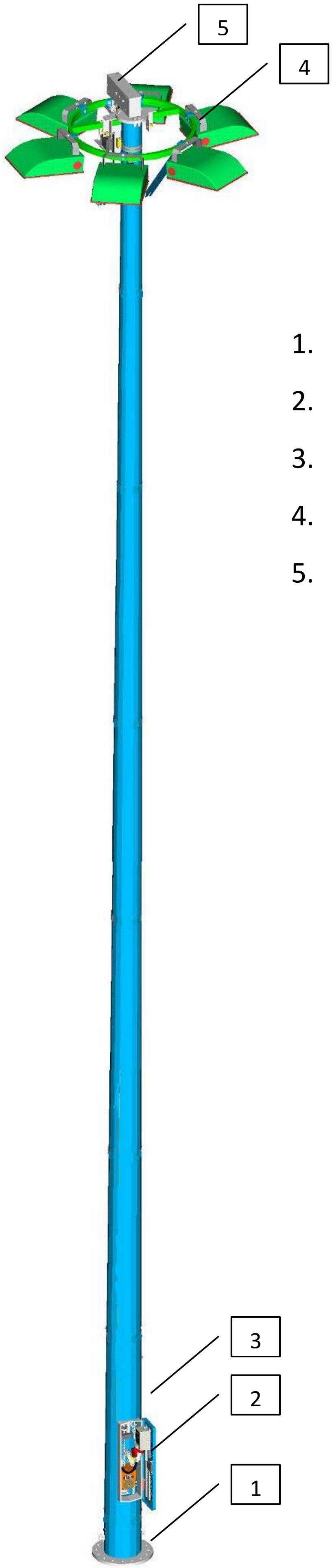
HIGH MAST WITH MOBILE PANNEL AND TROLLEY CHAIN ENGINE



1. Base plate
2. Inspection door
3. Interlocked switch
4. Mobile ring
5. Fixed head

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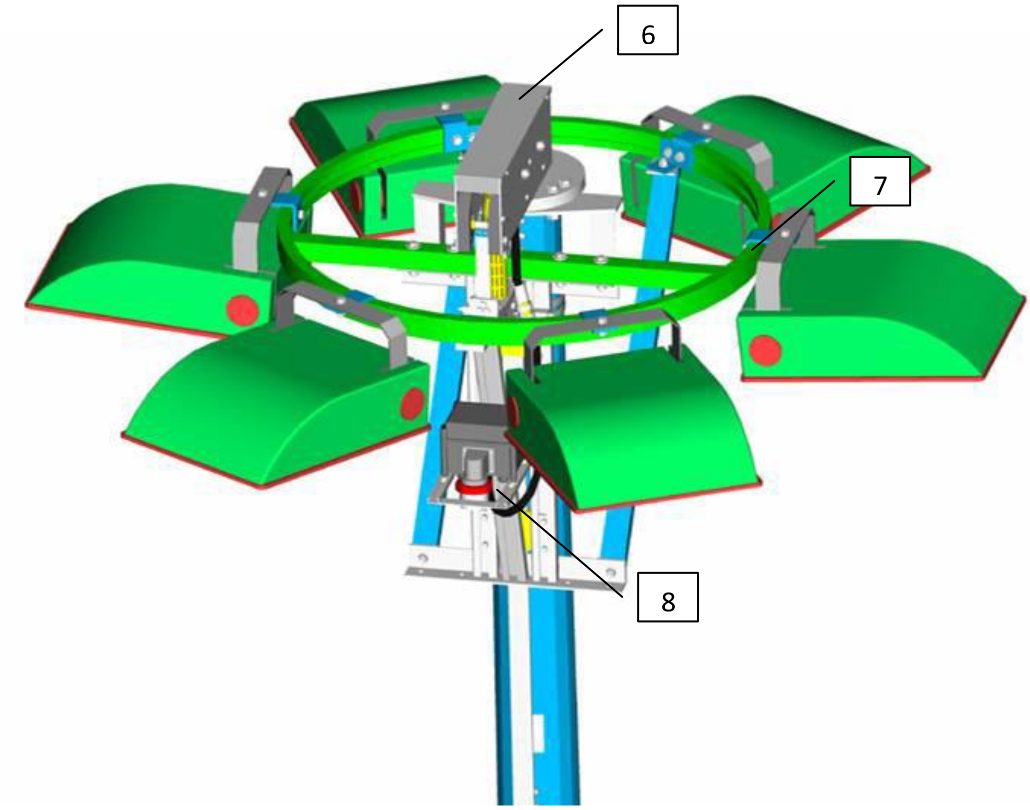
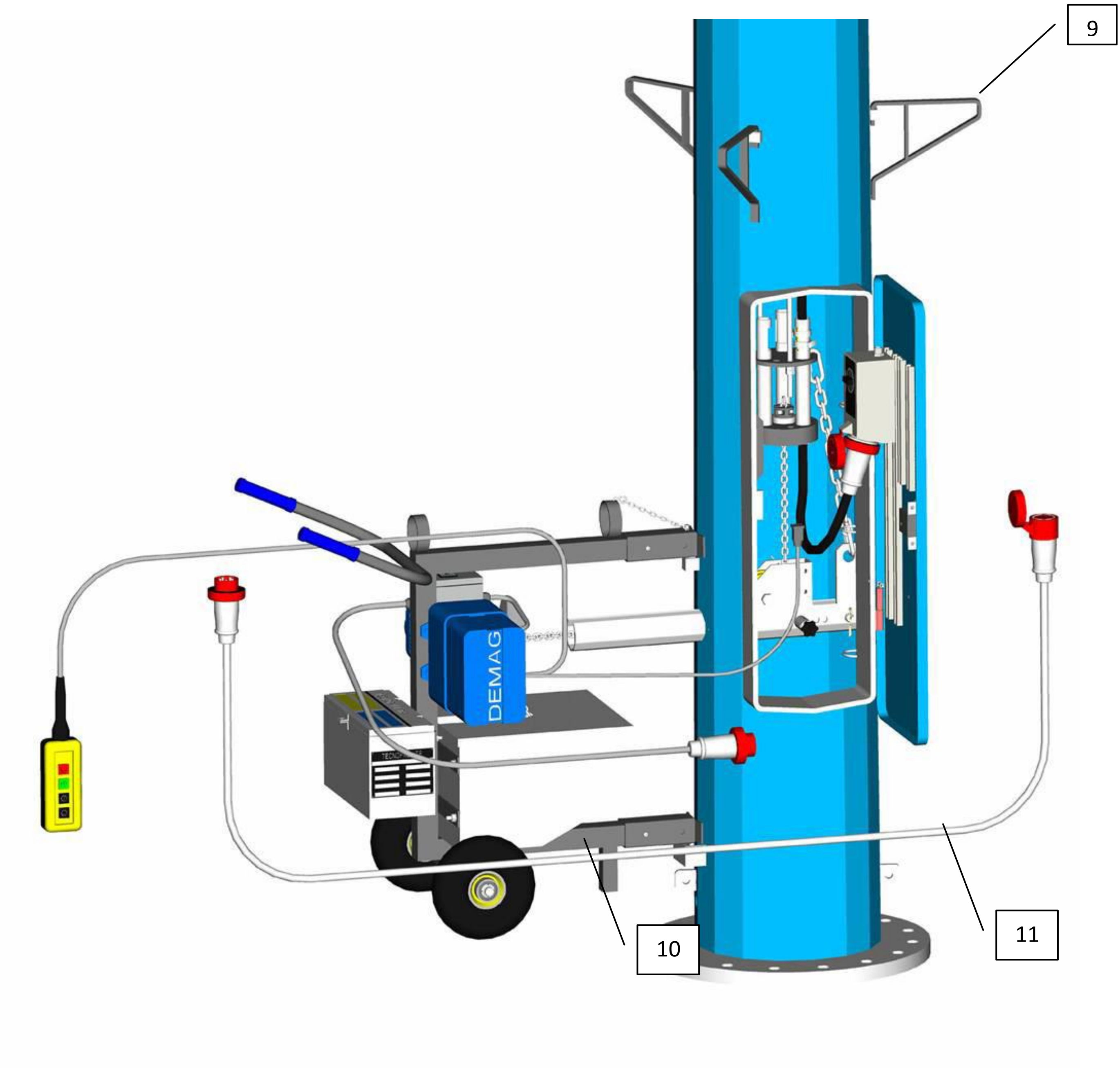
Top system



6. Fixed head
7. Mobile ring
8. Electric box

CHAIN HOIST TROLLEY-MOUNTED (EXTERNAL)

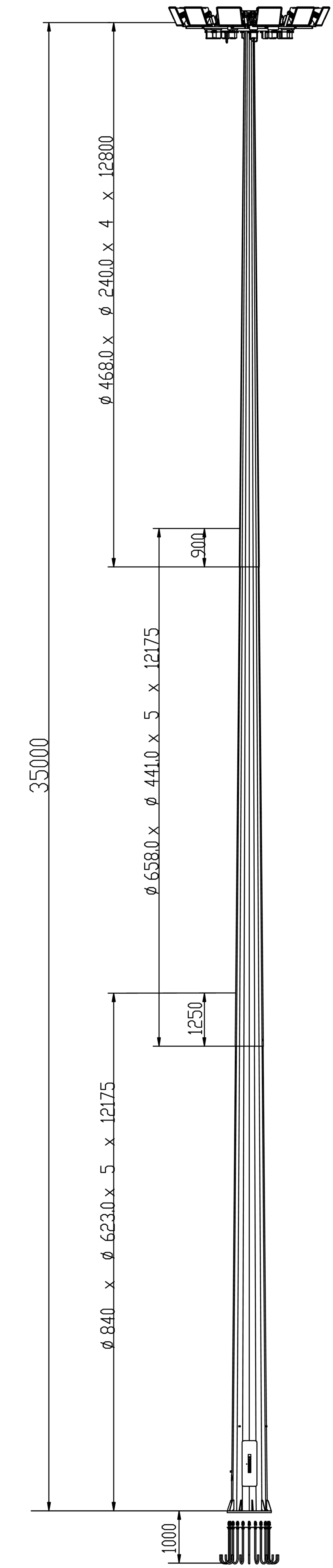


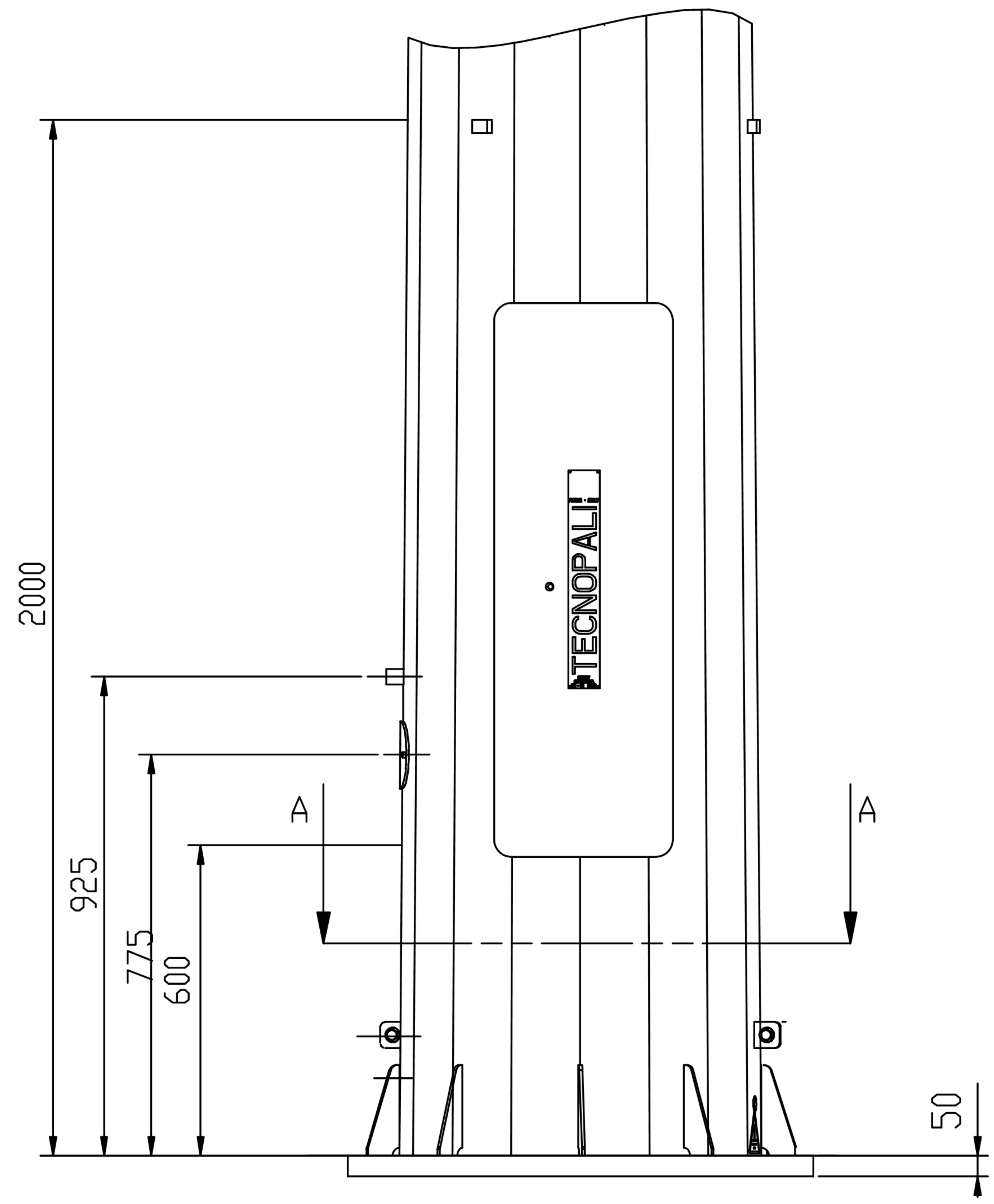
9. Mobile ring support arm (optional)

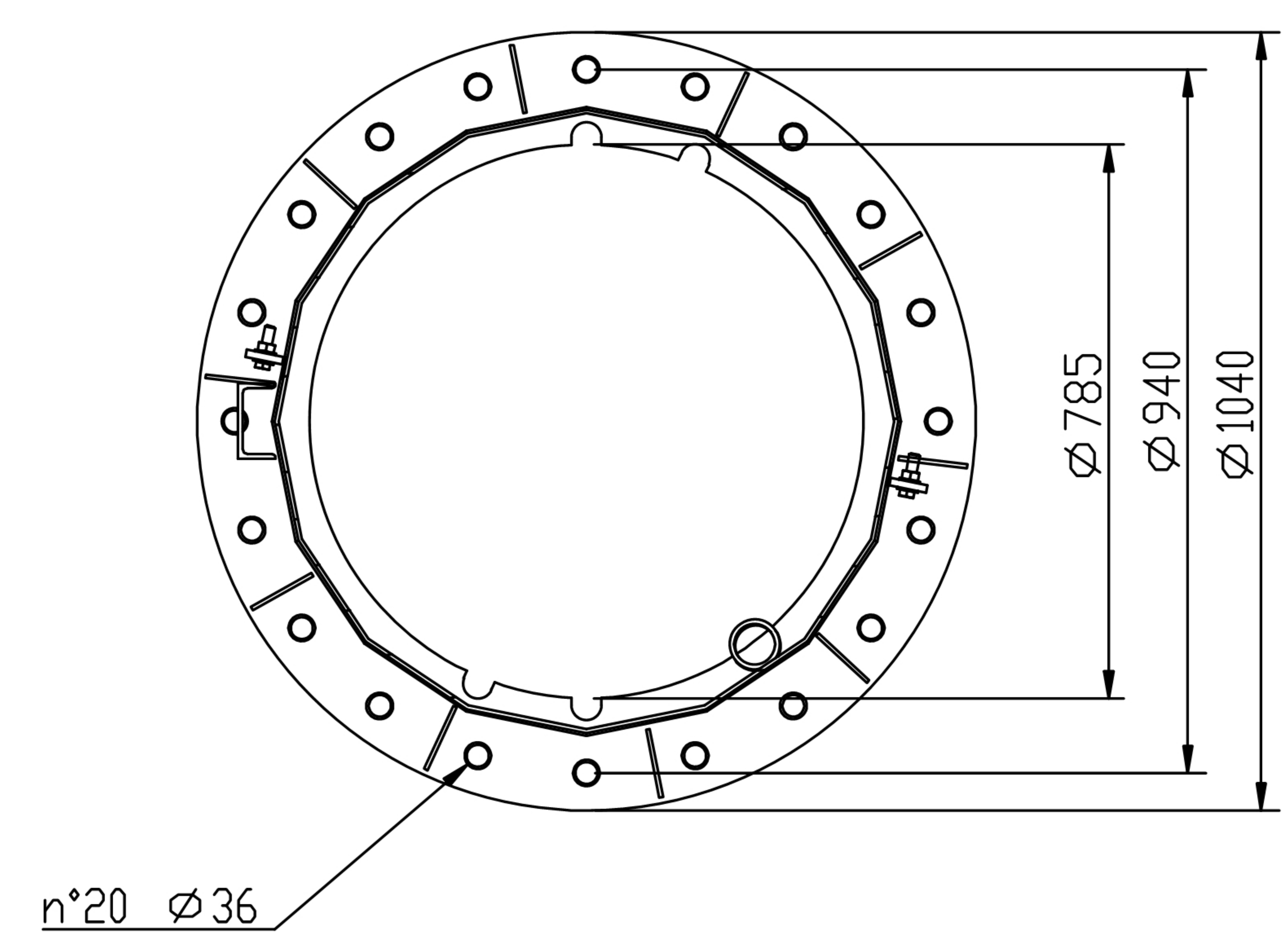
10. Trolley unit supports

11. Ground testing cable

PRELIMINARY DRAWING



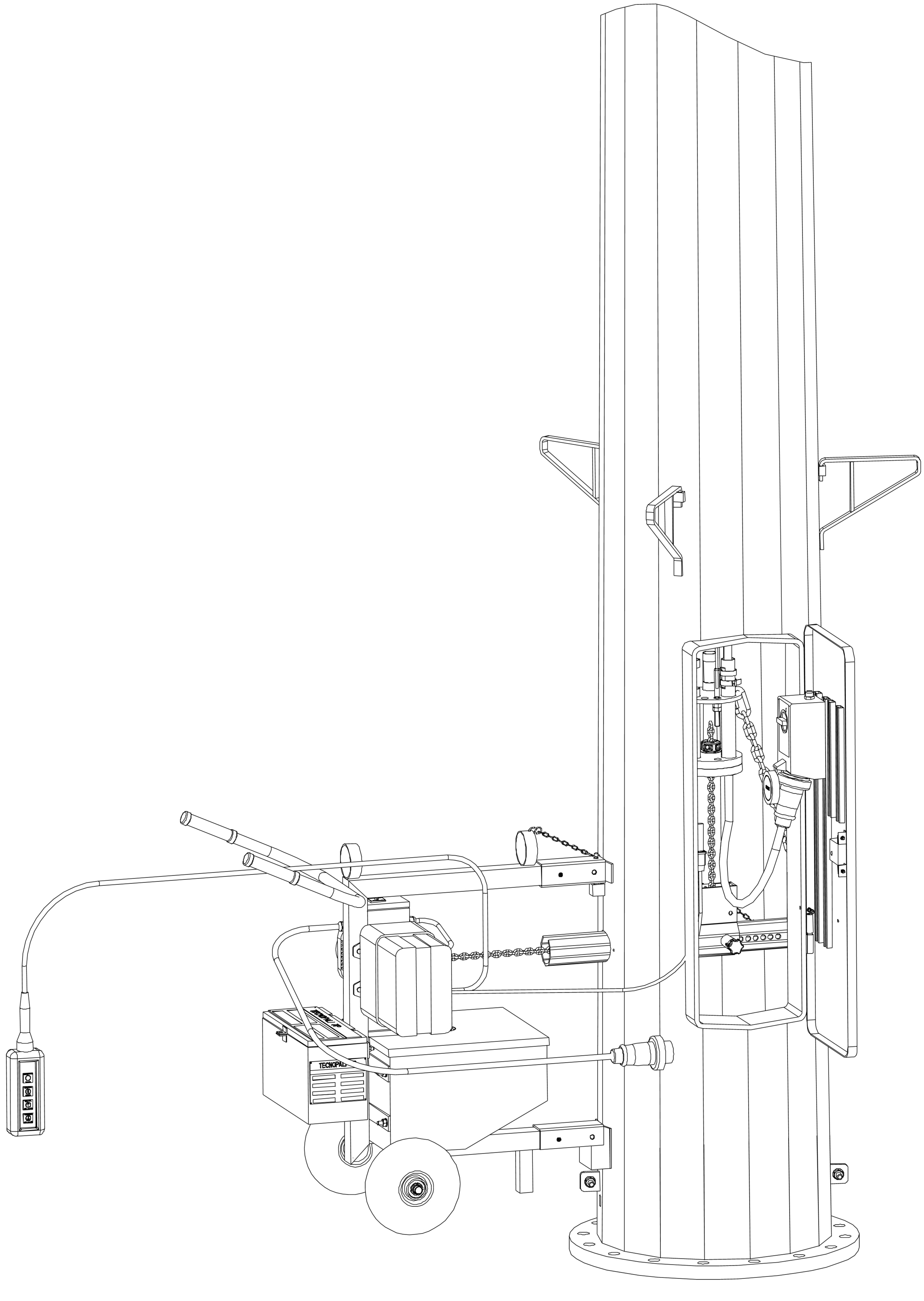




SECTION A-A

Dimensioni in mm

TECNOPALI
NORTH EUROPE



TECNOPALI
NORTH EUROPE

4V

Site: Europe
Wind speed: 24 m/s
Cat.: II

Verification of Stability

HIGH MAST R/L H=35m SUITABLE FOR N.8 Secom Invictum L FLOODLIGHTS on 360°

Verification of Stability Nr. 15748W – Rev 0
Project Nr. 39150 – Rev.2

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1. FOREWORD

1.1 Introduction

This document contains stability verifications of self-supporting monotubular steel pole, height 35m with tapered cross section, obtained by folded steel plates longitudinally welded. The cross section obtained is a regular polygon with 16 sides.

The mast consists of three sections site jointed with the “slip on joint” system.

Equipment: top R/L device with a maximum of 8 floodlights (Secom Invictum L S6400 58 126 74 ERA 3501, 35 kg; max 0.6 m²) installed on 360°.

Restraint to foundation by means of base plate and anchor bolts.

1.2 Dimensional parameters, structural materials, wind parameters

The structure is made in sheet steel, and overlapping is made by "slip-on joint" system; the mast has a polygonal 16 sides cross-section. The characteristics of the structure are the following:

Nominal height (mm)	35000
Steel characteristics	
Specific weight (kN/m3)	78.5
Young’s Modulus (MPa)	210000
Poisson’s Modulus	0.3
Yield limit stress (fyk) (MPa)	355
Characteristics for wind pressure calculation:	
Wind speed:	24 m/s
Height of site:	0m
Terrain category	II

Geometric characteristics of the shafts in this structure are:

High mast = 35 m

Shaft (#)	Øtop (mm)	Øbase (mm)	Lenght (mm)	Overlapping (mm)	Thickness (mm)	Shaft weight (kg)
3	240	466	12800	900	4	430
2	442	657	12175	1250	5	798
1	625	840	12175	0	5	1066

2. REGULATIONS

EUROCODE 1 – ACTIONS ON THE STRUCTURE - EN 1991-1-4:2005 Part 1-4: General actions - Wind actions

EUROCODE 3 – DESIGNING STEEL STRUCTURES - EN 1993-1-1:2005 Part 1-1: General rules and rules for buildings

EUROCODE 3 – DESIGNING STEEL STRUCTURES - EN 1993-1-5:2007 Part 1-5: Structural elements in sheet

3. CHARACTERISTICS OF MATERIALS

- Shaft: S355 EN 10025
- Base plate: S355 EN 10025
- Anchor bolts: S355 EN 10025
- Restraint to the foundation: flanged
- Welding procedures according to EN ISO 15614
- Surface protection: hot galvanization according to EN 1461
- Concrete assumption C 25/30

Al materials used will comply with EN 10025 standards.

4. METHOD OF CALCULATION

In this design calculation we have considered "semi-probabilistic limit state method."
The vertical and horizontal forces have been determined according to 'Eurocode 3.
In particular they are:

- Structural permanent loads (Steel polygonal mast weight)
- Not permanent structural loads (head frame, mobile crown, floodlights, accessories)
- Variable loads (wind pressure)

Horizontal loads due to wind have been determined in the following way:

$$\text{Wind pressure: } p = q_r \times c_e \times c_p \times c_s c_d$$

Where:

- q_r = reference kinetics pressure
- c_e = exposure coefficient
- c_p = form coefficient
- $c_s c_d$ = dynamic coefficient, which takes into account the reductive effects associated with the non-contemporaneity of the maximum local pressures and the amplifying effects due to the dynamic response of the structure = 1.221 (EC3)

The kinetics pressure reference q_r (in N/m²) is given by the expression:

$$q_r = 0.5\ \rho\ v_r^2$$

where:

- v_r = reference wind speed (m/s)
- ρ = air density conventionally assumed constant and equal to 1.15 kg / m³

The exposure coefficient c_e depends on the height above ground (z) of the considered point, on land topography, and on the exposure category of installation site.

- $c_e(z) = k_r^{\ 2}\ c_t \ln(z/z_0)\ [7+ c_t \ln(z/z_0)]$ if $z \geq z_{min}$
- $c_e(z) = ce\ (z_{min})$ if $z < z_{min}$

The force is determined by multiplying the pressure p with exposure surface of exposure of the wind.

Wind curve, according to EC3:

Height above ground (m)	C_e (z)	V_{ref} (m/s)	$q_{b,0}$ (daN/m²)	$Q_{ref,0}$ (daN/m²)
1	1,42	24,00	36,00	51,24
2	1,42	24,00	36,00	51,24
3	1,64	24,00	36,00	59,03
4	1,80	24,00	36,00	64,82
5	1,93	24,00	36,00	69,46
6	2,04	24,00	36,00	73,34
7	2,13	24,00	36,00	76,69
8	2,21	24,00	36,00	79,64
9	2,29	24,00	36,00	82,29
10	2,35	24,00	36,00	84,68
11	2,41	24,00	36,00	86,87
12	2,47	24,00	36,00	88,90
13	2,52	24,00	36,00	90,77
14	2,57	24,00	36,00	92,52
15	2,62	24,00	36,00	94,17
16	2,66	24,00	36,00	95,72
17	2,70	24,00	36,00	97,18
18	2,74	24,00	36,00	98,57
19	2,77	24,00	36,00	99,90
20	2,81	24,00	36,00	101,16
21	2,84	24,00	36,00	102,36
22	2,88	24,00	36,00	103,52
23	2,91	24,00	36,00	104,63
24	2,94	24,00	36,00	105,70
25	2,96	24,00	36,00	106,73
26	2,99	24,00	36,00	107,72
27	3,02	24,00	36,00	108,68
28	3,04	24,00	36,00	109,61
29	3,07	24,00	36,00	110,50
30	3,09	24,00	36,00	111,37
31	3,12	24,00	36,00	112,22
32	3,14	24,00	36,00	113,04
33	3,16	24,00	36,00	113,84
34	3,18	24,00	36,00	114,61
35	3,20	24,00	36,00	115,37

Where:
Z = height from the ground,
C_e = factor Exposure,
v_{ref} = wind speed reference,
q_{ref} = Wind reference pressure,
$Q_{ref,0} = q_{ref} \times C_e$.

5. LOADS ANALYSIS

5.1 Loads

VERTICAL LOADS - Equipments			
Type of load	Level (m)	Weight (daN)	Wind surface (m²)
R/L device + n.8 floodlights	35,00	700	5,10

VERTICAL LOADS - Shafts		
F(daN)	Level (m)	Type of load
1065,54	5,46	Shaft 1
797,65	16,56	Shaft 2
429,92	28,60	Shaft 3

HORIZONTAL LOADS - Wind on Shafts		
F(daN)	Level (m)	Type of load
495,12	5,46	Shaft 1
518,65	16,56	Shaft 2
425,28	28,60	Shaft 3

HORIZONTAL LOADS - Wind on Equipments		
F(daN)	Level (m)	Type of load
718,25	35,00	R/L device + n.8 floodlights

The above wind pressures already consider pressure coefficient (form coefficient) and dynamic coefficient. Forces at the considered levels are calculated multiplying wind pressure per exposed surface to wind.

Considered floodlights: n°8 floodlights Secom Invictum L S6400 58 126 74 ERA 3501, 35kg; max 0.6 m², installed on 360°.

5.2 Forces combination applied

The verification has been developed taken in consideration following formula:

$$F_d = \gamma_g G_k + \gamma_q Q_{1k} + \gamma_q \psi (Q_{2k} + Q_{3k})$$

where:

- G permanent loads;
- Q variables loads;
- γ_g factor amplifying permanent loads;
- γ_q factor amplifying variables loads.
- Ψ combination factor

LOAD CASE U.L.S : G x 1,35 Q x 1,5

Perm. Vertical Forces (G, daN)	Amplif. Coeff.	Permanent vertical forces (G * Coeff. Amplif. , daN)	
700,00	1,35	945,00	CONCENTRATED V.LOADS
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	
0,00	1,35	0,00	DISTRIBUTED V.LOADS
0,00	1,35	0,00	
1065,54	1,35	1438,47	SHAFTS WEIGHT
797,65	1,35	1076,83	
429,92	1,35	580,40	

Var. Horizontal Forces (Q , daN)	Amplif. Coeff.	Variable horizontal forces . (Q * Coeff. Amplif. , daN)	Height above ground (m)	Partial moment (daN*m)	
718,25	1,50	1077,38	35,00	37708,24	CONCENTRATED H.LOADS
0,00	1,50	0,00	34,00	0,00	
0,00	1,50	0,00	30,00	0,00	
0,00	1,50	0,00	26,00	0,00	
0,00	1,50	0,00	23,00	0,00	
0,00	1,50	0,00	20,00	0,00	
0,00	1,50	0,00	18,00	0,00	
0,00	1,50	0,00	16,00	0,00	
0,00	1,50	0,00	12,00	0,00	
0,00	1,50	0,00	10,00	0,00	
0,00	1,50	0,00	0,00	0,00	DISTRIBUTED H.LOADS
0,00	1,50	0,00	0,00	0,00	
495,12	1,50	742,68	5,46	4056,86	FORCES HORIZONTAL ON SHAFTS
518,65	1,50	777,97	16,56	12885,13	
425,28	1,50	637,92	28,60	18244,63	

LOADS ON FOUNDATION		
MOMENT =	**76633,79**	**daN*m**
SHEAR =	**3235,95**	**daN**
AXIAL LOAD =	**4040,70**	**daN**

LOAD CASE S.L.S.: G x 1,0 **Q x 1.0**

.

Perm. Vertical Forces (G, daN)	Amplif. Coeff.	Permanent vertical forces (G * Coeff. Amplif. , daN)	
700,00	1,00	700,00	CONCENTRATED V.LOADS
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	
0,00	1,00	0,00	DISTRIBUTED V.LOADS
0,00	1,00	0,00	
1065,54	1,00	1065,54	SHAFTS WEIGHT
797,65	1,00	797,65	
429,92	1,00	429,92	

Var. Horizontal Forces (Q , daN)	Amplif. Coeff.	Variable horizontal forces . (Q * Coeff. Amplif. , daN)	Height above ground (m)	Partial moment (daN*m)	
718,25	1,00	718,25	35,00	25138,83	CONCENTRATED V.LOADS
0,00	1,00	0,00	34,00	0,00	
0,00	1,00	0,00	30,00	0,00	
0,00	1,00	0,00	26,00	0,00	
0,00	1,00	0,00	23,00	0,00	
0,00	1,00	0,00	20,00	0,00	
0,00	1,00	0,00	18,00	0,00	
0,00	1,00	0,00	16,00	0,00	
0,00	1,00	0,00	12,00	0,00	
0,00	1,00	0,00	10,00	0,00	
0,00	1,00	0,00	0,00	0,00	DISTRIBUTED V.LOADS
0,00	1,00	0,00	0,00	0,00	
495,12	1,00	495,12	5,46	2704,58	SHAFTS WEIGHT
518,65	1,00	518,65	16,56	8590,09	
425,28	1,00	425,28	28,60	12163,09	

LOADS ON FOUNDATION

MOMENT =	**50442,96**	**daN*m**
SHEAR =	**2157,30**	**daN**
AXIAL LOAD =	**2993,11**	**daN**

6. CHECKS

In this design calculation it's considered, according to EC 3, the state elastic limit of the sections. It's necessary to have always the limit tension superior to project tension.

$$f_{yd} = \frac{f_{yk}}{\gamma_M}$$

- γ_g factor amplifying permanent loads;

Where γ_M = 1,1 according to EC3.

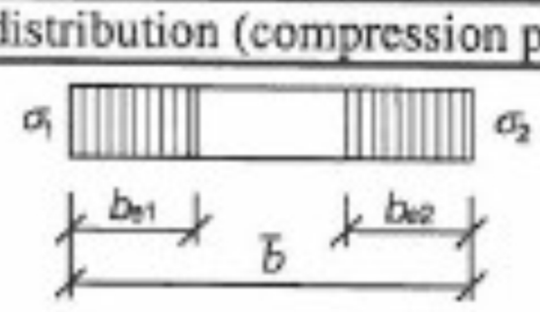
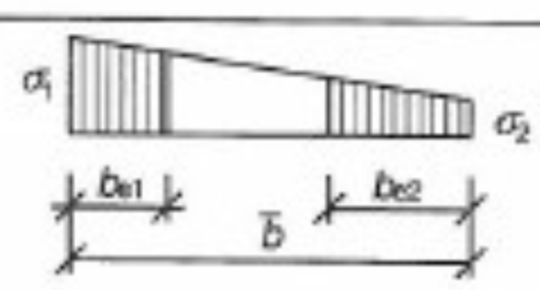
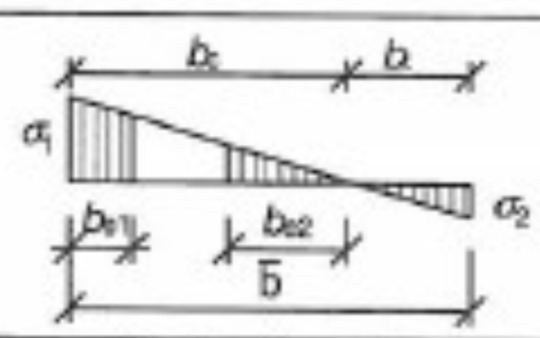
According to Eurocode 3, Part 1-5, if necessary, the limit tension can be reduced in consideration ratio between diameter / thickness (local buckling).

6.1 Local buckling

Cold-formed profiles can be subject to local instability phenomena, i.e. buckling of the plates following the presence of compressive stresses.

Local instability is studied according to the UNI EN 1993-1-5 standard. The table and formulas to refer to for the distribution of stresses on each side of the section is as follows:

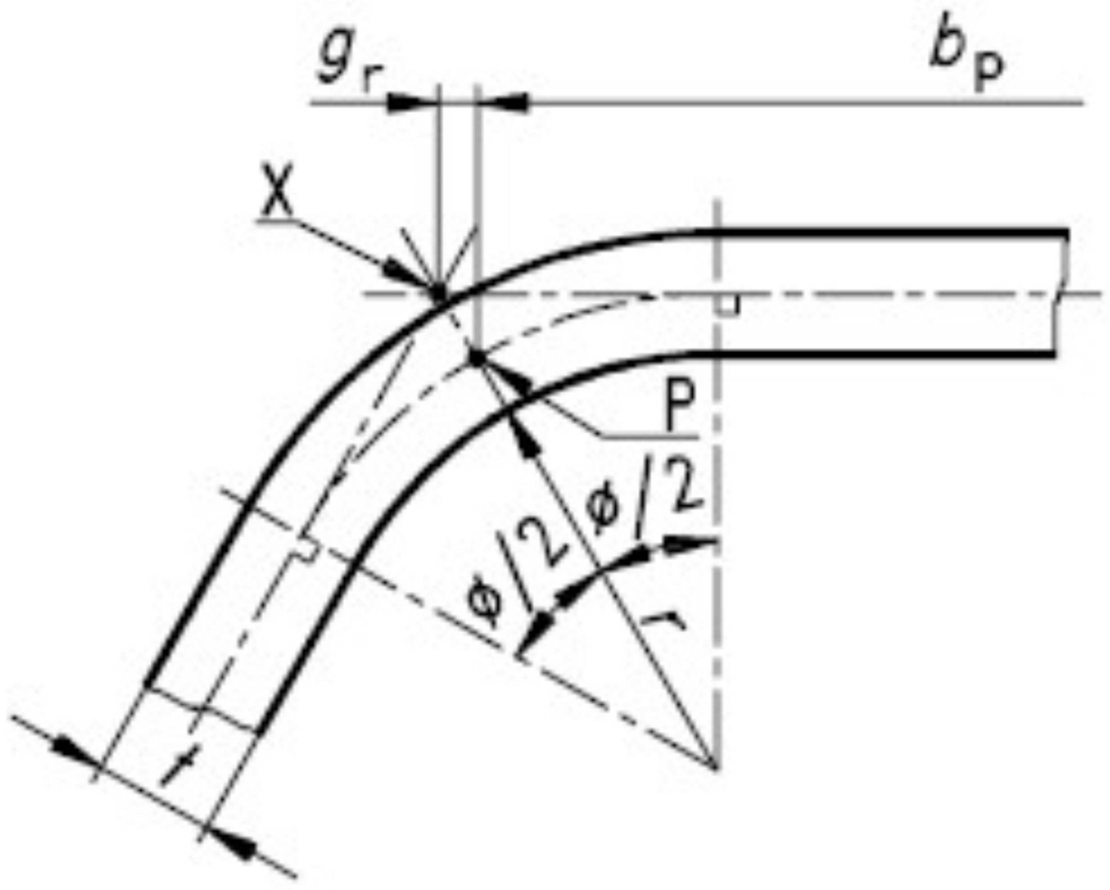
Table 4.1: Internal compression elements

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\psi = 1$: $b_{eff} = \rho\, \bar{b}$ $b_{e1} = 0,5\, b_{eff}$ $\quad b_{e2} = 0,5\, b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho\, \bar{b}$ $b_{e1} = \frac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, \bar{b} / (1-\psi)$ $b_{e1} = 0,4\, b_{eff}$ $\quad b_{e2} = 0,6\, b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi \geq -3$
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98\,(1-\psi)^2$

where $\lambda_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}}$ $\qquad \varepsilon = \sqrt{\frac{235}{f_y\,[N/mm^2]}}$

fy = 355 N/mm²;
b = side of the section;
b⁻⁻ = b' = side of the section not considering rounded edges as following scheme:



X = intersection of axes lines of the sides
P = midpoint of edge
$r_m = r + t/2$
$g_r = r_m$ [ta (Ø/2) – sin (Ø/2)]
t = s = thickness
r = 4t

If $\lambda_p < 0.673$, there is no local buckling:

Check for local buckling on shaft 1:

H above ground (m)	Ø (mm)	b (mm)	b' (mm)	b'/s (mm)	λp (ψ=1)	ρ (ψ=1)
0,00	840	162,88	162,71	32,54	0,704	0,976
2,73	792	153,47	153,30	30,66	0,663	1,000
5,46	744	144,06	143,89	28,78	0,623	1,000
8,19	695	134,66	134,48	26,90	0,582	1,000
10,93	647	125,25	125,08	25,02	0,541	1,000

Check for local buckling on shaft 2:

H above ground (m)	Ø (mm)	b (mm)	b' (mm)	b'/s (mm)	λp (ψ=1)	ρ (ψ=1)
10,93	657,10	127,20	127,03	25,41	0,550	1,000
13,74	607,32	117,49	117,32	23,46	0,508	1,000
16,56	557,55	107,78	107,61	21,52	0,466	1,000
19,38	507,78	98,07	97,90	19,58	0,424	1,000
22,20	458,01	88,36	88,19	17,64	0,382	1,000

Check for local buckling on shaft 3:

H above ground (m)	Ø (mm)	b (mm)	b' (mm)	b'/s (mm)	λp (ψ=1)	ρ (ψ=1)
22,20	466	90,12	89,98	22,50	0,487	1,000
25,40	410	79,10	78,96	19,74	0,427	1,000
28,60	353	68,07	67,93	16,98	0,368	1,000
31,80	297	57,05	56,91	14,23	0,308	1,000
35,00	240	46,03	45,89	11,47	0,248	1,000

Limit stress is reduced due to local buckling. Therefore, in base section:

sR,d = 315 N/mm²
sE,d = 294 N/mm² , sE,d < sR,d, 294 N/mm² < 315 N/mm²

Stress ratio: 0.93

6.2 Resistance check

VERIFICATION : U.L.S. GX1.35 QX1.5						
Level (cm)	**σ (N,M) N/mm²**	**τ (N,M) N/mm²**	**f ED (N,M,T) N/mm²**	**fy (N,M,T) N/mm²**	**CHECK**	**USE RATIO**
0,00	294,08	4,97	294,20	315,12	✓	**0,93**
273,13	294,45	5,56	294,61	322,73	✓	**0,91**
546,25	294,45	5,58	294,61	322,73	✓	**0,91**
819,38	294,39	5,58	294,55	322,73	✓	**0,91**
1092,50	285,85	5,57	286,02	322,73	✓	**0,89**
1092,50	277,04	5,47	277,20	322,73	✓	**0,86**
1374,38	273,41	5,44	273,58	322,73	✓	**0,85**
1656,25	274,89	5,42	275,05	322,73	✓	**0,85**
1938,13	266,23	5,43	266,40	322,73	✓	**0,83**
2220,00	258,38	5,48	258,56	322,73	✓	**0,80**
2220,00	309,73	6,71	309,95	322,73	✓	**0,96**
2540,00	287,58	6,80	287,83	322,73	✓	**0,89**
2860,00	245,34	7,03	245,64	322,73	✓	**0,76**
3180,00	166,05	7,49	166,56	322,73	✓	**0,52**
3500,00	3,21	8,35	14,82	322,73	✓	**0,05**

6.3 Base plate check

Adequate dimension for base plate in this high mast are the following:

$Ø_{external}$ = 1040mm
$Ø_{internal}$ = 785mm
$Ø_{bolts\ holes}$ = 940mm
Thickness = 50mm
Anchor bolts: N°20 M30 J (total length 1000 mm)
Gussets: N°10 Thickness 10mm; b=90; h=230mm.

Limit stress on anchor bolts, and minimum base plate thickness are checked:

Maximum bending moment (daNm)	M	76633,79
Maximum shear (daN)	T	3235,95
Base diameter (mm)	Db = Øs	840
Bottom shaft thickness (mm)	Sb	5,00
Sides number		16
Internal diameter base plate (mm)	Di	785
Anchor bolts diameter (mm)	Df = Øv	940
External diameter (mm)	De = Øe	1040
Limit stress base plate (N/mm²)	T f lim = Sig af	284,00
Limit stress anchor bolts (N/mm²)	T t lim	322,73
Leverage factor	K elev	1
Coefficient inaccuracies in assembly	K inc	1,1
resistive module base section (cm³)	Wp	2633,68
Ratio Dv/Dp		1,14
Base plate coefficient	Kfla	0,5869
Anchor bolts diameter (mm)	Dtf	30
Cross section anchor bolts (mm²)	SecT	561
Anchor bolts number	Nt	20
Cross section base (mm²)	As	13032,03

Design stress anchor bolts= $\frac{4 * M * Kelev}{SecT * Df * Nt} + \frac{T}{Nt * SecT}$ = **293,53** N/mmq

Minimum thickness base plate = $(Kfla * Kinc) * \sqrt{\frac{M / Wp * (Db - Sb) * Sb}{Tf \lim}}$ = **42,22** mm

Designed thickness base plate = **50,00** mm

*For higher safety standards, any gussets are neglected in the calculation

6.4 Connection length

The 35m high mast is a "Slip Joint" structure, i.e. made by coupling two shafts by overlapping, in which the guarantee of continuity of the structure and the transmission of efforts from one shaft to the other are entrusted to the frictional resistance established between the contact surfaces.

These joints are made before installing the support by forcibly inserting the upper shaft for a predetermined overlapping length "i", equal to or greater than a minimum value which must be such as to guarantee the stability of the tower.

International Standards, Prescriptions, Specifications agree in establishing a minimum overlapping length equal to 1.5 times the maximum diameter of the female shaft.

In this case, by applying the aforementioned rule, the following lengths are required:

1st overlapping: 657 x 1.5 = 985.5 mm → assumed 1250 mm
2nd overlapping: 466 x 1.5 = 699.0 mm → assumed 900 mm

7. LOADS ON FOUNDATION

U.L.S. CHECK , Gx1.3 , Qx1.5 (daN , m) LOADS ON FOUNDATIONS	
MOMENT =	76633,79
SHEAR =	3235,95
AXIAL LOAD =	4040,70

U.L.S. CHECK , Gx1.0 , Qx1.5 (daN , m) LOADS ON FOUNDATIONS	
MOMENT =	75664,44
SHEAR =	3235,95
AXIAL LOAD =	2993,11

S.L.S. CHECK , Gx1.0 , Qx1.0 (daN , m) LOADS ON FOUNDATIONS	
MOMENT =	50442,96
SHEAR =	2157,30
AXIAL LOAD =	2993,11

8. CONCLUSION

This document contains the structural safety checks of a newly installed self-supporting metal pole capable of supporting a fixed lighting system, specifically:

- Top R/L device with a maximum of 8 floodlights (Secom Invictum L S6400 58 126 74 ERA 3501, 35 kg; max 0.6 m²) installed on 360°.

The structural checks, carried out with the semi-probabilistic limit state method, confirmed that, in correspondence with the combinations of the loads considered significant, the bearing capacity of all the members is higher than the maximum calculation actions.