Traditionally, the Olympic Games have produced a number of exciting structural designs and have left the host city with a legacy of outstanding sports facilities. Often, this has resulted in an equally monumental debt. The Speed Skating Oval (see Fig. 1) for the 1988 Winter Olympics in Calgary, Alberta, is different.

The Olympic Oval features a unique precast prestressed concrete, segmental arch roof structure which marries a world class structure to a very austere budget. The maximum building budget for the entire facility was set at $30 million (Canadian funds in 1985). The actual construction cost was $27.2 million (Canadian). The precast prestressed concrete work (including post-tensioning) amounted to $3 million (Canadian).

The Oval, constructed as one of the venues for the 1988 Winter Olympic Games, will serve during the Games as the site for speed skating and, after the Games, as a multi-functional athletic field house. These functions and a general description of the complete building have been described in a previously published paper. This article will describe the design, manufacture and erection of the precast roof structure itself.
The Olympic Oval, a $27 million speed skating facility made for the 1988 Winter Olympic Games, was built using precast prestressed concrete. This report describes the concept, design, production and erection features of the roof structure.

Fig. 1. Panoramic view of Olympic Oval with Calgary skyline in background.
CONCEPTUAL DESIGN

An initial program for the building was prepared which defined the design criteria from a functional viewpoint. The essential aspects of the program affecting the structural design were the requirements for high quality and reasonable cost. Funding for the project was provided by the Government of Canada as part of their commitment to the Olympic Games and it was the desire of the owners, the University of Calgary, to maximize the use of these funds by constructing a facility with as multi-functional a purpose as possible.

In addition, although the initial capital cost was to be paid by the federal government, the maintenance and operating costs of the facility would be the responsibility of the University of Calgary. The structure, therefore, had to provide a long clear span; be economical to construct; and be low maintenance and have a long life.

The architects and structural engineers, although independent firms, worked very closely together in developing the building design. The architects, working from the owner’s program, defined a building “footprint” and cross section which adhered as tightly to the program as possible. In order to minimize both capital and operating costs, floor area and volume were reduced as much as possible.

The role of the structural engineers was then to develop a roof structure with the best possible fit to the architectural requirements and to use the shape of the building cross section to structural advantage.

The cross section was easy to accommodate. To provide maximum height over the playing surface yet low exterior walls, in order to minimally impact adjacent buildings, a very low arch structure was chosen.

The plan, however, was quite unusual having a long straight center section and a semicircle at each end. The traditional, parallel arch, barrel vault obviously would not be appropriate for these circular ends.

Three families of solutions were reviewed at this stage:
1. A barrel vault center section with a different framing system at each end. The end framing alternatives included cable suspended fabrics, steel trusses or partial domes.
2. An intersecting arch system spanning both the center section and the ends.
3. A steel space frame system.

The steel space frame system utilizing a proprietary, pre-engineered system was priced and found to be beyond the capabilities of the project’s budget.

The parallel and intersecting arch schemes were compared and the intersecting system was found to have several advantages over the barrel vault:
— A single spanning system could accommodate the entire building.
— The grid provides structural redundancy with alternate load paths to accommodate heavy point loads or snow drifts.
— The arch grid was stable during erection without lateral bracing.
— The arch grid is flexible and can accommodate thermal and volume changes without any expansion joints.
— Since the grid does not depend on the deck for stability, the deck could be detailed to “float” on the arches. This allows a variety of envelopes, including insulated fabrics, to be considered, independent of the structure.

Preliminary alternative designs were carried out for the intersecting arches utilizing triangular steel trusses and trapezoidal concrete box segments. While costs for the two alternatives were similar, the concrete option was chosen due to the simplicity of the node intersections, the clean appearance of the structure, and the perceived logic of using concrete for a structural system which is predominantly in compression.

Indeed, the economy of the entire roof
structure was developed by using the most logical structural materials for each component; from the preformed steel deck, long established as the most economical noncombustible decking material, to the short span open web steel joists spanning between the arches, to the concrete arches themselves.

**DESIGN DEVELOPMENT**

**Loads**

The structure was designed for dead loads; wind and snow loads; thermal effects; creep and shrinkage of the concrete arches; and for lateral movement of the foundations.

Each of these load effects is discussed below:

1. **Dead Loads**: These included the self weight of the arches, equivalent to a uniform roof load of approximately 2.5 kPa (50 psf); steel framing, metal deck, waterproofing membrane and inverted roof system, a further 0.5 kPa (10 psf); an allowance for miscellaneous loading from lights, sound system, ductwork, and potential future hanging loads of 0.5 kPa (10 psf).

2. **Wind and Snow Loads** (see Fig. 2): Since the shape of the cross section is

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Fig. 2. Snow and wind loading.
similar to that used for a curved roof, hangar type building, a preliminary estimate of wind and snow loads was taken directly from the National Building Code of Canada commentary for this type of building. Final loads were confirmed in a wind and snow study conducted by Morrison Hershfield Ltd. using a 1:400 model and are shown in Fig. 2. Additional drift loads, caused by filling of the valleys between the arches were also included and these loads were applied in alternate bays resulting in maximum torsion on the arches.

3. Thermal Effects: When final design first commenced there were two possible construction schedules, one requiring winter erection and one requiring summer erection. As a result, the structure was analyzed for temperature variations of plus 17°C or minus 45°C (plus 31°F or minus 81°F) from the erection temperature. Since temperature gain produced an effect opposite to that of creep and shrinkage, it was ignored in the final analysis.

4. Creep and Shrinkage: Long term creep and shrinkage were modelled as a further temperature reduction of 74.5°C (134°F).

5. Foundation Movements: As design was proceeding on the roof structure, the foundation design was carried out based on a series of lateral load tests on large diameter concrete piles. These load tests predicted a lateral movement of approximately 10 to 15 mm (0.39 to 0.59 in.) under working loads. The structure was analyzed for outward lateral displacement of 20 mm (0.79 in.) at all supports along the straight portions, where the substructure provided a stiff diaphragm to ensure that all movement would be similar, with reducing amounts of 15 and 10 mm (0.59 and 0.39 in.) at the ends where the arch spans were shorter. In addition, an arbitrary lateral displacement of 20 mm (0.79 in.) acting singly at the isolated pile caps at the ends of the building was input as a separate loading case.

Load Factors

Because the arches are designed essentially as slender beam-columns, subject to potential buckling effects; and because, in the event of an overload on the roof, it was considered desirable to ensure that the secondary framing would fail prior to the arches; a higher load factor was applied to the design of the arches than was used in the design of the secondary steel framing.

Dead and live load factors of 1.25 and 1.5, respectively, were applied in the design of the metal deck and steel joists while the factors were increased to 1.25 and 2.5 for the arches. Under the effect of uniform roof load this is equivalent to an increase in load factor of approximately 17 percent, a similar increase to that used historically in the design of slender beam-columns.

Structural Analysis

Preliminary arch designs during the conceptual development stage were carried out manually. The initial arch depth requirement was estimated from the flexural moments due to snow drift load on one-half of the span. The arch width was calculated from lateral buckling requirements assuming the arch to be laterally supported only at the node intersections.

The final analysis was carried out using a STRESS program. Fourteen different loading conditions and twelve combinations were analyzed. In addition, a unit load analysis was run in order to provide an easy reference for determining the feasibility of adding future loads to the structure.

Based on the results of the initial computer runs, cracked section properties were estimated for the arches in order to review the ultimate deflections and check for potential buckling. An iterative P-delta analysis was used for this purpose with three cycles of iteration.

The preliminary design assumed that
the arches would be reinforced with mild steel reinforcement. The final analysis indicated that there was insufficient dead weight in the structure to overcome the moments created by large snow drift loads and that flexural tension was controlling the design of the arch segments. Concentric post-tensioning of the arches was an effective method of increasing the compression in the section and providing tensile reinforcement.

At the conclusion of the computer analysis, interaction diagrams for the arches were prepared taking into account the effects of slenderness and biaxial bending. Two sets of curves were prepared; one set for the hollow arch itself and one for the solid nodes. The axial forces and moments due to the various combinations of load were plotted on these interaction curves to determine the reinforcement required.

**Thermal Effects and Volume Changes**

Provision for expansion joints in large roof structures can be a problem since the magnitude of movements occurring at these joints can lead to ongoing maintenance problems and potential roof leaks. The intersecting arch grid does not require expansion joints since the grid itself is capable of flexing to accommodate volume change movements.

The steel structure supported on the arches was provided with a series of closely spaced control joints by providing a guided joint at one end of each joist to slip on top of the arches. The movements occurring at each one of these “slip” joints is small enough to be accommodated simply by deforming of the steel roof deck so that a joint in the deck and in the roofing membrane is not required.

To accommodate movements in the substructure, control joints were placed all around the perimeter at two or three bay spacing. The precast concrete perimeter roof beam was jointed at the same locations as the substructure. The roof layout and control joint locations are shown in Fig. 3.

Because the arch forces do not always coincide in direction with the buttresses, lateral forces are placed on the perimeter building columns. To assist the columns in resisting these forces, very stiff tension ties consisting of large hollow structural steel sections were cast inside half of the perimeter beams and connected to the column/buttress heads with high strength threaded Dywidag bars.

**ROOF COMPONENTS**

**Typical Arch Segments**

The typical arch segment is shown in Fig. 4.

The segments consist basically of a trapezoidal thin walled box with 150 mm (5.9 in.) thick webs, a 170 mm (6.7 in.) soffit and a 200 mm (7.9 in.) top flange. Concrete was specified as semi-lightweight with a 28-day compressive strength of 40 MPa (5800 psi) and a dry unit weight of 1770 kg/m³ (2980 lb per cu yd). The typical segment length is approximately 24 m (79 ft) with a weight of about 48 tonnes (~3 tons).

In order to maintain constant elevations for the steel joist seats on opposite sides of each arch, one side of the arch was cast higher than the other. Also, since the slope of the segments would vary depending on their location in the completed arch the offset had to be varied between those segments near the end of the arch and those near the centerline of the building. Two basic sections were used, each with a “left” and a “right.”

The final axial reinforcing ratio was one percent. This reinforcement was concentrated close to the corners of the arch segments in order to be effective for bending in both the vertical and horizontal directions. (Horizontal bending
Fig. 3. Roof plan of Olympic Oval showing principal structural components and major dimensions.
occurs due to the tilting of the principal member axes due to the slope in the top flange.)

To resist very high torsions occurring in the arches due to snow drifts in alternate bays, closely spaced stirrups and a uniform distribution of longitudinal reinforcing were provided. Because this longitudinal reinforcing was also used to resist compressive forces, 6 mm (0.23 in.) ties were provided to prevent buckling of these bars.

The arches are concentrically post-tensioned with seven 16 mm (0.6 in.) diameter strands in ducts in each corner of the section. Since the maximum positive and negative moments in the arches were found to be almost equal, this post-tensioning layout was the most effective.

**Typical Nodes**

The arch intersections were referred to as “nodes.” Typical interior and exterior nodes are shown in Figs. 5 and 6, respectively.

At each node, all of the forces in the arch segment (positive and negative moments; shear; torsion; and axial compression) had to be transferred from one segment to the others. To accomplish this, all of the axial reinforcement and post-tensioning was fully developed by either mechanical couplers or by lap splicing. Because the arches change direction in the vertical plane at the nodes, the axial forces are not always normal to this joint and shear friction played a large part in transferring the axial forces and shear through the nodes.

To ensure good bond of the precast segments and the cast-in-place concrete, all of the surfaces of the precast element were given a heavy sandblast to an amplitude of approximately 6 mm (0.24 in.). No bonding agent was used but all surfaces were thoroughly wetted prior to placing the node concrete. The section was then cast solid using the same semi-lightweight concrete as used for the segments themselves.

Because the outside webs and soffits
of the precast elements were extended as far as possible leaving only a 150 mm (5.9 in.) gap, a very minimal amount of formwork was required in the field for each node.

**Perimeter Roof Beams**

The perimeter roof beams support a portion of the roof, the exterior fascia and, along the east side of the building, a sloped glazing structure over the administrative offices and VIP lounge. In addition, approximately half of the perimeter beams enclose a steel tension tie connecting the perimeter columns. The beams span approximately 18.5 m (61 ft) and are a hollow box section. A 75 mm (3 in.) precast fascia panel and 50 mm (2 in.) of insulation were cast as a sandwich panel on the lower half of each beam.

The typical beam is shown in Fig. 7. The beam is tilted 28 degrees from the vertical on the building to match the slope of the cast-in-place buttresses.
Since the beam is relatively shallow for resisting flexural moments about the weak axis the section was pretensioned to resist a combination of vertical and lateral moments.

The tension ties cast into the perimeter beams were required to be very stiff in order to be effective in transferring loads between the equally stiff cast-in-place perimeter columns (1500 mm (59...
in.) in diameter]. To achieve this stiffness, the tie consisted of a 300 x 200 mm (12 x 8 in.) hollow structural section with a wall thickness of 12.7 mm (0.5 in.). Additional stiffness for the ties at the extreme ends of the Oval, where the tie forces are at a maximum, was achieved by plating the 300 mm (12 in.) sides of the HSS with 20 mm (0.79 in.) thick plates.

Bearings

The thrusts from the arches are transferred to the perimeter columns and buttresses through very large disc bearings. The typical bearing is shown in Fig. 8.

Maximum axial force in the bearing was 8453 kN (1900 kips) [unfactored] while the maximum vertical shear force was 775 kN (174 kips) and lateral shear force was 1495 kN (336 kips). Shear forces occur due to slight variations in the thrust line of the arches depending on the loading condition on the roof.

The bearings are also designed for a rotation of two percent to allow for deflection of the arches.

It was originally intended that the bearings would be cast into the perimeter buttress heads prior to erection of the precast arches. Unfortunately, major delays in the manufacture of the bearings by the supplier resulted in a delay of approximately six months in the delivery. Fortunately, it was possible to modify the reinforcement of the exterior nodes to allow the bearings to be installed after erection of the arch segments with no overall loss of time in the construction schedule.

MANUFACTURE

The production aspects of the arch roof segments are given first followed by the perimeter beams.

Arch Roof Segments

The structural design of the arch root beams using the trapezoidal shaped cross section provided a very pleasing architectural appearance. This cross section also provided other advantages in the precast concrete production of the
beams. It permitted easy beam stripping, eliminated basic form setup, provided good “as cast” finish and permitted the cage, complete with plywood voids and end block-outs, to be preassembled and lowered into the form.

This procedure maximized repetition and economy by permitting all 84 precast concrete arch roof beams to be cast in one basic form. Although the beams varied in length from 14.2 to 25.4 m (46.6 to 83.3 ft), all were cast in one steel form which had 100 mm (3.9 in.) camber built into it. This form was also equipped with external vibrators to provide a dense smooth beam finish.

All beams had a 800 mm (31.5 in.) wide base but there were two basic cross sections each with a cross-fall on the top of the beam. The basic form was made to accommodate both cross sections.

For the most severe cross-fall, the form side heights were 1800 and 2019.
mm (71 and 79 in.). To produce girders with the smaller cross section, a 89 mm (3.5 in.) side rail was unbolted from the high side to produce a form with 1800 and 1930 mm (71 and 76 in.) sides.

Because weight was critical to the substructure design, semi-lightweight concrete was used. The beams were also voided using 4350 mm (171 in.) long plywood boxes. These boxes were chamfered on all sides and formed a part of the cage. This preassembled reinforcing cage also included the four post-tensioning ducts, protruding end reinforcing bars and inserts, and the end block form.

This end block-out form accommodated the variation in girder lengths and ensured a proper 150 mm (5.9 in.) gap between girders at each node. It also accurately positioned the 20 and 25 mm (0.79 and 1 in.) main reinforcing bars forming a very rigid cage assembly which could be lowered into the form, positioned and poured. By using superplasticizer additive in the 40 MPa (5800 psi) concrete and with adequate chamfer on the box voids, the beams were poured from the top in a one-pour sequence using both internal and external vibrators.

Girders were stripped and yarded with four 15 tonne (16.5 ton) overhead cranes. Low bed trucks with 12 wheel self-steering trailers were used to transport the beams to the site.

**Perimeter Bars**

Perimeter beams were produced in two stages. In the first stage the 75 mm (3 in.) architectural exterior cladding was cast complete with 50 mm (2 in.) S.M. styrofoam insulation backing and protruding anchors. These 1530 x 3750 mm (60 x 141 in.) panels were then positioned vertically in a steel form so that the structural portion of the beam could be poured to produce a composite sandwich structural perimeter beam with architectural finish. The structural portion of the beam was also loafed out using plywood voids within the reinforcing cage and cast using standard weight concrete.

Bulkhead variation was used to produce the various beam lengths [14.7 to 16.9 m (48.2 to 55.4 ft)] required to accommodate the non-typical units at the ends of the building. Of the 28 perimeter beams, 16 required large structural steel tubes complete with end connection hardware which were cast within the beams. These were used as tension ties and ran through the loafed out structural perimeter beams for their full length. The inside face of the beams contained electrical conduits for interior lights and were trowelled smooth.

**PEER REVIEW**

Due to the magnitude of the building and the unusual nature of the design, the structural consultants engaged other independent structural engineers to
Fig. 10. Perimeter roof beams showing steel tension ties.

Fig. 11. Typical arch segment being loaded for shipment.
carry out a review of the design.

The peer review was carried out at two stages: first, at the completion of conceptual design and finally, at the completion of the design and drawings. The concept review was considered valuable in ensuring that no judgmental errors might have been made which might result in potential increases in costs arising during the final design.

The check at the completion of the final design was carried out by running a completely independent computer analysis of the structure. The software used for this purpose was a space frame analysis program written by the reviewers.

Not only was the structure found to be safe, but the peer review actually saved the owner a substantial amount of money. Working from nearly complete architectural and structural drawings, the peer review team was able to analyze the structure without making as
Fig. 14. Aerial view at completion of structural framing showing nodes at various stages of casting.

Fig. 15. Close up of typical interior node prior to placing reinforcing steel and post-tensioning ducts.
many assumptions as are often required during the normal design process.

The final building configuration, dimensions, loadings, etc. were all well defined prior to this final analysis and the result was that the longitudinal arch reinforcing was reduced from approximately 2 to 1 percent, a minor reduction in design terms, but a saving of $120,000 when applied to all 84 arch segments.

The authors strongly support this process on any major project.

**ERECITION AND SCAFFOLDING**

Various phases of the production, shipment and erection of the precast components are shown in Figs. 9 through 18.

The precast components were transported to the site by truck and erected, using two cranes, by the precast supplier.

The temporary scaffolding to support the segments was designed and constructed by the general contractor. Two basic types of temporary support were required: 31 towers under the interior nodes, and 26 exterior steel frames to support the arches at the perimeter columns.

Each of the interior towers was required to support a load of approximately 1350 kN (300 kips). Maximum tower height was approximately 22 m (72 ft). The towers consisted of modified standard sectional scaffolding 2 m wide x 4 m long (6.5 x 13.1 ft). Each tower was supported on four temporary timber piles approximately 6 m (20 ft) long topped by a cast-in-place pile cap. A screw jack was located under each tower leg.

The towers were guyed in the long direction of the building. No guying was required across the oval since the towers were stabilized as erection proceeded by welding of the arch components.
The contractor achieved extremely accurate elevation control by precasting temporary head frames for the top of each tower. These head frames contained temporary steel bearing plates to receive the arch segments. Inverted steel angles were temporarily bolted to the underside of each arch segment at the precast plant to provide a "knife edge" bearing on the head frames.

The arch components were supported around the perimeter of the oval on custom steel trusses. Each truss supports a load of approximately 50 tonnes (55 tons). To complicate matters, the load had to be supported under the end diaphragm in the arch segments which occurred approximately 3 m (10 ft) away from the center of the perimeter column.

To allow for temperature changes, shrinkage, and elastic shortening of the arches due to post-tensioning, the perimeter trusses supported a sliding bearing seat inclined at an angle parallel to the centerline of the arch segment.

**SHORING REMOVAL**

After all components had been erected and the interior nodes had been cast, the post-tensioning strands were placed and stressed. Stressing was carried out from both ends.

The bearings were then set in place in the buttress heads and the exterior nodes cast.

The structure was now ready for removal of the temporary supports, a significant event for the designers and contractor.

The first step was the removal of the temporary steel angles supporting the segments on the perimeter steel trusses. This was accomplished by simply burning out the angles to allow the weight of the arches to be carried by the shear pin in the center of the bearings. Removal of the perimeter supports first would allow the arches to rotate as the interior towers were removed.

Although a screw jack was provided in
Fig. 19. Interior view of finished structure showing complete arch span.

Fig. 20. Public skating after formal opening.
each leg of the interior towers, it was anticipated that friction on the thread would be too great to allow this to be turned. Accordingly, a small bracket was welded to the side of each leg and a small hydraulic jack was used to lift the load off the screw while it was turned.

In order to prevent over stressing of the structure during lowering, each tower was dropped incrementally only 10 mm (0.39 in.) at a time. As the predicted dead load deflection was between 40 and 50 mm (1.6 and 2.0 in.) it was anticipated that four to five increments would be required at each tower. This required the contractor to move back and forth from one tower to another as he gradually moved from one end of the building to the other. The actual deflection at the first tower to be completely unloaded was approximately 43 mm (1.7 in.).

While this operation seems rather primitive, the lowering operation easily kept ahead of the crews dismantling the towers and, in fact, the towers were completely removed at the north end prior to completion of jacking at the south end.

CONCLUDING REMARKS

Construction began in March 1985 and the roof structure was effectively completed by June 1986. Finishing trades and interior work continued until April 1987.

The end result is indeed a most beautiful and functional structure (see Figs. 19 and 20).

Historically most long span buildings have been constructed of light materials, usually steel. Recently, tension structures and fabrics have received a substantial amount of both positive and negative publicity. Except for a relatively small number of domes and shells, concrete long span structures have been noticeable by their absence.

The intersecting precast concrete segmental arch system used for the Olympic Oval allows the precast prestressed concrete industry to compete in the long span building market without any new, expensive plant or erection technology. The system is economical architecturally exciting; and, with the advent of high strength concretes, capable of almost any span.

The Olympic Oval was a winning project in the 1987 PCI Professional Design Awards Program.

CREDITS

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REFERENCES