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COVERING THE FIELD – DESIGN AND CONSTRUCTION OF FORSYTH BARR STADIUM ROOF

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ABSTRACT

New Zealand Rugby Union winning the rights to host the 2011 Rugby World Cup provided a catalyst to proceed with a new totally roofed stadium on a new site close to Dunedin city to replace the previous Carisbrook Stadium, colloquially known as The House of Pain. The new stadium roof provides protection for the players and spectators from the ravages of the Antarctic winds and sleet like rain sometimes encountered in Dunedin. The solution adopted to meet the Client's enclosure requirements was a permanently closed fully transparent roof spanning between a major stand to the south and a lesser stand to the north, thus angling the roof for beneficial presentation to the sun's rays and minimising shadowing of the turf.

There were many challenges to designing such a structure. New Zealand is a seismically active country and the dynamic interaction between a rugby-field sized roof and its supporting stand structures is complex and critical to the structural design. The area is also subject to sub-zero temperatures with significant snow falls. Wide area wind effects had to also be considered. These loading considerations were challenging enough but at the same time it was also critical to minimise shadowing on the field by the supporting structure, both for health of the turf and the adverse effect of shadowing on television cameras.

Construction time was tight to have the facility ready in time of the Rugby World Cup. To build such a structure required the services of New Zealand's largest steel fabricator but this meant fabrication was in Auckland while the site was in Dunedin, placing severe programming logistics on the project.

1. Introduction

Dunedin's Carisbrook Park is the southern-most rugby stadium in the world, was home to the Otago Rugby Union and the scene of many All Black encounters. However due to its exposure to sometimes severe winter weather, it earned itself the nick-name 'House of Pain'. The New Zealand Rugby Union won the rights to host the 2011 Rugby World Cup and this provided the catalyst to proceed with a new roofed stadium on a new site close to Dunedin city.

Roofed stadiums are generally located in major metropolitan areas with large local populations and provide seating around 60,000, necessary to fund elaborate opening roof sections, mobile turf and the like. However, Otago is a relatively sparsely populated region (Dunedin 2010 population 125,000) and the new stadium business plan indicated that it could support only 20,000 permanent seats, expandable to 29,600 for special events.

A common problem for roofed stadiums is turf health. To maintain healthy turf requires both adequate sunlight and good air movement at turf level throughout the year. Dunedin's southerly location results in low

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winter sun angles with weak intensities and consequently it was essential to maximise the opportunities for sunlight to strike the grass.

The solution developed was a permanently closed transparent roof spanning across the field between a major stand to the south and a lesser stand to the north (Figure 1). This arrangement angles the roof towards the low sun's rays for beneficial presentation of light to the turf. The air movement requirement is achieved by having multiple and quite large openings around the sides of the stadium and at strategic locations at roof level so as to generate venturi/chimney effects.

Figure 1: *Under Construction - 16 Nov 2010*

New Zealand is a seismically active country and the dynamic interaction between a rugby-field sized roof and the supporting stand structures is complex. Also Otago is subject to sub-zero temperatures with significant snow-falls. With a roof so vast, wide area wind effects also had to be considered. These loading considerations were challenging enough, but at the same time it was critical to minimise shadowing on the field by the supporting structure, both for health of the turf and the adverse effect of shadowing on television cameras.

This paper describes the roof structure developed; the unique approach to seismic energy dissipation intended to ensure that seismic generated forces in the roof structure are never the critical load case for over-field structural members; consideration of snow and wind loads; the effect of significant thermal expansion. The paper describes the approach taken to fabrication and erection adopted to achieve completion within budget and to a very tight time-frame

2. Stadium Description

2.1 Stadium Planning

The stadium was planned as a multi-purpose venue around a rectangular grass playing field. This allowed the forward seating to be close to the field with all seats having an intimacy with the play. There are four stand zones arranged rectangularly around the playing field, the North, South, East and West Stands. The East and West Stands are at the goal-post ends with North and South down the side-lines.

The South Stand (Figure 2) is the major stand which also incorporates corporate, hospitality and back-of-house activities. The stand is five storeys on a 10m longitudinal structural grid and a variable transverse grid. The stand is substantially concrete construction utilising precast systems for beams, rakers, bleachers and flooring but with in situ-poured columns.

The North Stand (Figure 3) is a single tier spectator stand with the only facilities provided being those to serve the patrons of that stand. The rakers, columns and bleachers are all precast concrete. Longitudinal stability is primarily provided by concentric steel bracing between the rear (northward) concrete columns

Figure 2: *Main Roof, Primary Truss and South Stand*

Figure 3: *Main roof structure and North Stand*

The East and West 'Stands' are flat floor areas at the goal-post ends of the fields, with roof over and semi-enclosing walls on the three sides away from the playing field. The enclosing super-structures are constructed in lean-to form in cross section but with the open side supported by their own transverse truss portals spanning across the width of the field (but behind the dead-ball line) (Figure 4).

Figure 4: *Roof structure with East Stand beyond*

2.2 Roof and Main Facades

The roof (Figure 5) has five zones, generally referred to as the Main Roof, Primary Truss, South Roof, East and West Enclosures. There are also major hanging facades at the east and west ends, each side of and over the East and West Enclosures. Cladding is a mix of clear Texlon-ETFE inflated pillows, polycarbonate panels and conventional long-run profiled steel roofing, depending on the relative importance of the zone to sunlight incidence. In total there was 20,569 m2 of ETFE in almost 300 separate double layered pillows.

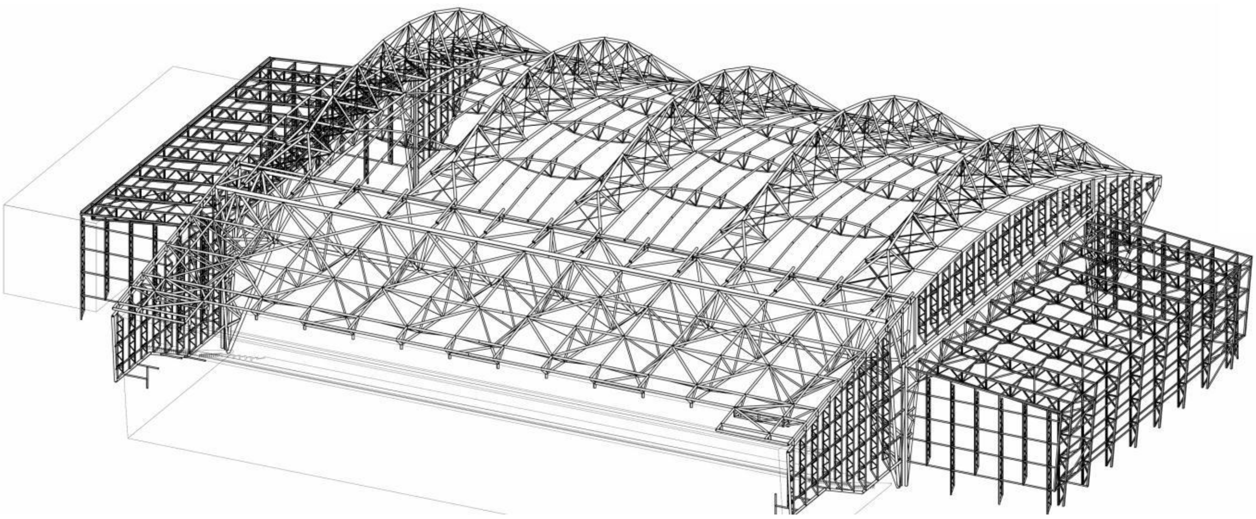

Figure 5: *Roof Steelwork viewed from South-East*

The Main Roof has five 10m-wide triangular trusses (Figure 6) that span 101m across the playing field and to the back of the North Stand. These are referred to as 'Arch Trusses' due to their shape but they work in truss action not arch action. The northern end of the arch trusses are on bearings fixed to the tops of the columns of the North Stand, free to slide N/S but constrained E/W (Figure 7). The southern supports of the arch trusses are by means of pin connections to the north top chord of the primary truss. Out-of-phase seismic response of the two supporting stands, some allowance for seismic ground spreading and significant thermal movement meant that connecting rigidly to both stands was undesirable.

Figure 6: *Arch truss being maneuvered across the site*

Figure 7: *Bearing at north end of an ETFE transom*

Spanning 20m between these five trusses there are planar 'Lens Trusses' at 20m intervals which in turn support steel tube transoms spanning north/south with three transoms between each arch truss plus one centrally placed between each set of arch truss lower chords. These jointly provide a 5m by 10m grid for support of the ETFE pillows. The roof clears the pitch centreline by 37m and externally reaches 47m above field level to its highest point.

The primary truss is a 10m-wide by 11m deep inverted triangular truss with clear span of 120m along the full length of the playing field with triangular truss end legs that enable portal action. The arch trusses are supported onto the northern upper chord and the south roof supported to the south upper chord. The lower chord is also braced back to the south roof for chord stability and torsional restraint.

The South Roof spans between the South Stand uppermost level and the primary truss and is formed from alternating 10m wide triangular trusses and planar trusses, such that there is a top chord on every 10m grid. Their southern supports are by way of bearings that can slide N/S but are constrained E/W. To achieve a rigid connection of N/S roof loads the southern roof trusses are connected to the stand structure via steel seismic energy dissipaters (Figure 8).

Figure 8: *Energy dissipater restraining South Roof*

Figure 9: *Two directional bearing between west hanging-facade and West Stand planar truss. Provides for vertical and longitudinal movement but lateral restraint.*

The East and West facades are largely hung from out-riggers on arch trusses 1 and 5 (Figure 5) but also have a gravity support to ground where they meet the East and West Enclosures. Lateral (wind) loads from the lower edge of these hung walls is gained through several horizontal props to the North and South Stands (where appropriate) but over the East and West Enclosures the facade and enclosures are interconnected for E/W lateral loads while releasing interconnection in all other directions via a complex box bearing that allows movement N/S, up/down and rotation in all directions but not E/W translation (Figure 9).

3. Roof Loads, Load Paths and Releases

3.1 Codes and Design Standards

Mostly the roof was designed to New Zealand Standards with particular reference to AS/NZ 1170 [1] Part 0 *General Principles*, Part 1 *Permanent, Imposed and Other Actions*, Part 3 *Snow and Ice Actions* and Part 5 *Earthquake Actions - New Zealand*. Part 2 *Wind Actions* was more or less followed but supplemented with specific wind tunnel tests to define more accurately the wind pressure level and distribution.

The primary material code was NZS3404 *Steel Structures Standard* [2].

3.2 Wind loads

While it was essential to ensure that any likely wind load combination was allowed for, it was equally critical that conservatism inherent in codes did not result in excessively large roof framing, both for budgetary and turf shadowing reasons. Therefore wind tunnel tests were conducted to provide ultimate (1000 year return period) wind loading for the stadium. A 1:400 model was created including all features greater than 0.2m and utilising 570 pressure taps. This was used with 30 separate wind load effects to establish a complete picture of wind load cases. The model was also used for a separate vibration and fatigue study for the roof structure.

Key structural outcomes from the wind testing and modelling were:

- Correlated wind load cases which removed conservatism and particularly provided verification for wind load distribution
- Additional loads due to the dynamic, resonant response of the structure under wind load were accounted for.
- Confirmation that structure is generally governed by load cases other than wind load e.g. snow.
- The refined wind tunnel modelling allowed some of the more minor, yet still significant, elements to have a refined design.
- Initially, it was estimated that the resonant response would increase the peak response by 25%. However wind tunnel results showed that the highest percentage increase in the peak response due to resonance is only 10%.

3.3 Snow loads

Although the roof is curved there are quite large regions that are near flat. Because of the low values of roof slope at various points the effect of accumulated snow was found to be a dominating gravity load case. The area is sub-alpine with a code defined ground snow load of 0.9 kPa. This was converted to a roof load with consideration of the roof height, slope, shape and re-entrant features. This led to snow loads of 0.63 kPa over the flatter regions of the roof dropping to 0.45 kPa over steeper sloping regions. The snow loads were considered for both fully distributed and alternate span situations to allow for uneven build-up due to eddy effects and/or snow sliding locally from the steeper roof slopes (Figure 10).

Figure 10: *Roof under light snow load*

3.4 Thermal

Thermal loads are imposed on the roof structure when the temperature of the elements varies from the temperature at which they were fabricated and constructed. For the purposes of analysis this benchmark was taken as 15^{o}C. Temperature data indicated extremes in this region to be +35.9^{o}C to -9.3^{o}C. The effects of solar gain and radiation were taken as 1000w/m^2 with a solar absorption factor of 0.5. These led to consideration of a temperature range of +45^{o}C and -25^{o}C from the 15^{o}C benchmark as load cases to be considered in combination with others.

In particular thermal expansion was provided for at the north stand support of the arch trusses (Figure 7) and also expansion differentials between the hanging facades and the east/west enclosure planar (front) trusses (Figure 9).

3.5 ETFE Pillow Loads

The ETFE pillows are inflated and exert a tensile load at their perimeter additional to the transmission of external loads (wind, snow). Other than at the roof perimeter, the tension effects of the ETFE pillow inflation on the support structure are equal and opposite, hence not imposing special loads on the support members. Careful consideration was given to the unbalanced load that would develop if a pillow were to fail for some reason or require replacement. This situation would result in the remaining adjacent pillows pulling laterally on the support member. This unbalanced load was taken into consideration in sizing the ETFE pillow support transoms.

3.6 Seismic Loads

New Zealand is a seismically active country but fortunately Dunedin is not located in the most severe seismic region. Special consideration was given to the interaction of the various structural parts, particularly between the roof and the supporting stand structures, but also to the structure-ground interaction. As the North and South Stands are independent of each other consideration had to be given to them moving out of phase.

4. Roof Seismic Analysis and Energy Dissipation

4.1 System Description

4.1.1 North-South

In the north-south direction the roof is restrained entirely by 14 ductile seismic dissipaters connected to the top of the South Stand (Figure 8). At the North Stand the vertical load of the roof is supported on the column tops but a sliding bearing frees the roof to move independently along the north-south axis (Figure 7). The roof members are designed and detailed for the over-strength of the dissipaters.

The steel tapered plate dissipaters are modelled on research by Tyler [4]. Usually this type of dissipater is included at the base of a base-isolated building or between a bridge deck and its buttresses. For the stadium, a novel approach was taken by including them between the stand and the roof. The logic of this location was to prevent or limit seismic dynamic response of the heavy concrete structure from causing high responses within the roof structure, thus causing an increase in critical roof member sizes

4.1.2 East-West

In the east-west direction the roof is laterally restrained by three primary systems. The south edge is restrained rigidly to the top level of the South Stand. The primary truss forms a portal frame which provides significant restraint on the line of the south edge of the sports field. The North Stand laterally restrains the north edge of the roof.

The portal frame formed by the primary truss is designed and detailed as a nominally ductile system except for the lower regions of the portal legs which have been detailed as ductile components.

4.2 Methods of Analysis

Three separate seismic analysis models were used: one for roof; one for the South Stand and one for the North Stand.

4.2.2 Roof

The roof was analysed in two stages to determine the design earthquake forces. In the first stage, a simplified lumped mass and stiffness model of the roof was analysed together with the South and North Stands by dynamic Time History methods. In the second stage a detailed model of the roof was analysed for equivalent static loads derived from the South Stand and North Stand models.

A key simplification of the roof structure was to analyse the horizontal dynamics of the north and south halves of the roof separately. This was possible because the main roof north has negligible diaphragm stiffness, thus the curved arch trusses transmit negligible shear between the South Stand to the North Stand. This means there is no dynamic coupling between the North and South Stands.

The North Stand model represented the tributary mass of the roof as a series of lumped masses which were active only in the east-west direction. This model provided the equivalent static base shear which was applied along the north edge of the detailed roof model. The peak east-west displacements for the roof were also derived from this model.

4.2.2 Earthquake Records Selected

No site specific seismicity study was carried out for this project. In the absence of such specialist advice, five earthquake records were selected from the PEER Strong Motion Database with a US Geological Survey soil category of C.

The selected strong motion records were:

Table 1*: Selected Earthquake Records used for Time History Analysis*

Record	***Earthquake***		***Magnitude M***	***Magnitude Ml***	***Magnitude Ms***
P0173	Imperial Valley	1979	6.5	6.6	6.9
P0267	Victoria, Mexico	1980		6.1	6.4
P0815	Landers	1992	7.3		7.4
P0920	Northridge	1994	6.7	6.6	6.7
P1125	Chi-Chi, Taiwan	1999	7.6	7.3	7.6

5. Roof Design

5.1 General

A general description of the roof has been given in section 2.2. The approach to seismic analysis of the roof was given in section 4. The roof steelwork design was carried out primarily in SKM Sydney office to take advantage of its extensive international large-span roof experience.

5.2 Load Combinations

To ensure that every loading situation was anticipated, a total of 27 primary load cases and 45 combinations were included in the analysis. Primary load cases included gravity, live, seismic, wind, snow, ETFE pillow tension, temperature and event loading and where relevant these were split by direction (N/S, E/W, up/down) and also full or skip loading.

5.3 Materials

The majority of the roof structure is tubular using standard hot rolled sections for available smaller pipe sizes and longitudinally welded tube for larger sizes. Steel generally was grade 300L15, selected specifically to be suited for ductile behaviour at sub-zero temperatures. However CHS sections are to BS EN 10219 grade S355JOH for wall thicknesses up to 40mm and BS EN 10025 grade S355J2H for wall thicknesses greater than 40mm. Rod bracing and associated fittings and plates are 400L15.

5.4 Member Sizes

The Primary Truss, as to be expected, demanded the heaviest steel sections with lower chords generally 711x32 CHS and top chords generally 711x40CHS. However at the lower chord node points and selected upper chord nodes short lengths (cans) of 711x60CHS were utilised to avoid the need for complex internal stiffeners at these critical intersections.

The arch trusses used a similar approach except that lower chord sizes ranged from 457x12.7CHS to 457x22CHS and top chords 508x25CHS to 508x32CHS. Again, heavier cans (457x32CHS) were used in selected lower chord node points.

5.5 Flanges

The various truss components were originally designed as all welded construction, as is common for major stadium roof structures. This, however, would have necessitated considerable site welding, placing demands on time, quality and painting requirements. The main contractor was appointed prior to completion of design and it made it very clear that to meet the tight time demands in a location such as Dunedin it was essential that all site joints were by means of fabricated bolted flanges. After much discussion it was recognised that there were a number of critical joints where flanges would not be sufficient and site welding was still required. These locations were primarily the primary truss lower chord joints. All other joints between fabrication assemblies were flanged (Figure 11). It is generally accepted that the project would not have been completed on time without this decision.

Figure 11: *Typical flange joint*

6. Fabrication and Transport

With over 3,700 tonnes of steelwork to be fabricated into 20,642 individual fabricated units on a very tight programme, the task was too large for any single New Zealand fabricator. To meet the challenge a joint arrangement was set up with NZ's largest fabricator, Grayson Engineering based in Auckland, taking the lead role and handling all the heavy engineering components, together with Christchurch based Pegasus Engineering handling lighter section fabrications. Even then Grayson's Auckland facility had to be extended specifically for this project. With the inclusion of bolted flanges for most joints all components were match fabricated with trial module assembly in the fabrication yard before being dispatched for painting and delivery. Altogether 8,578 shop drawings were required, developed on Strucad 3D software.

Figure 12: *Trial assembly of Primary Truss leg*

Once components were fabricated, trial fitted and painted they were dispatched to site by road transport, requiring several large articulated transporters in an almost continuous loop Auckland – Dunedin – Auckland, with the driver resting while the transporter was on the Cook Strait ferry. It is often queried why, since both Auckland and Dunedin have good harbours why sea transport wasn't utilised instead, potentially saving considerably on fuel costs. The answer to this lies in the relative logistics of the two alternative transport modes. As well as the double handling at each end that sea transport would have entailed this mode also would have required considerable stock-piling of components at the fabricator's yard and then extensive lay-down area on site to allow for large shipments arriving at once. Neither end of the supply chain could handle this storage and also it would have placed unmanageable demand on the fabricators to generate large shipments at a time. The road transport option, on the other hand, allowed shop-drawing production and review, fabrication, painting, delivery and assembly/erection to be carried out on an "as-required" basis, spreading the load on each step of the process. Thus the logistics of handling this quantity of steel over-rode the more fuel efficient approach of using sea transport.

7. Site Erection

The overall sequence of assembly and erection seems simple and logical, but nevertheless required substantial amounts of planning and coordination. The sequence in essence was: Primary Truss portal legs – Primary Truss main span - South Roof connections to South Stand - Arch Truss #5 (west end) followed sequentially by the other arch trusses and infilling between arch trusses as each pair was ready, west and east hanging facades and finally east and west enclosures. Of course each of these steps progressed in echelon with the preceding steps.

The legs of the Primary Truss were assembled at their respective ends of the site prior to erection. The Primary Truss main span was, logically, assembled more or less mid-field. Being a 350 tonne lift clearly manoeuvring the assembled truss needed to be minimised. However site logistics required each Arch Truss to be assembled at the east end of the site and then transported to the list location utilising two very large cranes (Figure 6).

To lift the Primary Truss main span required sections of the South Stand bowl seating bleachers and support beams to be delayed in order to enable the erection cranes to back into the space to minimise crane reach. This main lift commenced at about 5:00 am and was completed by nightfall. The splices to the already erected sections above the truss legs were to be site-welded which provided only 4mm clearance at each junction for lifting in the 350 tonne main span.

Figure 13: *Primary Truss assembly*

Figure 14: *Primary Truss lifting*

The Arch Trusses were fabricated and assembled precambered to match the predicted dead load deflections. In the case of trusses #1 and #5 (east and west ends) the trusses also supported the deadload of the hanging facades from outriggers on one side of each truss. Thus there was a torsional deadload deflection in addition to the normal vertical component, requiring these two trusses to be assembled with a twist. The twist created difficulties in that the infill lens trusses had to be erected prior to the deadload of the hanging facades being applied, but the lens trusses would not fit correctly while the twist existed. To overcome this steel cable pull-down guys were fitted to the truss outriggers down to deadmen anchors buried under the end enclosure slabs. These were tensioned up to equal the weight of the end facades and then progressively released as the mass of the facades was applied.

8. Conclusions

This challenging project has achieved a number of technical breakthroughs, principally in the methods of seismic energy dissipation and structural interaction. It is believed to be the first rugby stadium anywhere to have natural turf under a permanently closed transparent roof. It was constructed over a 25 month period and completed one month prior to the RWC2011 kick-off.

Figure 15: *Rugby World Cup 2011 - Ireland vs Italy*

9. Acknowledgements

A project of this scale and complexity invariably involves a large team of specialists. It is not possible to acknowledge every organisation but the following are the key participants in the roof structure for this stadium:

Client – Carisbrook Stadium Trust

End owner – Dunedin City Council

Project Managers – Arrow International Ltd

Architects – Populous / Jasmax Joint Venture

Structural Engineers – Sinclair Knight Merz

Quantity Surveyors – Rawlinsons Ltd

Main Contractor – Hawkins Construction Ltd

Steelwork Fabricator & Erector – Grayson Engineering Ltd (lead) & Pegasus Engineering Ltd

ETFE Cladding Contractor – Vector Foiltec

10. References

[1] New Zealand Standard (AS/NZS 1170), *Structural Design Actions;* Part 0 *General Principles*, Part 1 *Permanent, Imposed and Other Actions*, Part 2 *Wind Actions,* Part 3 *Snow and Ice Actions* and Part 5 *Earthquake Actions - New Zealand*, Standards New Zealand, Wellington

[2] New Zealand Standard (NZS 3404:1997), *Steel Structures Standard*, *including Amendment 2:2008.* , Standards New Zealand, Wellington

[3] Tyler, R.G.,1978 "Tapered Steel Energy Dissipaters for Earthquake Resistant Structures", *Bulletin of the New Zealand National Society for Earthquake Engineering,* 11 (4), 282-294,