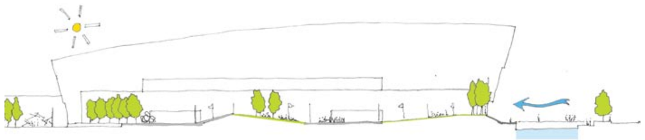

Figure 2-41. Section through Plaza showing cambered surfaces

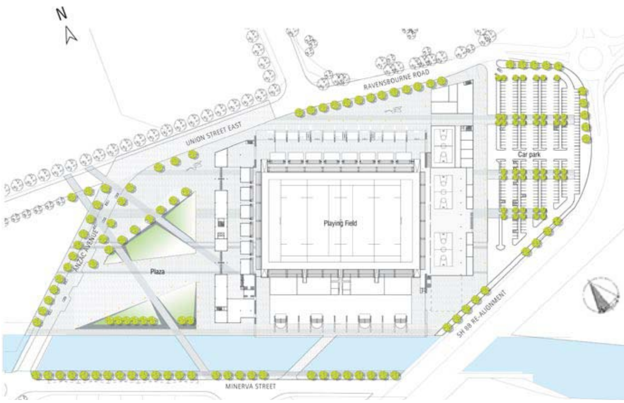

Figure 2-42. Landscape Plan (see Appendix for larger plan)

Figure 2-43. Trees used to define space – street edges, public spaces, thoroughfares

Landscape

The new stadium development precinct will be landscaped as an integral part of the development, enhancing the precinct and amenity for Dunedin City and the University of Otago. The focus of landscaping can address connections beyond the limitations of the site boundary with the University and Stadium acting as a promoter for intensification and redevelopment of Dunedin City.

The proposed landscaping plan addresses the following points of note:

a) Linking the City and the New Stadium

The design of the landscaping aims to link the City to Stadium by use of a 'Civic Mat' stretching out from the Plaza over Anzac Avenue, integration of the Leith River by use of link bridges and pontoon connections with the harbour side, and enhanced pedestrian connections and street trees.

b) Identify the relationship between the University and the New Stadium

The mixture of University and Stadium activities creates the need for a plaza in front of the main buildings that can accommodate the different needs of both user groups.

The plaza needs to be large enough to accommodate large crowds but it also needs to be used as an amenity space by the University.

The open green space in the plaza is proposed as split in two, allowing for a main entry route in from Anzac Avenue that can cater for large crowds, and is shown cambering towards itself creating a small amphitheatre for casual use by University users or smaller waiting crowds.

c) Consider the environment

The design of the plaza considers proximity to southerly winds and adjacent vehicular corridors. For this reason a ramped surface or 'wedge' is provided for localised shelter within the plaza. A shelter belt of trees to the South side of the plaza adds to this provision.

d) Address scale, the 'human factor'

The landscape is used as a means of breaking the scale from pedestrian to building. Objects are also places in which people can engage and react with. The predominance of the motor car is significantly reduced by the re-alignment of SH 88 and by the placement of the main car park away from the Plaza. Refer Figure 2-40 & 2-41.

e) Elements to define the space and guide its users

Trees are used to (refer Figure 2-42):

a. Define spaces within the plaza and providing shelter from predominant South-westerly winds.

b. Lessen the impact of a large facility with its surrounding environment

c. Tie the site wide landscaping into the existing street network.

d. Minimise the visual effect of a large area of surface car parking

Paved areas are provided at main entry points and will guide crowds towards them.

During non event times these spaces will provide a great urban activity amenity for the community and University.

The use of street and feature lighting is envisaged to ensure safety of users, as means of orientation, and to assist in the creation of a grand civic space (refer Figure 2-43 & 2-44).

The plaza is perceived as a big sculpture with inclined, grassed plains that form smaller scale human spaces. These spaces provide relaxation spaces with outlook to the Leith River and Logan park as well as back towards the rest of the University and City (refer Figure 2-45 & 2-46).

As part of the landscaping strategy it is envisaged that local artists be involved in the design of the Plaza and any art works to be placed in it (refer Figure 2-47).

Figure 2-44

Figure 2-45

Figure 2-46

Figure 2-47

Figure 2-48

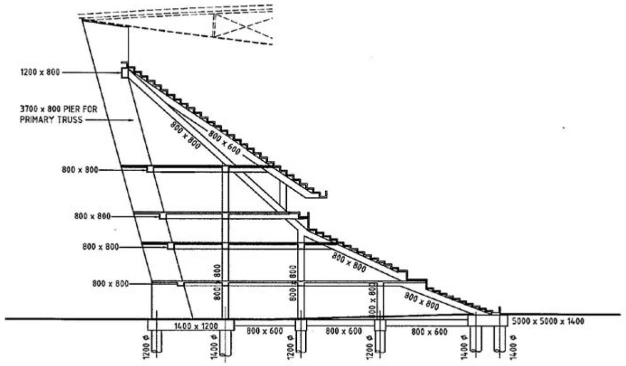

Figure 2-49. South Stand

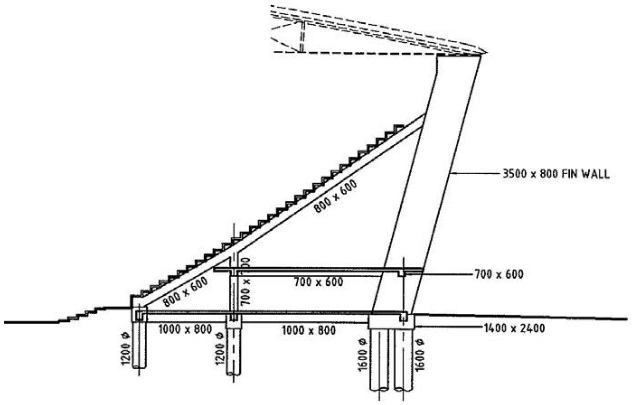

Figure 2-50. North Stand

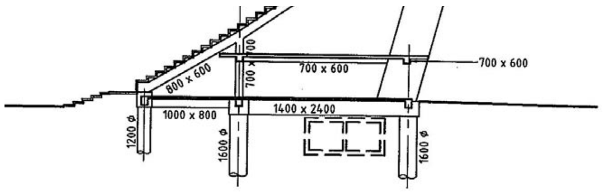

Figure 2-51. North Stand

2.3.10 Structure

The selected structural form for the North and South stands is strongly influenced by the very large anchorage loads generated by the roof structure options and by the poor geotechnical conditions of the site. The focus of attention has been for the "fixed roof" option with structures to support other roof options being a variation on this. The East and West stands are also required for anchorage of the roof structure but the loads are substantially less and thus the structural form is matched to the function and building geometry.

South Stand (Figure 2-49)
The very large roof cable loads are anchored at high level to large concrete "fin" walls on each grid. Site constraints prohibit consideration of any tie-back arrangement and hence the roof loads are carried in bending and shear in these walls. A raking strut has been introduced into the structure from as high on the fin walls as possible extending to ground level as far forward as possible. The triangle formed by the fin wall, raking strut and ground beam provides the dominant structural system and is intended to minimise bending action in the fin walls and net uplift on the foundation piles below the fin walls. However very large downwards foundation reactions result at the toe.

For earthquake considerations the dominant roof cable load consistently pulling in one direction means that a ductile response would be inappropriate and the raking strut also means that ductile structural behaviour would be hard to achieve. Therefore the structure has been analysed for elastic response to earthquakes. Longitudinal resistance is provided by frame action rather than by shear walls in order to maximise internal planning freedom.

Foundations will be bored concrete piles taken down to bear on the (assumed) boulder layer (see section 2.2.3). Skin friction has been ignored for downwards capacity. For uplift resistance belled piles were considered impractical (or at least very difficult and hence expensive) and at this time ground anchors have also been discounted (but could be further investigated). Thus uplift resistance is generated by the mass of the pile and skin friction based on a 10 – 15 degree cone of soil.

Generally insitu poured concrete has been assumed, though for many of the horizontal beams in either direction, precast concrete is an option. The fin walls are at a size that is impractical to precast and must be insitu-poured, possibly by slip forming. Floors will be either TT units or hollow-core depending on availability and cost at the time of design. The tiered seating bleachers will be precast and designed for satisfactory vibrational stiffness, above 8 Hz.

North Stand (Figure 2-50 & 2-51)
Although this stand is lower and simpler than the South stand most of the same considerations still apply. Its form stems from the dominant roof anchorage loads and consists of fin walls on each grid frame, propped at high level by the seating raker beam taking loads direct to ground at the toe.

Because the seating raker takes a high level of axial thrust (compression) concrete is considered to be more suitable than steel.

Because most of the rest of the construction is concrete, it has been assumed that the suspended floor will also be concrete utilising hollow-core floor units and precast beams. However steel deck on steel beams is also a viable alternative.

Several grids are intersected by the existing 6m x 3m stormwater culvert at shallow depth. For these it is necessary to relocate the affected piles and to transfer loads via beefed up ground beams.

East Stand (Figure 2-52)
In principal this is a large open lean-to shed supported on its open (field) side by a steel truss spanning the full 90m width of the building (hence field). However, in addition to its own wind loads this structure must receive the lateral wind loads from the closing wall below the main roof and also the wind friction forces of the wind blowing longitudinally E/W along the main roof. Consequently the member sizes are heavier than might normally be expected for the stand structure alone.

Steel is the obvious material for this structure and to limit the member sizes the roof diaphragm is to be cross braced throughout taking lateral loads to vertical braces or shear walls in the end walls. These will need to be coordinated with required wall openings.

Due to the lighter weight of this structure, compared to others, it is proposed to found the rear wall on a strip footing in the form of an inverted Tee, rather than piles. This decision will require review when better geotechnical information becomes available. Support to the main truss will be via the re-entrant major walls and will need to be pile supported.

West Stand (Figure 2-53)
The West Stand structure is incorporated as part of the University Building with the ground floor being primarily the stand access concourse, the raking tiers for the stand and the various suspended levels for academic use. The floor level with the top of the raking bleachers may also be used for stadium concessions at times.

The columns adjacent to the top of the raking tiers do not pass to foundation level, being terminated above ground floor level in order to give a more open concourse. Loads are taken to adjacent columns by storey-high braces.

Floors are 200 hollow-core and Unispan spanning East/West. The frame has currently been assessed as a steel structure, though concrete is equally viable.

The bottom of the raking member will be released to slide during earthquake events and as such a ductility value of 3 has been assumed. It may also be possible to incorporate an energy dissipating device at this release, though the small size of the building may make this an uneconomic option. It has not been allowed for in the scheme to date.

Re-entrant Walls
These walls are shown architecturally to be L shaped in plan, but in structural terms must incorporate a vertical seismic break at or near each corner in order to avoid seismically inter-connecting the various stands at their corners. Thus each wall "leaf" is supported by its adjacent stand.

Generally the walls form the end frame of each stand and/or University building and thus will match the relevant part of the stand structure. However large portions of some of the walls extend a considerable height and distance from the nearest support building and over these areas the walls are framed with precast concrete blade columns and walers. A steel space-frame may also be viable.

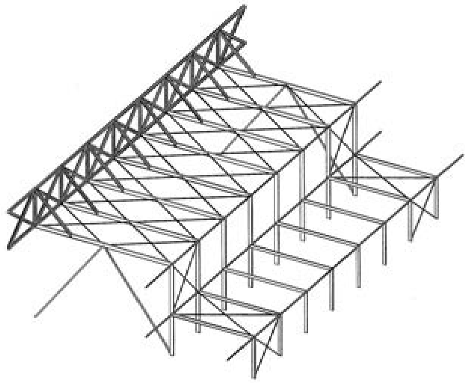

Figure 2-52. East Stand Structure

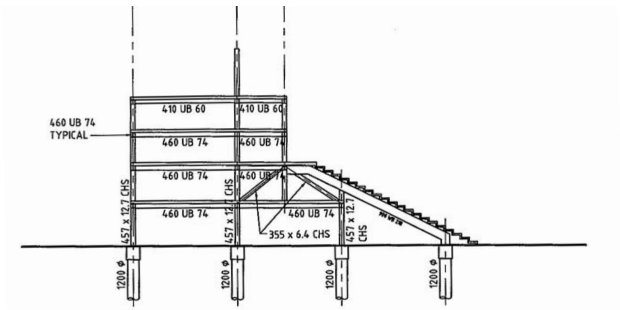

Figure 2-53. West Stand Structural Section

At the immediate corner the walls are required to support the considerable end reactions of the roof primary trusses. To achieve this 3m in each direction (i.e. as an L in plan) will need to be a solid concrete pier. The seismic break would be positioned beyond this zone, in the East and West direction.

The walls will need to be piled with a concentration of large piles under the truss support piers.

2.3.11 Roof

Roof Scheme A: No Roof over the Pitch

The Architectural scheme makes use of cables spanning 175m North-South to support the stand roofs. The cables would be provided in pairs and positioned at 5m centres. These cables span over the field of play and are anchored by the concrete fin walls positioned at the rear of the North and South Stands at 10m centres. The loads from the alternate cables between fin walls will be transferred by a local truss arrangement. This relationship is shown in Figure 2-54.

The cables are draped between the stands and the North and South Stand roofs are supported from the cables. Structurally this is called a catenary cable system and the cable force is related to the depth of the cable sag, the deeper the sag the smaller the cable.

When a roof is supported from such a system care is taken to deal with wind uplift on the roof as the system can be unstable when upwards loads are applied. To ensure the system does not go into uplift mass is used to weigh the cables down. In this design the concrete roof slabs are utilised for mass rather than a typical lightweight roof.

Figure 1 shows the geometry for the initial set up and the indicative deflected shape once the dead load is applied. The inwards facing arrows represent the tension load pulling the North and South stands towards one another. To achieve the shallow drape the reaction force transferred to the concrete fin walls is too high. To make the scheme viable a deeper drape will be required to reduce the force at each end of the cable to a manageable force. To make the solution feasible smaller cable tensions are required by reducing roof self weight based on a detailed wind analysis and a revised geometry with a deeper sag.

Figure 2-54 shows a preliminary detail for the cables, the relationship with the roof and the reinforced concrete roof slabs.

There is no real benefit of linking in the West and East roofs to this system and it is proposed these smaller roofs are an independent cantilevering design. If the West and East roofs were to be linked to the edge catenaries running North – South this would increase the horizontal load in the end catenaries and create the need for more weight to resist wind uplift forces.

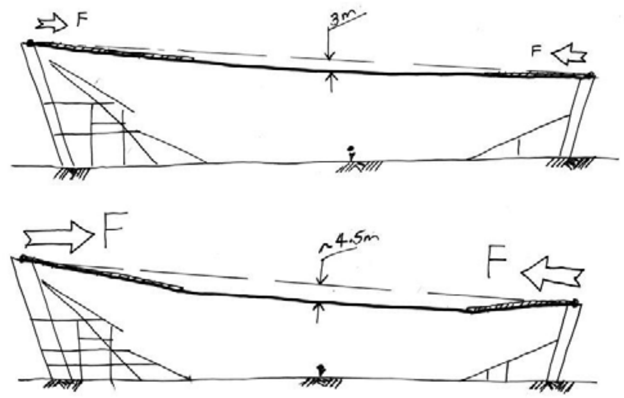

Figure 2-54. Cross Sections Showing Catenary Cable Geometry

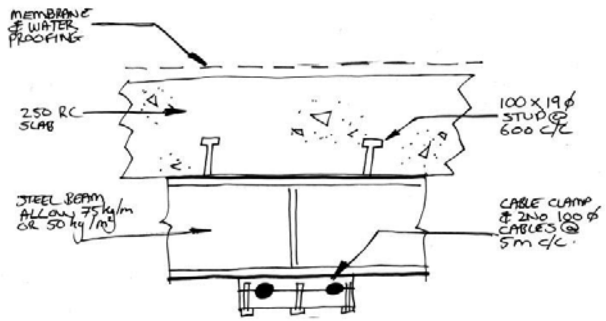

Figure 2-54. Typical Roof Detail

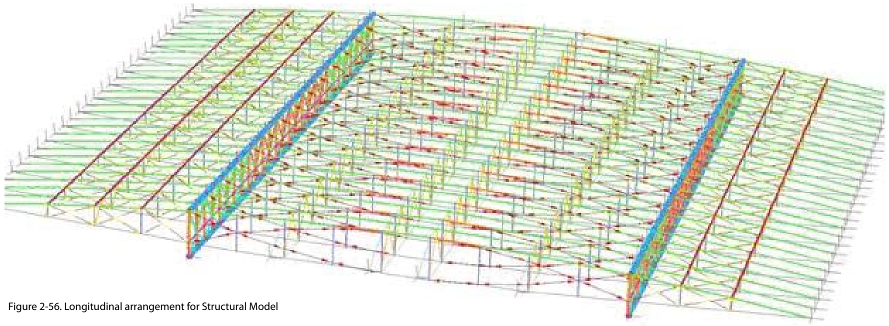

Figure 2-56. Longitudinal arrangement for Structural Model

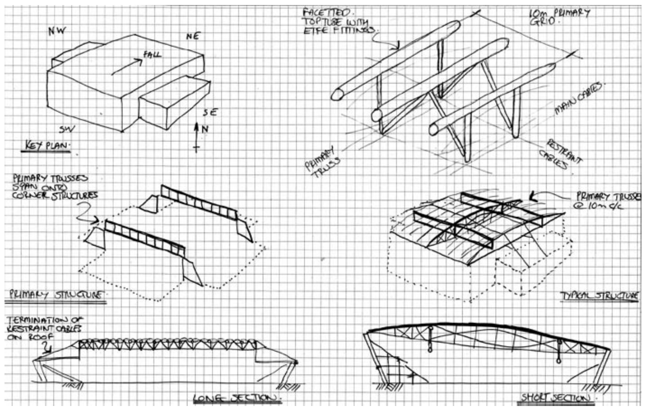

Figure 2-57. General Arrangement of Enclosed Roof

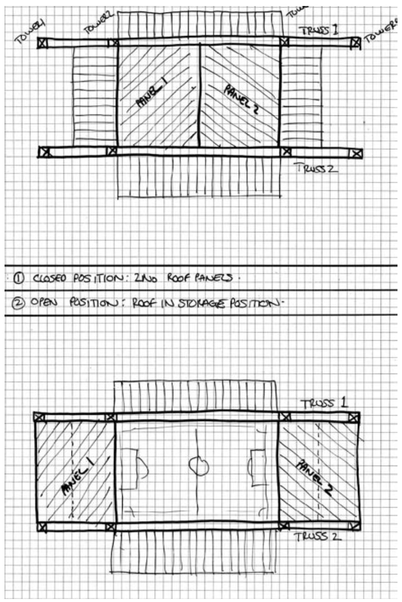

Figure 2-58. Option 1: Opening Roof Arrangement

Roof Scheme B: Fixed Roof over the Pitch

The Architectural scheme put forward considers ETFE cladding to enclose the roof. ETFE has been put forward on the basis that UV can pass through the material therefore making a natural turf pitch viable. ETFE is typically detailed as a two layer cushion that is continuously inflated with a low pressure pump. ETFE is considered a desirable material for this project. It is lightweight and offers an economic way of covering the roof.

Discussions with the ETFE supplier, Vector Foiltec have led to a maximum pillow width of 5m and a cushion depth of 1m. The structural framing has been drawn up to suit.

The Architectural scheme considers a roof spanning 175m North-South and a slender lens shape has been selected in elevation. Supporting structure is provided on a 5m grid, this consists of a cable truss and beam system located on an alternating 10m grid. Refer Figure 2-57 & 2-58 for the general arrangement.

The cable solution was selected to create a lightweight structure that did not create excessive shadowing on the field of play. In broad terms the upward arching members support wind uplift and the down arching members self weight and snow loads.

Where the roof structure meets the concrete fin walls on the North and South large tension forces are transferred to ground in the system described in section 2.3.9.

The anchorage capacity of the North and South stands form a constraint for the roof design, refer to section 2.3.9. Initially the truss was analysed to span 175m but the horizontal tension forces acting on the North and South concrete fins walls was considered too large to be feasible. To reduce the tension forces in the system two primary trusses were introduced spanning 140m each side of the field. These act as an intermediate support to the system and can be removed if the lens geometry is significantly increased in depth. Figure 2-56 is an image from the structural model which highlights the structural arrangement.

To provide a direct load path to the primary trusses internal "butterfly" cables are proposed. These transfer uplift and down forces over the field to the primary trusses.

Lateral restraint cables are provided in 4 no. locations adjacent to the top and bottom chords. These stabilise the truss chords to remain in a vertical plane and resist longitudinal wind friction forces. Restraint cables will be anchored to the rear of the West and East stands.

Figure 2-57 highlights the different components of the system and the junctions with the bowl structure. On the East and West perimeter of the roof a steel rather than cable truss is provided. This could be utilised to support the East and West roof stands, although for the purposes of the feasibility study the end stands are considered as separate structures.

To achieve the most economic roof structure it is recommended that a wind tunnel test be carried out. Experience has demonstrated this is a valuable exercise that allows a significant reduction in design wind loads. Such a study would be best carried out in collaboration with the cladding designer for an integrated design.

A preliminary sequence of construction has been developed where the structure is erected piecemeal and at height. However, discussions with specialist contractors

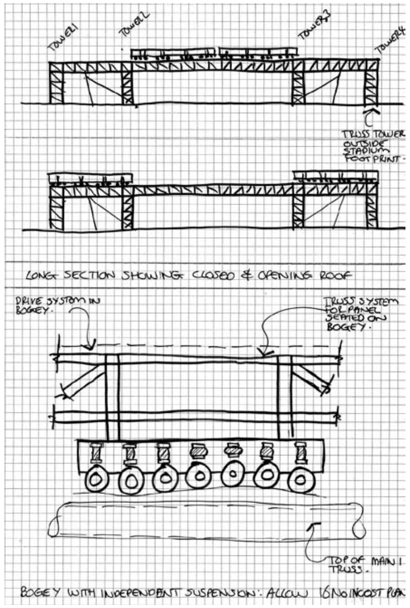

Figure 2-59. Option 1: Opening Roof Elevation & Bogey Solution

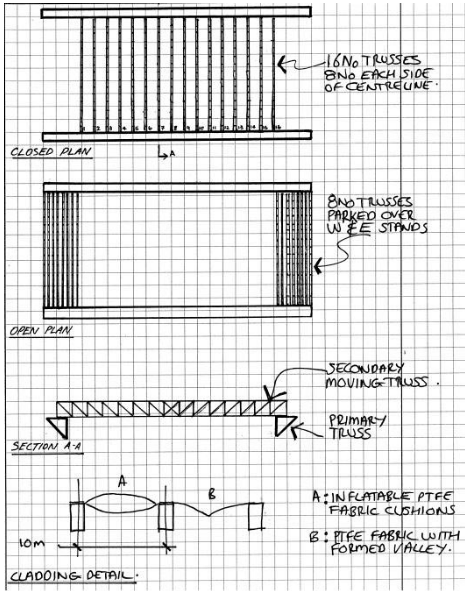

Figure 2-60. Option 2: Retractable Roof

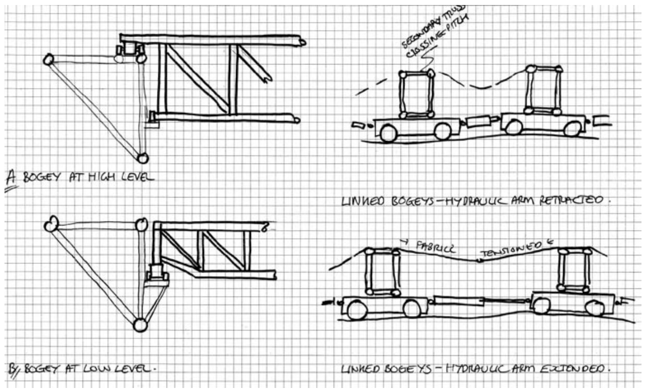

Figure 2-61. Option 2: Retractable Roof Truss & Bogey Solutions

may utilise the primary trusses to build the structure at ground level and then lift into place as one element or a series or large panels. In the next stage of works it is recommended that a specialist contractor is invited to join the team so that the structural design suits the most cost effective construction sequence.

Roof Scheme C: Retractable Roof over the Pitch

In line with the Architect's scheme for an opening roof two options were explored. The first was for two roof panels from trussed steelwork and a metal clad roof. The second was for a series of trusses spanning across the field with a PTFE fabric spanning between the trusses.

ETFE was discounted from the opening roof options as the material is inflated into cushions and if difficult to deflate for storage. PTFE put forward as an alternative is more robust and if detailed correctly can be repeatedly folded in the concertina system.

Figures 2-57 & 2-58 summarise the scheme for Option 1. The large trusses provided each side of the pitch support the roof panel and the guide rails for the mechanical system. The large size of each panel presents a storage problem in the open position as the panel is larger than the East or West roof and can not simply stow over the stand. This requires the primary trusses to extend outside the foot print of the stadium.

Due to the large structure required to store each panel in the open position this solution is considered less desirable than Option 2.

It is not practical to breakdown the panels into smaller sizes as the height to store the panels would be too large relative to the stadium size.

Figures 2-60 & 2-61 summarise the scheme for Option 2. The advantage of this system is that the storage required for the trusses is smaller as the roof concertinas and the fabric simply folding between each truss. The PTFE would most likely be inflated to create falls in the fabric.

Suggestions for typical solutions for the moving mechanisms for the roofs are presented in Figure 2-61.