EOS SPACE SYSTEMS

University of California Lick Observatory

Automated Planet Finder Enclosure

DESIGN ANALYSIS REPORT

STRUCTURAL ENGINEERING

CI No. DN – 07753-01

ELECTRO OPTIC SYSTEMS PTY LIMITED
111 Canberra Avenue Griffith ACT
A C N 008 587 451
TELEPHONE +61 2 6222 7900
FAX +61 2 6299 7687

DOCUMENT CONTROL

Prepared	Signed John Skurr / Francis Teng	Date 17 August 2005
Checked	Signed Adam Seedsman	Date 19 August 2005
Approved	Signed Adam Seedsman	Date 24 August 2005

Issue	**Date**	**Description**	**Checked**	**Approved**

Index	

CONTENTS

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 FS02985-01$_2$, 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.

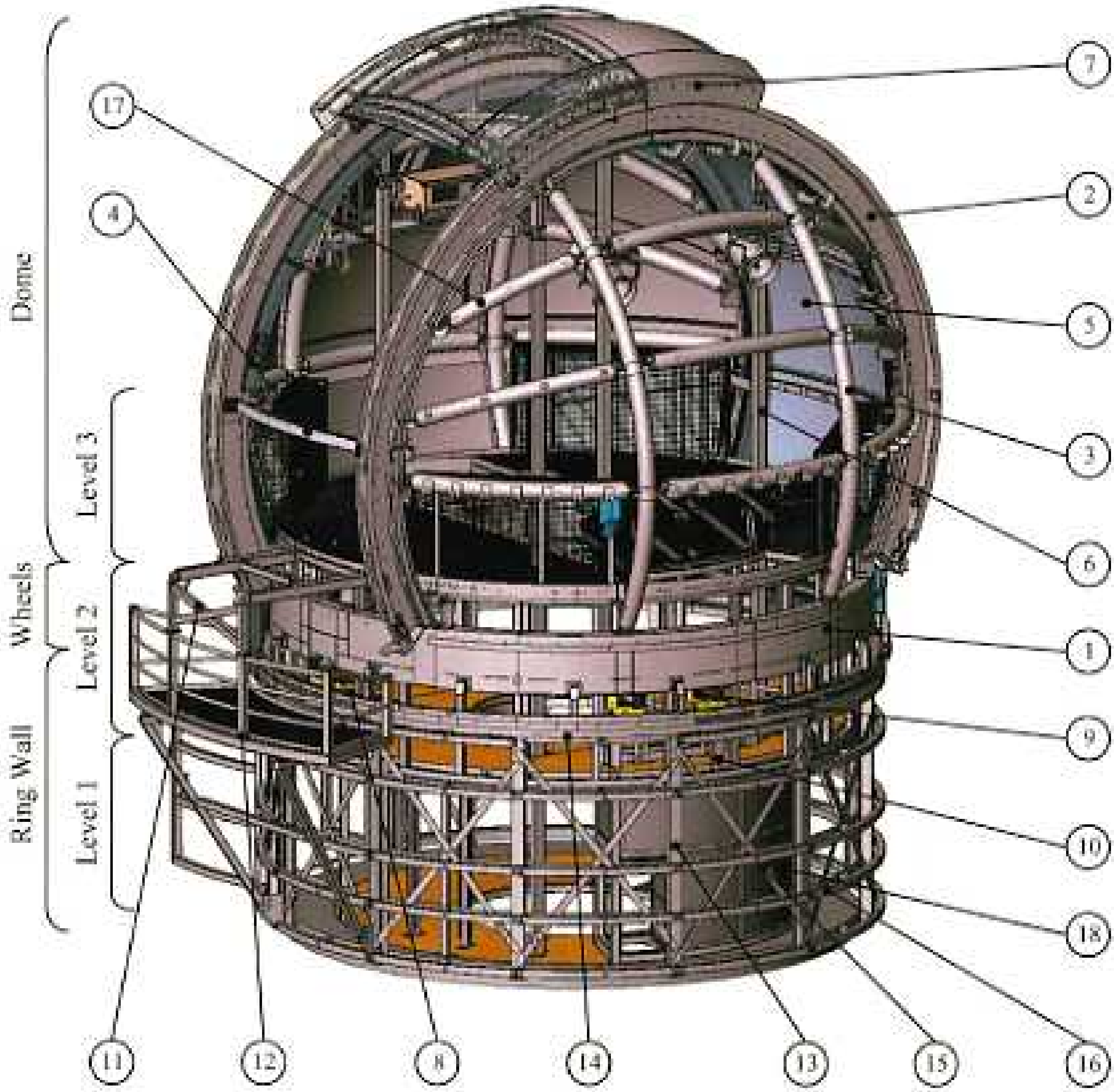

Figure 1.: LICK ENCLOSURE – STRUCTURES OVERVIEW

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 (S_D)

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