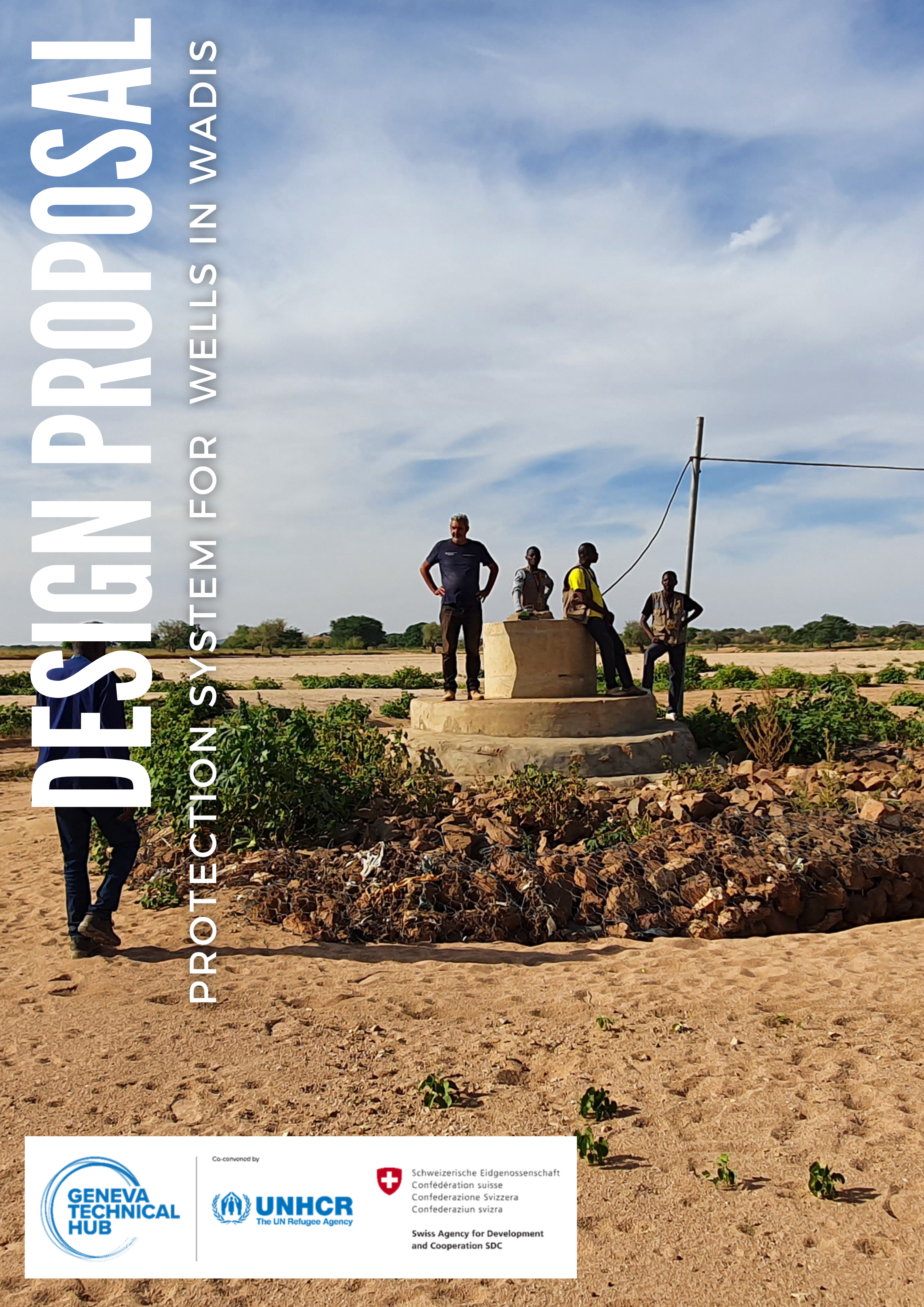
DESIGN PROPOSAL

PROTECTION SYSTEM FOR WELLS IN WADIS



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PROTECTION SYSTEM FOR WELLS IN WADIS

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1. High-level summary

Many of the key wells supplying piped water to refugee camps in Chad are inside or near the edges of wadis. With every runoff season, **critical infrastructure is menaced** threatening the water supply of over 200,000 refugees. The protection works are costly, in some cases nearly matching the cost of the wells meant to protect. They are also not durable. They need yearly rehabilitation that drains human and economic resources from other areas.

In the context of the well solarization initiative in Chad, constant repairs complicate the cost-recovery calculus of these systems. **Finding a durable solution is vital to have a clear solarization success story**.

But wadis are an extremely challenging place to build durable infrastructure. This document provides a **tentative design** to pioneer this durable, low-cost solution to be later deployed off the shelf in these contexts.

2. Problem description

Wadis are a notoriously challenging place to build infrastructure. The water flows dig around it, a process called scouring. They also carry away pipes and cables in the riverbed. The profile keeps changing season after season. Soft wet sand has one of the lowest bearing capacities of any soil, making structures easily unstable. And to cap it all, the high-water table and the enormous permeability of the sand make concreting or pipelaying at any depth very challenging in low-income remote locations.

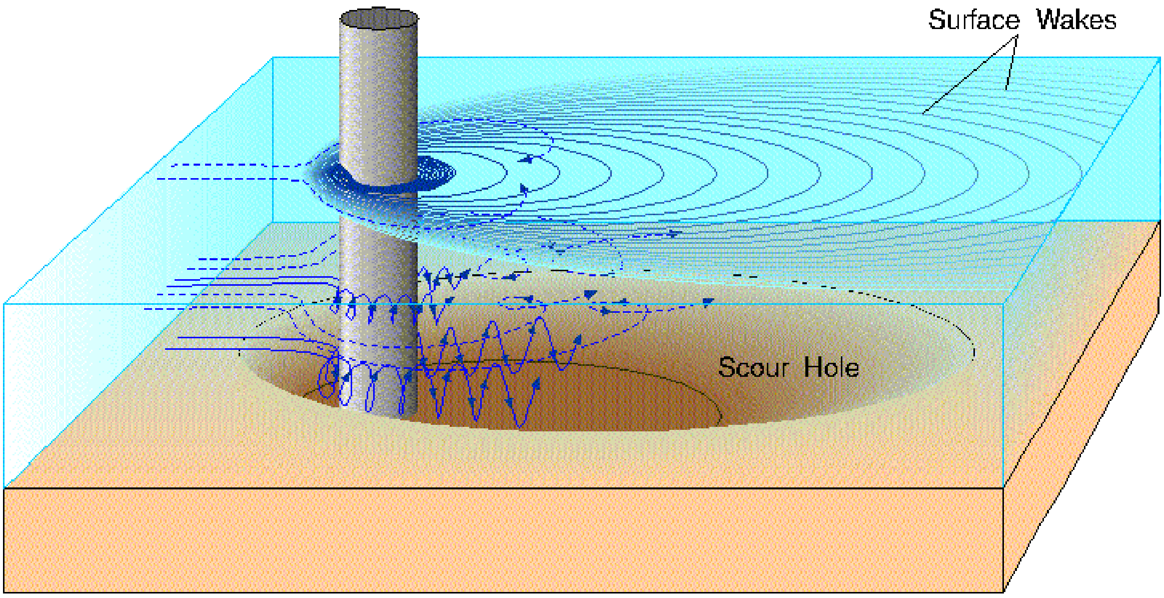


Figure 1. Scour at work. The erosion is cause by the deflection of the incoming water downwards in horseshoe vortices and the and the wake vortices downstream. Source: USGS

Unfortunately, **few tried and tested solutions available are available for contexts like Chad**. In bridges, for example, piers rest on caps and piles (very deep columns) that provide stability and accommodate erosion. The unavailability of such means and their cost in comparison to the goal value calls for testing durable lower cost options.

Figure 2. Location of the well built in 2019 and lost in the 2022 season.

Locally, several solutions have been tried that, while being partially effective in protecting the well, **need constant remediation works** making them cost-ineffective. Besides the need for better designs, the main reason is they **rely heavily on gabions** that are poor quality, poorly executed, and not the material of choice for the problem at hand:

Figure 3. Scour around a gabion wall has de-stabilized it. Ther lack of proper geotextile and mattress gabions as foundations meant there was no protection against scour. Durability of the cage wire is also a big concern. Hadja Hadid

Figure 4. De-stabilization and failure of the protection works, Treguine.

Figure 5. Broken pipe fittings in Bredjing. The pipelines crossing wadis need constant repairs.

The challenge is to find a design that is durable, cost-effective, and feasible in a low-income context.

This design **builds on some of the solutions already tried**, improving them by eliminating gabions, protecting the water pipes too, and giving hydrodynamic shapes that reduce scour and improve scour stability.

Figure 6. This well protection in Mile already captures most of the logic behind the solution.

3. Proposed solution: P2W

The proposed solution to field-test is an overhead line carrying the pipe and electric motor cable supported by steel lattice towers. A hydrodynamic geometry of mass concrete provides the stabilizing counterweight for the line forces and protection against scour. The protection is achieved by making the structure deeper than the depth at which scour stabilizes. The steel tower is the same for wellheads, mid-points, and end-points.

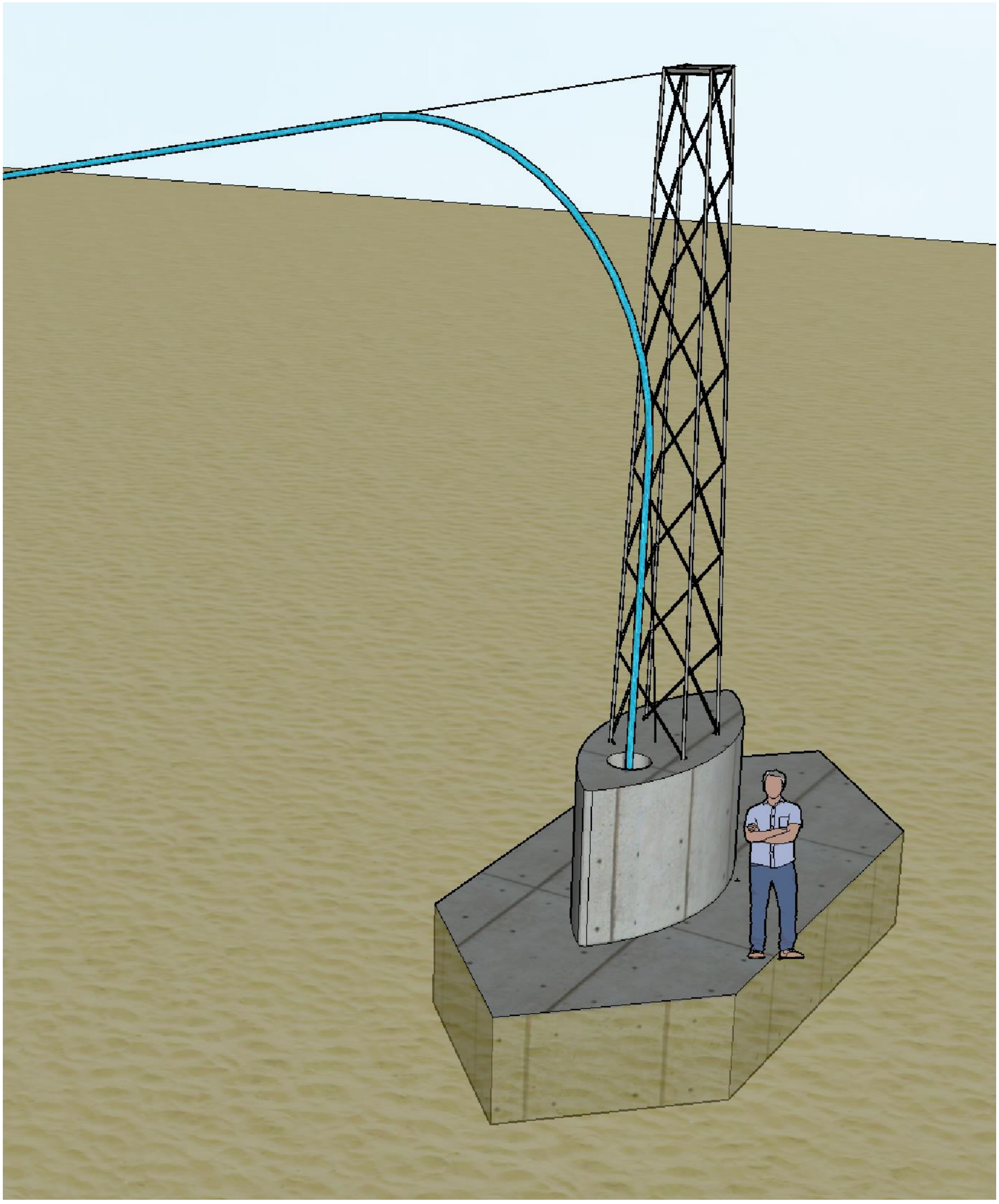


Figure 7. Wellhead. Start of the overhead line. ***The hexagon shape is buried and only the sail shows.***

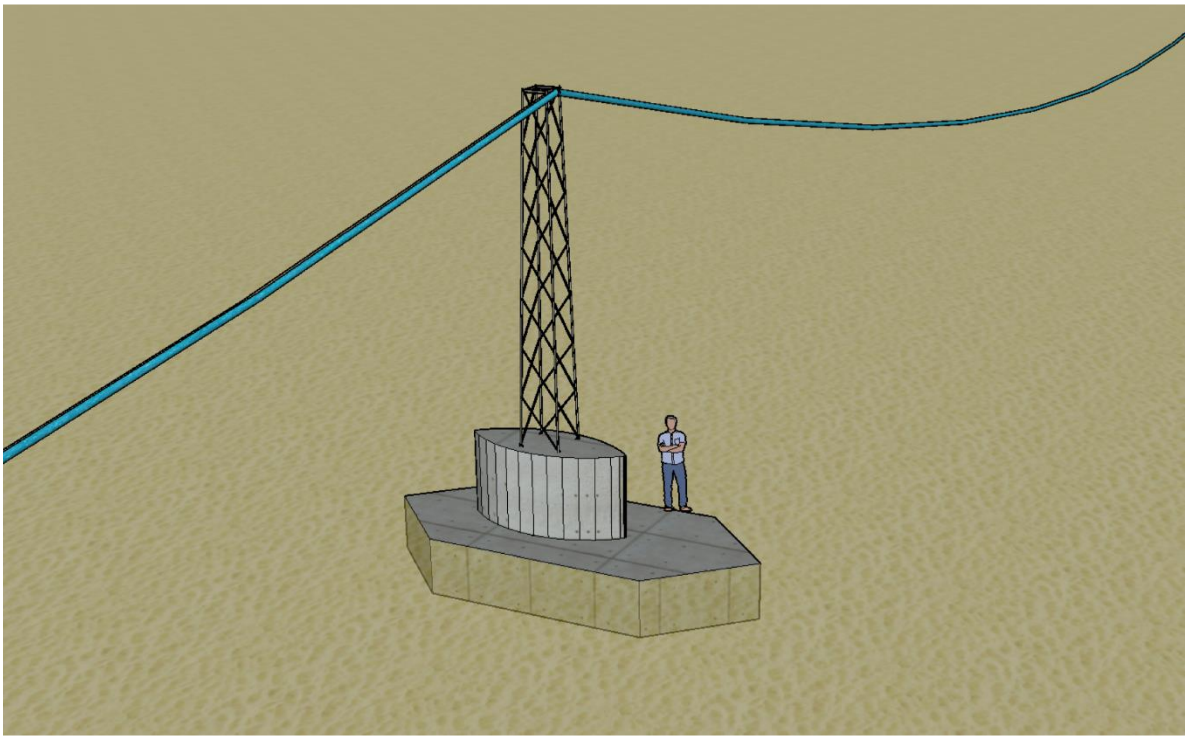


Figure 9. Mid-point support for distances over 50 m.

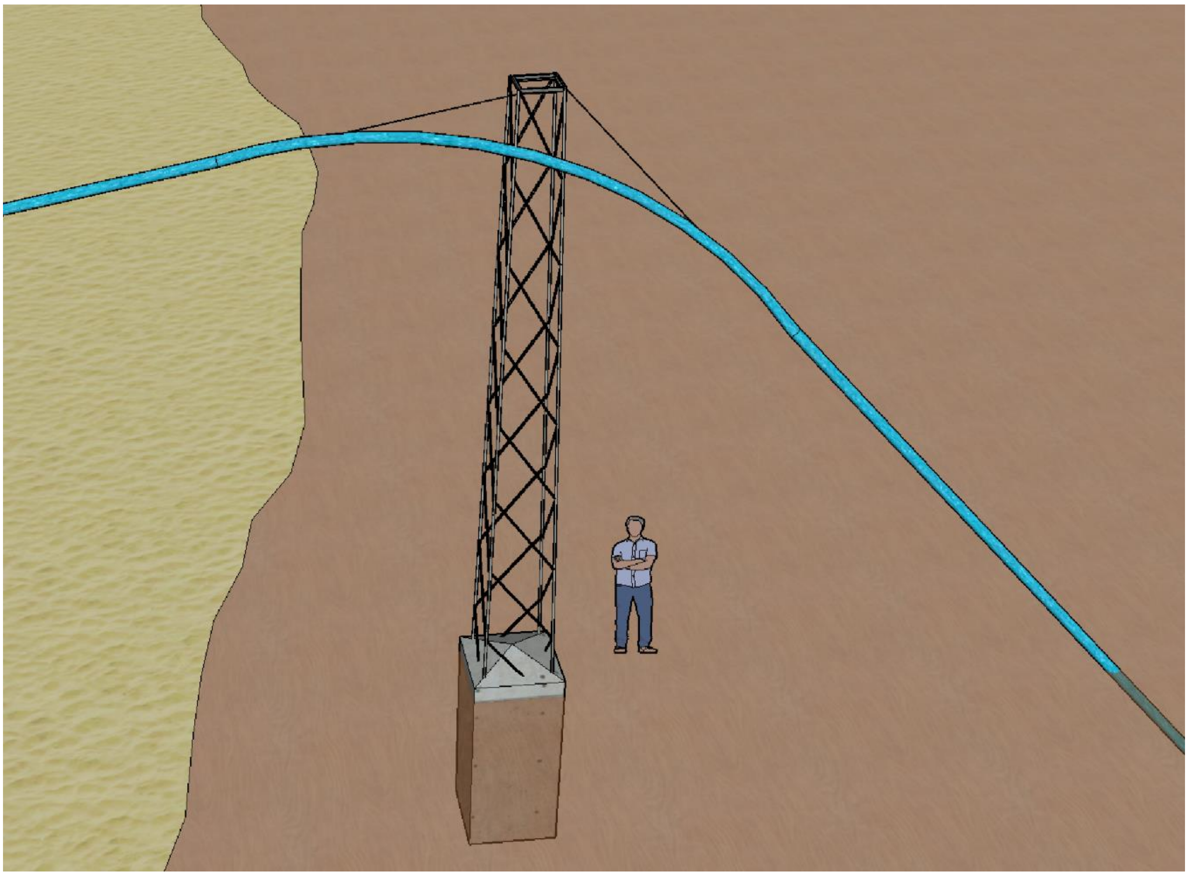


Figure 8. End-point, arrival outside of the wadi. This can be transformed into the previous if needed.

Key specifications:

- Single pipe system. Do not add extra load. Each well needs its own system.
- Maximum span between supports: 50 m. Do not exceed!
- Minimum cable sag: 3 m. Do not reduce!
- Maximum water depth: 1.5-2 m
- Minimum cable elevation: 3 m above maximum water level.
- Maximum pipe diameter: 110 mm.
- Maximum motor cable: $4x50mm^2$ (3.02 kg/m).

DISCLAIMER: This design is a proposal to be revised locally and adapted to local codes*. The lack of information on wadi hydrology makes it tentative and it may require refinement when more information is available.*

4. System description

A. Overhead lines

The pipe and motor cable are installed in an overhead line. The pull from the weight of the cable is the ruling load on structural and stability calculations. In a catenary, **forces increase spectacularly with the distance between supports or the decrease in sag (!)**. It is vital to respect the prescribed parameters below:

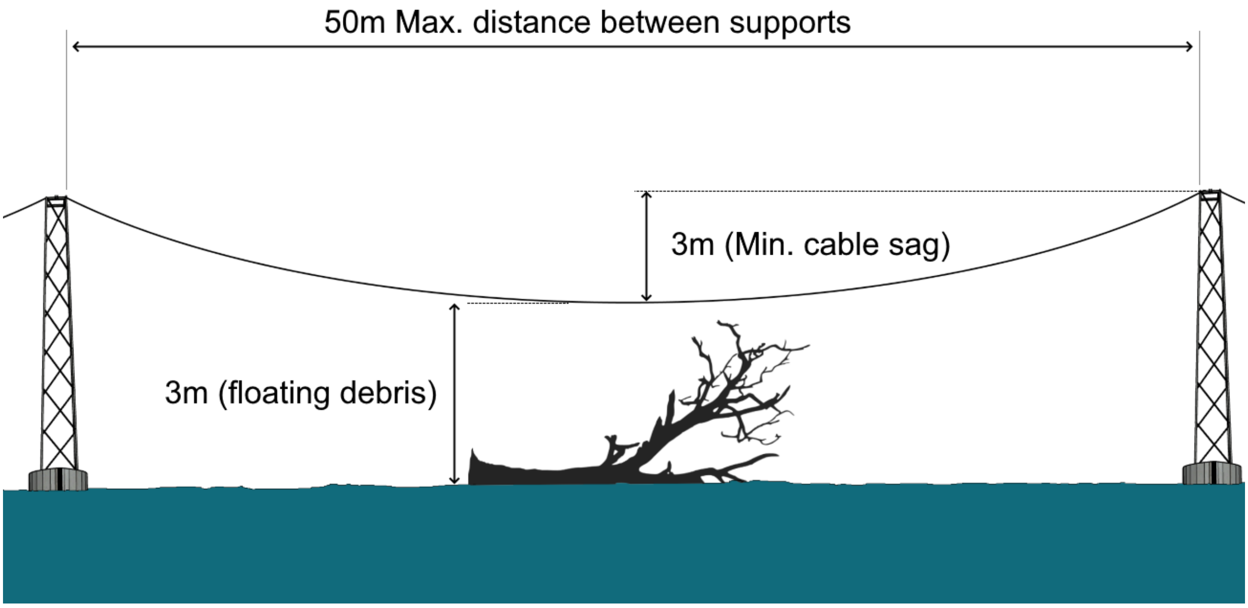


Figure 10. Key overhead cable geometric specifications.

B. Steel lattice tower

The S275 grade steel tower is made of common angle iron profiles in a standard configuration that should be easy to manufacture locally. It may even be readily available from electric suppliers, although loads on these electricity pylons tend to be lower.

The geometry needs to be strictly respected. The angle iron profiles are among the most common, but bigger or thicker profiles can be used if these can't be found locally, i.e. L45.5 profiles instead of L40.4 or L70.7 instead of 70.6. Ideally, galvanized profiles and specific welding electrodes are used.

Usually, angle iron profiles come in 6-meter lengths. The facilitate transportation the tower is split into two spans that can be bolted together on site. See details at the end of the document.

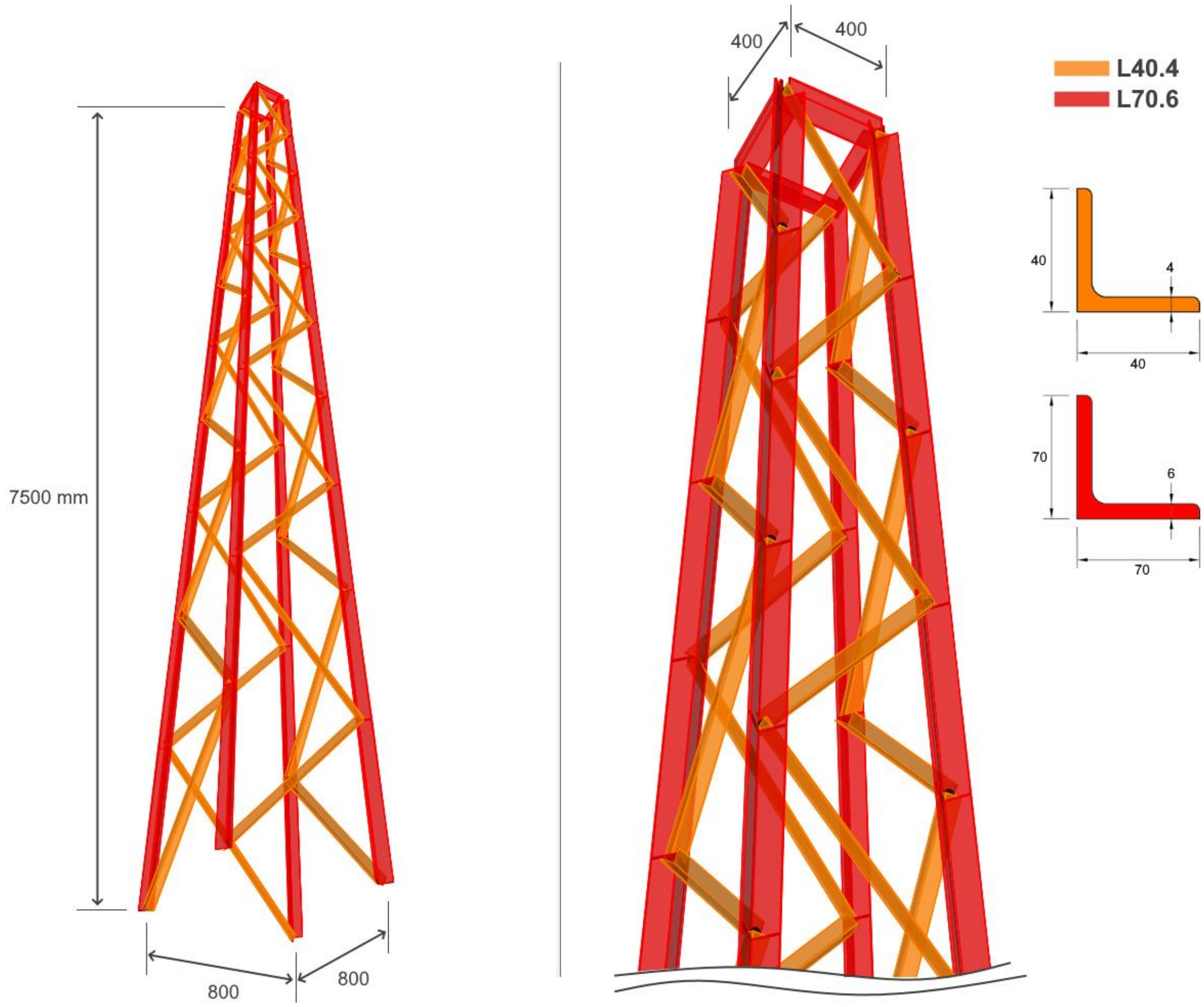


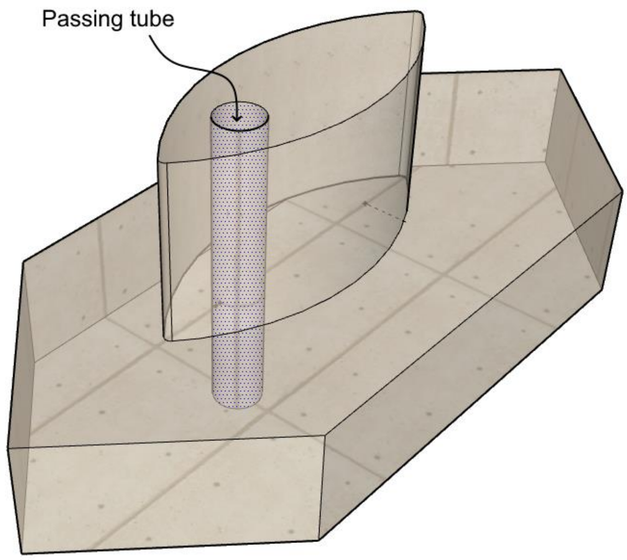
Figure 11. Lattice tower proposed. The first 5 stages are 800mm apart and the last five are 700 mm for a total height of 7.5 m.

The tower must be above the water load at all times **protected from impacts** of floating debris by the concrete sail.

C. Mass concrete counterweight

The mass provides scour depth protection and counterweight like a roly-poly toy. The **scour depth** is one of the important pieces of information missing and may need to be increased after testing.

This mass should be oriented to cut the flow as cleanly as possible. **The hexagonal part lies buried under the soil**. The oval shape (sail) covers the complete water depth to protect the tower from impact.



This mass should be able to move freely from borehole tubes to **accommodate the likely tilting and sinking** and repair work. A 40 cm diameter passing tube should be placed when casting to avoid transmission of forces to borehole pipes and other elements.

This is the most expensive part of the protection to build. To **reduce the costs, cyclopean concrete** (plum concrete) is used. This reduces the volume of concrete needed by about 40% and allows us to recycle existing gabion rocks. Care must be taken when building this plum concrete. Simply **dumping unclean, unwet, fractured, and poorly placed rocks in the mix will result in very poor-quality concrete.**

ANNEX 1. Hydraulic calculation

Calculation basis

Existing wellhead parameters (generalized)

Construction geometry: Round with concrete rings stacked on each other

Diameter: 1.3 m

Head height over soil: 2 m

Wadi hydrogeologic parameters

Unfortunately, wadi hydrodynamic data is not available and the possibility of measuring it in different sites with the available human resources is not a real possibility.

Maximum flow speed (u): 2.5 m/s

Murky water density ($\rho_{wm)}$: 1200 kg/m^3

Load determination

Hydrodynamic drag

Hydrodynamic drag (F_D) is given by the following equation (SI units).

$$F_D\ (N) = ½·\rho·C_{D·}\ u^2·A$$

For existing concrete ring wells:

A. Data input		
Density	1200	kg/m3
Drag coefficient	0.64	*Short cylinders (L/D≈1)*
Velocity	2.5	m/s
Area	1.95	m2
B. Result		
Force (N)	4680	N
Force (Kg)	478	kg

For a teardrop profile like the turret of a submarine:

A. Data input		
Density	1200	kg/m3
Drag coefficient	0.1	Teardrop cylinder
Velocity	2.5	m/s
Area	1.2	m2
B. Result		
Force (N)	450	N
Force (Kg)	46	kg

The reduction of force is considerable in comparative terms, but not very consequential. **The main interest of the shape comes from the reduction in turbulence and hence scour.**

Scour

There is not enough data to perform scour modeling.

Gabions have quality issues locally, have performed poorly, and require frequent maintenance.

The protection model proposed is based on **scour depth**. Rather than protecting against scour, scour is allowed to occur until an equilibrium is reached. If the structure goes deeper than the scour depth, then it is not destabilized by it.

Scour depth

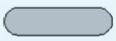
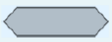
The maximum scour depths observed in the sites were 50 cm with shapes that were not hydrodynamic: barriers, cubes, rhomboids…

Increasing the structure depth has a very practical limit, not only in terms of cost but also the high water table in the wadis. Dewatering to concrete creates logistic and economic challenges.

With this in mind, a **tentative structure depth of 1 m** is to be field tested.

Hydrodynamic shapes

The shape of the structure is key to limiting the scour depth, see, for example, the following test results from lab models[1] that have been used for guidance:

Pier Shape		V=0.18	V=0.25	V=0.3
		Measured scour depth (cm)	Measured scour depth (cm)	Measured scour depth (cm)
Circular		3.9	6.1	6.9
Rectangular		4.3	6.8	7.6
Octagonal		4.2	5.2	5.9
Joukowsky		4.7	5.5	6.1
Chamfered		4.1	5.9	6.7
Oblong		4.1	4.6	5.8
Elliptical		3.6	4.9	5.6
Sharp nose		3	4.5	4.9
Hexagonal		2.8	3.6	4.1
Streamline		1.9	2.6	3

[1] Al-Shukur, Abdul-Hassan & Hadi Obeid, Zaid. (2016). EXPERIMENTAL STUDY OF BRIDGE PIER SHAPE TO MINIMIZE LOCAL SCOUR. 7. 162-171.

The practicality of the construction is also taken into consideration:

- For the foundation and counterweight, the underground mass of cyclopean concrete is given a **sharp nose profile**, the best performing of those easy to build with normal formwork and that provide a long straight tilting edge for stability.

- For the structure protecting the well and the tower from flow and impact, **a streamlined shape** is taken.

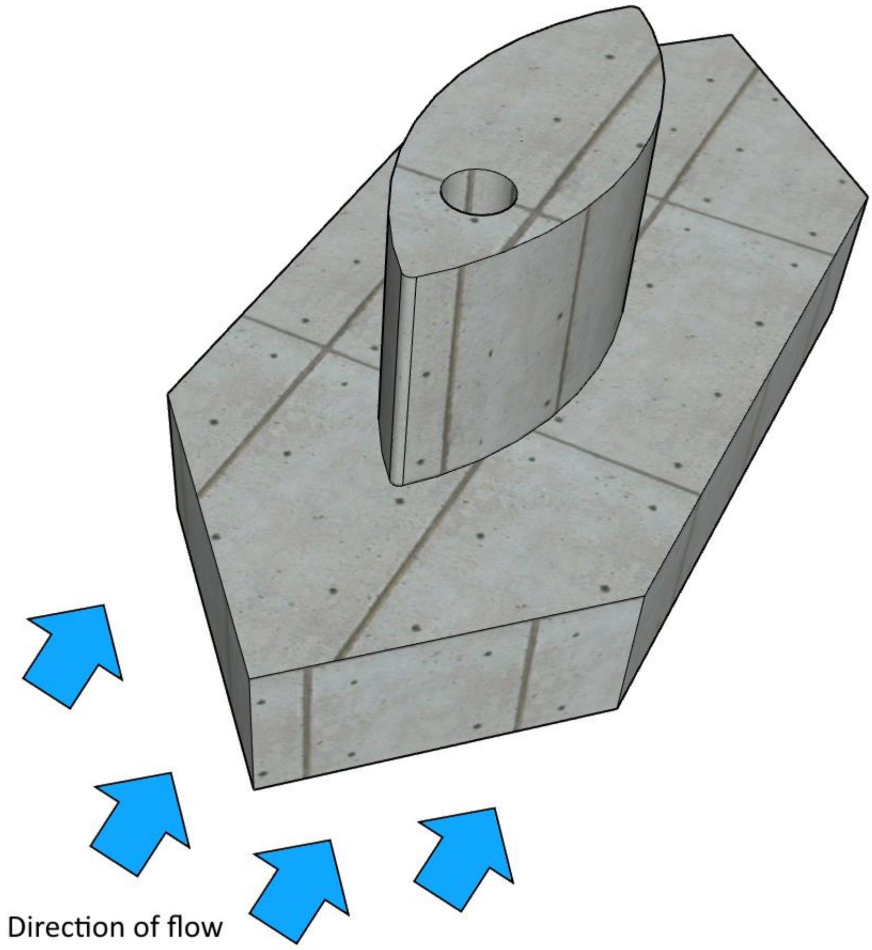


Figure 12. Hydrodynamic shapes of counterweight foundation and well head (sail).

ANNEX 2. Structural calculation

Methodology

The tower is modeled as a 3D steel structure in SAP2000 using kg, mm, °C as units. LRFD approach is adopted for design. Loads and combinations are as defined by ASCI 7-16 code with the site conditions according to the referenced sources of information for Chad when available.

Steel code ANSI/AISC 360-16 is used for the steel sections. EHE-08 for the concrete slab.

Calculation basis

Material properties and resistance factors

Concrete: HA - 25 - f_c = 25 N/mm^2

Rebar: B - 400 - f_y = 400 N/mm^2

Cold laminated profiles: C235 f_y = 235 N/mm^2

Resistance factor as per codes.

Design codes

- ASCI 7-16
- ANSI/AISC 360-16.
- EHE-08
- BS 6349-2

Load combinations

Those prescribed by ASCI 7-16 section 2.3.2:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

A_k = load or load effect arising from extra ordinary event A
D = dead load
D_i = weight of ice
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
F_a = flood load
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load
S = snow load
T = self-straining load
W = wind load
W_i = wind-on-ice determined in accordance with Chapter 10

Snow, ice, and earthquake loads will not be considered for locations in Chad which makes some load combinations redundant (5, 6, 7).

Design hypothesis

H1. Dry foundations. The riverbed has no flowing water, there are no flotation forces.

H2: Foundation submerged underflow. Buoyancy effects.

H3: Pipe partially filled. Water live load weight acting only on one side on intermediate poles.

Load determination

The overhead line is supported by a four-sided pyramidal steel lattice tower. Since the same tower is to be repeated in several locations in Chad, the least favorable conditions are taken.

For **catenary loads**, the length-to-sag coefficient is 50/3=0.06<01, therefore, the cable can be approximated to a **parable uniformly loaded with horizontal loads**:

$R_{1x} = R_{2x} = q L^2 / (8 h)$; $R_{1y} = R_{2y} = q L / 2$

Where:

$R_{1x} = R_{2x}$ = horizontal support forces (lb, N) (equal to midspan lowest point tension in cable)

q = unit load (weight) on the cable (lb/ft, N/m)

L = cable span (ft, m)

h = cable sag (ft, m)

$R_{1y} = R_{2y}$ = vertical support forces (lb, N)

The angle θ can be calculated as

$\theta = \tan\text{-}1(R_{1y} / R_{1x}) = \tan\text{-}1(R_{2y} / R_{2x})$

The length of the sagged cable can be approximated to

$s = L + 8 h^2 / (3 L)$

where:

s = cable length (ft, m)

See the wire rope and sizing section for the results.

1. Dead loads

Steel cable (10 mm): 0.40 kg/m

Electric motor cable ($4x50mm^2$): 3.02 kg/m

Pipe (HDPE 110mm PE100): 2.27 kg/m

For steel profiles, 7850 kg/m^3.

The overhead catenary dead load is: 0.4 + 3.02 + 2.27 = 5.69 kg/m

2. Live loads.

The weight of the water in the pipe (110 mm): $W= \rho \cdot l \cdot \phi^2/4 = 1000 \cdot 1 \cdot 0.097^2/4= 7.39$ kg/m

This value is also the overhead catenary live load.

Service load: 300 kg is considered incompatible with wind load.

3. Wind loads.

The basic wind speed (V) is taken as 120 km/h or 33.33 m/s.

The wind directionality factor for both trussed towers of angle iron profile and round pipe is K_d = 0.95 (Table 26.6-1).

Velocity exposure coefficient, for exposure D and 12 m height, K_z = 1.16 (Table 29.3-1).

Abrupt changes in topography are not expected in mostly flat terrain K_{zt}= 1

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 \text{ (lb/ft}^2) \qquad (29.3\text{-}1)$$

$$[\text{In SI: } q_z = 0.613\, K_z K_{zt} K_d V^2 \text{ (N/m}^2\text{); } V \text{ in m/s}]$$

q_z= $0.613 \cdot 1.16 \cdot 1 \cdot 0.95 \cdot 33^2$= 751 N/m^2 = 751 $N/m^2 \cdot 0.10197$ kg/N = 76.52 kg/m^2

3.1 Wind on cable

The wind is considered horizontal and perpendicular to the cable.

$F_{wc} = q_z \cdot d \cdot l = 76.52.5\ kg/m^2 \cdot 1m \cdot 0.11m = 8.42\ kg/m$

3.2 Wind on the trussed tower

For a square trussed tower area, assuming L 50.5 profiles, ϵ=0.1586 and C_f= 3.16

F = 0.751x0.85x3.16·.1 = 0.48 kN/m or 20.4 kg/m of tower at 90°.

For diagonal winds, this is multiplied by 1+0.75 ϵ = 1.012

F_{45° = 20.4*1.012= 20.65 kg/m of tower

For simplicity, only perpendicular winds will be considered (20.4 kg/m) since their effect on the planned construction is the most adverse.

4. Snow and ice loads.

The snow and ice loads are not considered in Chad for climate zones that range from tropical to arid.

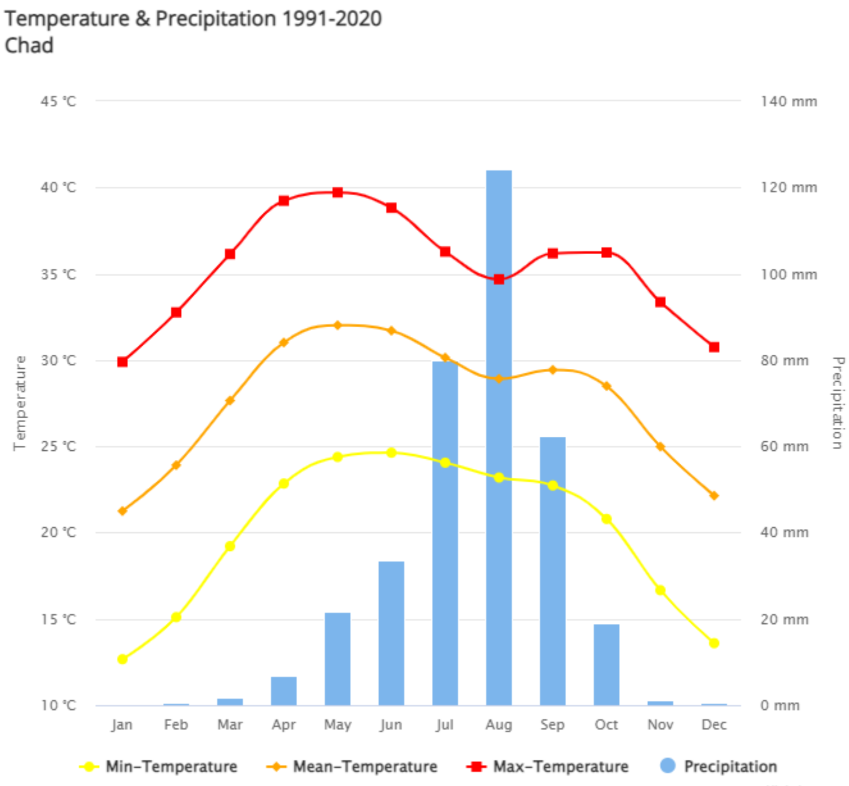


Figure 13. Source: World Bank Climate Change Knowledge Portal.

5. Earthquake

Ground acceleration (g) is below 0.04 g, not requiring earthquake calculations.

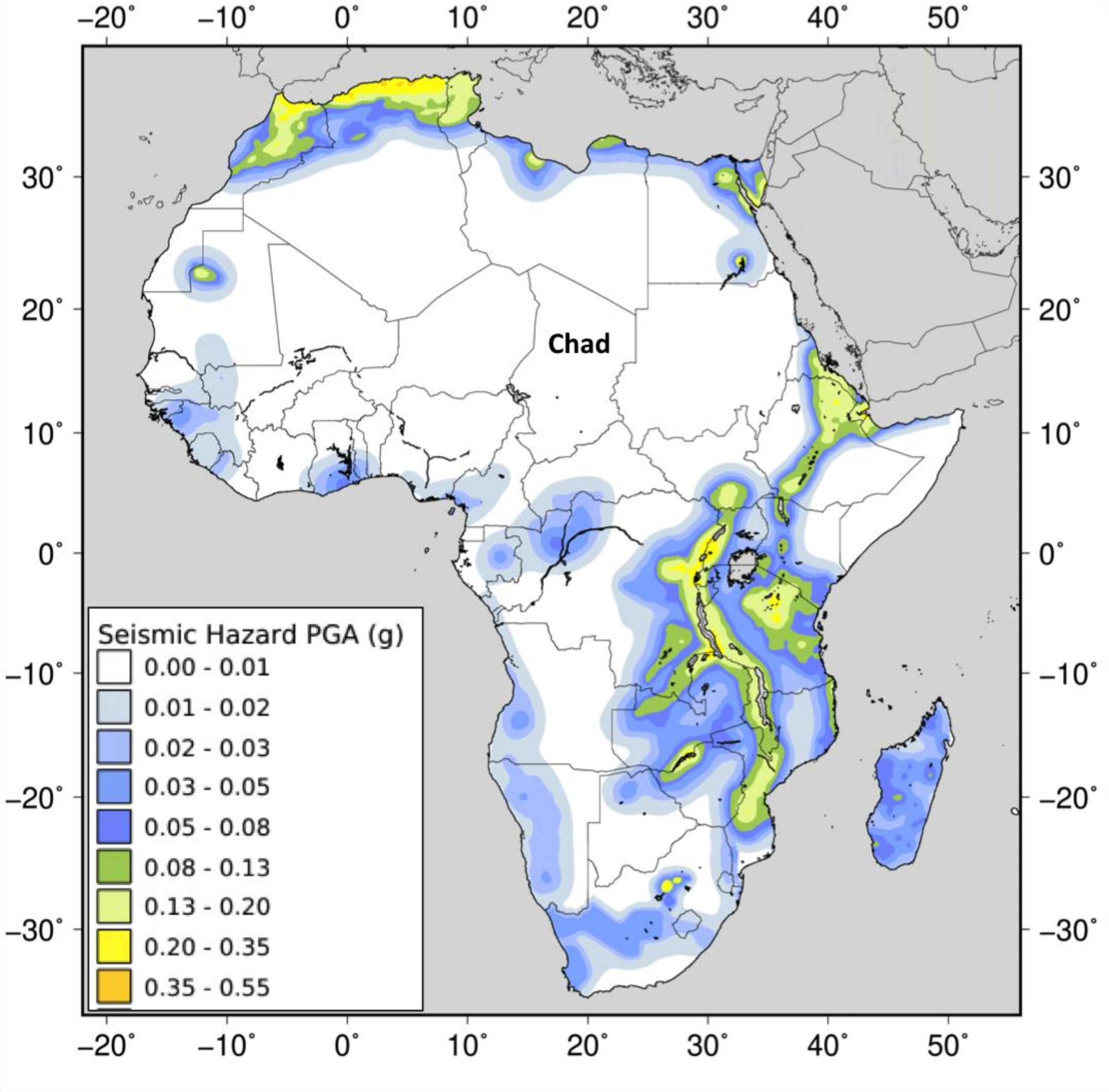


Figure 14. Africa Earthquake Hazard and Risk Model. Source: Global Earthquake Model Foundation.

6. Self-straining

Temperature self-staining is not considered due to the small size of the structure. The single slab foundation does not allow differential settlements.

7. Impact and entanglement

The impact of floating debris is low mass and low velocity, easily absorbed by the concrete part of the structure. The concrete apron above the water line protects the tower steel profiles from impact deformation. **Cables will be kept at least 3 m above the maximum expected water level** to avoid entanglement and impact from floating debris, mainly trees. In very complicated settings this distance can be reduced to two meters.

Further protection is not considered cost-effective.

8. Load summary

A. Data input		
Max. catenary cable length	50.5	m
Tower height	7.5	m
Cable sag	3	m
Crosswind on cable	8.42	kg/m
Catenary dead (-z)	3200	N
Catenary dead (x)	55.76	N/m

B. Secondary input		
Catenary dead	5.69	kg/m
Catenary live	7.39	kg/m

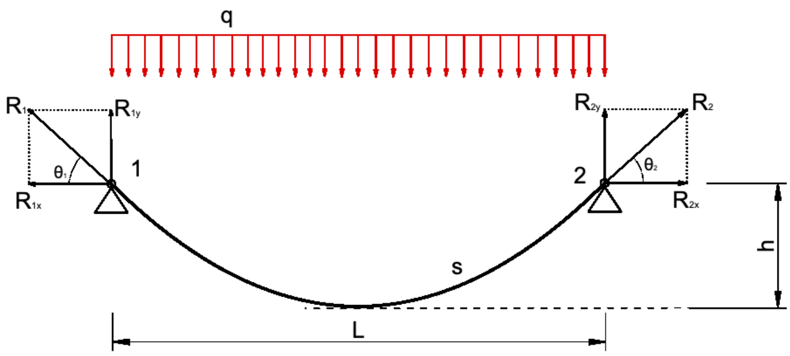
C. Load per case		
Crosswind on cable (y)	425.21	
Catenary dead load (-z)	287.35	kg
Catenary dead (x)	592.05	kg
Catenary live (-z)	373.20	kg
Catenary live (x)	777.57	kg
Service load (-z)	300.00	kg

Wire rope load and sizing

The total dead and live loads on the cable are:

Unit weight	**13.08**	kg/m
	7.39	water
	2.27	pipe
	0.4	steel cable
	3.02	Electric cable
Safety coefficient	1	
	13.08	kg/m
	128	N/m

- R_{12x} (N, lb): **13333**
- R_{12y} (N, lb): **3200**
- R_{12} (N, lb): **13712**
- theta (degrees): **13.5**
- s (m, ft): **50.5**
- Sag: **3 m**



The wind load is 8.42 kg/m. For 50.5 m, the load on the cable is 4171 N.

The total load on the cable is $T_{max} = \sqrt{Tcat^2 + Twind^2}$= 14.332 kN or 1461 kgf.

Tentatively, 10 or 12 mm cable is recommended to be checked with manufacturers to see what is locally available (safe load has a safety factor of 5 that may not be necessary). Wire rope accessories need to be standard for the cable size.

Rope Diameter		Minimum Breaking Strength		Safe Load		Weight	
(in)	*(mm)*	*(lb$_f$)*	*(kN)*	*(lb$_f$)*	*(kN)*	*(lb$_m$/ft)*	*(kg/m)*
1/4	6.4	5480	24,4	1100	4.89	0.11	0.16
5/16	8	8520	37,9	1700	7.56	0.16	0.24
3/8	9.5	12200	54,3	2440	10.9	0.24	0.36
7/16	11.5	16540	73,6	3310	14.7	0.32	0.48
1/2	13	21400	95,2	4280	19.0	0.42	0.63
9/16	14.5	27000	120	5400	24.0	0.53	0.79

Figure 15.Minimum breaking strength and safe load for Bright wire, uncoated, fiber core (FC) wire rope, improved plow steel (IPS). Source: Engineering Toolbox

Lattice tower

A 7.5 m by 0.80 cm pyramidal truss tower with axial loads only. Chords are made of S275 steel 70.6 angle iron and bracing consists of 40.4 S275 angle iron. See figure 22.

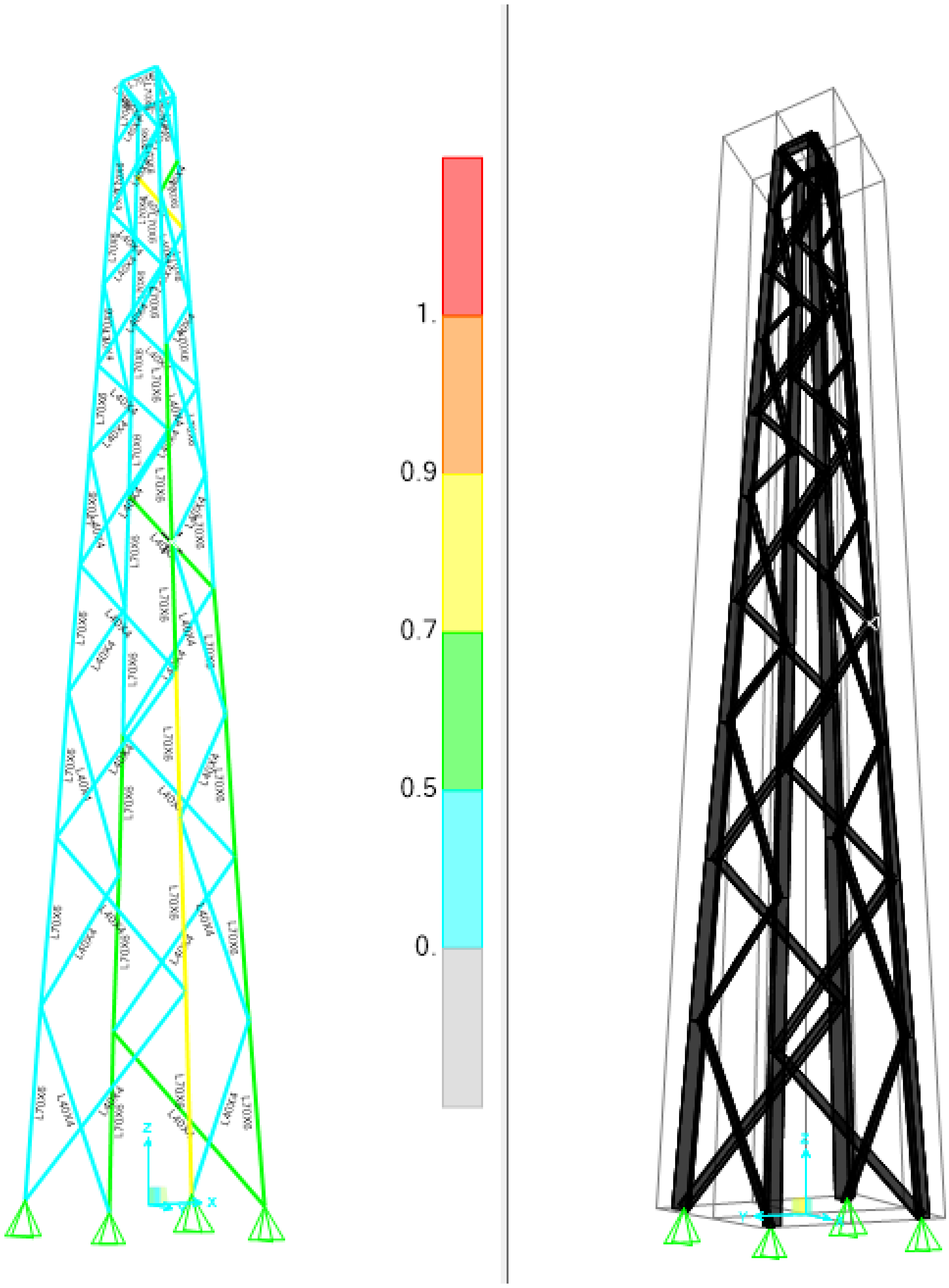


Figure 16. Ground view of the lattice tower (right) and member ratio (left) for the least favorable load combination.

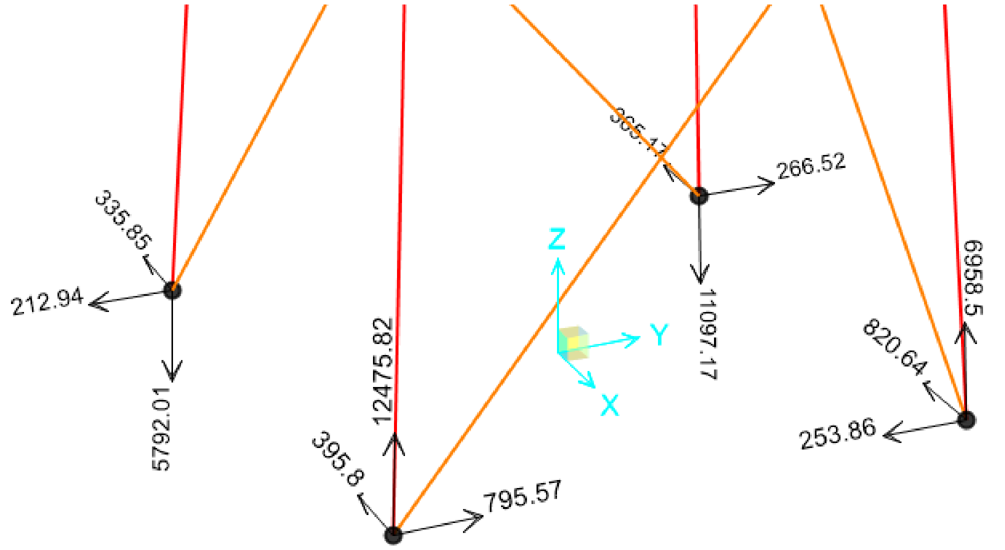


Figure 17. Reactions on tower's supports in kgf (maximum envelope).

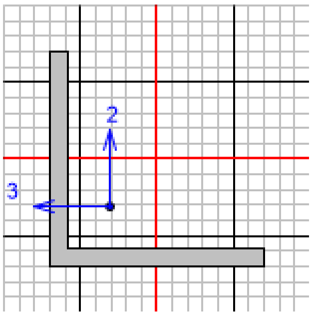
Frame element results

TABLE: Steel Design 1 - Summary Data - AISC 360-10				
Frame	**DesignSect**	**DesignType**	**Ratio**	**Combo**
Text	Text	Text	Unitless	Text
112	L70X6	Column	0.769832	DSTL1
113	L70X6	Column	0.760744	DSTL1
172	L40X4	Brace	0.72088	DSTL1
114	L70X6	Column	0.700414	DSTL1
115	L70X6	Column	0.674922	DSTL1
144	L70X6	Column	0.650324	DSTL5
143	L70X6	Column	0.605781	DSTL5
116	L70X6	Column	0.58977	DSTL1
100	L70X6	Column	0.584226	DSTL1
141	L40X4	Brace	0.577431	DSTL6
117	L70X6	Column	0.555008	DSTL1
145	L70X6	Column	0.550503	DSTL5
146	L70X6	Column	0.547257	DSTL5
98	L70X6	Column	0.540909	DSTL1
164	L40X4	Brace	0.540749	DSTL5
99	L70X6	Column	0.527699	DSTL1
168	L40X4	Brace	0.510792	DSTL1
170	L40X4	Brace	0.49969	DSTL1
166	L40X4	Brace	0.484397	DSTL5
82	L70X6	Column	0.484094	DSTL3
96	L70X6	Column	0.477992	DSTL1

81	L70X6	Column	0.452501	DSTL3
148	L70X6	Column	0.45068	DSTL5
97	L70X6	Column	0.450267	DSTL1
118	L70X6	Column	0.444215	DSTL1
196	L40X4	Brace	0.440037	DSTL1
152	L70X6	Column	0.427708	DSTL6
83	L70X6	Column	0.427412	DSTL3
147	L70X6	Column	0.422668	DSTL5
119	L70X6	Column	0.418838	DSTL6
137	L40X4	Brace	0.416084	DSTL6
139	L40X4	Brace	0.411701	DSTL6
94	L70X6	Column	0.391216	DSTL1
84	L70X6	Column	0.388036	DSTL3
150	L70X6	Column	0.373972	DSTL5
95	L70X6	Column	0.370701	DSTL4
171	L40X4	Brace	0.355817	DSTL1
135	L40X4	Brace	0.35479	DSTL4
85	L70X6	Column	0.335403	DSTL3
133	L40X4	Brace	0.323667	DSTL6
132	L70X6	Beam	0.321314	DSTL6
197	L40X4	Brace	0.321202	DSTL6
86	L70X6	Column	0.309033	DSTL3
149	L70X6	Column	0.305483	DSTL5
92	L70X6	Column	0.279127	DSTL4
103	L40X4	Brace	0.277288	DSTL5
140	L40X4	Brace	0.263563	DSTL6
169	L40X4	Brace	0.258205	DSTL1
87	L70X6	Column	0.254059	DSTL3
105	L40X4	Brace	0.252325	DSTL3
93	L70X6	Column	0.241646	DSTL4
120	L70X6	Column	0.231909	DSTL1
167	L40X4	Brace	0.23133	DSTL5
198	L40X4	Brace	0.218358	DSTL3
109	L40X4	Brace	0.215945	DSTL3
88	L70X6	Column	0.21035	DSTL3
121	L70X6	Column	0.2084	DSTL6
70	L70X6	Beam	0.205755	DSTL5
138	L40X4	Brace	0.200229	DSTL6
107	L40X4	Brace	0.198202	DSTL5
165	L40X4	Brace	0.196812	DSTL5
79	L40X4	Brace	0.186175	DSTL5
102	L40X4	Brace	0.185451	DSTL10
101	L70X6	Beam	0.180932	DSTL5
136	L40X4	Brace	0.180116	DSTL6
151	L70X6	Column	0.178969	DSTL5
134	L40X4	Brace	0.175985	DSTL6

91	L70X6	Column	0.165133	DSTL1
163	L70X6	Beam	0.16234	DSTL5
75	L40X4	Brace	0.161644	DSTL5
73	L40X4	Brace	0.159984	DSTL5
89	L70X6	Column	0.15723	DSTL3
77	L40X4	Brace	0.154102	DSTL3
110	L40X4	Brace	0.15226	DSTL5
104	L40X4	Brace	0.14643	DSTL5
78	L40X4	Brace	0.140149	DSTL5
195	L40X4	Brace	0.133835	DSTL5
72	L40X4	Brace	0.131474	DSTL5
106	L40X4	Brace	0.130931	DSTL5
74	L40X4	Brace	0.12939	DSTL5
76	L40X4	Brace	0.1293	DSTL5
71	L40X4	Brace	0.12366	DSTL3
108	L40X4	Brace	0.122526	DSTL5
90	L70X6	Column	0.065544	DSTL5

Frame check column




```
AISC 360-10 STEEL SECTION CHECK    (Summary for Combo and Station)
Units  :  Kgf, mm, C

Frame :  112       X Mid:  386.69     Combo:  DSTL1          Design Type:  Column
Length:  800.      Y Mid:  -386.69    Shape:  L70X6          Frame Type:  SMF
Loc   :  800.      Z Mid:  399.557    Class:  Compact        Princpl Rot: 45. degrees

Provision: LRFD   Analysis: Direct Analysis
D/C Limit=0.95    2nd Order: General 2nd Order         Reduction: Tau-b Fixed
AlphaPr/Py=0.528  AlphaPr/Pe=0.253  Tau_b=0.997        EA factor=0.8      EI factor=0.8

PhiB=0.9          PhiC=0.9          PhiTY=0.9          PhiTF=0.75
PhiS=0.9          PhiS-RI=1.        PhiST=0.9

A=812.7           I33=368800.       r33=21.302         S33=7271.293       Av3=420.
J=9648.           I22=368800.       r22=21.302         S22=7271.293       Av2=420.
Ixy=-224632.84    Imax=593432.84    rmax=27.022        Smax=11989.154
Rot= 45. deg      Imin=144167.16    rmin=13.319        Smin=5226.339
E=21414.04        Fy=28.042         Ry=1.1             z33=13540.
RLLF=1.           Fu=43.848                            z22=13540.
```

STRESS CHECK FORCES & MOMENTS (Combo DSTL1)

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
800.	-12043.524	11575.219	10245.681	-11.841	-14.701	-57.165

PMM DEMAND/CAPACITY RATIO (H2-1)
D/C Ratio: 0.77 = 0.72 + 0.045 + 0.005
= fa/Fa + fbw/Fbw + fbz/Fbz

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H2-1)

Factor	L	K1	K2	B1	B2	Cm
Major Bending	1.	1.	1.	1.	1.	0.601
Minor Bending	1.	1.	1.	1.	1.	1.

	Lltb	Kltb	Cb
LTB	1.	1.	1.

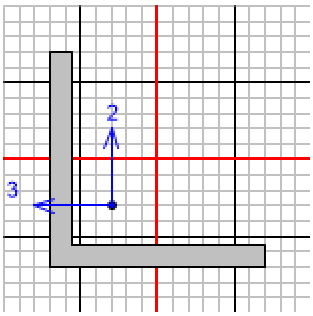
	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	-12043.524	16728.807	20510.903

	Mu Moment	phi*Mn Capacity	phi*Mn No LTB	phi*Mn Cb=1
Major Moment	15429.707	341722.192	341722.192	441674.854
Minor Moment	-940.125	197853.313		

SHEAR CHECK

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check
Major Shear	11.841	6359.97	0.002	OK
Minor Shear	14.701	6359.97	0.002	OK

Frame check brace



AISC 360-10 STEEL SECTION CHECK (Summary for Combo and Station)
Units : Kgf, mm, C

Frame : 172	X Mid: 11.646	Combo: DSTL1	Design Type: Brace
Length: 791.786	Y Mid: 185.383	Shape: L40X4	Frame Type: SMF
Loc : 791.786	Z Mid: 6442.855	Class: Compact	Princpl Rot: 45. degrees

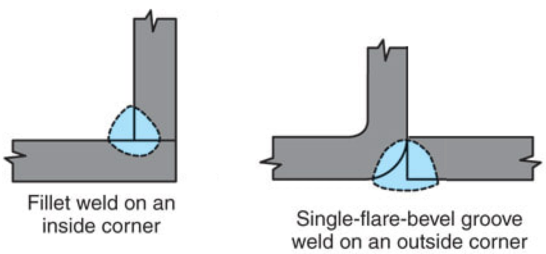
Provision: LRFD	Analysis: Direct Analysis		
D/C Limit=0.95	2nd Order: General 2nd Order	Reduction: Tau-b Fixed	
AlphaPr/Py=0.309	AlphaPr/Pe=0.453 Tau_b=1.	EA factor=0.8	EI factor=0.8

PhiB=0.9	PhiC=0.9	PhiTY=0.9	PhiTF=0.75
PhiS=0.9	PhiS-RI=1.	PhiST=0.9	

```
A=307.9             I33=44720.          r33=12.052          S33=1552.778        Av3=160.
J=1621.             I22=44720.          r22=12.052          S22=1552.778        Av2=160.
Ixy=-27277.79       Imax=71997.79       rmax=15.292         Smax=2545.506
Rot= 45. deg        Imin=17442.21       rmin=7.527          Smin=1088.729
E=21414.04          Fy=28.042           Ry=1.1              z33=2910.
RLLF=1.             Fu=43.848                               z22=2910.


STRESS CHECK FORCES & MOMENTS (Combo DSTL1)
    Location                 Pu         Mu33         Mu22          Vu2          Vu3           Tu
    791.786            -2664.54     3592.739     1238.171       -3.126        0.426       12.283

PMM DEMAND/CAPACITY RATIO    (H2-1)
    D/C Ratio:     0.721 = 0.634 + 0.047 + 0.04
                        = fa/Fa + fbw/Fbw + fbz/Fbz

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
    Factor                    L           K1           K2           B1           B2           Cm
    Major Bending            1.           1.           1.           1.           1.           1.
    Minor Bending            1.           1.           1.           1.           1.           1.

                           Lltb         Kltb           Cb
    LTB                      1.           1.           1.

                             Pu      phi*Pnc      phi*Pnt
                          Force     Capacity     Capacity
    Axial              -2664.54     4202.925     7770.773

                             Mu       phi*Mn       phi*Mn       phi*Mn
                         Moment     Capacity       No LTB         Cb=1
    Major Moment       3415.969    73442.509    73442.509    87762.684
    Minor Moment      -1664.932    41215.968

SHEAR CHECK
                                                               Vu      phi*Vn       Stress       Status
                                                         Force     Capacity       Ratio        Check
    Major Shear           3.126     2422.846        0.001           OK
    Minor Shear           0.426     2422.846           0.           OK

BRACE MAXIMUM AXIAL LOADS
                              P            P
                           Comp         Tens
    Axial             -2674.003           0.
```

Welding strength

There will be two types of welds:



Φ= 0.75 (LRFD)

Flare bevel groove weld

Not planned.

Fillet weld

$$R_n = F_{nw} A_{we}$$

$$F_{nw} = 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta), \text{ ksi (MPa)}$$

For the L40.4 profile:

The effective throat is 70% of t, 2.8 mm. F_{Exx} is taken as the same as the metal.

0.75 R_n =0.60* 235 *1.5 * 2.8 mm^2 * 0.75 = 444.15 N/mm or 0.444 kN/mm LRFD strength

Ω corrected flare bevel groove weld contribution= 100* 0.444 kN / 2 = 22.2 KN Ok

For the L70.6 profile:

The effective throat is 70% of t, 4.2 mm. F_{Exx} is taken as the same as the metal.

0.75 R_n =0.60* 235 *1.5 * 2.4 mm^2 * 0.75 = 666.3 N/mm or 0.444 kN/mm LRFD strength

Ω corrected flare bevel groove weld contribution= 100*mm 0.666 kN / 2 = 33.3 KN Ok

Frames should be welded at all their contact surfaces. See construction details.

Bolted joints

The bolted joints of these lattice structures are well known from practice, no calculations are deemed necessary.

For the tower span unions, two bolts are required for redundancy. Bolts are to be **M14 grade 5.8** or higher on a 16 mm hole. The distance to the edge of the profile would e at least 18mm and the separation between bolts 7 cm.

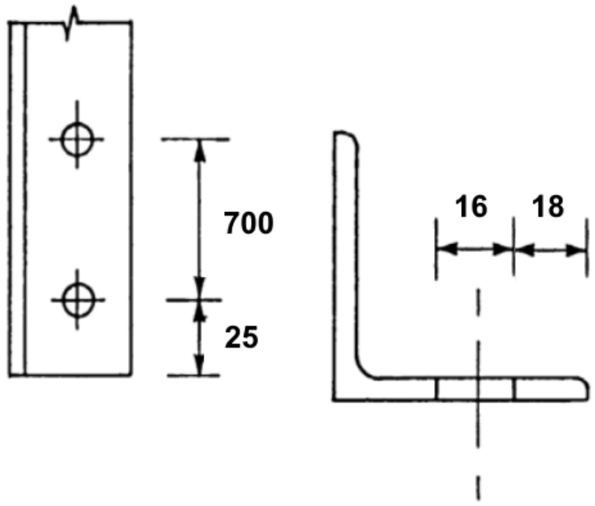


Figure 18. Span bolted joints construction details. Bolts are M14 grade 5.8 or higher on a 16 mm hole.

Stability calculations

A. Pylons in wadis

The structure is laid on very loose, fine river sand. Wet values are used:

- Bearing capacity: 0.15 kgf/cm^2 (J. Montoya).
- Friction angle as low as 15°.

Sinking

Using Tergazhi's method to determine the soil-bearing capacity of wet loose sand:

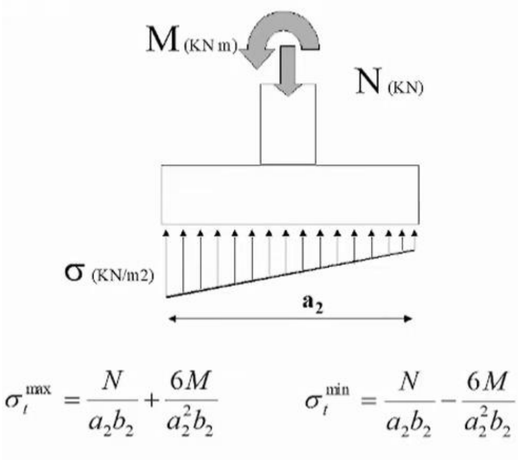
A. Input

Soil type	Wet loose sand		
Depth (Df)	1	m	
Width	3	m	
Cohesion (c')	0		
γ	18	kN/m3	
Safety factor (FS)	3		
ϕ	Nc	Nq	Nγ
15	12.86	4.45	1.52
20	17.69	7.44	3.64
26	27.9	14.21	9.84
30	37.16	22.46	19.13

B. Results

ϕ	Qu (kN/m2)	Qnet (KN/m2)	Qnet (kg/cm2)
15	112.932	37.644	0.38
20	212.544	70.848	0.72
26	468.324	156.108	1.59
30	817.488	272.496	2.78

The load distribution of a 3x3 m square foundation (end triangles and circles ignored for simplicity) is the following:



A. Data input

N	119.30	kN
M	112.5	kN/m
a2	3	m
b2	3	m

B. Results

Eccentricity (e)	0.94	m
Tension (σ_{max})	144.30	kN/m
Tension (σ_{min})	94.30	kN/m

144.30 < 156.1 Ok for soils with an angle of friction equal to or greater than 26 °. Those lesser than that may require a geometric modification if the safety factor is to be kept at 3.

Overturning

The least favorable case is when the wadi is flowing adding a flotation force that reduces the weight from 2.5 tn/m^3 to 1.5 tn/m^3. The safety coefficients are 1.5 (destabilizing) and 0.9 (stabilizing).

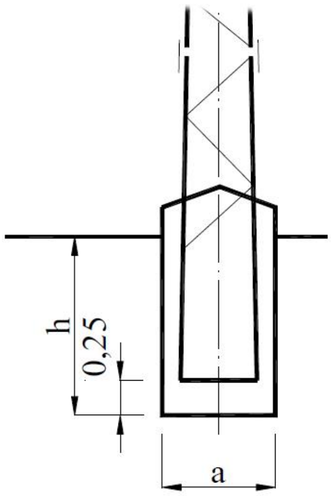
Stabilizing		F (kN)	d(m)	M (kN/m)
e1	Foundation weight	119.30	1.5	178.95
e2	Catenary dead and live weight	6.47	1.5	9.70
e3	Wellhead structure weight	10	1.5	15
De-Stabilizing				
d1	Cable pull	15	7.5	112.5
		Ratio	1.63	>1.5 ok

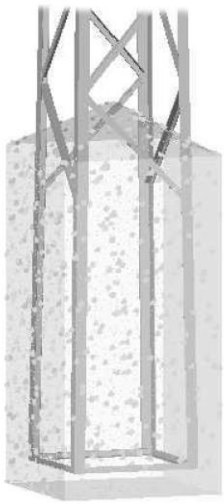
Sliding

Friction angle	15	
Stabilizing (Vs)	38.90	kN
Destabilizing Vd	12.74	kN
	3.05	>1.6 Ok

B. Pylons outside wadis

The foundation consists of an HB-20 mass concrete block with a square section. The top is finished pyramidal to avoid standing waters:





The dimensions are calculated using the **Sulzberger method** with a 1.5 safety coefficient.

$$M_V = F \cdot \left(H_L + \frac{2}{3} h \right) = F \cdot \left(H - \frac{1}{3} h \right)$$

$$M_e = \frac{b \, . \, h^3}{36} C_{t'} \, . \, tg\alpha + P \, . \, a \left(0,5 - \frac{2}{3} \sqrt{\frac{P}{2 \, . \, a^3 \, . \, C_{t'} \, . \, tg\alpha}} \right)$$

Simplification for tnα= 0.01

$$M_e = 139 \,.\, k \,.\, a \,.\, h^4 + 2200 \,.\, a^3 \,.\, h \,.\, 0{,}4$$

$$C_s = \frac{M_e}{M_v} \geq 1{,}5$$

For a foundation that is 2,25 m deep and 0.8 m wide on both sides the results are shown below:

A. Data input			
Load (F)	1500	kg or daN	
Free standing height (HL)	7.5	m	
Block height (h)	2.25	m	
Side (a=b)	0.8	m	
Soil compressibility coefficient (k)	8	kg/cm3	Poor soils
Safety coefficient (Cs)	1.5		
B. Secondary calculated data			
Post height (H)	9.5	m	
Block weight (P)	3168	kg or daN	
C. Results			
Stabilizing moment (Me)	23813	Kg/m or DaN/m	
De-stabilizing moment (Mv)	13125	Kg/m or DaN/m	
Mv*cs	19687.5		

23813>19687 Ok

Alternatively, <u>if narrow pits cannot be dug wider pits with at least 2m depth can be dug</u>.

A. Data input			
Load (F)	1500	kg or daN	
Free standing height (HL)	7.5	m	
Block height (h)	2	m	
Side (a=b)	1.3	m	
Soil compressibility coefficient (k)	8	kg/cm3	Poor soils
Safety coefficient (Cs)	1.5		
B. Secondary calculated data			
Post height (H)	9.25	m	
Block weight (P)	7436	kg or daN	

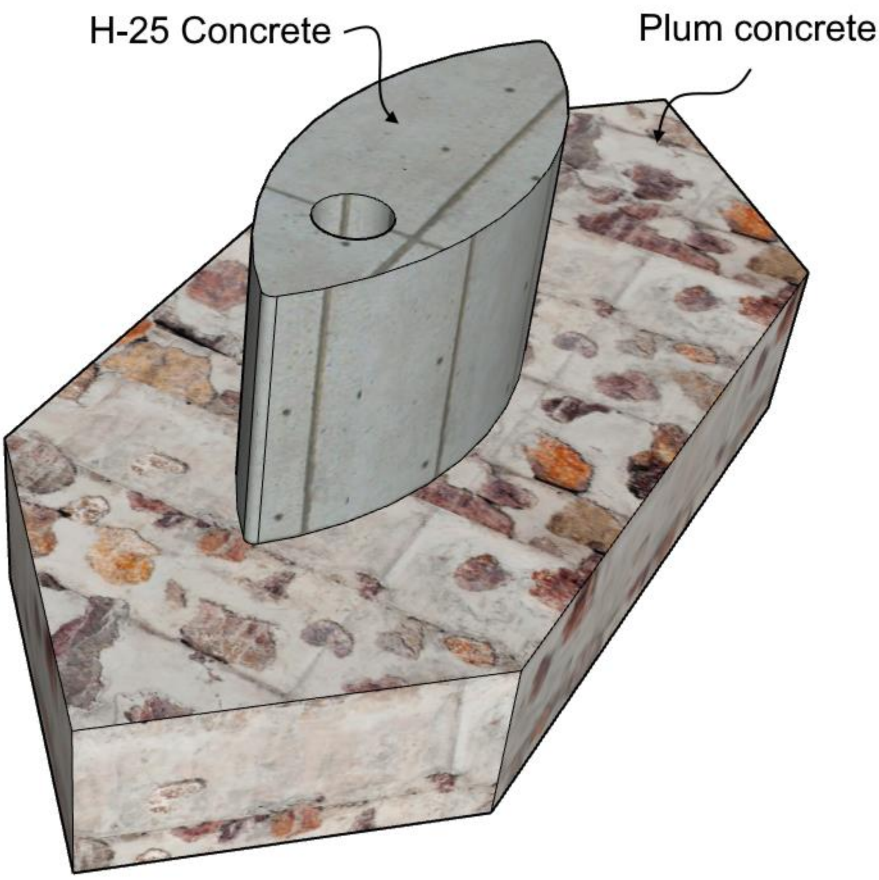
C. Results

Stabilizing moment (Me)	26996	Kg/m or DaN/m
De-stabilizing moment (Mv)	12875	Kg/m or DaN/m
Mv*cs	19312.5	

26996>19312 Ok.

Mass concrete reinforcement

The structure is made up of mass concrete **reinforced to prevent cracking**. Using cyclopean concrete in the hexagonal part (plum concrete) saves 30-40 % of concrete by weight and allows to recycle of existing gabion protection. The sail is made of standard H-25 concrete.



To prevent the excessive use of geometric reinforcement in big sections of mass concrete, only the outer 25 cm are computed in the formula as prescribed in BS 6349-2.

Minimum geometric reinforcement (EHE): (2‰)

$\rho = A_s/A_c > 0.002$ $0.002 \cdot (25+25) \cdot 100/2 = 5\ cm^2/m$

Reinforcement required: **5ø12 at 0.2 m** ($5.65\ cm^2 > 5\ cm^2/m$ Ok)

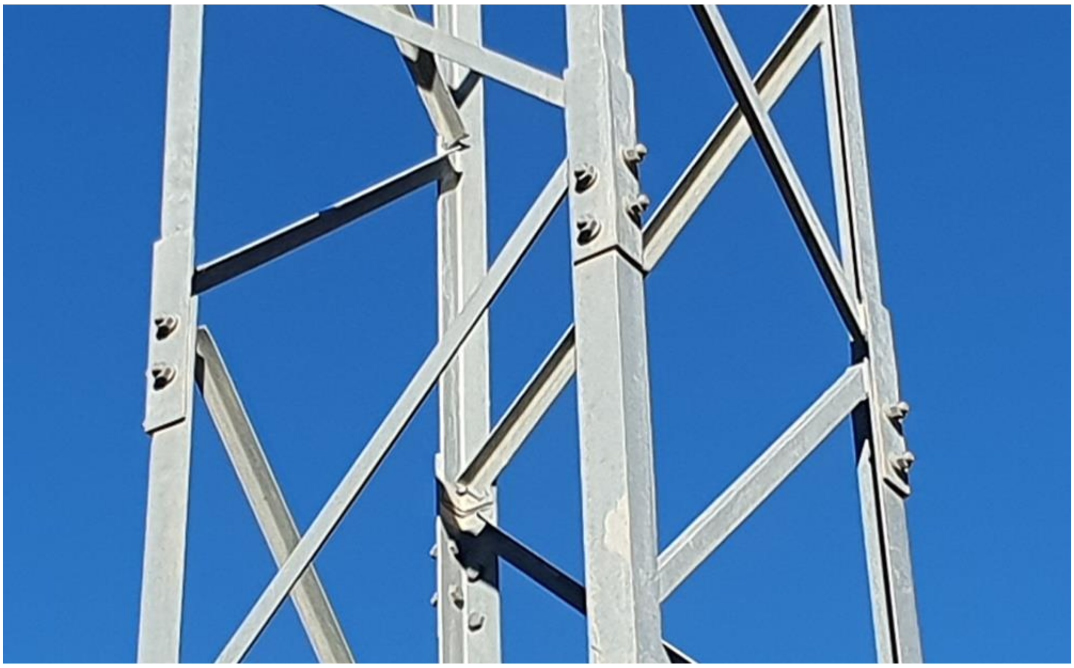
The reinforcement cover is 5-7 cm.

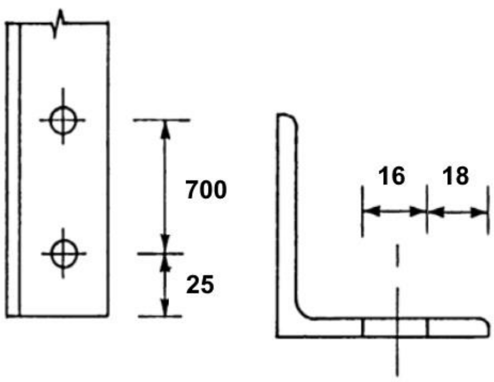
The reinforcement is shown in figure 21.

ANNEX 3 Construction details

- The Foundation slab is to be given a top pyramid shape to avoid standing waters that can corrode the chords:

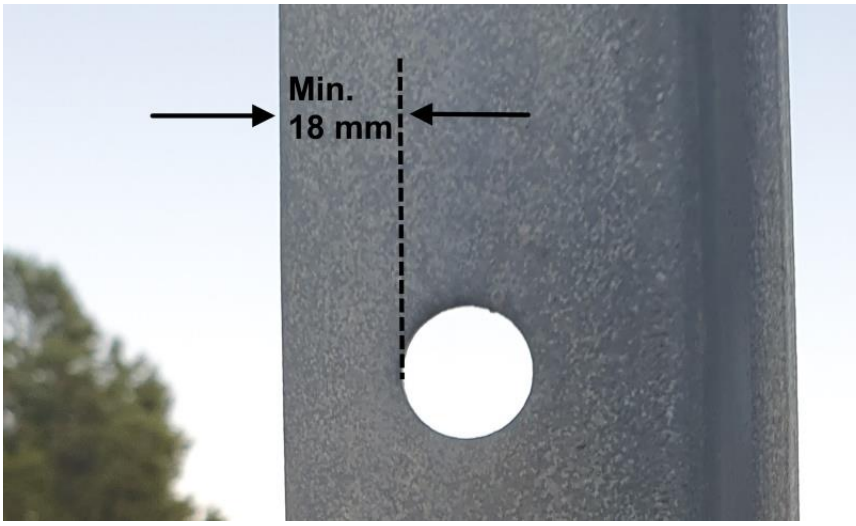
- Tower spans to be joined with a bolted plate with two M14 grade 5.8 or higher bolts:




700
25
16
18

- Bolt threads need to be flattened to avoid accidental loosening with vibration. Use thick washers if they are available:

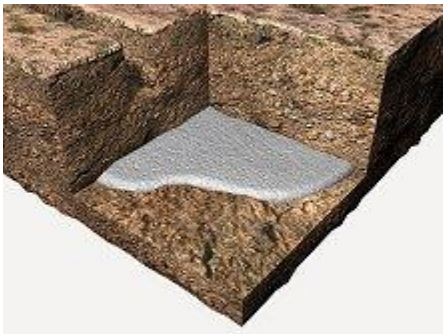
- Respect bolt diameter and tear-out margin:



- Use galvanized profiles and weld with galvanic electrodes rather than painting. Weld all contours (see minimum length calculations).

- Sample pipe cable clamp accessory:

- Use 10 cm of **lean concrete** before any proper concreting work, to level, avoid contamination, place rebar supports and keep humidity conditions.



ANNEX 4. Guiding drawings

Detailed drawings in the French language are to be finalized locally. These schematic drawings are meant to convey key information to understand the proposed system.

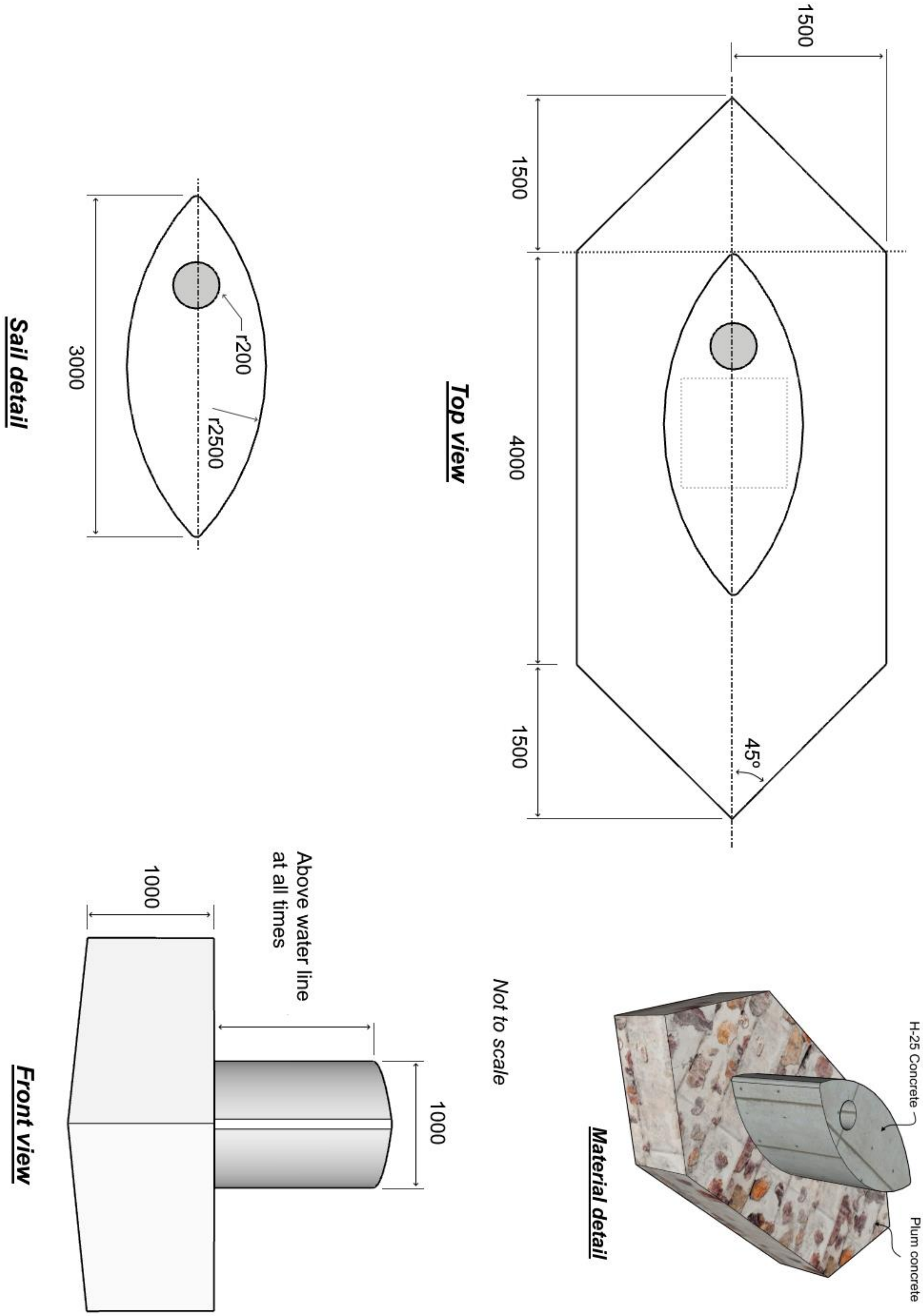


Figure 19. Counterweight base specifications.

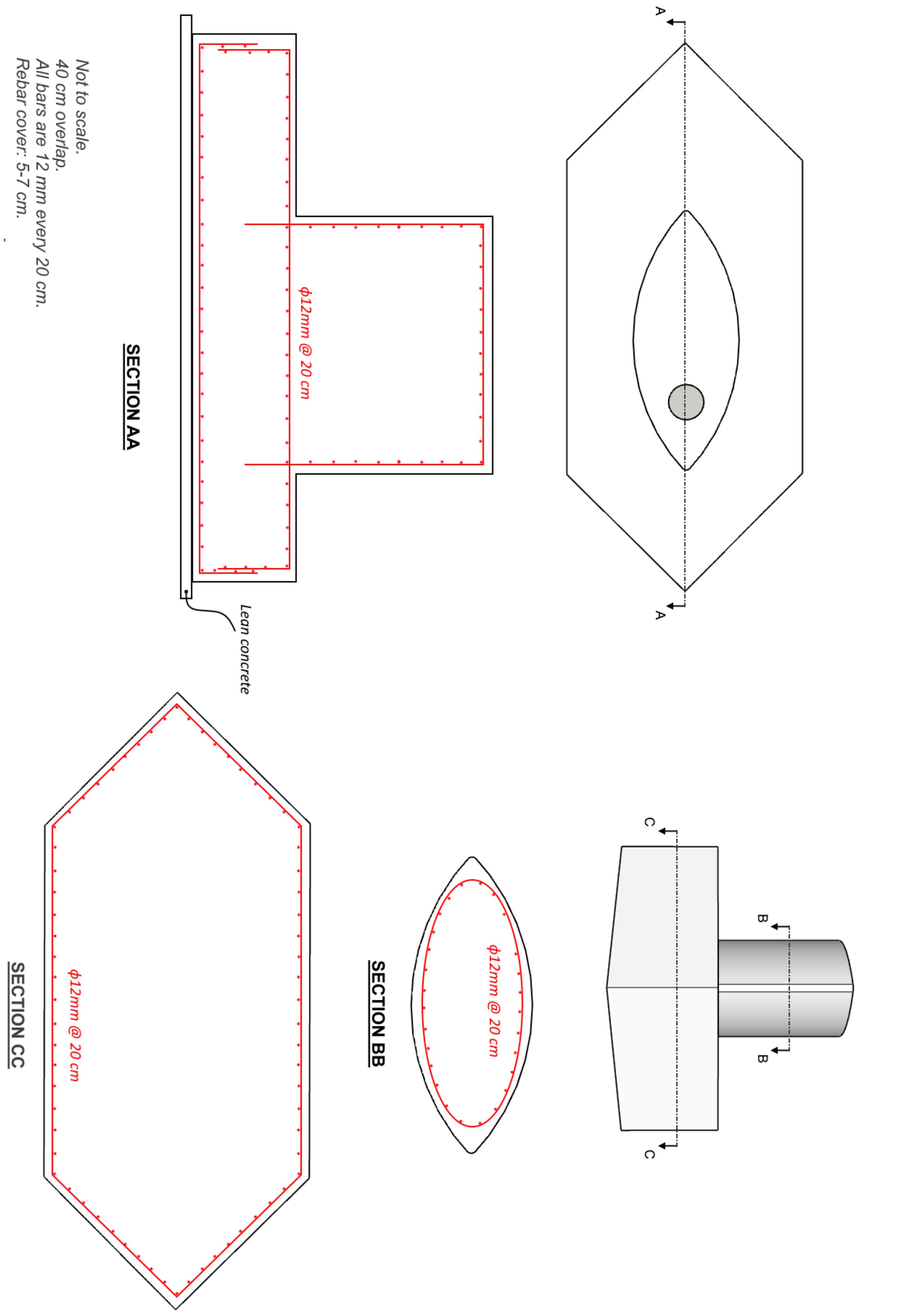


Figure 20. Counterweight base reinforcement.

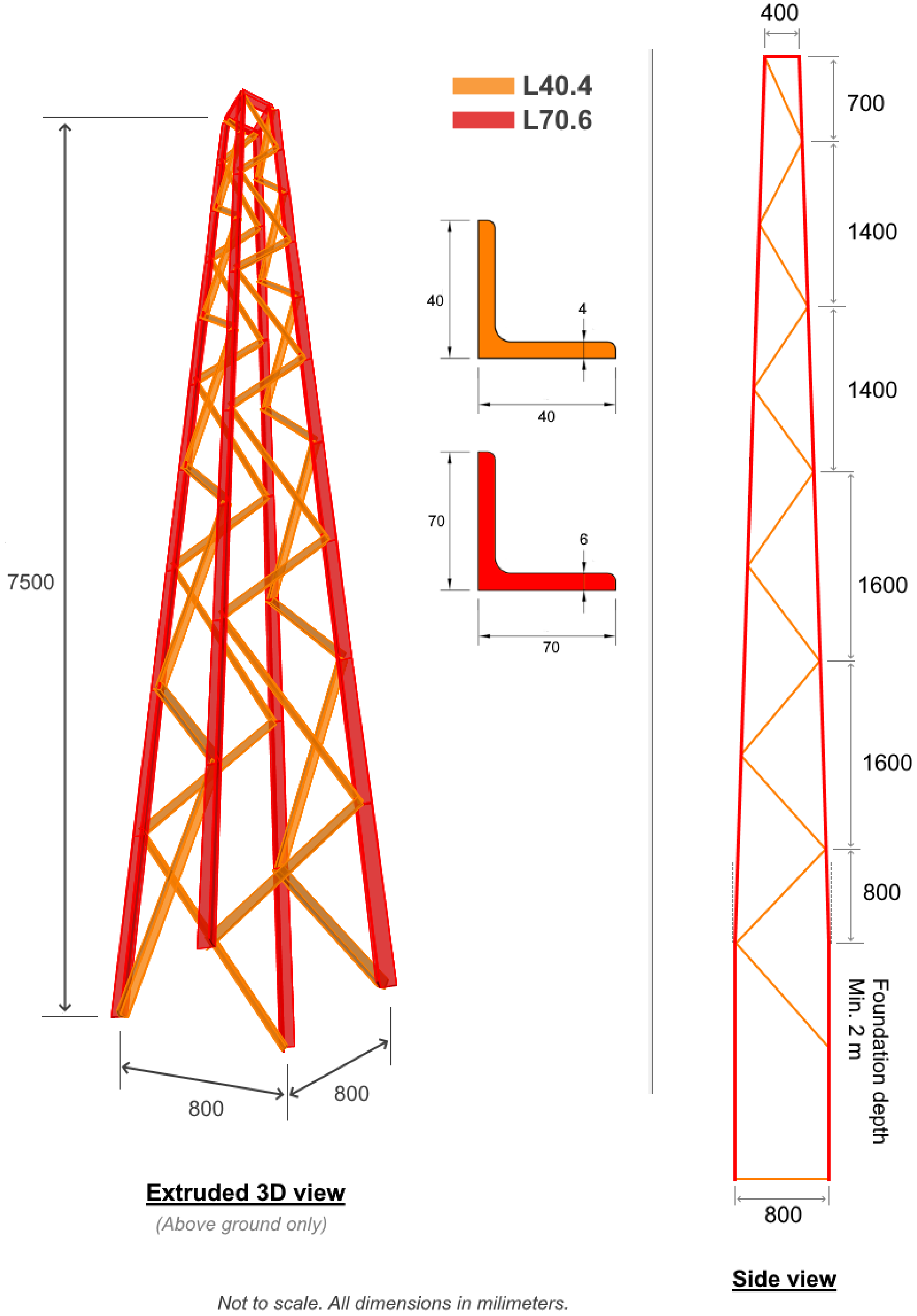


Figure 21. Steel lattice tower details. Steel grade S275 or higher.