Singapore Changi Airport MRT Station
Design & Construction of Glass Atria

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ABSTRACT: The glass atria at Changi Airport MRT station are some of the station's most dramatic features. Tension cable facade trusses span vertically, supported by steel structural works which involved some particular challenges in design and construction. Design was a combination of conceptual work by consultants, in-house detail design, and a design and construct subcontract for the trusses and glazing. This paper explains the principles and some of the unusual features of the design, and describes the construction methods developed to suit the confined working spaces.

1 INTRODUCTION

The glass atrium buildings on each side of the new Changi Airport Station, 60m long and 36m tall, give the appearance of twin towers and represent a new airport landmark. The atria house escalators from Terminals 2 and 3 arrival and departure levels and accommodate two-tier walkways leading to lifts down to platform level.

Figure 1. The atrium at Terminal 2, as seen from the upper (departure) level of the airport

Inclusion of such spectacular features in an MRT station is, of course, a far from straightforward utilitarian approach to creating passenger access to the station, but was justified for this particular station because of its location as the gateway to Singapore for arriving visitors.

The complex and sophisticated structural system relies on pretensioned cable trusses to support the glazing, stretching between lower anchorages cast into the concrete structure and a fabricated steel box beam spanning centrally along the atrium roof between a lift core at one end and inclined steel tubes at the other. Some of the trusses turn horizontally at ground level walkway anchorages to form a skylight admitting daylight to the station platform.

The concept design for the atria was created by the New York office of Skidmore, Owings and Merrill. Engineering input was provided by Ove Arup and Partners, also from their New York office, to develop the design to tender stage. Following award of the main works contract, the facade trusses and glazing were packaged as a specialist design and construct subcontract. This was undertaken by YKK with design input provided by Meinhardt Facades in Australia. Detailed design of the structural steelwork and the RC structure of the station was completed in-house by the LTA.

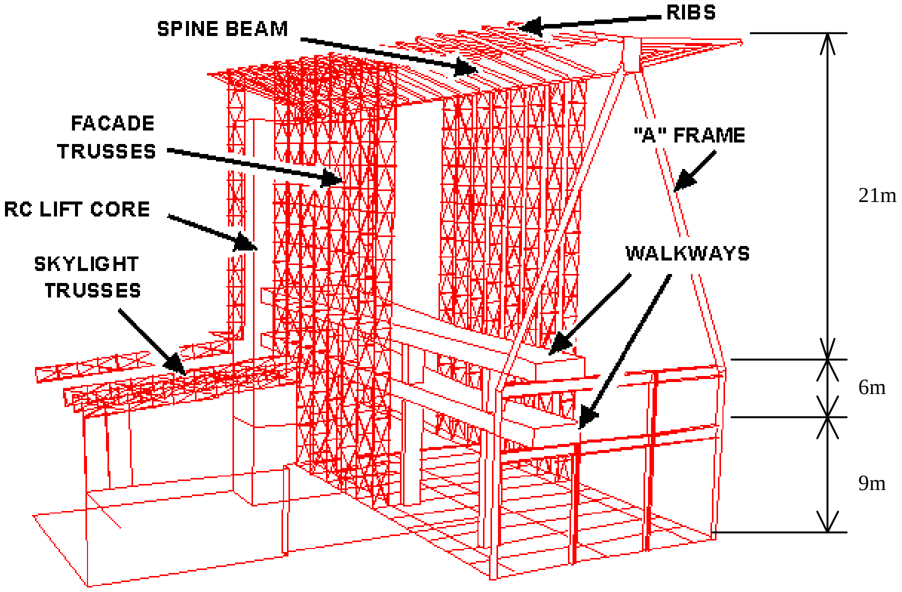

Figure 2. Components of each atrium.

Figure 3. Terminal 3 atrium approaching completion

2 DESIGN

2.1 Facade Truss Support

The distinctive features of the design are the facade trusses, whose cables are maintained in a permanent state of tension, and the interaction between these trusses and the supporting structure to enable this to be achieved in the most efficient way. Maintaining tension in the cables, and also in the diagonal truss members, is the key to minimizing their section size to create the slender overall appearance.

The supporting structure has to sustain this tension which is initially induced by prestressing of the truss cables, the prestress in each truss having to be calculated and set individually. In service, the initial prestress is modified mainly by lateral wind loads and also by creep and temperature changes in the truss members and supports. The effect of lateral wind forces is principally to redistribute the forces in the cables and diagonals of a particular truss, the overall anchorage force in the longitudinal direction of the truss not being greatly affected. This is not true for creep and temperature effects, however, as these act on all members of a truss together and modify its overall length. When this change in length occurs, the longitudinal stiffness of the truss interacts with the spring stiffness of its supports to modify the prestress in the truss members. If the support is relatively flexible and has experienced appreciable movement when cable prestress was first applied, there is effectively a self-correcting action on the truss tension so that an increase in cable length due to elevated temperatures, for example, does not result in a directly equivalent loss of prestress. The important consequence of this is that a truss with a relatively "springy" support does not require such a large initial prestress to cater for losses under service conditions as is needed for a more rigidly anchored truss.

In practice, this means that trusses anchored to the roof level ribs close to midspan of spine beam can be given less prestress than those close to supports, which of course is of considerable benefit in reducing overall loads in the main steelwork. The principle was also made use of in designing bottom anchorages for the high level trusses in the ends of the atria above the open access links to the airport terminals. Instead of being anchored directly to the concrete roof of the airport buildings, they are mounted on fabricated beams spanning the width of the atrium, allowing pre-stress in the middle trusses to be lower and resulting in a useful reduction both in uplift loads on the airport building roof and in axial loads in the tubular legs supporting the spine beam. The beam at the Terminal 2 atrium, subsequently concealed by cladding in the completed structure, is shown in Figure 4.

Figure 4. Lower anchorage to high level trusses

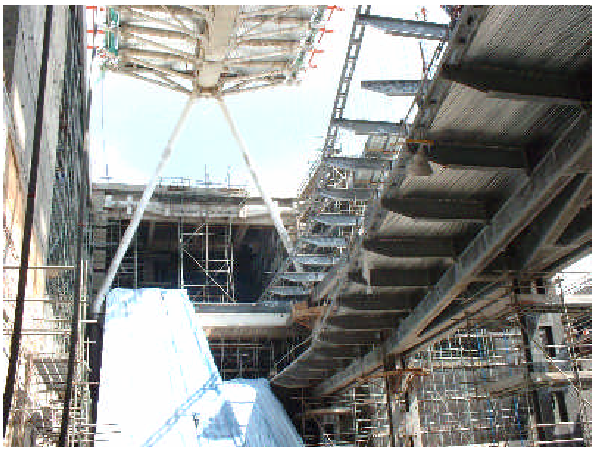

Figure 5. Atrium roof and walkways.

2.2 Walkways

Ground and upper level walkway structures, corresponding to airport arrival and departure levels, have their main framing formed of fabricated 700 x 450 box sections (Figure 5). The ground level walkways provide lateral restraint to the facade trusses to reduce their overall span, and also provide a structural anchorage where the pre-stressed vertical cables from the facade trusses meet the pre-stressed horizontal cables from the skylight trusses. To accommodate the large vertical cable forces, the ground level walkways are pre-cambered downwards in a sagging position.

2.3 Lift Core and Spine Beam

The lift core at one end of each atrium is the only concrete structure above ground level, and is designed as a conventional shaft with 300 thick walls. The original design intent was for the spine beam to be rigidly connected to the lift core so that it would act as a propped cantilever, leading to the structural rationale of a beam 4.5m deep at the lift core tapering to 2.5m deep at the link to the airport terminals. As the atrium design was developed in detail between the structural and facade designers, concerns arose with this approach particularly in regard to the possible effects of concrete creep. The concrete lift core would have been quite highly stressed in bending and thus prone to significant creep movements, with the complication that this would have included horizontal deflection at the top of the lift core in addition to rotation and axial shortening. The effects of this on the glazing seemed potentially serious and the decision was therefore taken to modify the spine beam lift shaft support to a simple fabricated rocker bearing (Figure 6) which would load the lift core axially. At this stage in the project, it was considered too disruptive to other design disciplines to alter the proportioning of the spine beam and it has remained deeper at the lift core than structural logic would now dictate.

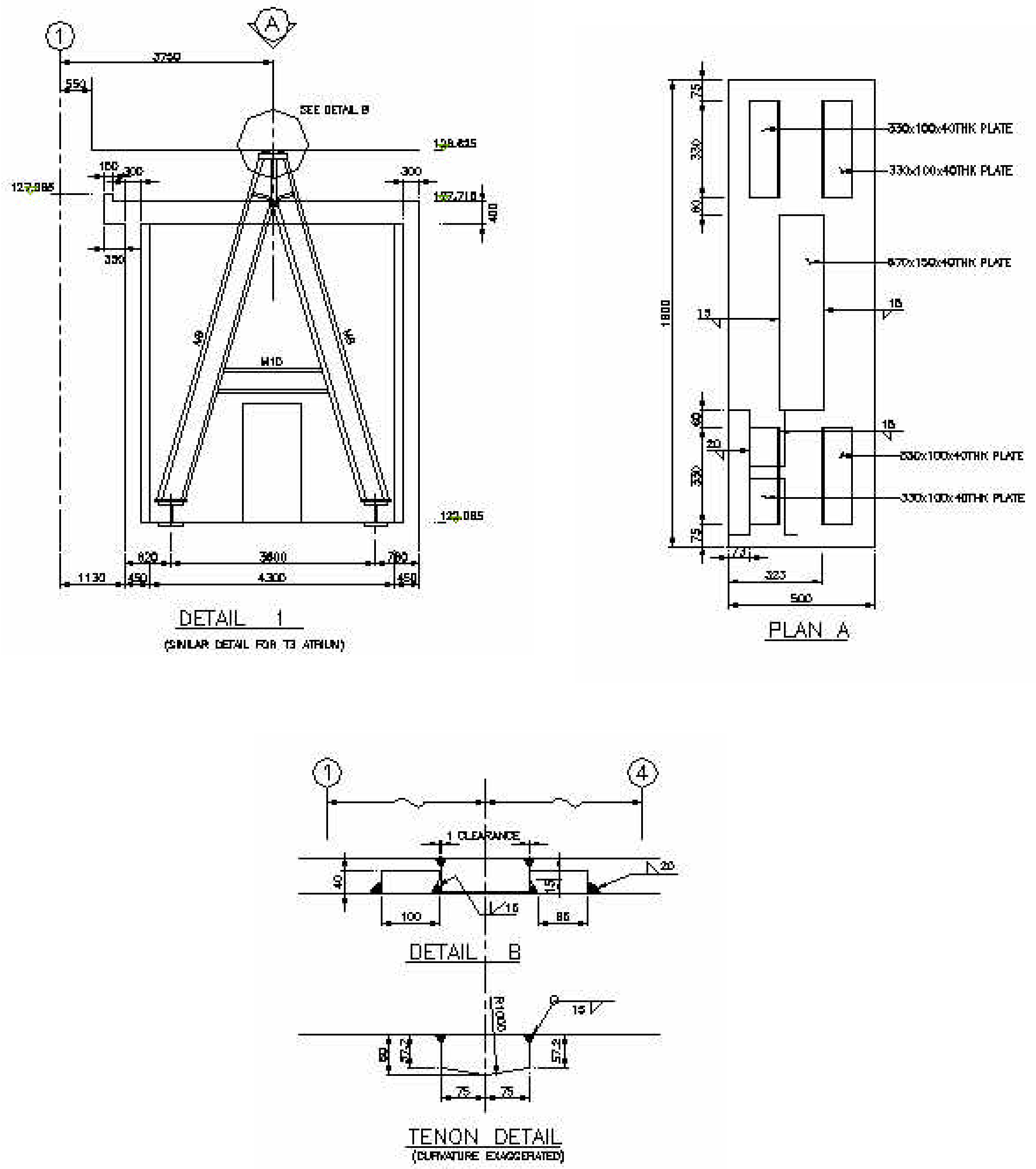

Figure 6. Spine beam to lift core connection

The spine beams are fabricated from 40, 60, and 100mm thick plate, each weighing around 500 tons. The 2.5m deep shallow ends abutting the airport terminal buildings are supported by A-frames, inclined 610mm diameter x 50mm thick hot formed tubes with horizontal ties. These are highly stressed and to maximize their compressive capacity are concrete filled. Ribs formed of 355mm diameter tubes at 3m spacing extend to the truss anchorages and also support the roof slab of 150mm concrete cast on permanent steel decking.

2.4 Facade Trusses

The facades are designed to act predominantly as trusses subject to lateral wind load, with a clear span of 25m from the roof anchorage to the lateral restraints at ground floor walkway level. Temperature and creep effects are also important design considerations, a peak temperature of 60ºC being allowed for in design to represent direct sunlight on the cables.

The majority of the trusses continue below ground level to the station mezzanine floor 9m underground, forming glazed walls against the airport terminal buildings on one side and admitting daylight and providing views of the sunken gardens on the other, and giving a total facade height of 36m. Above the escalators to the station platform, nine of the facade trusses turn horizontally at the walkway level anchorages to form a skylight feature. All main truss cables are made from galvanized spiral strand cable with a specified minimum breaking force of 1770 N/mm^2, the diagonal rods from precipitation hardened martensitic stainless steel of 0.1% proof strength 1000 N/mm^2, and the cross struts from austenitic 316 grade stainless steel with 0.2% proof strength of 190 N/mm^2. Connection fittings are all cast stainless steel (Figure 7).

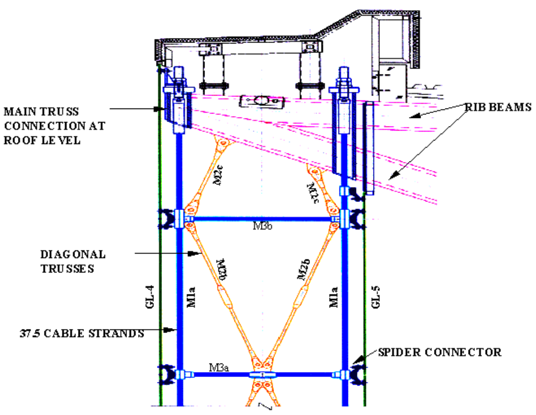

Figure 7. Diagram of truss construction - top.

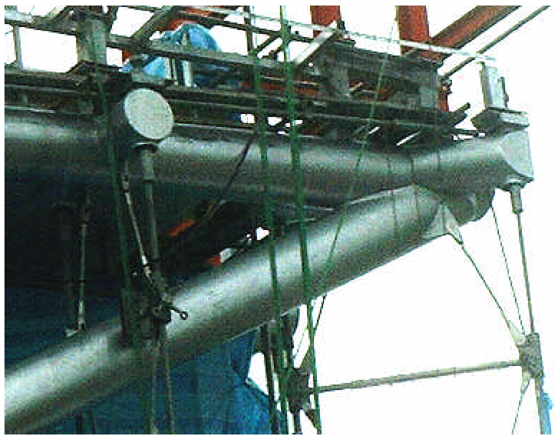

Figure 8. Truss upper anchorage at corner of atrium. Enlarged rib assemblies are used to provide anchorages for the end wall trusses.

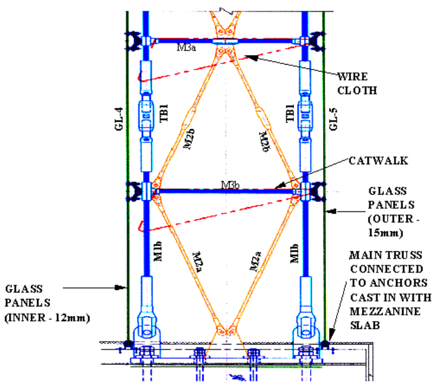

Figure 9. Diagram of truss construction - bottom.

Figure 10. Truss base anchorage

The trusses are pre-tensioned such that under all loading conditions the vertical cables and diagonal rods are never allowed to go into compression, and can thus be made more slender and graceful in appearance than if they were required to resist compressive buckling. Only the 50mm diameter cross struts are required to carry compression.

Truss behavior dictates that under lateral load, large forces develop in the diagonal members at the anchorages and have to be transferred to the vertical cables. As the cables are continuous throughout the length of the truss the only mechanism for transmission of these forces, which under extreme design conditions can be over 100kN at an individual node, is by friction through the cable clamps. Design of these clamps therefore assumed considerable importance and in the absence of established rules for guidance, the design was supported by an exhaustive series of prototype tests carried out by McCalls in Sheffield, UK. The use of a chemical adhesive to enhance the bond was seriously considered but eventually rejected, partly because of concerns about difficulties which would arise if clamps required repositioning during erection or subsequently. After experimenting with various finishes for the clamp bearing surfaces, the best performance was achieved with continuous 0.5mm deep saw-tooth serrations, these being found to produce noticeable indentations in the galvanizing to the cables resulting in an effective friction co-efficient of between 0.25 and 0.3. The clamping force is maintained by load-indicating bolts ("Smart Bolts"), most of the clamps being cast as four or eight bolt fittings with ten bolt fittings being used for the most heavily loaded clamps at the top of the trusses. The two halves of each clamp are not closed tight together by bolting, to ensure that pressure on the cable strand and thus the clamp's friction capacity are preserved.

Glazing support at the corners of the atria has to recognise the inherent high stiffness of two facades meeting at a right angle, which if restrained only by the comparatively flexible trusses would attract unacceptable in-plane forces to the glass. Use is therefore made of the horizontal catwalks as structural elements to carry the lateral wind forces, which are then transmitted to anchorage points on the concrete structure.

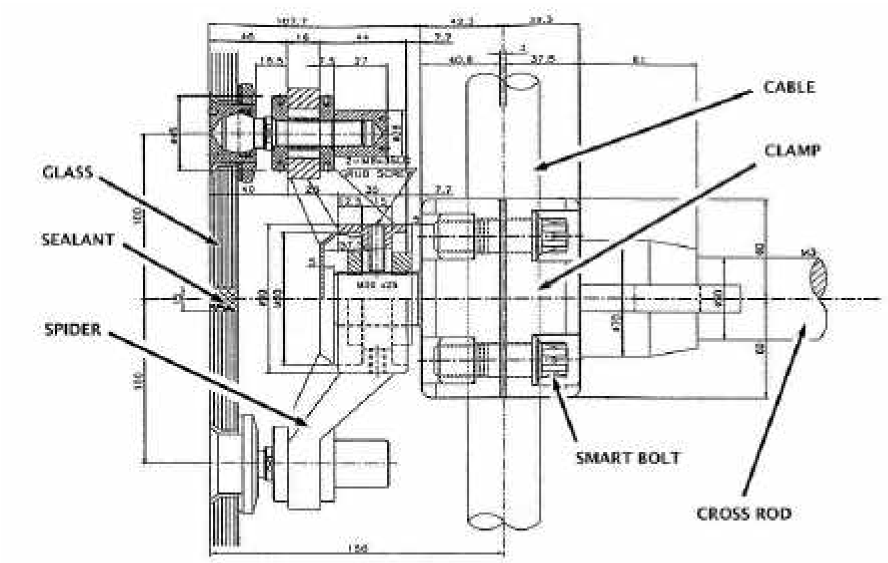

Figure 11. Detail of cable clamp and 'spider' fixing

2.5 Catwalks

Continuous catwalks are provided at each cross rod level. These serve as working platforms for maintenance access, and also function as sun shading louvres to enable the required OTTV (Overall Thermal Transmission Value) to be achieved. An open weave wire mesh wrapped around a triangular structural frame was chosen to provide the necessary compromise between facilitating a flow of cooling air and providing an adequate barrier to control radiant heat. The catwalks also had to be arranged so that visibility through the glass would not be compromised more than necessary.

2.6 Skylights

The gently sloping skylights are formed of trusses essentially similar to the vertical facades. As the lower edge of the skylights is around 1m below ground level, it has not been possible to provide a gravity connection to surface drainage and rainwater is discharged through station pump sumps.

2.7 Temporary provisions for Terminal 3

At the time the atria concept was developed it was envisaged that the MRT station and Terminal 3 would be constructed more or less concurrently. In the event, station construction has preceded the airport building and it has been necessary to provide a temporary means of enclosing the large access openings to the atrium, and also to provide temporary structural restraints to facade trusses which will ultimately be anchored to the Terminal 3 building structure. This has been achieved by triangular braced steel frames, the vertical elements of which will remain in place within the building columns so that the necessary structural restraint to the atrium facade will not be disturbed.

2.8 Glass

The main vertical facades are assembled from two glass skins mounted 1.5m apart on either side of the trusses, the space between them forming a heat insulation gap. 15mm thick frit glass is used for the exterior skin and 12mm plain glass for the interior, both being tempered and heat soaked. Conventional silicon sealant is used for all joints on the exterior skin, the interior skin being left with open joints. The skylight uses 21mm thick laminated glass as an exterior skin with no glazing on the underside of the trusses. Glazing panes are fixed using ‘spiders’ to give a point fixing at each corner which provide out of plane restraint but allow each pane to rotate independently, the top two fixings on each pane carrying the weight of the glass

To identify any possible problems with the glazing system and with the assembly and fit-up of the trusses, as well as providing a visual impression of the completed structure, a 12m high mock-up of a corner section was constructed off site. A sealed plenum was constructed around the inside face so that the glazing could be pressurized to simulate the effects of wind load. Pressure of 1.5 times the design wind load was applied which resulted in very noticeable distortion of the glass panes, but no shortcomings were apparent necessitating any redesign.

2.9 Tolerances

Glass is not a forgiving material if it has to be forced into place during erection and it was recognized to be of vital importance that the glazing fixings were accurately located prior to fixing of the glass panes. The spider fixings attached to the cable clamps have slotted holes to allow some measure of adjustment, but this is limited to about 5mm. Thus comprehensive provisions for adjustment to accommodate tolerances in fabrication and erection were a major consideration in detailing of the main steelwork and truss anchorages. The spine beams and walkways were fabricated as single units with site welding as necessary, but the brackets and other members to which the cables themselves are fastened are all provided with bolted connections to facilitate initial adjustment. Extensive use has been made of shim plates and enlarged holes, with the general aim of allowing 25mm adjustment in any direction. The top cable anchorages are formed of oversize tubes welded through the ends of the rib beams, through which the swaged cable terminations pass with nuts threaded on above to provide the means of applying the cable pre-stress. Brackets to form lower cable anchorages are attached to bolts cast in to the concrete structure, each bracket being fabricated individually to suit the surveyed as-built locations of the bolts. Oversize holes were also used in the base-plates to provide further adjustment.

2.10 Design Analysis

Analysis of the structures was carried out using frame models, with some use of plates to represent the spine beam and roof slab. There was no overall analysis combining all the components of the atrium in a single model as this was felt to be an unnecessary complexity. Detailed design of the supporting structures, which was carried out in-house by the LTA, made use of independent models for the spine beam and for the walkway, truss loads for these models being provided by the facade designers. The deflected shape of the ground level walkway under the action of cable anchorage forces from the trusses above the skylight is shown in Figure 12, and the facade truss model in Figure 13.

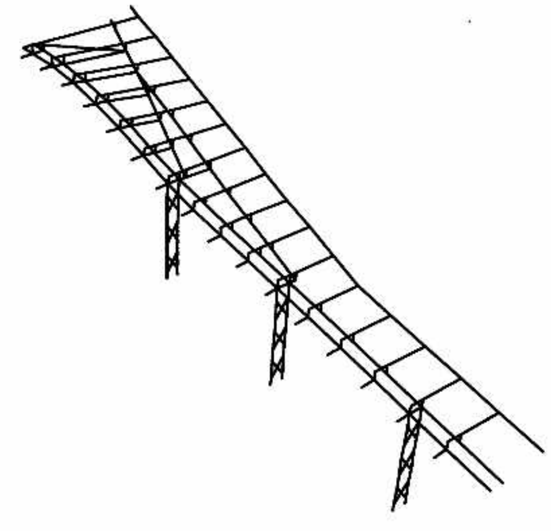

Figure 12. Analytical model of walkway.

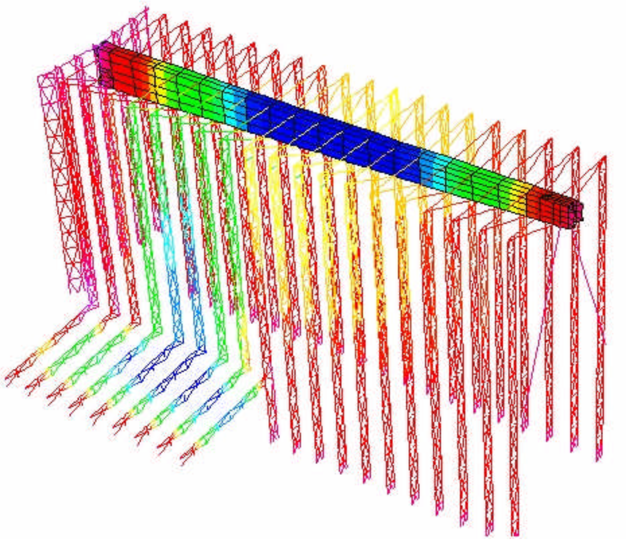

Figure 13. Analytical model of spine beam and trusses

3 FABRICATION AND ERECTION

3.1 Quality Control

A Quality Plan was submitted by the manufacturer and approved by the Authority prior to commencement of fabrication. Regular QA/QC inspections were carried out at the fabrication yard to cover the following areas:

- Dimensional checks
- Check of groove angle, root opening and visual examination of weld
- Chemical analyses and non-destructive tests on raw material and welding
- Blast cleaning according to Swedish standard SA 2.5 before applying the primer coat
- Quality of paint work

3.2 Spine Beam

Fabrication of the spine beam was carried out off-site in Singapore (Figure 14 and 15), the grade 50 steel plates being fitted up and full penetration butt welded to form 7 box girder modules for transportation. Flux core arc welding was adopted with about 96 passes for the thickest 100mm plates. Table 1 summarizes the spine beam properties and fabrication process module by module.

Table 1. Spine Beam Fabrication Summary

Segment No	Weight (t)	Ribs (t)	Sub total (t)	Welding	Purpose
1	77	50	127	FP FCAW (ASW)	To Weld 100mm thk Plates
2	43	18	61	Semi-Automatic	To Form Box Girder
3	51	18	69	FP FCAW	To Weld Segments on site
4	48	26	74	FP FCAW & SMAW	To weld ‘A’ Frame and Ribs CHS
5	55	35	90	**Paint**	**Purpose**
6	40	26	66	Zinc rich primer	Protects the parent material from corrosion
7	37	39	76	Micaceous Iron oxide	Anticorrosion and over coat purpose
Grand total	**351**	**212**	**563**	HS Epoxy	Anticorrosion and over coat purpose
				Polyurethane finish	Provides aesthetic appearance

Figure 14. Semi-automatic welding of spine beam.

Figure 15. Spine beam connection to A-frame

3.3   Ribs and A-Frame

The rib beams and 'A' frames tubes were fabricated from hot finished circular hollow sections with full penetration butt welds, using a combination of flux core arc welding and shielded metal arc welding depending on the angle formed by the joint.

3.4   Steelwork Erection, Terminal 2

After completion of the lift core, steelwork erection commenced with the spine beam as described in Figure 16.

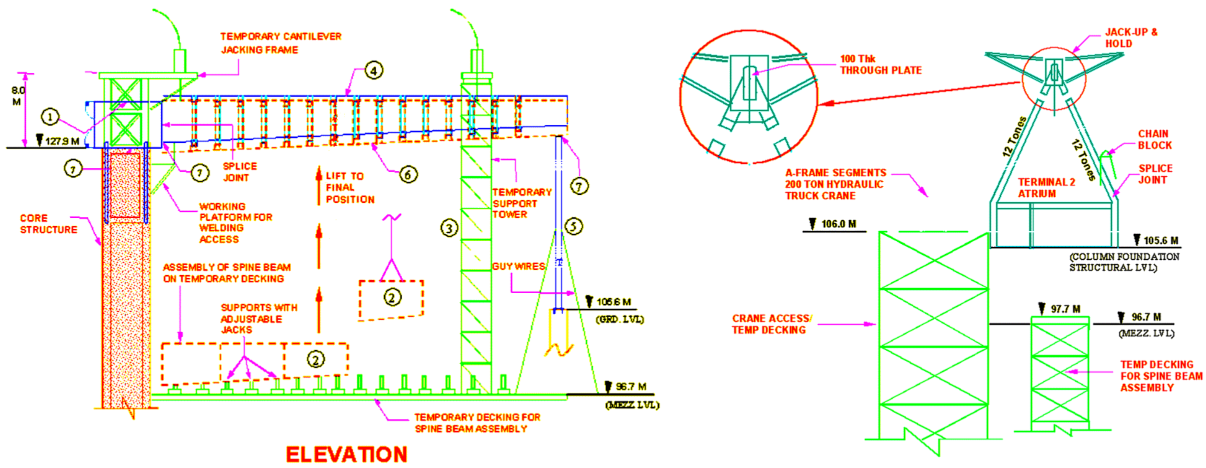

Figure 16. Spine beam erection, Terminal 2 atrium

The site layout shown in Figure 17 highlights the limited space available due to the surrounding Terminal 2 buildings, departure level viaduct and boulevard. Because of these constraints, a temporary deck had to be erected inside the underground station box to support the central five of the seven spine beam modules for final welding. The modules were welded from welding cages and on completion raised for joint welding to Module 7, which was positioned at ground level due to the limited space available as shown in Figure 20. Figure 19 shows the preparation for a flange weld.

Module 1 was lifted in advance of the other modules and positioned directly on top of the lift core, using two VSL SLU 120 strand jacks as shown in Figure 18. Jacks were also tested at this time prior to lifting the remaining pre-assembled modules 2 to 7. Temporary towers were erected with two VSL SLU 220 strand jacks installed, for the initial lift of modules 2 to 6 to ground level followed by the main lift of modules 2 to 7 to roof level. Lastly, modules 2 to 7 were lifted about 300m above final level and slid towards segment 1 for welding as shown in Figure 21.

Once all the modules were assembled the ‘A’ Frame was erected and filled with concrete. After the concrete had reached the required strength, the spine beam was lowered and welded to the 'A' frame as shown in Figure 21, following which the temporary towers were removed.

The rib beams were then lifted individually using a long arm hydraulic crane from ground level, and bolted to the spine beam using hanging access cages as shown in Figure 22.

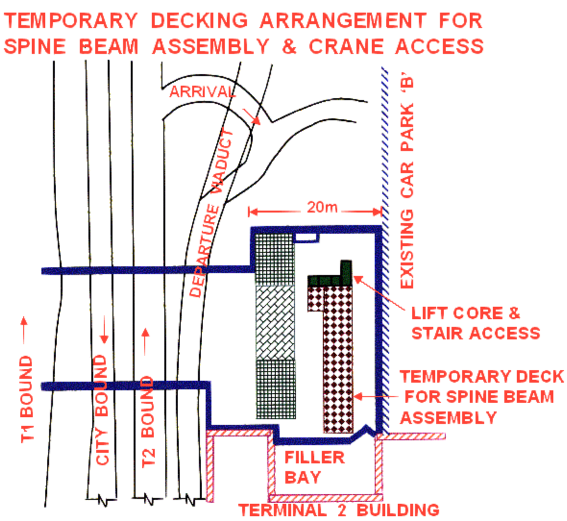

Figure 17. Site arrangement, Terminal 2 atrium.

Figure 18. Erection of 1st module

Figure 19. Spine beam butt weld preparation

Figure 20. Modules 2 to 7 under assembly on temporary deck

Figure 21. Terminal 2 spine beam erected.

Figure 22. Installation of ribs

3.5 Steelwork Erection, Terminal 3

Sufficient working space was available at the Terminal 3 atrium site to enable the entire spine beam to be assembled at ground level prior to lifting, complete with most of the ribs. The erection was not carried out directly beneath the beam's final location, however, as this would have excessively obstructed other works inside the station box. Instead, the beam was assembled at a 45º rotation to its eventual position so that the work was largely outside the station structure as seen in Figures 23 and 24. The seven modules were assembled progressively on a launching rack and skidded towards the lift core to free working area behind.

Figure 23. Terminal 3 atrium spine beam on ground.

Figure 24. Spine beam top view

In preparation for lifting, the strand jack apparatus used on top of the Terminal 2 lift core was transferred to the Terminal 3 side. The lift core end of the spine beam was then suspended from these jacks so that all the intermediate temporary supports could be removed, leaving only the far end supported on a temporary traversing trolley. This trolley was then progressively rotated by jacks on a curved sliding track until the spine beam was aligned to its final position.

Temporary towers were erected on both sides of the shallow end of the spine beam, and a tandem lifting operation performed using two 500t hydraulic mobile cranes operated simultaneously with the strand jacks at the lift core top, as shown in Figure 26. Sliding of the complete assembly over the lift core was then achieved with a sledge at the lift core end, which was jacked along a cantilever frame installed on the lift core roof whilst combined slewing and extension of the crane booms at the far end passively followed the sledge movement. A temporary extension tail piece was added to the spine beam to enable the temporary towers to carry its weight until installation of the 'A' frame was complete. Installation of the remaining ribs and casting of the roof slab followed with the spine beam in its final position.

Lifting using strand jacks was always configured to simulate a 3 point statically determinate lift, which was made possible by the formation of a hydraulic hinge within the jacking configuration.

Figure 25. Sliding Terminal 3 spine beam.

Figure 26. Lifting spine beam to its final level

3.6 Tension Cables and Bracing

Following completion of each atrium roof structure, including the casting of the concrete slab, installation of the trusses could commence. One of the challenges was to maintain the structure in equilibrium during the pre-stressing process, as the positions of the pre- tensioned cables and the bracing system would have a direct impact on the designed sizes of the glass-panels. Since the working tolerances were so stringent, the contractor engaged a team of stressing experts from Japan to carry out all the stressing and adjustment works. Their works were so precise that no glass panel had to be discarded due to incorrect cable positioning.

Installation and prestressing of the facade trusses had to be carefully sequenced because the flexibility of the spine beam and other anchorages meant that each truss could not be fully stressed independently, as stressing of the later trusses would cause relaxation of the earlier. Tensioning of the cables was divided into ten equal stages, commencing from roof mid-span where vertical deflections would be greatest. The stressing or tensioning was carried out at the roof level, while the anchors were cast into the concrete structures at mezzanine level. The first phase of the process was to install the cables only, without the diagonals and cross struts, and apply excess tension to them equivalent to the total designed prestress in the completed trusses In this way the steel supporting structures could be brought to their final loaded condition and hence to their final deflected positions. To achieve this, tension was applied to the cables progressively in steps of 25%, the earlier cables to be tensioned at each step being overtensioned so that they would relax to their intended tensions as the supporting structures deflected under the action of the subsequent cable stressings.

The interaction with the walkway structures during the stressing of horizontal cables on the skylight required special consideration, as cable tension on the facade adjacent the skylight was increased when the horizontal cables were tensioned. It resulted in the floor walkway floor structure deflecting upwards, so that level adjustments to the finished and the floor mounted air-conditioning diffusers were necessary.

Once all cables had been stressed to a load equivalent to the design load for the completed trusses, the second phase commenced. This involved trusses being assembled one at a time and then tensioned to their design prestress level before moving on to the next truss, so that as each truss was being worked on the change in deflection of the supporting structure would be minimal. Original thinking for installation of the adjustable diagonals had envisaged that this would be a tedious operation involving stressing of each diagonal individually using calibrated torque wrenches, strain gauges or similar means. In the event, however, a much simpler procedure was devised. Positions of clamp fittings were first marked off on the stressed cables. The cables were then destressed to a calculated value, and the entire diagonal and cross-bracing system installed in a tightened but not stressed condition, except for the topmost diagonals which were initially omitted. Bracing was pre-fabricated in modular lengths and lifted into position with a winch. The cables were then restressed at the top anchorages so that the required forces were automatically induced into all the diagonals and cross struts. Slight overtensioning was carried out to allow the top diagonals to be installed and connected to their upper anchorages, with subsequent relaxation of the overtensioning to induce the required tension into these upper diagonals. The cable tensions at each step were in practice slightly modified to allow for the dead weights of glazing and catwalks which were to be installed after completion of the stressed trusses.

4 LONG TERM DEFORMATION, STRESS AND TEMPERATURE MONITORING

It is critical to the long-term performance of the atrium structural system that the cable trusses remain in a state of tension under all conditions. Complete loss of this prestress would be likely to result in excessive deformation of the trusses under wind load with unacceptable movements of the glazing. Although steelwork is not expected to be subject to creep to the extent that concrete would be, it was nevertheless thought prudent to make comprehensive provision for monitoring of the structural elements together with means of subsequent adjustment should the need arise. In view of Singapore's tropical climate, it was also thought prudent to monitor temperatures to confirm that the design assumption of a maximum steel temperature of 60º is valid.

An automated monitoring system has therefore been installed. Data is gathered primarily from vibrating wire strain gauges installed on selected cables and also on some of the roof ribs. All gauges are wired to a central data logger which has a remote download capacity by dial in modem so that the system can be monitored without having to visit the site. The gauges are provided with a facility for temperature recording enabling an appropriate compensation to be applied when determining the changes in measured strains.

If necessary, adjustment of truss tensions can be carried out at a future date using the screwed cable anchorages at the roof ribs, which are accessible from above.

Other features installed into the structural system are load indicating bolts (“Smart Bolts”) in some of the critical clamp locations. These bolts make use of a probe and gauge to give an instantaneous reading of the tension state of the bolts, to indicate whether a loss of clamping force and hence frictional shear capacity is taking place.

5 CONCLUSIONS

The design and construction of the glass atria successfully combined an engineer’s design of the supporting steelwork with a specialist design and construct contract for the facades themselves. The unusual nature of several of the design features, which were often highly stressed to achieve the overall lightness of appearance of the atria, was addressed by a programme of comprehensive testing during construction and monitoring after completion. An important aspect of the design was a recognition of realistic tolerances which would need to be accommodated in the fit-up of components, and making appropriate provisions for these in the detailing. As a result erection was able to proceed smoothly, with the structures being completed to a tight programme.

REFERENCE

Beckwith-Smith, J., Roche, M., and Thung, S.K 1999 Strucural Design of Atrium Building and Mezzanine Bridge in Changi Airport Station. *International Conference on Rail Transit, Singapore*.