THE REDEVELOPMENT OF THOMOND PARK, LIMERICK

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SYNOPSIS

Thomond Park, the historic home of Munster Rugby, has undergone a major redevelopment which has seen its capacity rise to 25,756. This paper describes the design and construction of the new stand and terrace structures. Particular emphasis is given to the development of the architectural and engineering concepts, the wind tunnel testing and dynamic analysis undertaken during detailed design, and the fabrication and erection of the two 150m span ‘rainbow’ roof trusses.
1. INTRODUCTION

The love affair between the public in Limerick and Rugby began very early in the history of the game. It has been played with passion and pride ever since the first club was founded in the city in 1876 and, uniquely in Ireland, has been as popular among working men (and now women) as among the professional classes. Undoubtedly the moment that epitomised all the passion and skill of Munster rugby was when the final whistle blew on 31st October 1978, when for the first and only time in rugby history a fully representative All Black side failed to score and were defeated 12-0.

Prior to the current re-development the official capacity of Thomond Park was 11,091, with 1,242 seated in the West Stand and a further 9,849 standing on terraces on all 4 sides of the ground (Figure 1). This was augmented by up to 2,000 temporary seats which were installed for major matches. While a major stadium upgrade was hoped for at various times over the years, undoubtedly the Heineken Cup victory by Munster in 2005/6 was the final impetus that released the energy and confidence to progress the project. The 100% record of Munster over an 11 year period of European Cup Competition at Thomond Park lent a very special edge to the experience of watching Munster compete at the venue. This proud record finally fell with a 13-6 defeat to Leicester in the very last game before the ground was closed for redevelopment.

Project Team

Michael Punch & Partners (MPP) have been involved with various developments at Thomond Park through Joe Murphy and Robert Tackaberry in particular over the past 30 years. Joe led the team that designed the West Stand back in 1989 (see front cover of ‘The Structural Engineer’, 5th December 1989), the substantial upgrade to North, South and East Terraces for the 1999 Rugby World Cup, as well as numerous other improvements. This team was also involved in the preliminary option studies carried out during 2005 for various configurations of new stands within Thomond Park, some of which involved reorientation of the main pitch.

The Design Team for the recent redevelopment works was appointed in February 2006 following an interview process led by Client Representative and former Irish international rugby player Pat Whelan. The team included Steve Cunningham & Associates as Project Manager, architects AFL of Manchester and Murray O’Laoire (MOLA) of Limerick, Michael Punch & Partners as Civil and Structural Engineer, Don O’Malley & Partners Mechanical and Electrical Engineer, Bruce Shaw Partnership (BSP) as Quantity Surveyor, Michael Slattery & Associates as Fire Safety Engineer, Joe Burke Consulting Engineers as Project Supervisor Design Process and Arup as traffic consultant.

Following a short listing of tenders and interview process, PJ Hegarty & Sons were announced as preferred bidder in February 2007 and later appointed as Main Contractor in a Design Build Contract. AFL and MOLA as the Architects and MPP as the Civil and Structural Engineer were then novated to PJ Hegarty & Sons. Banagher Concrete, as sub-contractors to PJ Hegarty & Sons, were responsible for the detail design and construction of the precast concrete frame elements. The steelwork sub-contractors were Foynes Engineering who manufactured the primary trusses. The secondary trusses were sub-contracted to O’Dwyer Steel, Dundrum.

Brief

Prior to appointment of the design team for the new stadium Arup were commissioned to undertake a site selection study including site options other than Thomond Park. Following this study Thomond Park was confirmed as the preferred site.

Early form studies showed how the required capacity could be provided by building two single tier stands, one on each of the long sides of the existing pitch. On this basis a challenging brief was laid down by the Munster Branch of the Irish Rugby Football Union:

- Accommodate 26,000 spectators or about double the existing capacity;
- Accommodate UL Bohemians Rugby Club within the new facility;
- Introduce a large amount of support facilities in line with modern standards;
- Maintain the existing pitch and keep it off limits during the construction period;
- Commence construction work after the pool stages of the 2006/07 Heineken Cup;
- Make the ground available for the 3 pool stage matches of the 2007/08 Heineken Cup with a capacity not less than the existing;
• Complete the ground in time for the 2008/09 Heineken Cup;
• Maintain the existing practice pitch (this would be made available during the construction period) and the existing Shannon Rugby Club Pavilion.

The retention of the existing pitch and existing terraces along 3 sides of the ground ensured that the famed spirit and atmosphere of the ground would be maintained. As sightlines on the existing West Terrace were sub-standard however, the opportunity was taken to rebuild this section to modern standards.

 Provision of a main stand along the east side of the ground was conditional on the acquisition and demolition of a number of houses along the Knockalisheen Road (Figure 1). As the houses were outside the original stadium curtilage, the change of land use from residential to stadium use required rezoning. This process occurred in parallel with the design phase and after the required notice period and a meeting of the elected public representatives in January 2007 both the rezoning and notification of a decision to grant planning for demolition of the houses were approved.

3. CONCEPT DESIGN

Geometry

Two generic forms for the main stands emerged from the early form studies. The first was a simple rectangular plan form of constant cross section while the other was a segment of a disc with the curved edge on plan at the rear (Figure 2).

A rectangular plan gives rise to sub-standard viewing positions in the upper corners. In the case of Thomond Park the gable end elevations for the rectangular stands would have been very imposing at the Cratloe Road end of the ground where they overlooked the public road and housing beyond, and at the north end where they directly overlooked housing. A rectangular form would also have given rise to stands of different depth, i.e. a smaller West Stand would be required to prevent encroachment on the training pitch leading to a bigger stand and gable on east side. It was felt that this geometry presented a serious risk to the project in planning terms (Figure 3).

A curved plan however locates all spectators within the optimum viewing circle, i.e. nobody is more than 90m from centre point or 150m from any point (Figure 2). This leads to a curved stand on plan, which in turn leads to a curved stand on elevation. This curved option naturally gives rise to a variable roof profile which drops in height towards the ends. The end elevations for this curved option presented a much more modest appearance to those outside the ground (Figure 3).

![Figure 1 Aerial view of site illustrating land available for redevelopment of the stadium (MOLA)](image-url)
Viewing requirements for terraced stands lead to a gradually increasing riser height with distance from the pitch. The steel formwork reflected this with 6 different riser height groups being used.

Comparisons of cost did suggest that the rectangular option was marginally cheaper but taking account of the planning, functional and aesthetic impact the curved option was chosen (Figure 4).

**Roof Design**

In addition to the two stand plan form options, a number of roof structure options were also evaluated. Having discounted membrane solutions on cost grounds, principally two generic roof types emerged: simple cantilever and ‘goalpost’ or arch solutions, both in steel.
The rectangular form lent itself more to the cantilever option with good repeatability on every grid and a shorter overall cantilever length. However the huge moments transferred into the stand superstructure at the root of the cantilever generated significant knock on costs within the superstructure frame (Figure 5).

In contrast the curved plan option also generates a gently curved geometry in section longitudinally and gives much greater scope for modelling the form of the 150m long primary trusses. The simple cantilever roof on the other hand was shown to be practically unworkable for the curved option due to the large cantilever spans midway along the stand.

Beam, arch and portalised options were all considered. An arch, at approximately 100 tonnes of steel, was clearly the most efficient design, however the springing points could not be brought down to ground level due to the restricted space available. With the arch springing points at high level the large horizontal reactions gave rise to massive abutments.

The concrete abutment solution was adopted at the Sydney Olympic Stadium (Figure 6). The corresponding abutment is sketched for Thomond Park and compared to scale with a typical building for the Georgian Quarter in Limerick (Figure 7).

The ‘rainbow’ truss profile was therefore developed in very close conjunction with the architects, and reflects both structural and aesthetic judgements (Figure 8a).
Figure 5  a) View of Croke Park from rear;  b) View of old East Stand, Lansdowne Road

Figure 6  Sydney Olympic Stadium

Figure 7  a) Typical building from Georgian Quarter, Limerick;  b) Sketch of concrete abutment required at Thomond Park for arched truss;  c) Abutment at Sydney Olympic Stadium
Given the proposed stand geometry and compact nature of the ground, a portal frame solution allows the truss to terminate quite high off the ground. The large reduction in horizontal reactions corresponds to a similar reduction in the size and visual impact of the support pedestals (Figure 8b). While requiring approximately 140 tonnes of steel, this portal option proved to be the most efficient due to the trade off between truss weight and end pedestal cost.

Main Frame

The use of concrete and steel for the main superstructure frame was examined during design development. Early studies by the Quantity Surveyor in consultation with fabricators and pre-casters suggested that steel was cheaper and the tender design was progressed on this basis.

Contractor Alternatives

During the tender process PJ Hegarty & Sons offered a pre-cast concrete proposal for the main frame. The precast frame alternative was modelled on the steel tender design. This concrete option was embraced enthusiastically by the team and was accepted by the Client. A typical section through the frame is illustrated in (Figure 9).

PJ Hegarty & Sons also offered an alternative rear support option for the raker beam with some cost savings. The proposal was to bring the rear support columns ‘back on grid’ (Figure 10a), therefore avoiding the use of the diagonal ‘branch’ supports (Figure 10b). In order to fully evaluate the visual impact of these alternatives MPP produced 3D renderings of these options using SketchUp (Figure 10a); however these options were not pursued.

The importance of the branched articulation for the exterior elevations facing the city can be clearly gauged from Figure 11 and Figure 25.
Figure 9  Typical cross section through East Stand

Figure 10  3D renderings of rear stand revelations a) un-branched option;  b) branched solution

Figure 11  a) 3D rendering of un-branched option;  b) Completed view of branched solution
Architecture – Impact on Structure

Two key architectural initiatives clearly have a major impact on the finished building form:

- The curved plan which generated a curved elevation and drove the roof structure in the direction of the long span arch-truss-portal family of solutions;
- Off setting the rear line of columns by half a grid. This opened up the load path, generating a fan-like pattern of diagonal ‘branches’ at the rear of both stands.

The evaluation of these initiatives on purely cost or structural criteria does not do justice to the end result as anyone who has seen the nearly complete structure can testify. While keeping to a prudent budget was always a concern to the project team, and that has been achieved, there is little doubt that, within the budget constraints, the option chosen by Client Representative Pat Whelan is of the highest design quality.

4. DETAILED DESIGN

Stability

A typical section through the East and West Stand is illustrated in Figure 9. Stability to the building is provided by 6 reinforced concrete shear walls in each stand. Wind load on the exposed seating/raker units is transferred to the shear walls via the floor diaphragms of the concourse areas.

Foundations

Foundations to the West Stand generally consist of reinforced concrete pads on weathered limestone. Foundations to the East Stand are located on firm clay, with the exception of two shear walls in the basement area which are located on rock.

The need for additional mass in the bases of the shear walls to resist overturning meant that those founded on rock were not significantly smaller than those founded on clay. Rock anchors were considered as an alternative to the large volumes of concrete, though their use was discounted after discussions with the contractor.

The tendered design for the primary truss pedestal bases utilised rock anchors to resist the large overturning moments generated. Post-awarding of the contract, the main contractor requested that the possibility of designing out the rock anchors by upsizing the bases be investigated. This resulted in large eccentric rectangular bases. The considerable horizontal component of the force from the primary truss is resisted by shear keys. The open excavations of those to be founded on rock were examined individually to ensure that the exposed face was of a quality which would restrain the base if poured directly against it.

Wind Tunnel Testing

Given the scale of the roof structure whose design would be dominated by wind load, wind tunnel testing was considered by MPP early on in an effort to reduce design loads and hence structure costs. Reducing the design wind load was seen to have a potential beneficial effect on the primary and secondary truss design, purlin size and spacing, and the foundation design of the shear walls. After investigating the matter in detail and based on projected wind load reductions relative to code values the proposal was judged cost effective and three tenders were sought. BRE were the successful tenderer.

A 1:250 model was constructed and tested with all buildings and major topographical features within a radius of approximately 220m represented (Figure 12). Only one grandstand roof was instrumented. The location of the instrumented stand was then switched to derive wind pressures for both the East and West Stands. In total 108 pressure taps were installed. Pressure differences were recorded for a simulated 1 in 50 year return period storm event, with correct atmospheric boundary layer structure and turbulent airflow.
Results of the wind tunnel test became available during the tender process, with significant benefits when compared to values recommended by the current code of practice\(^1\). The greatest benefit applies to the primary truss as the load is averaged over a larger area, hence the dramatic reduction in primary truss load.

In view of the reduced wind load, the non-wind imposed load became design critical for the primary truss. MPP therefore investigated the possibility of reducing the default minimum imposed load value of 0.6kPa for the primary truss design. This was initially thought appropriate due to the low local basic snow load, the absence of drift considerations and the fact that maintenance load will only apply very locally. Having consulted widely, including with a member of the original BS 6399 code drafting committee, a sound basis for reducing the default minimum could not be established and hence this value was used in the design.

**Crowd Induced Dynamics**

Relatively modern grandstand structures in the UK have been known to respond dynamically to rhythmic crowd movement, particularly when stimulated by music at events such as pop concerts. This dynamic response does not usually affect the structural performance of the grandstand, however in severe conditions the discomfort felt by the spectators can and has led to panic. It was thought prudent to therefore ensure that the stand structures are capable of hosting such concert events.

A full 3D frame model of the East Stand was constructed using the STRAP software package (Figure 13). The first natural frequency of the statically designed structure was determined to be 7.4Hz in the vertical direction. This was above the recommended minimum levels of 3.5Hz for sports usage and 6Hz for concert usage\(^2\).

Table 1 Comparison of wind load values derived from wind tunnel test and BS 6399

<table>
<thead>
<tr>
<th></th>
<th>Code Value (Uplift) (kN/m(^2))</th>
<th>Wind Tunnel Test Value (Uplift) (kN/m(^2))</th>
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<tbody>
<tr>
<td>Primary Truss (overall)</td>
<td>1.51</td>
<td>0.70</td>
</tr>
<tr>
<td>Secondary Truss (overall)</td>
<td>1.67</td>
<td>1.00</td>
</tr>
<tr>
<td>Local Zones</td>
<td>A  1.90  B  3.50  C  3.80</td>
<td>A  1.35  B  1.60  C  2.40</td>
</tr>
<tr>
<td>(refer to BS 6399 for details)</td>
<td></td>
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A sensitivity study showed that modelling one-half of the structure with allowance for the roof mass only was acceptable, hence a reduced computer model of the West Stand was constructed.

The first natural frequency of the statically designed West Stand was determined to be 4.3Hz which, while more than adequate for sports usage, fell below the recommended limit for concerts. This reduced natural frequency is due to the larger span of the raker beam between the concrete column and steel ‘branch’ support (see Figure 9 and Figure 14). The lateral flexibility of the taller columns is a noticeable feature of the fundamental mode shape.

Significant stiffening of both the rakers and rear columns was required to achieve a natural frequency of 6Hz, i.e. a 30% increase in raker depth and 50% increase in column depth.

Investigation in this area suggested that the limits recommended by the interim guidance aimed to ensure that the structure would be outside the excitable range of the first two harmonics of crowd jumping for concert use. A more detailed ‘Performance Based Assessment’ of the grandstand was therefore carried out in an effort to quantify the dynamic response that would arise in a concert scenario. Recommendations on modal analysis and dynamic load factors for crowd jumping were based on the very helpful paper by Wilford.

Figure 13  3D computer model of East Stand

Figure 14  Typical mode shape of West Stand
The range of accelerations which are likely to result in spectator panic is suggested as 35-70% of gravity by Kasperski. Wilford suggests that vertical accelerations of raker/seating units in stadium structures should be limited to 30% g. The only code of practice where limits for accelerations in stadium structures are specified is The Canadian National Building Code. The values given appear to be based mainly on the work of Kasperski, and accelerations in the range of 10-18% g are said to be acceptable. This range was therefore adopted for Thomond Park.

Damping values used in the analysis were taken to be in the range of 2-3% of critical. Accelerations results are quoted so as to reflect this range.

For the statically designed structure, peak accelerations of between 13-19% g were determined to occur during concert conditions in a small area of the stand (equal to approx 5% of stand’s spectators). The critical load case occurred however when a crowd was concentrated in the rear section of the stand only – on the ‘positive side’ of the raker mode shape (Figure 14) – resulting in accelerations of between 21-31% g (Option 1 - Figure 15).

A number of stiffening options were investigated to provide satisfactory dynamic performance. The adopted solution caused minimal impact and involved increasing the depth of only 5 of the central rear columns from 850 to 1050mm (Option 2 - Figure 15).

The natural frequency of the structure increased to 4.6Hz and ensured accelerations in the acceptable range of 10-16% g. This acceleration range is conditional on the crowd being ‘managed’, i.e. ensuring that the crowd is located not only to the central rear area of the stand. This crowd management issue was discussed with and accepted by the Client.

![Figure 15](Image)

Figure 15  Graph of maximum raker accelerations for various structural options
Table 2  Details of 3D digital models required for design and construction

<table>
<thead>
<tr>
<th>Company</th>
<th>Software</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Michael Punch &amp; Partners</td>
<td>SketchUp</td>
<td>Visualisation</td>
</tr>
<tr>
<td></td>
<td>CADS 3D</td>
<td>Static structural analysis and design</td>
</tr>
<tr>
<td></td>
<td>STRAP</td>
<td>Dynamic structural analysis</td>
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<tr>
<td></td>
<td>AutoCAD</td>
<td>Geometric model as input to STRAP</td>
</tr>
<tr>
<td>Banagher Concrete</td>
<td>SketchUp</td>
<td>Construction sequence model</td>
</tr>
<tr>
<td></td>
<td>PROKON</td>
<td>Precast concrete analysis and detail design</td>
</tr>
<tr>
<td>Foynes Engineering</td>
<td>StruCad</td>
<td>Structural steel fabrication</td>
</tr>
<tr>
<td>O’Callaghan Visualisation</td>
<td>3D Studio Max</td>
<td>Architectural context visualisation for planning</td>
</tr>
</tbody>
</table>

3-D Modelling

A total of 8 3-D digital and 1 physical models of the proposed stands were constructed for various purposes as detailed in Table 2. The notion of a single integrated 3D model for all stages of planning, design and construction while attractive in principle does not reflect the reality of the very disparate requirements of various parties in the supply chain.

5. CONSTRUCTION

Construction began with the demolition of the West Stand and Terracing. The West Terracing was immediately reconstructed in precast concrete, thus providing the required capacity for the first three pool games. In the knowledge that the West Stand would not be fully constructed in time for the first match, the West Terrace was designed as a completely independent structure.

Concrete Frame

The pin-jointed concrete frame consists of precast concrete beams and columns designed as simply supported elements. The frame uses C40/50 self compacting concrete to achieve high strength and quality with a consistent finish. Beams are supported by corbels, cast integrally with the column sections and tied using rebar dowels which are grouted into position after erection and levelling.

The columns contain dowel tubes at the base to receive starter bars cast into the pad foundations (Figure 16a), a similar arrangement to that used to connect the head of the column to the raker beams overhead (Figure 16b). The floor plate consists of 100mm precast prestressed Flood Flooring widelslab planks with a 125mm C30/37 structural screed.

The vomitories which provide access to the terrace, their associated flank walls, stair flights & landings, and indeed smaller items such as weathering upstands were all precast.

The Client requirement to maintain the pitch in playing condition meant that all craneage for the frame had to be undertaken from the rear of the stands. In the case of the East Stand the crane was located on the basement roof. This rail-mounted crane could move up to 25m in either direction from a central point and required significant back-propping to the slab to ensure flexibility of lifting positions to the Contractor.

The erection sequence for the primary truss was contingent on support from two temporary steel towers constructed at the third points of the stand. For this reason the construction of the central portion of each stand was prioritised above the outer sections with the first precast columns on the east side being erected in the last
week of July 2007. At this point the shear walls and lift cores had been jump-formed to their full height and could be used for bracing of the precast elements in their temporary state (Figure 17).

Figure 16 Images from Banagher works: a) cast in dowel tubes for starter bars; b) steel form, reinforcing and dowel tubes in place prior to pouring of raker beam

Figure 17 Erection of first precast concrete columns

Figure 18 Munster 36 - Clermont Auvergne 13, Thomond Park 18\textsuperscript{th} November 2007
Fabrication and erection of the 150m span primary steel trusses was one of the many challenges posed to the construction team. Prior to tendering the job, MPP engaged with a number of steelwork fabrication companies to establish their views and preferences for fabricating and erecting such a large piece of structure. The overriding view expressed was to perform all welding in the fabrication shop and for the assembly to be bolted together at ground level on site into 3 segments which would finally be craned onto 2 temporary towers at the third point locations.

The final scheme proposed by the successful fabricators, Foynes Engineering, decided to make one significant change to the tendered proposal. All of the truss lacers to the main chords were directly welded on site thus eliminating bolted connections and the requirement to weld on stub sections to the primary chords in the fabrication shop. Overall the level of welding was the same but the elimination of the bolted connections significantly reduced the cost of plated fabrication work and resulted in a much more pleasing product aesthetically. However the significant amount of site welding proposed did require a significant site inspection and testing regime.

Due to the varying geometry of all of the truss nodes and the very complex end cuts on the tubes, the fabricators chose to get all of the pieces cut by laser profile and curved in the UK. It should be noted that overlapped joint detailing was used in accordance with the CORUS ‘Design of SHS Welded Joints’ guide which added to the geometry complexity of the end cuts (Figure 20a). Pre-fabricated 24m segments of the bottom chord ladder arrived on site shot blasted and primed, ready for site-welding with other pieces being delivered individually.

To avoid any potential erection pitfalls, a detailed truss erection document was compiled following numerous meetings and discussions between all parties. The agreed construction sequence first involved lifting the centre third of the truss into place and supporting it on temporary towers (Figure 21). The end segments were then lifted into position and the main chords welded together at high level. All temporary configurations were modelled and checked. The main chord welds were tested in-situ by x-ray (bottom chord) and ultrasound (top chord).
Figure 20  a) View of typical end cut detail at end of main truss diagonal;  b) Rear column to raker connection

Figure 21  First segment of the East Truss in position

A full time lapse sequence of the erection can be seen at www.standupandbuild.com.

Lateral stability in the temporary case was provided by the temporary towers, four secondary trusses and tension cables located towards the ends of the truss. All the temporary conditions were modelled and analysed as illustrated in Figure 22.

In the final case, the lateral stability of the primary truss is provided by the secondary trusses which convey horizontal forces through a cantilevered steel column at the tip of the rakers to the main frame. The rear column to precast connection is therefore critical (Figure 20b). Achieving moment fixity and allowing for dimensional tolerance at the interface between two sub-contractors required a double base plate detail with a site weld.
6. INSPIRING FUTURE CIVIL AND STRUCTURAL ENGINEERS

Thomond Park with such a strong visual presence on the Limerick skyline provides a great example of creative and innovative engineering design to the wider public (Figure 25). MPP clearly identified the project as means for reaching young would-be engineers and raising the profile of the profession as a whole.

As part of a nationwide Engineers Ireland campaign MPP became involved in raising awareness among children of primary school age to the possibilities of a career in engineering. With the newly erected trusses in the background over one hundred children between the ages of 10 and 12 gathered in the nearby Limerick Institute of Technology and set about constructing their own interpretation of what a modern stadium could be as part of the MPP K'NEX Challenge.

The event was attended by a number of the Munster players and, as illustrated in Figure 24, the level of enthusiasm generated by the re-development project was tangible in the extent and variety of the models produced.
7. REFERENCES