SPACE SYSTEMS

University of California Lick Observatory

Automated Planet Finder Enclosure

DESIGN ANALYSIS REPORT

STRUCTURAL ENGINEERING

CI No. DN – 07753-01

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1. INTRODUCTION

1.1 General

This report covers the structural design, the determination of the critical design load case and the critical design condition for each set of structural members in the UCO-Lick APF Observatory Enclosure.

The 'Environmental Requirements' section of the KECK Outrigger Observatory Functional Specification CI No $FS02985\text{-}01_2$, pages 12 – 17 are used to determine the critical loads and load cases for each structural member. KECK loading is more severe than the conditions at LICK which is at 5000 ft elevation, Mt Hamilton near San Francisco, California. The conditions at Lick are checked against each condition at Keck for comparison purposes (see Table 1 below).

Condition	KECK	LICK
Elevation	4146m	1520m
Wind Velocity at site, V_p	73 m/s	53 m/s
Equivalent wind velocity at sea level	60 m/s	49 m/s
Air density relative to sea level	68 %	86 %
Basic wind pressure	2.0 kPa	1.4 kPa
Dead load on azimuth wheels including equipment - upper bound	500 KN	550 KN
Dead load on azimuth wheels – lower bound	300 KN	350 KN
Telescope dead load	TBC	210 KN
Centre of gravity of telescope above base of scope	TBC	3.4 m

Earthquake Zone	4	4
Pseudostatic horizontal acceleration	0.25	0.25
Snow top 45°	2 kPa	2 kPa
Snow 45° - 70°	0.6 kPa	0.6 kPa
Snow on Balcony	3 kPa	3 kPa
Ice load	0.7 kPa	0.7 kPa

Table 1: KECK and LICK – Condition comparison

To avoid confusion with management, designers and fabricators, **working stress loads, (resulting bending moments, shears, axial forces etc are un-factored)** are considered i.e.; actual expected maximum loadings.

Hand calculations were backed up by ANSYS-version 8 and 9 Finite Element Analysis (FEA) program results run by Francis Teng.

1.2 REFERENCES

This document references the following specifications and standards

#	CI / Reference	Title
1	TBA	Co-Rotating Observatory, Dome Control System, Technical Specification
2	TS-02848-02	Outrigger Enclosure Technical Specification (9m Co-rotating Observatory)
3	DN-02985-03	Outrigger Enclosure Critical Design Review Notes
4	AS 1170.3-01	Australian Standard : Wind Loading Code
5	TBA	USA Uniform Building Code Volume 2 -Structural

6	DN-03158-02	Design Analysis Report : Cladding
7	DN-03157-02	Design Analysis Report : Cooling and Ventilation
8	DN-03159-01	Design analysis report Structural Engineering (Preliminary)

1.3 LICK ENCLOSURE – STRUCTURES OVERVIEW (SEE FIGURE 1)

The enclosure broadly consists of two main structures: - fixed ringwall and rotating dome structures. Please refer figure 1. Telescope pier structure was designed by EarthTech and coordinated by Matt Radovan, from the University of California Observatories LICK.

The ringwall consists of an approximately 8m diameter base ring wall with 3.5m high, which supports the dome and the cylindrical internal structure below the upper dome hemisphere housing the three floors. It also supports the service balcony (refer FIG1-item 11) and the APF mirror removal rail (refer FIG1-item 12).

The rotating dome consist structures of:

(a) Large azimuth support beam (refer FIG1-item 1) , which is approximately 700mm high x 200mm wide and supports all of the rotating structures and has the azimuth support wheels underneath.

(b) Two arch beams on each side of the slit (refer FIG1-item 2), this is approximately 400mm high and 250mm wide. The beam supports the shutters the four main floor hangers and supports the dome sausages and separation ties. These members are important to support the dome for wind, snow and ice loads.

(c) Cladding Frame Assembly (refer FIG1-item 3), – these members support the side of the dome and are made of 219OD 4.8 thick CHS members.

(d) Separation ties/struts (refer FIG1-item 4), which hold the two halves of the dome from peeling apart in winds and also to transfer loads between two hemispheres.

(e) Fiber Reinforcement Panel (FRP) (refer FIG1-item 5), provides enclosure to the dome structures and also provides wind resistance and thermal insulations to environmental loadings. .

(f) Three main floors (refer figure 1), are supported by the floor hangers (see FIG1-item 6) and the azimuth support beam (FIG1-item 1).

(g) Shutter (refer FIG1-item 7), – consists of two shuttles, which operate separately.

The rotating dome is driven by 2 drive wheels at 180° spacing (refer FIG1-item 8) and supported by 20 vertical support wheels (refer FIG1-item 9), flexibly mounted in removable bogies, and equispaced between the drives. These rollers are mounted on the underside of the azimuth support beam.

14 horizontal guide wheels (refer FIG1-item 10) position the dome laterally between the vertical support wheels. The guide wheel assemblies are mounted at the top onto the azimuth support beam and the bottom onto the level 2 floor.

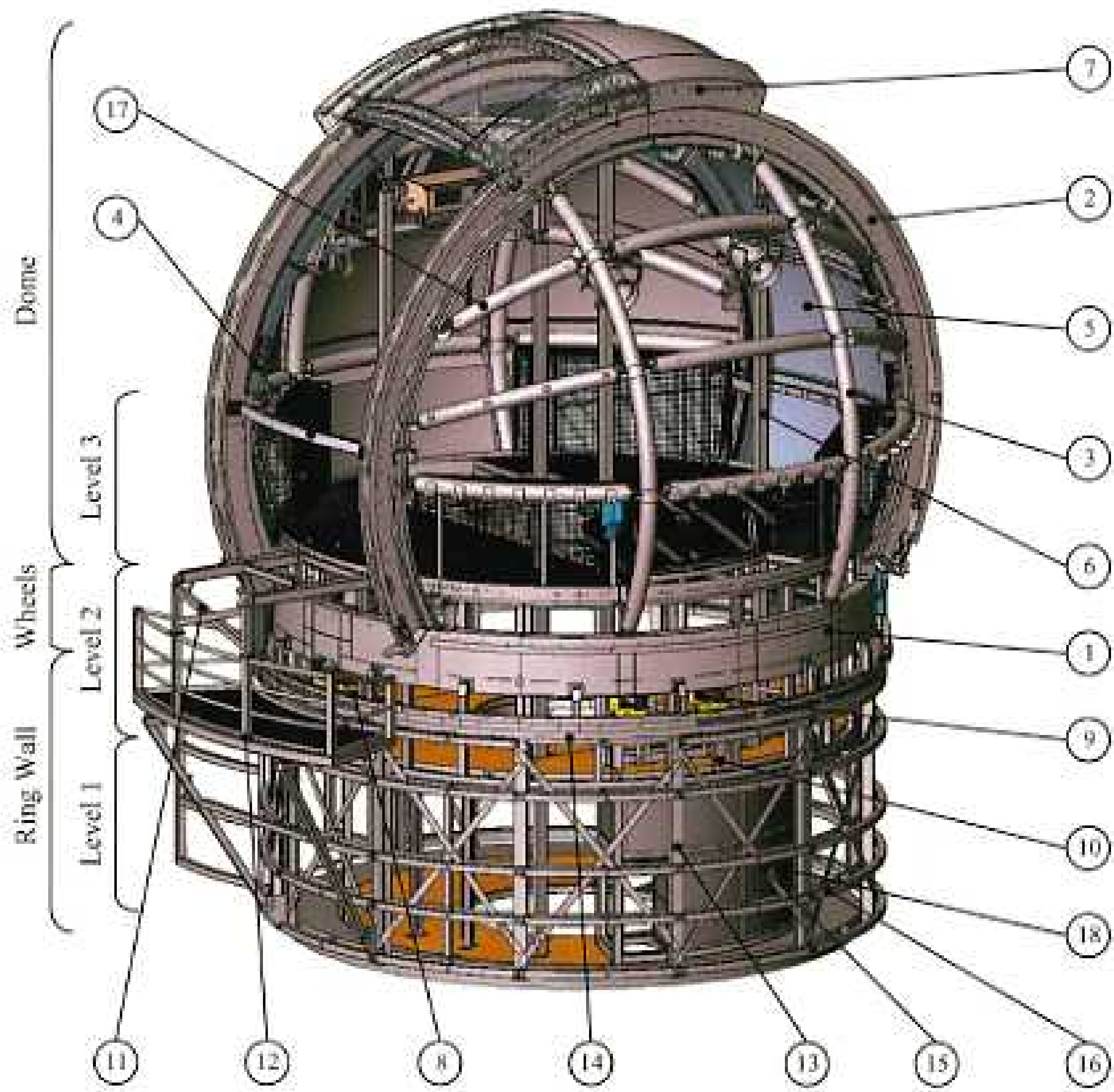

Figure 1.: LICK ENCLOSURE – STRUCTURES OVERVIEW

2. STRUCTURAL DESIGN LOAD CONDITIONS AND MATERIALS

2.1 Summary of Load Conditions

The structural design condition is firstly determined for the critical structural load for each set of structural members.

2.2 Dead Load (D)

Dead load of the steel structure for the design is taken as 350kN lower bound and 550kN upper bound with 300kN lower bound and 400kN upper bound for the weight of the structure on the azimuth wheels. Dead load of equipment is taken as 100kN upper bound on the azimuth wheels giving an upper bound total load on the azimuth wheels of 500kN.

The telescope dead load as taken 21 tonne with a centre of gravity 3.4m above the base of the telescope.

2.3 Live Load (L)

The live load on the floors for strength calculations is taken as 3kPa UDL and 3kN point load over a 100mm by 100mm area. For stability calculations and load on azimuth wheels, a live load factor of 0.4 is used, giving 1.2 kPa UDL on all floors. The level 3 floor has an additional load of a 6 tonne mirror including 0.5 tonne mirror cart. These mirror loads are increased by 50% for dynamic load. The maximum point wheel load of the mirror cart is taken as 50kN to allow for unusual conditions.

2.4 Wind Load (W_D)

From P14 of the KECK functional specification (ref 2) the survival wind gust is 67m/s (150mph) for a three second gust to ASCE 7-95.$_{TM}$ at 4146m – 13,600 ft elevation. This design speed was increased by 10% to 73m/s (165mph) to ensure the safety of the structure. This 10% extra procedure is normally reserved for post disaster facilities as hospitals and essential services buildings. As this is not applicable here, the extra 10% wind speed becomes an extra safety factor. This translates into an approximate 20% increase in wind pressure over the 67m/s (150mph)-wind pressure required in the design.

The steel support structure consists of short, stiff members, therefore dynamic harmonics are ignored.

The dome is considered to be completely sealed for this 73m/s (165mph) wind load i.e.: the slit closed, no external doors or vents open and all air intakes sealed under the extreme wind gust condition.

The equivalent design condition at sea level is 55m/s (125mph) and at LICK 5000ft (60m/s 135mph).

Using 73m/s (265Km/h) (165mph) at 4146m elevation and designing to AS1170.2 Wind Load Code $_{10}$ results in a basic pressure of 3.25kPa if the installation were at sea level. As the air pressure at site elevation of 4146m is 60% of sea level, this gives a basic design wind pressure at all sites of 2.0kPa. The basic wind pressure at LICK is also taken as 2kPa. This is conservative and was reduced for the slot shutter cover design to the local LICK conditions giving 2.5kPa suction instead of the 3.5kPa used elsewhere on LICK.

Using Fig. A4.3 (see attached) the external pressure coefficients can be determined giving the following local design pressures;

- The maximum external suction and uplift pressure is 1.5 x 2.0 = 3.0kPa. The design URS 3.5kPa and is therefore concluded as being adequate. Internal pressures are ignored as it is assumed that the dome slit is closed as are the external doors and vents. If the condition of the vents open and the maximum wind then the maximum pressure on the external members could be as high as 5kPa.

- The maximum direct pressure is (0.9 + 0.2) x 2.0 = 2.2kPa. The 2.5kPa used in design is therefore concluded as being adequate.

- The lee side suction is 0.4 x 2.0 = 0.8kPa. When added to 1.7kPa allow for internal pressure. The 2.5kPa used in design is therefore concluded as being more than adequate. This condition allows for the shutters being partially broken or not closed.

These pressures are all permissible stress design or working stress design pressures. The values should be conservative, as the dome will dissipate some of the pressure in the third dimension. Refer to AS1170.2.

For overall stability, structural loads and calculation of the wheel forces, drag and uplift forces from AS1170.2 Appendix $A4_{10}$ are used. A horizontal drag factor and an uplift factor of 0.7 is used from equation A4 (2) and assessment of Fig A4.2. These factors result in pressures of 1.4kPa. The 2.5kPa used in the design therefore is concluded as being adequate with extra safety possible with area reduction factors.

The uplift force calculated on the structure is 100kN. The horizontal force on the 9.5m diameter dome for the section above the spring line is $35m^2$ by 2.0kPa being 70kN applied 2.0m above the spring line. The horizontal force on the 3.3m high equivalent barrel section also conservatively taken as 9.0m in diameter is 60kN. Combining these two forces gives 130kN shear and 470kN-m overturning moment at the azimuth dome rail level.

The horizontal force on the 3.5m high base ring wall taken as 8m in diameter is 56kN. Combining these three forces now results in 190kN base shear, and 1020kN-m overturning moment at the base of the base ring wall.

2.5 Earthquake Load

The earthquake area is seismic zone 4. The steel dome structure is flexible, less than 15m high therefore the initial design uses the simplified pseudostatic approach specified in USA UBC Code $_{11}$ Vol. 2 1629.8.2. Zone 4 Z = 0.4, foundation unknown, therefore use Ca = 0.4 Na, structural importance factor 1.0, no irregularities, R = 5.6, omega 0 = 2.2, therefore we conform to the height limit of 48m. See Table 16-0 B ap = 2.5 Rp = 3.0 Na = 1.0

Therefore from 1630.2.3.2 the base shear maximum is 0.21W. The structural design conservatively uses 0.25W.

The estimated maximum dome weight above the azimuth rail and equipment, is a maximum of 40 tonne, therefore the maximum base shear is 100kN. For comparison the base shear for wind at the dome rail level is 130kN, which is more than the earthquake base shear.

2.6 Temperature Load

The structure is able to expand in all directions therefore it is not necessary to take temperature forces into consideration for the steel frame design. The modulus of the cladding is an order of magnitude lower than that of the steel framing and therefore too soft to induce much load. Thermal stresses in the cladding itself are not significant compared with other loads (see DAR "Composite Cladding" $_{11}$) and the thermal loads on the frame are negligible.

2.7 Snow Load (Sd)

The KECK snow load is 2kPa on the top 45 degrees and 0.6kPa on the ring between 45degrees and 70 degrees (refer to P14 of the Functional Specification $_{2}$).

The area of the dome above 45 degrees is approximately 6.7m in diameter giving 35sq m at 2kPa, which gives 70kN. The area of the dome between 45 degrees and 70 degrees is a ring of approximately 1.2m wide and 7.8m average diameter, giving 30sq m at 0.6kPa gives 18kN.

These values give a total vertical load of 88kN, due to snow, to the azimuth rail. Adding 6sq.m balcony at 3kPa, giving a total 106kN to the base of the ring wall.

2.8 Ice Load (Id)

The KECK ice load is 0.7kPa on the entire surface of the dome and walls. The surface area of the 9.5m diameter hemisphere above the spring line is 141sq m and the 8m diameter barrel, 3.3m high, has an area of 82sq m.

This gives a total vertical load of 143kN to dome rail level.

The 8m diameter base ring wall 3.5m high gives another 88sq m or 62kN.

Adding the platform and handrail (10kN), to two faces of the ring wall makes the total load at the base of the ring wall 215kN.

2.9 Load Combinations / Critical Loads

(a) The 3.5kPa wind suction load Wd on the external walls and dome members is the most critical load for the design of individual external members.

(b) The critical load for the stability of the structure is taken as 2.5kPa horizontal wind pressure.

(c) The critical load for the design of the internal floor members is 3kPa UDL and 3kN point load over a 100mm by 100mm area.

(d) The critical load case for the design of the maximum vertical load on the azimuth support rollers is dead load upper bound + 40% factored live load (see above) + snow + ice +wind 50%

I.e.: D + 0.4L + Sd + Id + Wd/2.

- The critical load case for the horizontal guide rollers at the azimuth rail is wind.
- The wind load causes uplift of 7.4kN/m on one side at the azimuth rail and downward load of 7.4kN/m on the other side.

- The lower bound dead load on the dome rail is 13kN/m and the upper bound is 17kN/m. The live load factored 40% adds 5kN/m. The lower bound weight at azimuth rail is more than the maximum wind load.
- The ice and snow load at dome rail level is 9kN/m.
- The critical load on the dome rail is therefore 35kN/m, being dead load upper bound plus the live load factored snow and ice load, plus 50% of wind. This results on a maximum wheel load on 22 wheels of 45kN with wheels stationary.
- The critical horizontal load at azimuth rail level is 130kN taken over 4 effective wheels giving 35kN maximum per wheel.

(e) The two critical loads for the foundations of the ring wall are

i. Dead load upper bound plus 40% factored live load, snow, ice and wind

$$D_u + 0.4L + Sd + Id + Wd$$

This gives 550KN dead load, plus 120KN by factored live load, plus 106KN snow load, plus 215KN ice load with 1020KNm wind overturning moment. The maximum load on the base of the ring wall is 991KN, giving 39KN/m and wind bending moment gives 20KN/m, therefore the maximum combined forces is 60KN/m, giving approximately 120KN per column maximum. Design Note: $A=25e3\ mm^2$, $I=200e9mm^4$.

ii. Dead load lower bound and wind.

$$0.9D_L + 1.5Wd$$

Using the dead weight lower bound multiply by 0.9 gives 18KN/m, therefore as the maximum wind bending moment multiply by 1.5 gives total 33KN/m, therefore the structure will need to be held down by 16KN/m weight allowing for 0.9 factor. This is equal to approximately 35KN per column.

(f) The critical load case for the horizontal guide rollers at the azimuth rail is wind.

2.10 Materials

The steel materials are grade C350 for circular and square hollow sections and grade C300 for universal beams, columns, angles, rods etc.

2.11 Painting

The steelwork painting system specified is IZS2 inorganic zinc silicate system to AS2312 with a polyurethane top coat. The polyurethane top coat is masked out for Ø 60mm at each bolt hole. So, the friction grip for the 8.8 grade bolt is able to be mobilized. This is to reduce vibration slip. The actual first coat Ameron D9 65-75 micron an inorganic zinc silicate over an abrasive blast clean 2½, 2nd intermediate coat Amercoat 385 100-125 micron with two top coats Ameron ISO free 977 polyurethane 50-75mm each.

In come cases, IZS3 inorganic silicate system only is used on components that are not visible externally on the finished enclosure. There is on top coat.

3. RINGWALL

3.1 Description

Ring wall, 8m diameter x 3.5m high, supports the dome and encloses the lower cylindrical structure below the dome hemisphere housing the three floors.

The ringwall is vertically supported by 12 support columns 150UC 23.4 (refer item 13) to resist the maximum dead weight, snow and ice, and a bending moment from wind.

Referring item 14 in figure 1 at the top the base ring wall is the main dome rail (azimuth ring beam) – Refer COF03294, consisting of a composite 200UC46 universal column section and a 200PFC bent as a band.

The base ring wall is thoroughly braced with 90x12 flat bar (refer item 15) to take the horizontal wind load as a cantilever. Please see figure 1.

3.2 Azimuth Track (Ring Beam)

(MDD03089, MDD03294) - (refer FIG1-item 14)

The vertical design load on the ring beam at the top of the 3.5m high ring wall is 35kN/m and the design horizontal load $\frac{130kN}{8\pi}+\frac{3.5}{2}x3.5kPa$ = 11kN/m

The vertical columns are spaced at approximately 2.1m centers with a 2.9m span at the door. This gives a secant length of 0.27m. The maximum vertical bending moment therefore is a maximum of 40kN-m allowing for 45kN wheel point loads 1.15m apart and 80% of simple support and a twist of 15kN-m. The proposed 200UC 46 and 200PFC composite beam, having a Zxx of 640E3, results in a bending stress of 50Mpa and a torsion stress of 45Mpa, which is satisfactory, with an expected deflection of approximately 1mm.

The horizontal bending moment on the rail was difficult to assess. For the design, a model of a simply supported beam of 6.5m span with 11kN/m was used giving 58kN-m. The 200UC 46 with a 200PFC welded parallel to it has a Zyy of 406E3 resulting in a stress of 145MPa which is satisfactory. There are three joints in the azimuth track to be site welded to allow for construction and transport.

3.3 SUPPORT COLUMNS

(MDD-03087, MDD03093, MDD03348) - (refer FIG1-item 13)

The 12 support columns, which are 3.5m high, will have a maximum vertical load of approximately 120kN under wind load and maximum dead weight, snow and ice, and a bending moment from wind of 2.5kPa of 10kN-m. The 150UC 23.4 columns selected give a 40MPa axial stress on 2980m² and a bending stress of 60MPa Zxx = 166E3 which is very acceptable. The maximum uplift per column is ultimate 35KN which is easily satisfactory.

The axial stresses at the bases of the columns will be slightly more, due to an additional 6kN ice load on the base ring wall, but are still well within acceptable limits. Also the columns will have an increased load of 70KN maximum due to bracing forces (see below). This gives a maximum total compressive stress of 63MPa which is satisfactory. The base and cap plates are 20mm plate to match the 20mm hold down bolts, both of which are sized as standard for an installation of this type. Note: ECO Engineers change order ROSP replaced by Zinc Silicate IZS2 A52312.

3.4 BRACING

(MDD03128) - (refer FIG1-item 15)

The critical horizontal wind load is braced (refer item 15) by diagonal 90x12 flat bracing between the main columns and truss bracing, two cross braces, on one side of the main double entry door. This horizontal load consists of the 2.5kPa load on the dome above the spring line of 90kN, the load on the dome to the azimuth rail of 74kN and half the load on the base ring wall of 50% of 35kN giving a total of 200kN. This force can be reduced by overall drag factor to 0.63 x 200=126kN. Refer to AS1170.2 C5.2.1 paragraph 2.

If we assume two braces are working in tension each side, the required load to be taken is 50kN maximum per brace.

Based on a maximum bracing load of 60kN, the stresses in the 90x12 flat is 55MPa. Two M20 8.8 TB bolts each end have a capacity of 62kN each working. Friction gap bolts using zinc silicate system would be better. Checking back to PANSTARS max force 60MPa x 90 x 12 = 65kN.

3.5 GIRTS

(MDD03090, MDD07990, MDD03091) - (refer FIG1-item 16)

The girts (refer item 16) are spaced at approximately 750mm maximum centers for a maximum local pressure of 3.5kPa. The maximum span is approximately 2.1m, therefore the maximum bending moment is 1.5kN-m.

The bending stress on the 100SHS 4 sections proposed is therefore only 35MPa. These members are selected to be consistent with others and to provide sufficient space for the bracing connections. They will also stiffen the base ring wall, thereby reducing both the brace and azimuth rail loads. The girts are supported using a standard M12 bolt – in - tube spanner hole detail, which has been used by EOS on several observatories.

These connections have a capacity of at least 4kN, depending on the washer diameter, the load is 3kN maximum under wind of 3.5kPa.

The inside wall girts are 100 x 6 flat and are supported on the girts at midspan. They therefore only to span 1m. Allowing 2.5kPa internal pressure gives a bending moment of 15kN-m and a stress of 260MPa, which is satisfactory for this non design condition of the doors fully open in a hurricane. At LICK the forces are approximately 70% giving a stress of 182MPa which is acceptable. Next project this should be 8mm plate giving 140MPa for 2.5kPa pressure.

4. WHEELS

4.1 Azimuth Drive/Support Wheels

(AD03623, AD07593) - (refer FIG1-item 8 and 9)

As outlined in section 2.1 above, the maximum expected vertical load on the vertical azimuth wheels is 3.0 tonne/meter length of the dome rail. The total number of vertical wheels is 22 being 20 support wheels and two drive wheels. The spacing between the wheels is approximately 1.4m therefore the maximum vertical load per wheel is 45kN.

The wheels used are 3623 Vulkollan wheel C45 Part No173/246/102/65x19.8 with whole wheel capacity is 65KN, with the weakest strength at taper bearings LM29748 with factor of safety of 1.9, allowing 0.5 factor gives 20KN for some axial loads at bearings. As the extreme loads are rare, 65KN capacity wheels are deemed easily appropriate and will handle a 2m maximum spacing without exceeding their continuous use static rating.

The design of wheel housings and axles uses standard statics to determine the plate and rod sizes. The wheel axles are sized at 38mm to suit the selected wheel, giving 45MPa shear. If we assume that the 50kN is a point load on the centre of the axle (refer to MDD03577) and that it spans 120m give 278MPa. The steel used is 4140 high tensile steel with the yield of 760 and ultimate of 860MPa. 80MPa bearing on the side rails (refer to MM07594) A lateral load of 10% of 50kN is allowed for 900 to the wheel force. This gives 119 MPa on Ø40 pivot shaft. The hinge axle is sized at 50mm OD 36mm ID tube giving 30MPa shear. The 90mm by 12mm side rails have 2.5kN-m each for the 400mm spacing between the support spring and the hinge axle, giving 155MPa bending stresses.

4.2 Azimuth Guide Rollers

(AD08103) - (refer FIG1-item 10)

Allowing for the resolved angles are proportion to the Cosine of the angle of the load to the wheel. 14 wheels therefore angle 25°, P +2Pcos25°+2PCos50° = 4P for each wheel give 35kN per wheel, indicating that 44 KN capacity wheels are acceptable. The weakest strength is at ball bearings 6208-2RS with factor of safety of 1.3, allowing 0.5 factor gives 17KN for axial loads at bearings.

The azimuth guide wheel assembly therefore is designed for a horizontal load of 40kN. The wheels are RAEDER VOGEL part no. 173/140/076/5/40. The wheel axles are held in a wheel box (refer MDD08118) with 12mm plate both sides via diameter 38mm axle sprung approx 100mm. Allowing for a point load of 40kN at the centre of the axle shaft giving 1kN-m and a stress on the shaft of 185mPa which is easily satisfactory. The bearing on the 12 plate sides is 81MPa is easily satisfactory. Bending on side plate is 40kN÷2x175÷4 is approximately 1kN-m and the stress on the 100 high 12mm plate is approximately 60MPa therefore easily satisfactory. The force on the four bolts is approximately 40kN÷4 = 10kN and therefore Ø16 8.8 bolts TF to old A51252 capacity is 22kN. Referring to MDD08104 the arm design these bolts bear on 12mm plate which is 52MPa is easily satisfactory. Bending on the main arm 150x100x4RHS is 40kNx1m÷4 = 10kN-m. Allowing for the 80x130 hole and for the 12mm platex200x100 double plates each side gives 160MPa for the RHS bending and the y axis, if the bending moment is taken by the sides only of the RHS and the two 12mm plate doubles the stress is 187MPa which is satisfactory especially as the two flanges each side are ignored. The top connection to the azimuth rail is 20kN distributed into two halves = 10kN each 12mm plate and M16 8.8 bolt is easily satisfactory for the other end the single M20 8.8 in tension takes 20kN units 87kN capacity which is easily satisfactory.

The connection to the azimuth support beam (refer to MDD08052 and MDD07826) are 12mm plate at top taking 10kN each onto the top and bottom of a box 12mm plate conservatively taken therefore on a 12m x 12m area gives only 70MPa and is therefore easily satisfactory. The bottom is supported on the 2nd floor perimeter beam with a gusset via 16mm plate shown on MDD08119. Taking a cantilever load of 20kN over 60mm vertical height gives 1.2kN-m, resulting in a stress of 351MPa over

80mm width. This plate is OK for LICK when the horizontal width force is 60%. On the level 2 the 5mm plate is too thin to take any bending so the 16mm plate bracket onto the 100SHS 4 floor beam results in approx 150MPa and the stress in the 100SHS 4 is only 15MPa therefore is satisfactory.

The supports for these wheels are currently being modified to suit the revised wheel beam which was a 200SHS which was found to be unable to be rolled and has been superseded by a 200UC46 with a 12mm side plate.

4.3 Azimuth Guide Roller Attachment

The vertical offset of Level 2 floor complicates the transfer of the horizontal base shear load from the dome structure to the base ring wall. The wheels are housed on the composite 200UC46 and 12mm side plate.

5. DOME

5.1 Description

As described in section 1.3, the dome structure consists of

(a) Large azimuth support beam

(b) Two arch beams on each side of the slit

(c) Sausages

(d) Separation ties/struts

5.2 Main Arch Beam

(MDD07811, MDD07812, MDD07813, MDD07814) - (refer FIG1-item 2)

Arch beam are located on each side of the slit and consist of a composite beam made up of 10mm thick plate, rolled 150 UC30 and stitch welded together.

Wind Load

Using 73m/s (265Km/h) (165mph) at 4146m elevation and designing to AS1170.2 Wind Load Code $_{10}$ results in a basic pressure of 3.25kPa if the installation were at sea level. As the air pressure at site elevation of 4146m is 60% of sea level, this gives a basic design wind pressure at all sites of 2.0kPa. For impinging wind, a factor of 20% is used to allow for local billowing & results in wind pressure of 2.5kPa. From AS1107.2, coefficient for external pressure on circular wall @ 90° is 1.5, therefore the suction & uplift pressure is 1.5 x 2.0 = 3.5 kPa. Two cases of FEA analyses were performed; wind direction is in parallel with the slits, and also is in perpendicular to the slits.

Snow & Ice Load

From the LICK specification, the loading of snow & ice is combined to give a design pressure of 1.2kPa from zenith to 45°. However, the wind load causes uplift on one side and downward load on the other side. Worst case scenario assumes maximum wind uplift suction 3.5kPa without the downward loads from the snow and ice.

Mechanical Forces

Each short 150 UC30 welded to Arch Beam carries 100KN vertical loads from floor hangers, which supports all the floors. Chain overlays for shutters along the tracks of Arch Beam gave 0.05MPa pressure acting radically inwards, with 40KN pulling forces at each end to maintain the tension in the chain.

Results – Refer figures 5.2, 5.3, 5.4, and 5.5

First analysis with wind direction in parallel with the slits - FEA-w1 (refer figures 5.2 and 5.4) shows maximum localized stress 157MPa at chain tensioner cutouts. The gussets at supports experience 67MPa and the bending stress is in arch bending is 70MPa. Both arch beams deflect about 10mm in the direction of the wind.

Second analysis with wind direction normal to the slits – FEA-w2 (refer figures 5.3 and 5.5) shows maximum stress 180MPa at flange of 150 UC30, welded to the 20mm end plate. The arch beam deflects about 40mm in the direction of the wind. This is considered satisfactory.

5.3 Side Support Beams

(MDD07786, MDD07787) - (refer FIG1-item 3)

These are the two large circumferential beams placed vertically and at right angles to each side of the slit at 3m centers. The approximate bending moment is taken as a 3m width, with 3.5kPa wind load, spanning 5m giving 30kN-m.

Hand calculation shows the resulting bending stress of approximately 200MPa under wind load for the selected 219CHS 4.8 is satisfactory.

Wind Load

Basic design wind pressure used is 2.0kPa. For impinging wind, local billowing wind pressure is 2.5kPa. This positive impinging pressure is used on structures where angle of wind drag is less than 40deg. The suction & uplift pressure is 3.5 kPa and is used for angle of wind drag larger than 40deg.

Results

FEA – w1 (refer figures 5.2 and 5.4) shows maximum stress is 45MPa at spring line level. 3.5kPa suction pressure causes normal relative displacement of 7.4mm from lower support to top arch beam attachments.

FEA – w2 (refer figures 5.3 and 5.5) shows maximum stress is 155MPa at the front of the dome, where there is only one separation tie between arch beams. 2.5kPa suction pressure causes relative displacement of 23mm from lower support to top arch beam attachments.

It is considered satisfactory with C350 minimum.

5.4 Other Cladding Support beams

(MDD07788, MDD07789, MDD07790, MDD07791, MDD07792, MDD07793, MDD07794, MDD08101, MDD08102) - (refer FIG1-item 17)

The same member size as the main cladding support beams has been selected for all the other cladding support beams. This is to provide common detailing, even though the bending moments in these beams are less, so stresses will be lower in these beams than in the main cladding supports.

Results

FEA – w1 (refer figures 5.2 and 5.4) shows cladding support beam that experiences the unbalance of the impinging and suction forces, the maximum stress is 40MPa. Other support beams have stress below 20MPa.

FEA – w2 (refer figures 5.3 and 5.5) shows no significant stress above 40MPa.

5.5 Strut/Ties (Arch Beam Separation Tie)

(MDD07784, MDD07785) - (refer FIG1-item 4)
The structure is kinked upwards to clear the swing radius of the telescope. These members take the horizontal wind load trying to peel the two part hemispheres apart. The load is taken down onto the main cladding support beams, approximately 1m above the reinforced Level 3 floor.

Hand calculation shows the required axial load to be taken is 50kN, making the 100x5 SHS, with an effective length of 5m satisfactory.

Wind Load

Working wind pressure 2.5kPa is used on structures where angle of wind drag is less than 40deg. The suction & uplift pressure is 3.5 kPa for angle of wind drag larger than 40deg.

Snow & Ice Load

Wind load causes uplift on one side and downward load on the other side. Worst case scenario assumes maximum wind uplift suction 3.5kPa without the downward loads from the snow and ice.

Results

FEA – w1 (refer figures 5.2 and 5.4) shows the maximum stress 160 MPa is in 150x50 RHS at the front of the dome. 100x4 SHS experiences maximum bending stress 116MPa. The mid span deflects about 20mm.

FEA – w2 (refer figures 5.3 and 5.5) shows the maximum localized stress 200MPa is in 150x50 RHS at the top of the dome. 100x4 SHS experiences maximum bending stress 165MPa. The mid span deflects about 32mm.

The FEA result on ties is conservative as any stiffness from the fiberglass cladding and seven 200x100x5 angle bars along the span, which connects to middle infill panel, have been neglected.

The grade of the RHS and SHS sections are C350 minimum, therefore 200MPa is 58% of yield and therefore is satisfactory.

5.6 Patch Plate Connections for Dome Cladding

The patch plate connectors are for the dome cladding connections (stirrups) and are placed at 800mm centers on the outside faces of the cladding support members (dome girts and circumferential beams) to provide the attachment for the fixed cladding panels $_{11}$.

The largest span of the skin is less than 3m, giving a one way span reaction for 2.5kPa at 400mm centers of 3kN. The patch plate is a 90mm x 50mm x 10 plate, with an M12 nut welded to the underside. This results in 120N/mm perimeter force on the plate and a maximum bending stress of approximately 50MPa, which is satisfactory given that local effects could double the wind forces to 5kPa.

The function of the plate is to distribute the force more evenly into the supporting skin of the 219 CHS 4.8. Allowing for a width of 100mm and an equivalent fixed ended beam for CHS of span 200mm gives a bending moment of 75E3N-mm and a wall stress in the 200 CHS 4.8 of 200Mpa, which is reasonable as the assumptions are all conservative.

5.7 Azimuth Support Beam

(AD07798, AD07799, AD07780, AD08051) - (refer FIG1-item 1)
Azimuth support beam is made as part of the level 2 quadrant, which the main function is to carry the weight of the whole rotating dome and transfer the loads to base ringwall via the azimuth wheels. It is made up of rolled 6mm plate and with 12mm local reinforced plate at arch beam support, and welded to level 2 1m spacing SHS. Each quadrant, with span 90deg, has three 12mm plate local reinforcements to increase the buckling strength, giving the factor of safety (FS) for linear buckling of 14. Please refer figure 5.1 for first linear buckling failure mode.

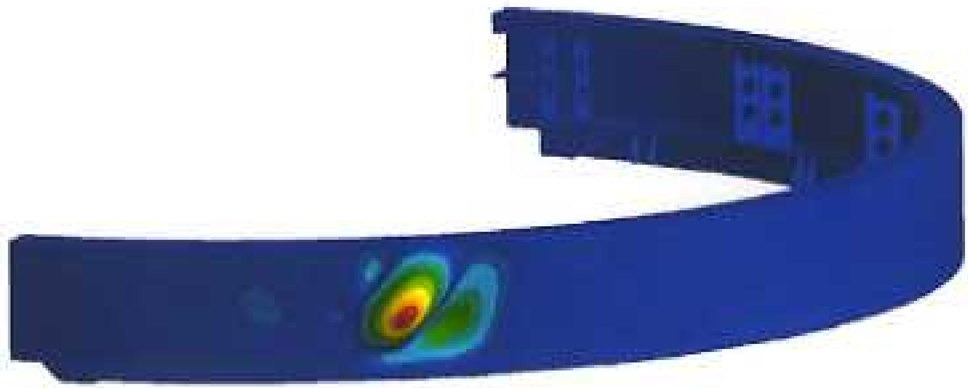

Figure 5.1: Azimuth Support Beam – first buckling failure mode with FS ~14

The horizontal force of 2.0kPa on 35m^2 above the spring line produces 70KN above 2.0m above the spring line and also on 3.3m high equivalent barrel section is taken as 60KN. Combining the two forces produces an overturning moment of 470KNm and shear of 130KN at the rail level of azimuth support beam.

Initial FEA analysis showed that with estimate 30 tonne weight of the structure on azimuth rail level, the horizontal wind load of 130KN is not sufficient to overcome the weight of the structure to cause over lifting. Further assembly analysis with spring supports 10KN/mm per support, which simulates the stiffness of the azimuth wheels, did support the finding.

However, as precautionary action, 14 circumferential hurricane hooks, which connect to azimuth guide roller and the ringwall ring beam, are welded to the azimuth support beam to stop the azimuth support beam from ever uplifting. This, in effects, if engaged, will reduce the net overturning moment to far less than calculated 460KNm maximum overturning moments.

FEA-w1 (refer figures 5.2 and 5.4) shows maximum stress 95MPa at the arch beam connection and the corner of the arch housing the driving wheel. The relative deflection is 7mm between front and rear of the azimuth support beam.

FEA-w2 (refer figures 5.3 and 5.5) shows maximum localized stress 215MPa at the main cladding support beam connection. The stress is 135MPa at the junction between perimeters vertical 6mm and bottom 12mm plate. The relative deflection is 10mm between windward and leeward of the azimuth support beam. It is considered satisfactory.

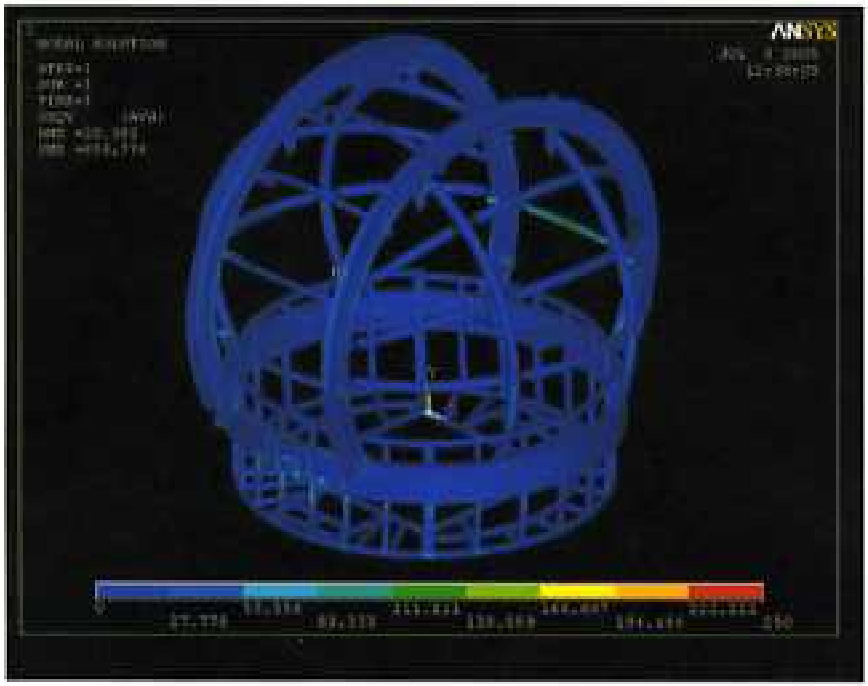

Figure 5.2.: Wind direction in parallel with the slits - FEA-w1 – Von Mises Stress

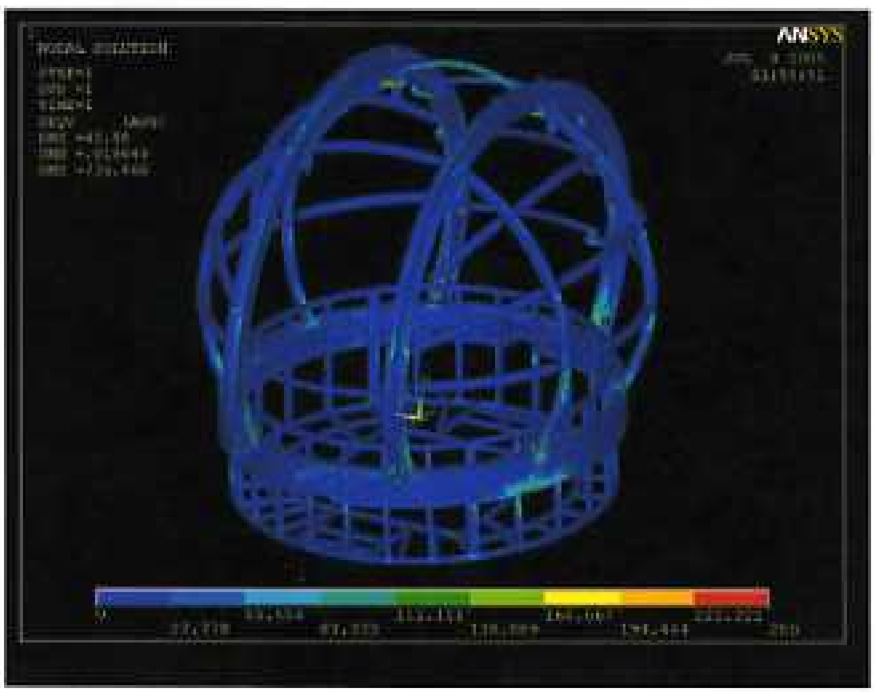

Figure 5.3.: Wind direction normal to the slits - FEA-w2 – Von Mises Stress

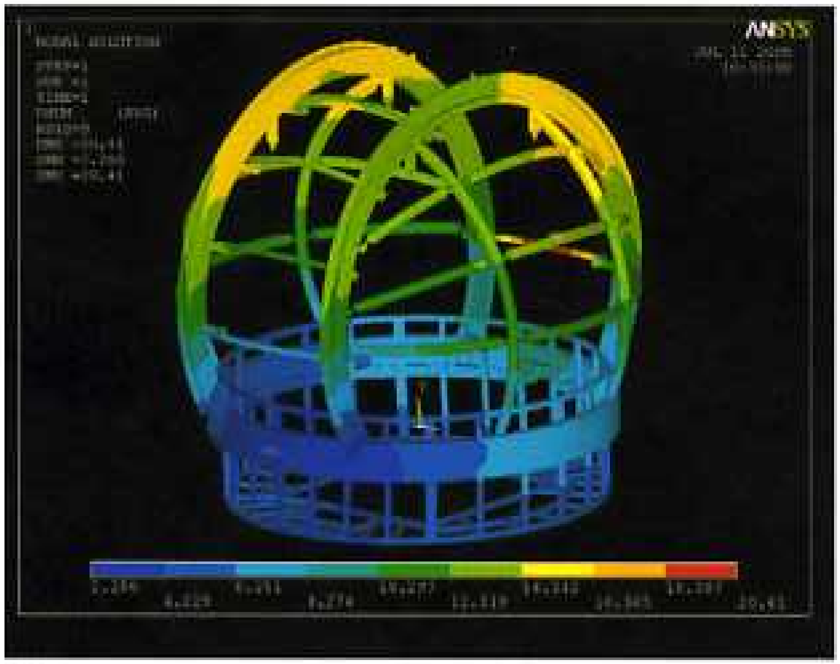

Figure 5.3.: Wind direction in parallel with the slits - FEA-w1 – Displacement Plot

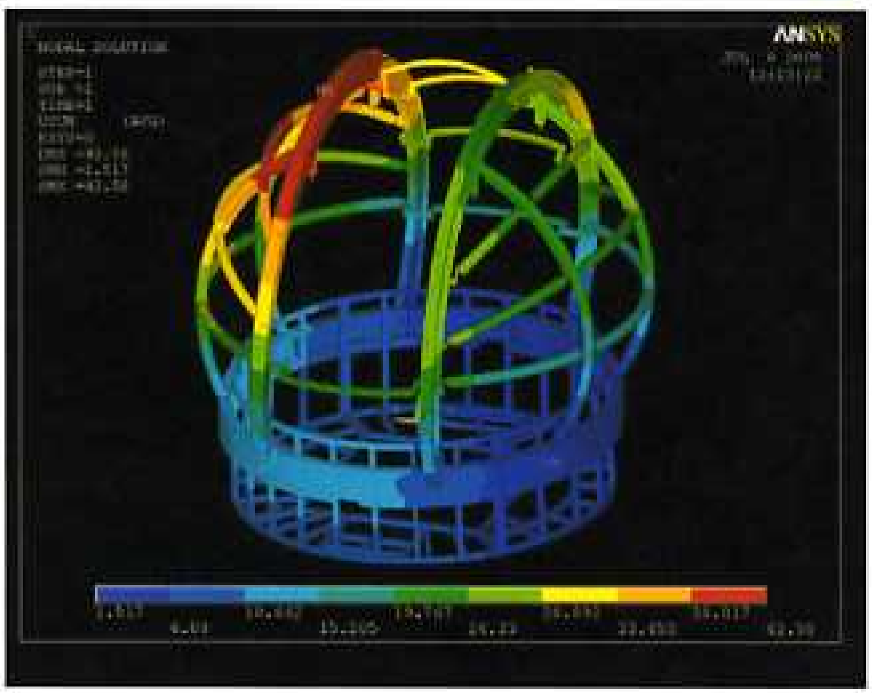

Figure 5.4.: Wind direction normal to the slits - FEA-w2 – Displacement Plot

Shutter Frame

(AD07715 and AD07716) - (refer FIG1-item 7)
This shutter frame is site specific. FEA analysis is considered conservative because it assumes wind, snow and ice load distribution act collinearly to produce the maximum in plane force at each transverse beam. Also any stiffness from the fiberglass cladding has been neglected.

Wind Load

Survival LICK wind speed is 120Mph (53m/s) for a 3 second gust. Designing to AS1170.2 Wind Load Code, this equates to a pressure of 1.7kPa, derating for altitude (5000ft) results in a basic site wind pressure of 1.36kPa. For impinging wind, a factor of 20% is used to allow for local billowing & results in wind pressure of 1.63kPa. From AS1107.2, coefficient for external pressure on circular wall @ 90° is 1.5, therefore the suction & uplift pressure is 1.5 x 1.63(note additional 20% is still used) = 2.5 kPa.

Snow & Ice Load

From the LICK specification, the loading of snow & ice is combined to give a design pressure of 1.2kPa from zenith to 45°. This 1.2kPa is used for the shutter design.

Forces

For design purposes, the snow & ice loading pressures are assumed to be acting over the full FRP surfaces & the wind loading pressure is acting over the worst case projected area. Total area is 21m^2 & gives a total of force of 58kN.

Member Sections & Strength

All material used for the shutter frame is aluminium 6061-T6 grade. This material has a UTS of 290MPa & a σyield of 241Mpa. However once welded, the parent material around the weld is no longer tempered & its strength is reduced. AS1664.2 stipulates the weld affected zone to be up to 25mm of the weld. If the member is not heat treated, the UTS of the weld affected zone falls to 165MPa & σyield is 138Mpa. If the member is heat treated, then full T6 properties are returned to all of the parent material. All tapped holes have helical thread inserts & all fasteners are zinc plated high tensile steel to class 8.8 minimum. All members are finished in a 3 coats, two pack isocyanate free coating system.

Longitudinal beams: These are a composite 180 x 90mm box section, fabricated from 6mm thick plate. These members are too long to be successfully heat treated to a T6 temper. Because of the composite fabrication, it is assumed that all of this member is weld affected & has a σyield of 138Mpa.

Transverse beams: These members are fabricated from an edge rolled 100 x 50 x 6mm RHS. A 12mm thick interface plate is welded to each end for attachment to the longitudinal beams. An additional 12mm thick reinforcing plate has been used to spread the highly localized stresses at this point. Again these members are too long to be successfully heat treated. Therefore a σyield of 138Mpa is assumed at the weld affected zones & a σyield of 241Mpa for the RHS away from this zone.

Drive plates: These plates support the drives & drive loading & torque is transferred through these plates.

They are 16mm thick plate with a 80 x 6mm SHS welded cantilever beam to support the outboard bearing.

These plates have been heat treated to attain the full T6 temper, therefore a σyield of 241Mpa is used.

Outrigger arms: These arms support the outrigger rails & are fabricated from a 50 x 6mm SHS welded to a 12mm thick interface plate. These members have been heat treated to attain the full T6 temper, therefore a σyield of 241Mpa is used.

Outrigger rails: These members support the FRP flare panels on the outboard sides. They are a rolled 75 x 6 EA. There are no welds on the rails so the T6 σyield of 241Mpa is still retained.

Critical survival condition produces an overall effective Von-Mises stress of no more than 85MPa with highly critical stress of 90MPa at the both end supports of the transverse beam. A 12mm thick aluminium reinforcing plate is welded at each corner between transverse and longitudinal beams to ensure the spread of the highly localized stresses. This gives a SF of 1.5 at this highly localized point. Transverse beams experience 40MPa bending tensile and compressive stresses at bottom and top surface respectively. The maximum deflection is approximately 28mm at mid span. For comparison, hand calculation results for a straight, both ends fixed, uniformly loaded beam are, 65Mpa at the fixed ends & 32 Mpa at mid span. This compares favourably with the FEA results as shown in figure 5.6, away from the highly localized stress area, as the longitudinal beams are allowing slight rotation at the ends.

Figure 5.6.: Shutter Survival Condition – Von Mises Stress

The outriggers arms, which are made of the 50x6 RHS, experience 85MPa bending tensile and compressive stresses at supports. Hand calculation shows, assuming even force distribution over the 4 arms each side, the stress in the arm is 62MPa.

The shutter drive has been designed so that the shutters are still able to operate on one chain only. For this condition full drive forces & torque are been transferred through one drive plate only. Single chain operating condition with 800Nm torque produces maximum effective stress of 150MPa as shown in figure 5.7. This results in a SF of 1.6 & is satisfactory in the unlikely event of failure condition. Hand calculations show a stress of 66MPa in the cantilever arm. Operational condition is considered conservative as working stress are much lower than this critical stress.

Figure 5.7.: Shutter Operating on one chain – Von Mises Stress

For the shutter uplift condition, the dead weight of the shutter itself has been neglected. Shutter uplift condition produces critical tensile stress of 90MPa in hurricane hook. The transverse beam experiences 65MPa tensile and 60MPa compressive stresses on top and bottom surfaces respectively as shown in figure 5.7. This results in a SF of 2.7 for this condition.

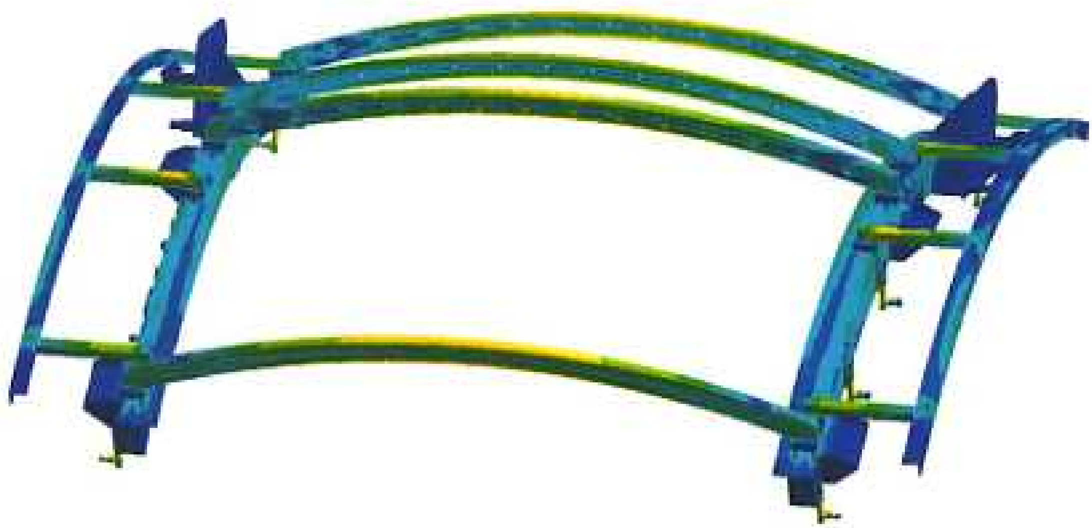

Figure 5.8.: Shutter uplift condition – Von Mises Stress

5.8 Vent Door Frame

A similar construction is proposed for the vent panel frames as is used for the shutters. Wind forces will be assessed by transform Composites design team as the detail design for these components are finalised.

6. FLOORS AND MISCELLANEOUS

6.1 DESCRIPTION

The top "observing" floor (level 3) is approximately 0.5m above the top of the Azimuth support beam, the intermediate "maintenance" floor (level 2) is approximately 0.75m below the rail and the bottom "equipment" floor (level 1) is approximately 2.5m lower again. All floors are part of the dome structure and co-rotate with the telescope.

Level 2 floor is the main support floor, and it is supported on the azimuth support beam on the outside and the four floor hangers on the inside. The level 3 floor is supported on the level 2 quadrant on the outside and the floor hangers on the inside. The level 3 is also supported onto main arch beam and the main sausages. This also ties the sausages onto the structure and reduces the stresses.

The Level 1 floor is also hung from these hangers and from a row of columns suspended from the level 2 floor above.

Stairs from each level connect each floor and removable hatches are provided in level 3 floor.

6.2 LEVEL 3 "OBSERVING" FLOOR

(AD07756, MDD500067-MDD500079, MDD500094-MDD500100)
(AD500187, MDD500185, MDD500186) - (refer FIG1-level 3)

This floor consists of serrated Expamet Preslock $_{TM}$ flooring code P30B-325 to allow free passage of air from the silt to the vent doors. This steel grating flooring consists of 32mm x 5mm serrated vertical load bars at 30mm centers, connected with 6mm twisted cross bars at 50mm centers. The serrations make the floor non-slip, even at temperatures below freezing.

The recommended maximum span is approximately 900mm. The grating weight is approximately 48Kg per m ^{2}.

The support beam maximum span is approximately 3m, with a maximum of 1m of floor supported. Using a 3kPa live load plus 3kN point load gives a maximum bending moment of 5.6 kN-m. The selected 100 SHS 4 floor beams result in a bending stress of 124MPa and a deflection of approximately 10mm.

All quadrants are bolted with at least three 3xM12 bolts, which are sufficient to overcome the 3kPa live shearing loads.

The quadrant supports door hatches are made of the curved 100x50x4 RHS, 100x4SHS and 100x10EA bar. Design moment by 3KN point load and 3kPa live load on door hatches is 2.25KNm, which gives bending stress on the curved beam to 85MPa. This quadrant is connected to neighboring quadrants by 4xM12 bolts, which is required to counteract the moments. This quadrant also has the welded supports for gas struts to assist the opening and closing of the door hatches, which weighs approximately 80kg each.

For M1 mirror maintenance purpose, level 3 quadrants are designed with mirror removal rail to carry its 6 tonne weight. Design takes into consideration that 5 tonne of its weight could rest solely on any one rail of the quadrant involved. This load is then spread through to the floor hangers, which support the level 3 quadrants, in addition to the columns down to level 2 quadrant and then to the azimuth support beam.

To spread the short term 60KN M1 mirror point load more efficiently to two of the floor hangers, a transverse beam with 3.2m span between the floor hangers. 60% of short term 60KN point load multiply with dynamic factor 1.5 gives 55KN concentrated load gives 78KN-m bending moment with 3.0m span between floor hangers. This produces a working stress of 178MPa, which is considered satisfactory.

50KN M1 mirror concentrated load, 3KN point load and live load of 3kPa over an area of 3.75m^2 produces 38KNm bending moment with 2.5m span. To carry these loads, two 100x4 SHS and a 100x9 SHS, which are bolted at six places, are required to form a composite beam with section modulus Z_e 225.6x10^{3}mm^3, giving the bending stress of 167MPa. The mid span deflection is calculated to be 4.9mm.

10mm plate reinforcement on both ends of 100x4 SHS is required for connection with the mirror rail removal support. High tensile and hardened alloy steel rod AS1444 4140 with yield strength of 680MPa, 850MPa tensile strength is used as pin connection between quadrant and removal support, giving the shear stress of 105MPa and bending stress of 70MPa for 27mm diameter size. A minimum of 7mm gap between ends plate of 100x9SHS and mirror removal support is allowed for simply supported condition.

The Level 3 floor also takes a large horizontal load from the dome wind load. The corners between the four posts and the outside walls have been braced to distribute this load into the structure and the floor panels integrated to the largest possible extent, consistent with the shipping dimensional constraints. The Webforge grating will also distribute this lateral loading.

6.3 Level 2 "Maintenance" Floor

(AD07713, MDD07922, MDD07923, MDD08091-MDD08096) - (refer FIG1-level 2)

The flooring for this floor is 25 mm Structural ply, which is waterproof, plastic coated plywood and attached to the floor beams with min M6 Teks $_{TM}$ at 150mm centers. This will brace up this floor well. The maximum span is 600mm resulting in a bending moment due to the 3kPa and 3kN floor load of 0.6kN-m over a 600mm length. This results in a 20 MPa short term stress, and deflection of approximately 3mm. The supporting beams again span approximately 3m with 1m of contributory floor width making 100SHS 4 beams, as selected for Level 3 floor, satisfactory.

6.4 Level 1 "Equipment" Floor

(AD07712, MDD07775, MDD07776, MDD07779, MDD07780) - (refer FIG1-level 1)

This floor is also attached to the four main columns. It consists of similar 100 SHS 4 floor joists at 500mm centers, with 25 mm Structural ply flooring as for the level 2 floor.

The ply provides good lateral bracing. The joists are supported by 100 SHS 4 curved beams to provide a ring floor approximately 1m wide from 4m to 6m diameter. These beams are held up to the level 2 floor at the outside perimeter with eight 100 SHS 4 hangers, equispaced at 45 degrees.

No lateral bracing is required due to the stiffness of the four central hangers (150SHS).

6.5 MAIN FLOOR HANGERS

(MDD500004-MDD500007) - (refer FIG1-item 6)

Floor hangers held all floors from arch beam at 4 places, and carry upper bound of 100KN each one, which making 150x5 SHS with 35MPa acceptable.

Two Floor hanger are increased to 150x9 SHS thick to allow for the bending stress 170MPa as it experiences 60KN load upon impact from the telescope, given the full rotational velocity of telescope is 4Deg/s with azimuth moment of inertia of 42211.86kgm^2 and effective dynamics mass of 13,362kg.

With 2.5m span between level 2 and level 3, this creates 34.5KNm design moment at impact. However, this estimate is fairly conservative as the telescope will hardly reach its full rotational velocity in service before hitting the floor hanger.

6.6 STAIRS

(AD07848, MDD500108) - (refer FIG1-item 18)

The stairs from level 2 to level 3 are constructed with serrated Expamet Preslock $_{TM}$, flooring code P30B-325 for safety in icy conditions, with 200 x 12 plate stringers segmentally curved to suit the inner and outer radii.

Handrail posts are welded to the PFC stringers at maximum 1.5m centers to support the 42OD x 4 CHS handrails.

6.7 SERVICE PLATFORM (BALCONY)

(AD07841, MDD07816, MDD07817, MDD500127, MDD500128, MDD500153, MDD500154) - (refer FIG1-item 12)

A 1.5m x 4.3m service platform is provided for servicing the vents and removing M1 mirror cell. The balcony is double the area initially proposed. The size had been increased as a result of a review of door handling by staff.

Entrance balcony covers an area of 10.7m^2. Total vertical distributed load from snow, ice solid infill, solid ice falling from top of dome, and live load produces a total of 6kPa.

50KN point load from MI mirror with bending moment 28.5KNm is supported by 150x100x6 RHS, giving the bending stress of 250MPa. This hand calculation is very conservative as two others members in parallel will reduce the bending stress.

The hand rails have been designed to clear the rotating dome by at least 100mm clearance.

Distributed load over 1.25m^2 area, cantilever 1.0m away from the balcony support, produces local load of 7.5KN. 150x100x4 RHS gives 115MPa, which is satisfactory.

100x4 SHS beam at centre section carries 3.5KN/m over 3.0m span, which gives total load of 10.5KN and 3.0KN point load. This produces bending moment of 6.25KNm and bending stress of 140MPa. For 3.5m span, bending moment is 8.2KNm. 150x100x4 is satisfactory with 103.8MPa.

This balcony also constructed with serrated Webforge $_{TM}$, flooring code P30B-325 for safety in icy conditions, and to prevent snow build up. The balcony frame is made up of 100 SHS 4 sections and handrail posts of the same size are provided to take the bending moment for the extreme condition of wind local pressure on a **solid** infill (should this be fitted in the future). This also guarantees safety from build up of snow and ice, which may act as a solid infill.

The verandah frame also supports the intake plenum (FCU) for the level 1 ventilation system, as described in the DAR "Cooling and Ventialtion"[12].

6.8 APF M1 Mirror Removal

(AD500129, MDD500130, MDD500131) - (refer FIG1-item 11)

For worst case situation, structure is selected to take on 5 tonne weight of the M1 mirror. 50KN point load at mid span of main support beam produces 23.75KNm bending moment. 150x100x6 is selected with bending stress of 145MPa tensile and compressive. For 5mm deflection design, the minimal second moment of area needed is $7.15 \times 10^6 mm^4$, the selected 150x100x4 has $8.17 \times 10^6 mm^4$.

The vertical structure is made of 100x4 SHS with 550KN axial compression strength, which is far larger than the design 50KN axial load. This is considered satisfactory.