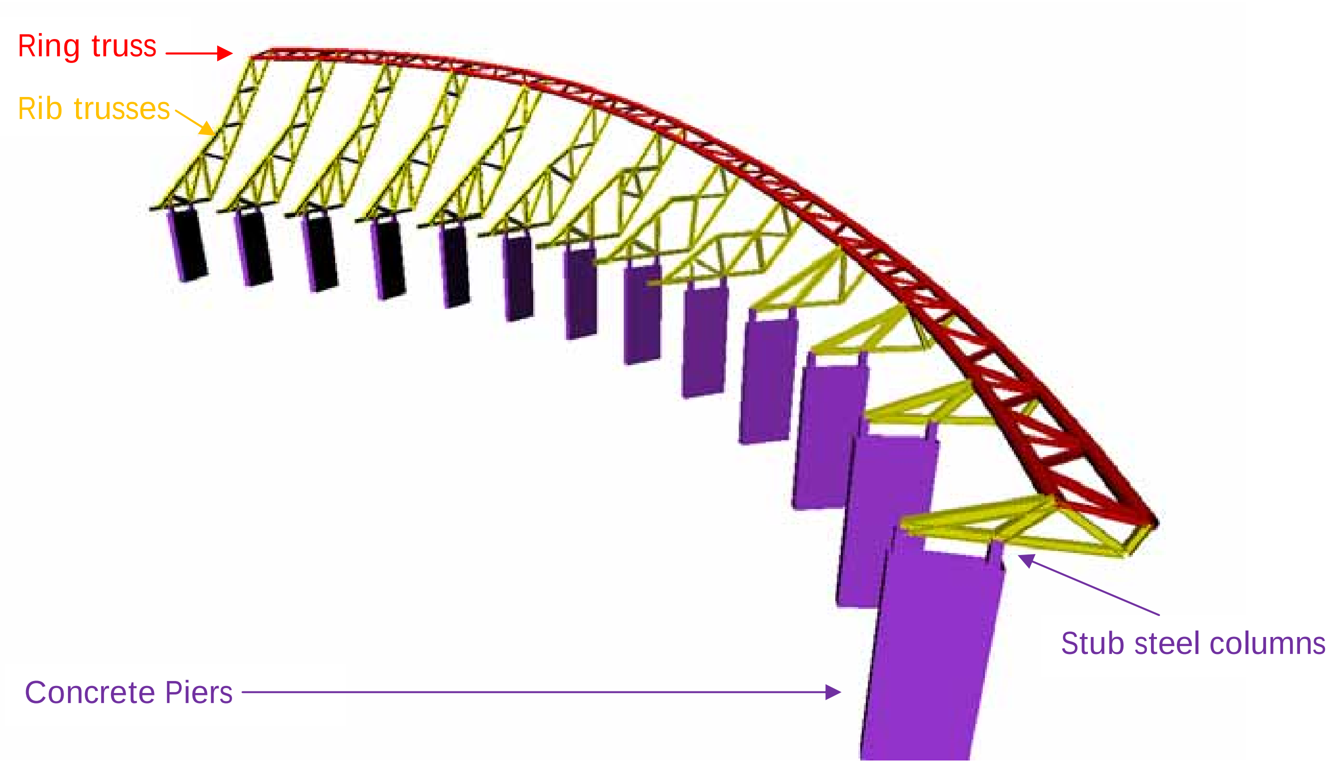

Stub steel columns in fabrication

2.4 Bracing

Diagonal bracing members are provided between the rib trusses. These members provide lateral stiffness to the upper bowl, but also contribute to distributing horizontal loads around the upper bowl.

2.5 Concrete Piers

The 48 concrete ribs support the rib trusses and take all roof and upper tier loads to the foundations. The piers also support part of the concourse slab. They rise from foundation level to the underside of the upper tier, 2.2m above concourse level.



There are two sizes of piers: 3200m long by 750mm wide, and 2800mm long by 600mm wide. All piers have a 400mm deep rebate to one side; these are used as service risers. All piers are cast with a high quality fair faced finish.

Some piers are post tensioned to carry tensile loads directly from the rib trusses to the foundations.

2.6 Concourse Level Slab

The concourse slab is 300mm thick. The slab has a flat soffit throughout, which is finished to a high quality finish (class 1) in many locations. To avoid issues of differential settlement between the slab and the bund, the slab is supported wholly by structure founded on the main building foundations. Proprietary shear connections ensure that the external slab acts independently to the internal slab. The external slab contains radial movement joints to limit thermal effects. Pour strips mirroring those in the ground slab below are formed in the internal slab in order to manage the slab movements.

2.7 Foundations and Pile Caps

The large overturning forces in the 48 piers are resisted by individual pile caps arranged radially on the main building gridlines. Overturning is resisted by a combination of self weight of the building and by self weight of the pile caps themselves. The higher loaded pile caps have enlarged heels to increase restoring forces.

2.8 Piles

Piled foundations are used for the main structures given the relatively large loads involved. In addition infield, track support and plant slabs are piled in order to limit deferential settlements. Three types of piling are used in the Velodrome:

- For higher loaded foundations continuous flight auger (CFA) piles taken into the Thanet Sand are used. These have been taken to have working loads of 1.6 and 3.2MN respectively.
- For lightly loaded foundations 270mm square precast concrete piles are used in the design. These bear into the River Terrace Deposits and have a working load of 200kN.

- In order to minimise cost and programme permanent sheet piles have been utilised in the basements and infield ramps. This are used to support vertical loads and retain soil. These sheet piles are driven into the River Terrace Deposits and have varying working loads.

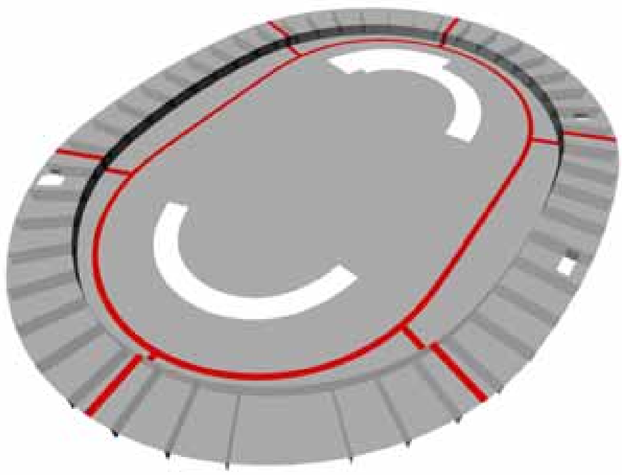
2.9 Ground Floor Slab

The ground floor slab extends around the whole perimeter. The reinforced concrete slab is 250mm thick with a thicker bay in the western extension, to support the rainwater harvesting tanks. An edge thickening is provided to support the perimeter block work walls and cladding loads.

2.10 Substructures

The substructure is of monolithic construction in order to provide propping to the main pile caps. These props ensure that lateral loads from the roof are generally resolved at ground level rather than requiring the foundations to resist high lateral loads. Movement joints would prevent the structure acting in this manner and as such are not provided.

During construction a series of pour strips were left uncast until a short time before the roof cables are installed. This allowed a proportion of the concrete shrinkage to occur before the full substructure is locked together.



Pour strips of the velodrome slab

The individual elements of the substructure are discussed below.

a. Basements

The two basements, are formed using permanent sheet pile walls. The base slabs, 300mm thick, are formed with waterproof concrete cast against the sheet piles with load transferred through shear studs welded to the in-pans of the sheet piles. The sheet piles are topped with a concrete capping beam which is used to tie the sheet piles into the main structure, provide propping and allowing vertical loads to be taken by the sheet pile wall.

b. Infield Ramps and Doping Suite

Leading up from the basements are ramps formed with sheet pile walls. These have a 300mm thick slab and waterproof concrete facing walls tied into the main structure through the capping beams. As the retained height becomes smaller the sheet piles stop, beyond this, the ramps are supported on pre-cast piles and the doping suite is constructed. This is formed under the track safety zone and is a waterproof concrete box, 300 thick slab with 250mm and 300mm thick walls, founded on pre-cast piles.

c. Infield Slabs

A reinforced concrete slab at infield level extends across the whole structure bounded by gridline A. The slab, 250mm thick, forms the infield slab, track safety zone support, track slab and plant slab, where it ties into the plant wall. Suspended from the infield slab are a number of service trenches and pits all formed from waterproof concrete. A pour strip adjacent to gridline W helped manage the movements of the monolithic reinforced concrete substructure.

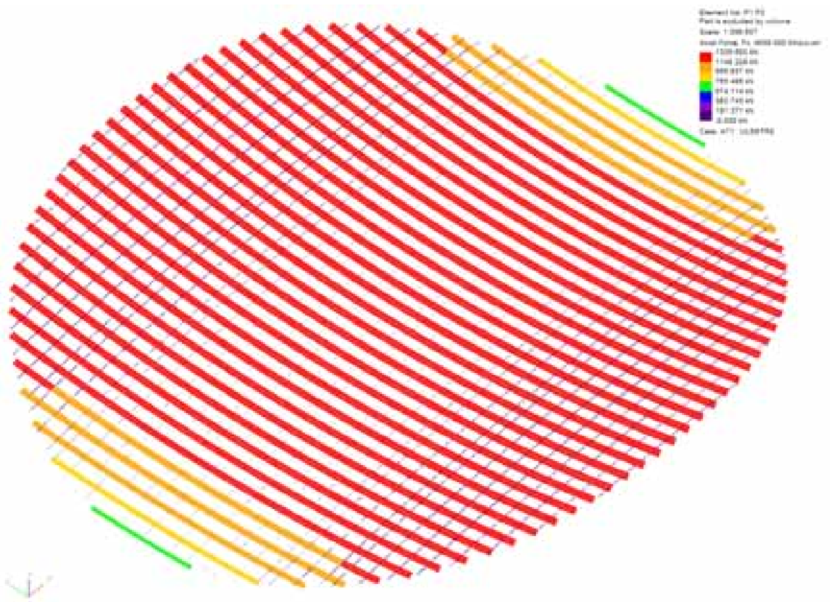
3.0 Structural Behaviour

This section of the report briefly describes the structural behaviour of the significant and more unusual elements of structure.

3.1 Roof

The shape of the roof has been form-found to maximise efficiency of the cables. The prestress in the roof cables has been set to a level where no cable becomes slack under any SLS load combination.

Under the highest Ultimate Limit State (ULS) vertical uniformly distributed load the maximum cable tension is 1340kN per pair, as shown below. Due to the roof being form found, the cables forces do not vary significantly across the roof.



Cable forces in the main sagging cables under the worst case ULS uniform roof loading.

Under the worst case uplift the tension forces in the sagging cables reduce to approximately 75kN tension.

The cables do not permit adjustment to the free lengths once fabricated, and therefore the prestress state of the cables depends wholly on the as-installed locations of the pins at each end. Recognising the difference in construction tolerance between the cables and the general steel bowl, these connections are designed to be able to be adjusted (before the cables are installed) in the direction of the cable, to accommodate the main bowl being out of the necessary tolerance of 5mm.

3.2 Horizontal Load paths from roof to foundations

The large horizontal forces applied by the cables to the ring beam are resisted via a number of distinct load paths:

- The ring truss and other circumferential members within the upper bowl retain some of the load at high level, acting as a compression ring. This behaviour can be further broken down into loads that are retained within the ring truss alone, and those that are transferred down to lower circumferential steelwork by the diagonal braces.
- Loads are transferred to the tops of the rib trusses by the ring beam and transferred directly down to the foundations through the truss and concrete supporting piers.

- Loads are transferred to the tops of the rib trusses by the ring beam and transferred directly down to the concourse slab where horizontal loads are balanced across the structure.

The contribution of each load path also varies around the building: in some areas approximately 80% of the local cable forces are carried by the rib trusses directly to the foundation. This is due to the relatively greater stiffness of the vertical load paths with respect to the ring truss. Towards the Eastern and Western ends the contribution of the ring truss increases greatly.

3.3 Buckling resistance of the ring truss

As described above, the ring truss transfers loads locally to the adjacent rib trusses and also transfers forces around the building at high level. The truss is generally in compression under all combinations, with ULS chord compression forces up to 4.6MN.

The effect of global buckling of the ring truss has been accounted for by the use of a design method where the geometric stiffness matrix is used for both direct and eigen analyses. (note that this method is only applicable where member loads that give rise to buckling can be calculated with sufficient accuracy using first order analysis and that material behaviour is linear elastic). For the members that are checked using this method this assumption is valid.

The method used allows for:

- The increased response of the perfect structure to loading
- The additional response of the structure due to imperfections

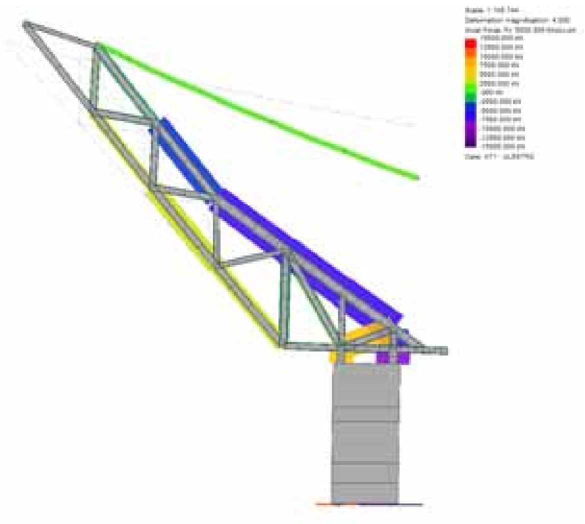
Imperfections were based on EC3 guidance.

The analysis has shown that the effective length of the ring truss is approximately equal to the distance between rib trusses. This implies that the rib trusses are suitably stiff to provide positional restraint to the truss.

3.4 Structural behaviour of the rib trusses

The rib trusses act as inclined cantilevers and structural behaviour is generally as would be expected.

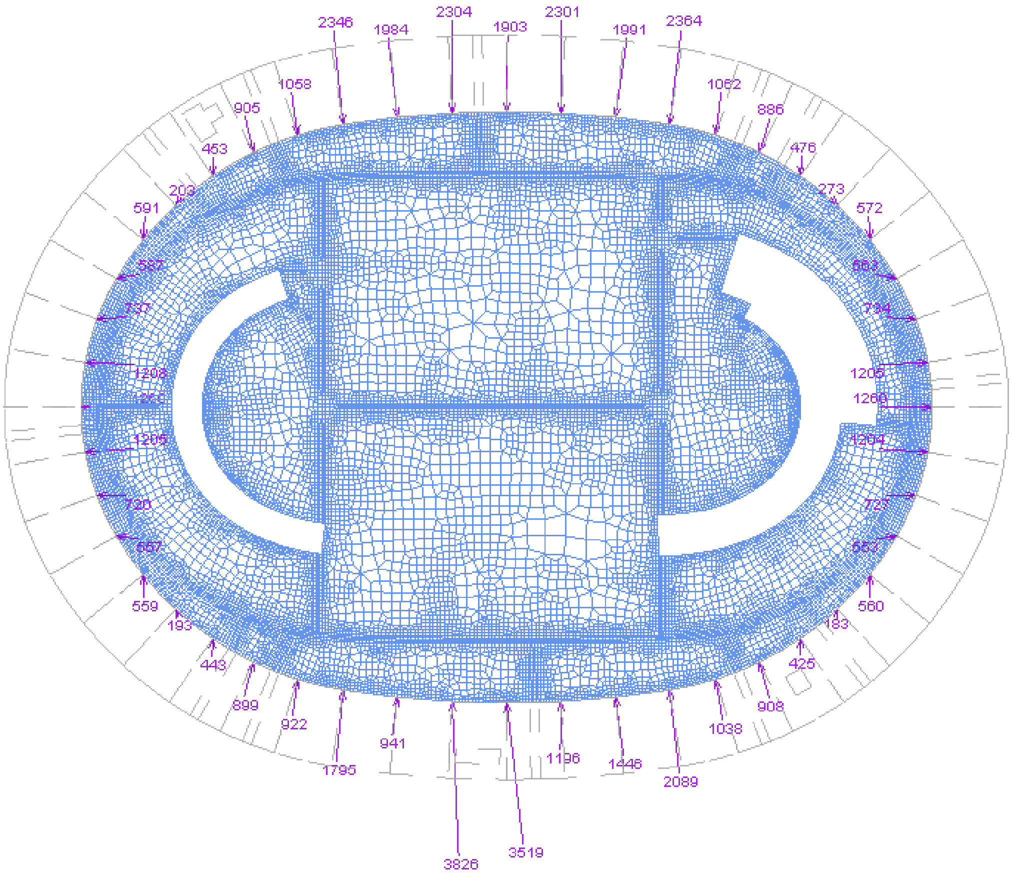
Under all permanent load cases the inner truss chords of the northern and southern trusses are subjected to compression forces. The lower part of the chord (under the pre-cast seating tiers) is braced by longitudinal steel member. No benefit has been taken from the pre-cast seating to act as lateral braces as the curvature of the bowl induced relatively high compression forces on these members, which was difficult to take into the member.



Axial forces in rib truss on gridline 13 under the maximum roof ULS load.

3.5 Structural behaviour of the Infield slab

As described above the infield slab is used as a strut to resist lateral loads resulting from the horizontal thrusts from the roof. The peak horizontal thrusts are applied by the pile caps along the northern and southern sides. The infield slab is continuous between these two sides resulting in a very direct load path.



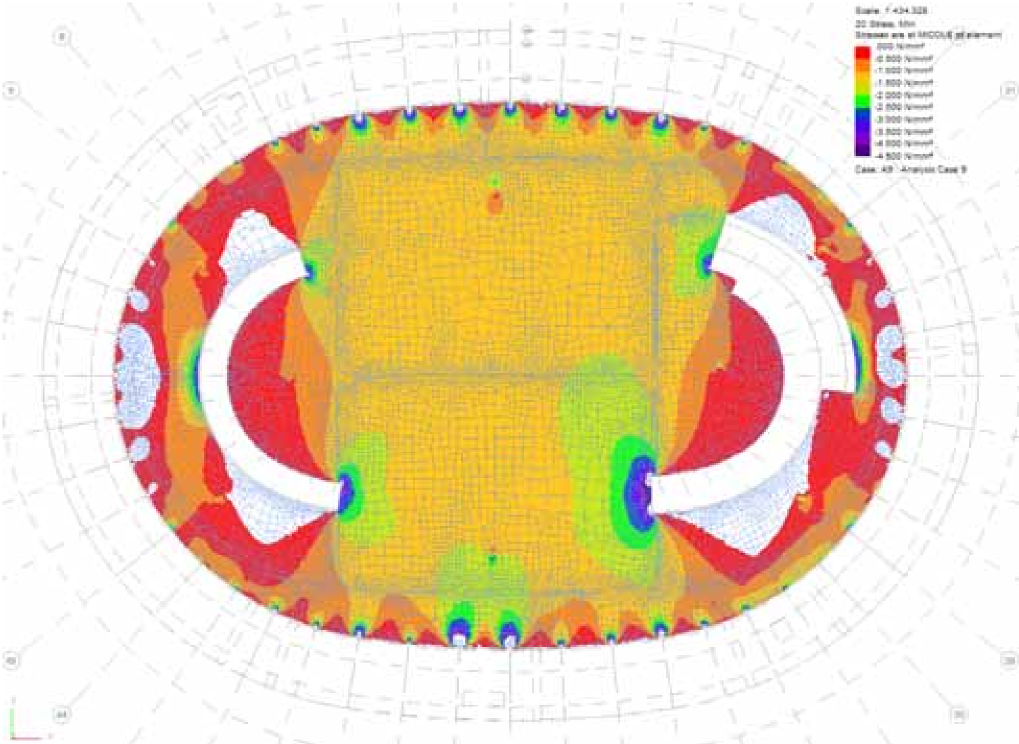
Typical lateral loads applied to infield slab (kN, ULS)

The Eastern and Western ends of the infield slab are broken by the two infield ramps which means that it is not possible to take thrusts applied at these ends directly into and through the main infield slab. In these cases the curved slabs beyond the ramps act as deep horizontal arch/beam. Although the slab is continuous with the ground slab beyond, for the purposes of design it has been assumed that only the infield slab is effective.

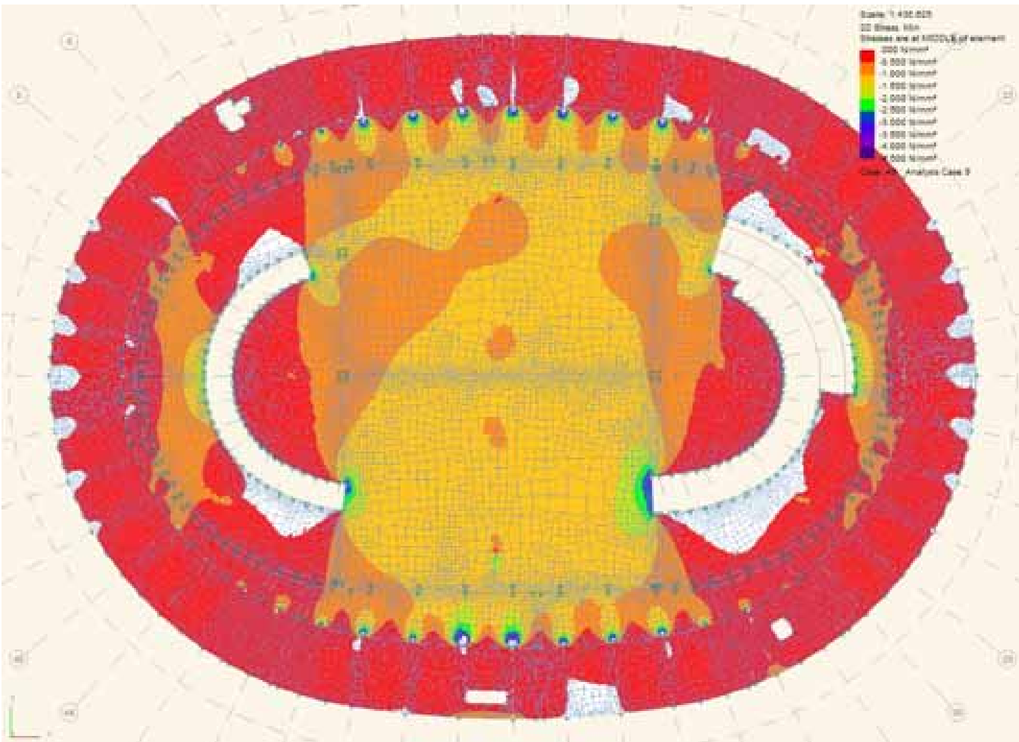
The restrained long term drying shrinkage strains are likely to place the infield slab into tension. However, the addition of the large compression stresses between the northern and southern sides means that the infield area is compression. If all shrinkage strains are assumed to be unrestrained, and therefore do not create any tensile forces, maximum average ULS stresses away from pile caps are in the order of 1.2MPa.

The bucking capacity of the infield slab has been calculated using a geometrically non-linear model. As the piles have been assumed not to have a tensile capacity they behave in a non linear manner: they resist downward movement but upward movement of the slab is only resisted by the self weight of the slab. Therefore simple eigen buckling is not valid.

Whilst it has been assumed that the infield slab alone (i.e. within gridline A) restrains all lateral loads at pile cap level a further assessment has been made in which the additional stiffness provided by the plant wall and ground slabs has been combined. The two images below show the difference in the stress field: typically infield stresses are unchanged, while the bending stresses in the Eastern and Western ends are reduced due to the greatly increased effective depth of section.



Maximum compressive stresses in infield where only the infield is assumed to resist lateral loads.



Maximum compressive stresses in infield where composite behaviour of the infield and ground slabs are assumed to resist lateral loads.

3.6 Structural behaviour of the substructure in terms of shrinkage

As described above, the ground and infield slabs are monolithic from the time the roof cables are stressed. Whilst this provides a benefit to the distribution of lateral loads, the long term shrinkages have been studied and a construction sequence proposed in order to prevent unrestrained shrinkage strains displacing the tops of the piles beyond their capacity.

A staged analysis has been carried out in order to estimate long term movement. As the effects of restraint, creep and shrinkage are difficult to predict, conservative values have been taken. The pile design (a contractor designed element) incorporates the effects of these displacements. As would be expected pile reinforcement is heavier than that which would be expected for piles that are not subject to lateral movements at the head.

3.7 Vertical load paths

A summary of vertical load paths is as follows:

- The vertical component of the roof cable tensions are transferred by the ring truss to the tops of the rib trusses.
- Roof level vertical loads are transferred down to the concrete piers through a combination of the rib trusses and the diagonal bracing. Generally downward roof loads are beneficial to the overturning stability of the structure as they resist the cable overturning forces
- The vertical loads from the upper tier are carried by the trusses to the tops of the concrete piers. Generally the upper tier loads are beneficial to the overturning stability of the structure as they resist the cable overturning forces.
- Vertical loads in the concrete piers are taken directly down to the pile caps.

6.8 Horizontal load paths

A summary of horizontal load paths is as follows:

- Cable forces are carried to the infield slab and pile caps in the manner described above.
- Imbalanced lateral loads applied at high level, including wind loads, roof pattern loading and upper bowl dynamic lateral loads, are transferred to the tops of the concrete piers through the diagonal bracing and longitudinal steel members.
- Lateral loads are transferred to pile cap level through major axis bending of the piers in line with the load resultant and minor axis bending of the piers perpendicular to the resultant.
- The stiffest load path is through major axis bending of the piers and some loads are therefore transferred around the ring truss, upper bowl bracing, upper bowl longitudinal members and the concourse slab.
- A degree of minor axis bending of the main piers still occurs and these have been designed accordingly.