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Design and construction of a high and heavy lattice tower for 380 kV transmission

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SUMMARY

In the scope of the so-called energy transition "Energiewende", TenneT upgrades the old 220 kV transmission line between the substation Stade and the substation Landesbergen to a 380 kV line. This transmission line of about 155 km will increase amongst others the power capacity for wind energy from the north to the south. The development of an out of ordinary lattice tower (N.19) belongs to this line, customized to the location restrictions and project requirements. This tower has a 4-circuit double barrel shape, called in German "Doppeltonne", and reaches a total height of 99.5 m. For such a decisive height, the use of suspension towers is preferable because of the significantly smaller loading. Such an example is the Elbe Crossing 2 (227 m height) in Northern Germany [1] or the Tucurui-Macapa-Manaus in Northern Brazil [2].

However, due to the sharp deviation angle of 106.8° in this location and the need of bearing end loads (dead-end tower), the only possibility was to use a tension pole. These technical requirements result in characteristic forces of steel legs of about 11 MN in compression and 8.6 MN in tension, which correspond to the double forces in comparison to the other poles of the transmission line. The development of this tower was realized by TenneT in cooperation with the engineering partner EQOS, who is a specialist in the power sector. The challenge of this design was the development of a rigid and robust construction, produced of as much as possible common steel profiles. Although the profiles are common (L-angles), the final design cannot be calculated with the standard software, because it is an elaborated lattice structure. The paper describes the combination of calculating methods and the assumptions made for the dimensioning of the structural construction, as only one conventional method was not enough. The focus lays on two technical aspects; on one side the design and the structural analysis of the tower (part A); on the other side the construction details and technical challenges (part B).

The experience gathered from this development can be used for future constructions of similar loading and complexity. The interest of this design focuses more on the stability and robustness of the construction than the aesthetic. The advantage is that this tower is a self-supporting construction (no guyed), which bears heavy loads, needs not too much land easement, and can be realized with common steel profiles. This kind of poles is suitable for cases like river or highways crossings, for regions with tight area restrictions or for branch lines where a tension pole is needed.

KEYWORDS: high tension pole, heavy loads, challenging constructions, river crossing, load path, rigid tower

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1. Introduction

As part of the 380 kV transmission line LH-14-3110, which will replace the existing corridor 'Stade-Landesberger' in Northern Germany till 2026, a heavy tension tower has been developed at the site position number 19 (named "Tower 19"). Due to high demands on ground clearance of the project, the Tower 19 reaches the height of 99. 5m. The deviation angle of 106.8° at this place caused by the line junction required a tension pole with the possibility to bear the loads only one-sided of cross beams and/or one-sided in the line direction (dead-end tower). At the same time there was a land easement restriction for the base which led to the bottom width of only 20 m. This restriction resulted in heavy axial forces of $F_{d,compression}$=15,020 kN and $F_{d,tension}$=11,605 kN at the main legs, and made the reinforcement of the legs necessary, otherwise a base width of about 26-28 m would be needed. This heavy construction weighs about 600 ton.

2. Part A: "Design and structural analysis"

2.1. Materials

The Tower 19 is a steel construction of grade S355J2 for the profiles and S355J2+N for the flange plates. All parts are hot dip galvanised to avoid corrosion. For the bolting connections metric bolts 5.6 are used. The bundle-conductors are a combination of aluminium and steel called Finch 565-AL1/72-ST1A according to DIN EN 50182/12.01.

2.2. Model

The Tower 19 has been modelled as truss structure, which means that all the bracings have been modelled like discrete elements, transferring only axial forces, and the joints have been modelled as pin connections. This approach deviates slightly from reality since the joints are indeed semi-rigid and not pinned. The bending moments, which appear at the joints, are relatively small and according to the German Standard DIN EN50341 it is allowed to be neglected [3]. Moreover, the designer of the structure tried to minimize the eccentricities, by intersecting the system lines of vertical, horizontal, and diagonal trusses (Fig. 1).

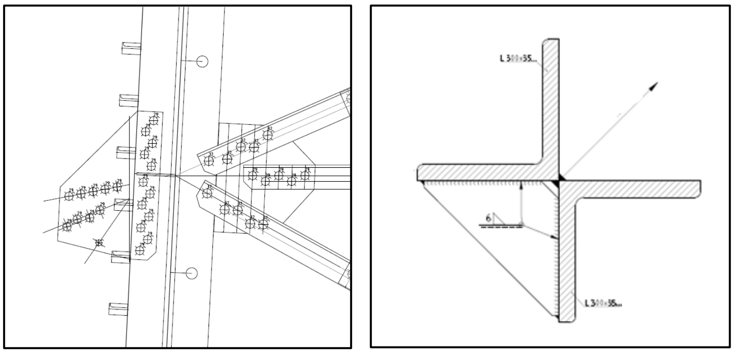

Figure 1 : (a) Intersection of system lines (b) Double L-angle welded in cross shape, the so-called ‘butterfly profile’

The whole structure is divided into upper (l=44.5 m) and lower part (l=55 m). The upper part is a common lattice tower of equal-leg angles in single and double arrangement (so-called ‘butterfly profiles’). For the lower part, the use of common L-angles was infeasible due to heavy axial loads. To bear these loads and secure the stability of the tower, an inter-reinforced structure has been designed. This structure consists of 4 external L-angles (4L-legs) connected to each other with welding plates and bolting (Fig. 2 & 3b).

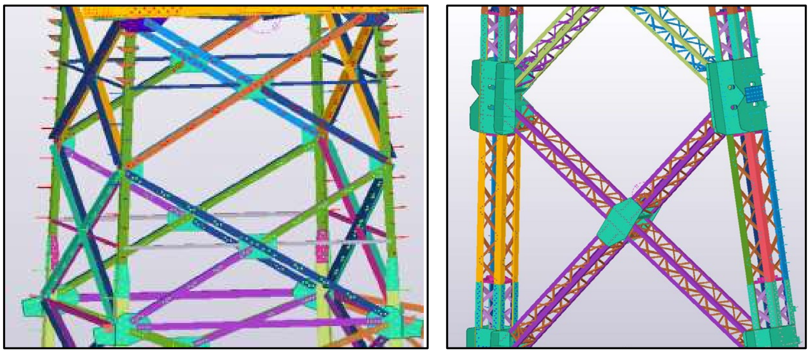

Figure 2 : Steel framework (a) Upper part (b) Lower part

The interconnection between upper and lower part was a big challenge for the designer because the joint should be neither completely rigid nor too soft. A pure welding construction would be inappropriate, as too rigid and a common bolting connection would be too soft. For that reason, a combination of both has been used by replacing the common bolts with fitting bolts DIN 7968 with tight tolerances (H11). The butterfly profiles of the upper part are enclosed to a welding construction, which is screwed to the 4L-legs at the edges (Fig. 3). Thus, the axial forces from the upper part are induced through the welding and bolting to the 4 L-legs at the edge. The welding plates serve also to bear the shear forces at this area.

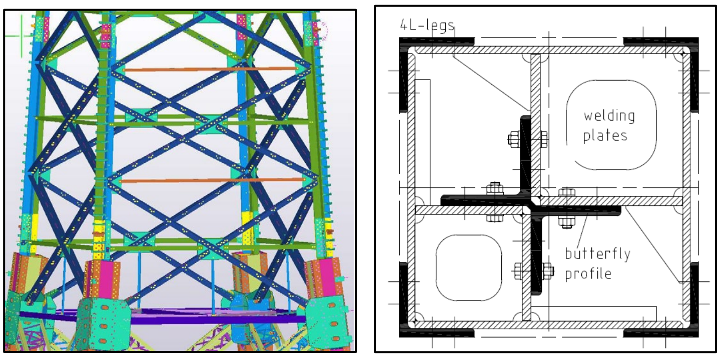

Figure 3 : (a) Transition between upper and lower part (b) Cutting section at the transition point

2.4. Loads

The Tower 19 has been calculated both as tension and dead-end pole. The extreme line deviation leads to excessive stresses caused by sheaving of conductors and more specifically at the dead-end situation. To present all the load cases in this paper is of course too extensive and of low interest, hence only the loads, which have an international interest, will be referred.

Wind loads

Due to strict project requirements for ground clearance, the height of the tower should be 99.5 m. In the international standard IEC60826 [4], there is no information about the wind loads at such a height. There is a limitation till 60 m. Even in the European standard the height for the wind loads is limited. The German standard [3] provides profiles of the dynamic wind pressure until 300 m above ground. These profiles are based on the logarithmic equations:

$$q_p(z) = 1.5 \times q_0 \qquad \text{für } z \leq 7 \text{ m}$$

$$q_p(z) = 1.7 \times q_0 \times \left(\frac{h}{10}\right)^{0.37} \qquad \text{für } 7 \text{ m} \leq z \leq 50 \text{ m}$$

$$q_p(z) = 2.1 \times q_0 \times \left(\frac{h}{10}\right)^{0.24} \qquad \text{für } 50 \text{ m} \leq z \leq 300 \text{ m}$$

The Tower 19 has been placed in the wind zone 3 in Germany inland, which corresponds to a basic wind velocity of q_o=470 N/m². The wind dynamic pressure has been calculated by using the above equations, exactly at the required height until the top of the tower q_o (99.5 m) = 1,713 N/m² (Fig.4).

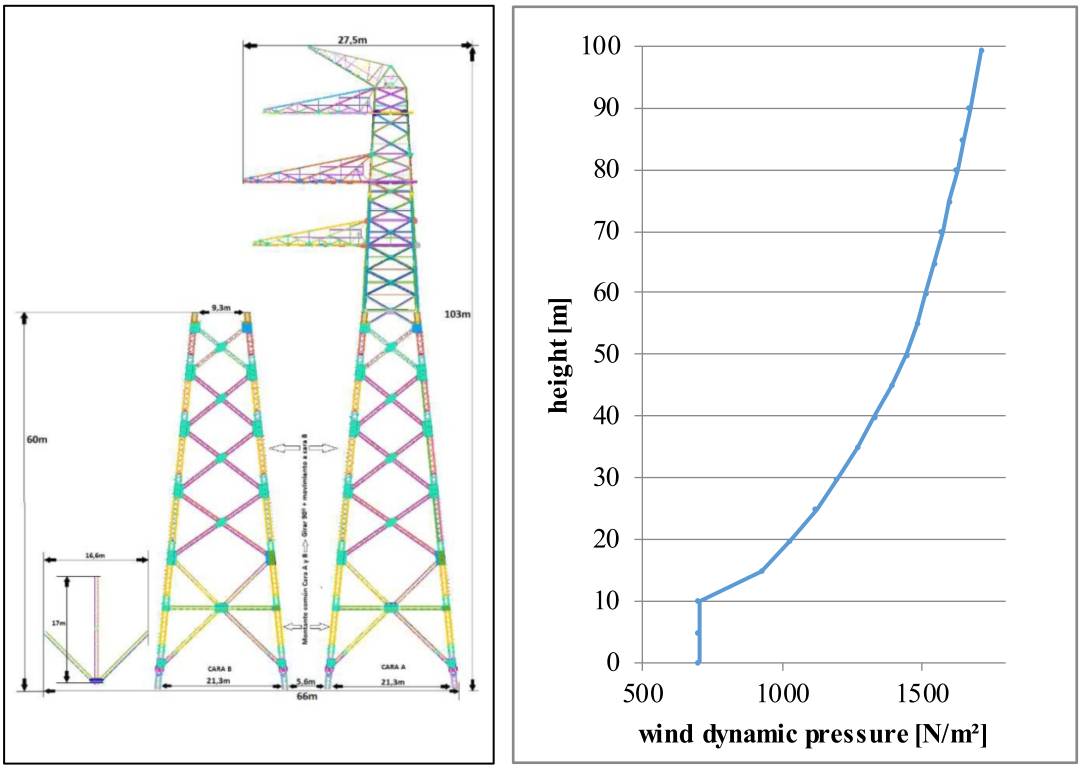

Figure 4 : Wind dynamic pressure till 99.5 m

The wind calculations have been performed according to DIN 50341-2-4. The lower part has been simulated with the overall width of the 4L-legs with an in-between width distance of 100 cm. The drag factor, based on the shape of the structure, is assumed to be 2.8 both for the front and back side. The used software could not automatically identify this factor for the lower part, because it is not a common structure. Thus, a post-test calculation should be performed to define the normal force coefficients for the 4L-legs, based on the standard 1993-3-1/Annex B (Eq.1). The wind loads have been adapted to these values.

$$c_{f,0,f} = 1.76\ C_1\ [1 - C_2 \varphi + \varphi^2] \qquad (1)$$

Conductor loads

As mentioned above, the conductor forces were decisive for the dimensioning of the tower. These conductors' forces result from the combination Ice-Wind (Load case D-F) and correspond to a maximal conductor tension of F_d=358 kN (σ_d=140 N/mm²) per conductor.

2.5. Methods

The structure of the lattice tower is a statically indeterminate structure, calculated with Finite Element Method Analysis (FEM). The calculations have been realized with the software Turrix, a software self-developed from the engineering office. The lower part has been simulated as one truss per main leg with the respectively stiffness of the inter-connected construction. The calculations occurred according to EC3 [5] based on second order analysis. While the common FEM analysis determines in accuracy the stress contribution of the structure, it provides less information about the load transfer at joints. For that reason, the FEM analysis has been combined with the load path analysis method, to optimise the structure and especially the joints. The article explains how the load path analysis has been applied and how unnecessary use of materials has been avoided.

Load path analysis

The load path analysis describes the trajectories which the load transfer follows from the point of load application to the point of reaction. The load path for a structure, constructed from truss elements such as a lattice tower, can be created by following force resultants from member to member across the domain [6]. The load paths in structural design have been studied by many scientists over the years. Its application has begun firstly at the airplane structures (Kermode, 1964; Osgood, 1970; French, 1992). The aim was to minimize the stress concentration and therefore the bending at the edges. For a better explanation, a mathematical equation was proposed in [7]. Through the years, the load path analysis extended to the pinned loaded connection structure (Kelly and Tosh, 2000).

Its application at the Tower 19 can be recognized at the joints of the diagonal bracing to the main legs at the lowest part of tower, just above the ground level. On one side, the axial forces are extremely high at that point ($F_{d,x}$=735 kN), on the other side this is a crucial position of structure, whose failure would lead to total collapse of the tower. The load path, marked in orange (Fig.5), shows that the load transfer of the upper diagonal occurs via the stiffest route, which is beneath the slotted hole. The load transfer of the lower diagonal occurs through the bracing to the main legs, without any resistance. That way the flow is facilitated, and it is not needed any local strengthening.

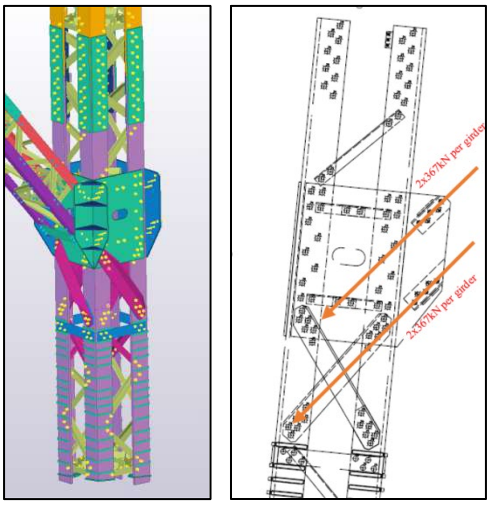

Figure 5 : Load transfer at the lower joint

The same concept has been used also for other connections, where the load path has been planned to be left and right from the slotted holes (Fig.6).

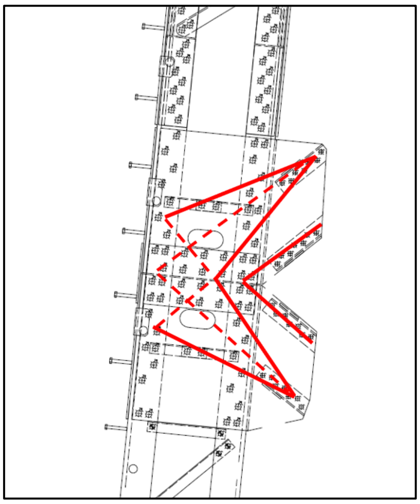

Figure 6 : Load transfer at the joint

Another critical position was the transition piece between upper and lower part (Fig. 7a). At this position, high compression-tension loads, and shear loads should be transferred. For this design joints of category A have been chosen predominantly, where the bolts are stressed perpendicular to their axis and work in shear and compression (Fig. 7b). The joints should not be totally rigid and at the same time the holes should not have big tolerances, otherwise the interaction of the bolts at load transfer could not be guaranteed. For this reason, there are used fitting bolts, in total 112 fitting bolts PM30, 28 bolts per bracket. The small tolerances of the holes (H11) and the avoiding of thread in the shear plane guarantee that the slippage is considerably smaller than the normal bolts. The thought of alternative rigid joints only with welding was unfavourable against this ductile connection. Thus, this connection is not the weakest point of the structure, and it ensures a ductile failure under overload.

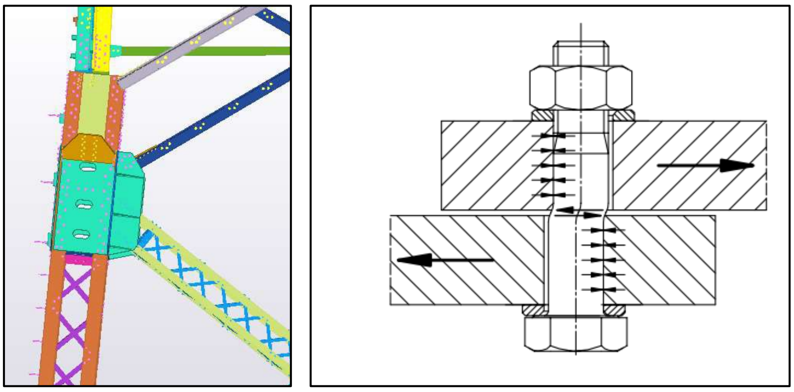

Figure 7 : (a) Transition upper-lower part (b) Shear and bearing connection

The initial idea was that the loads, transferred from the upper part to the lower part, should be distributed uniformly to the 4L-legs. However, this was not possible due to change of tapering from 100 mm/m to 200 mm/m and the attendance of shear hole connections outside. For that reason, it has been decided to have some structural reserve especially for the buckling verification of the plates (utilisation factor of the plates = 27 %,), while the utilisation of the rest structure amounts to about 90 %.

3. Part B – "Construction details"

3.1. Construction

The Tower 19 has a height of 99.5 m. Due to rolling, transportation and handling restrictions, it should be divided into 11 segments, 5 segments for the upper part and 6 segments for the lower part.

Tower - Upper part

The upper part is a common four-legged lattice tower with equal angle profile sized from L60x60 till L300x300. For the main legs, the double L-profile LL300x35 has been used, which shapes a cross via a fillet welding. The joints are constructed by using gusset plates and bolting for the bracing.

Tower - Lower part

Both main legs and diagonal bracings were constructed as an inter-jointed steel framework of isosceles L-profiles (Fig.8). The internal diagonals have an inclination of about 45 °, to achieve symmetry in the structure. The joints have been constructed in the same concept of upper part, a combination of gusset plates and bolting.

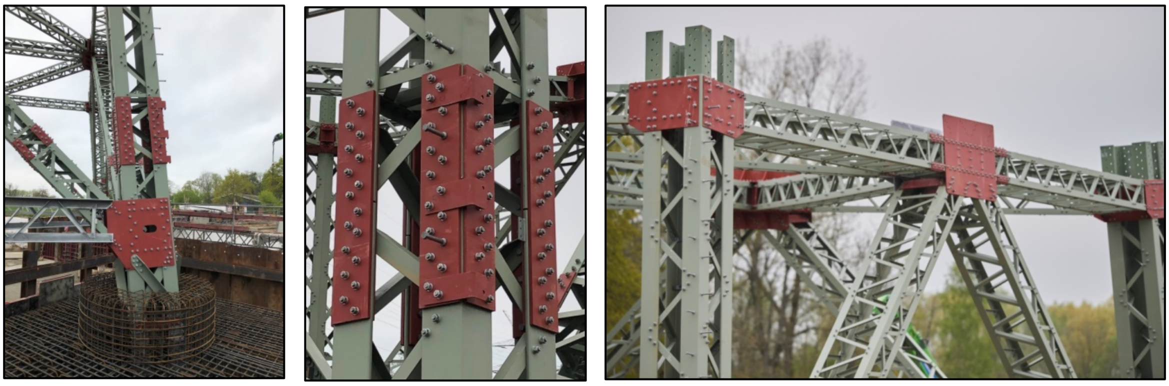

Figure 8 : Inter-jointed steel framework (4L-legs)

Transition piece

The combination of bolting and welding served to connect the main legs (LL300) of the upper part to the inter-jointed profiles of the lower part (Fig.9).

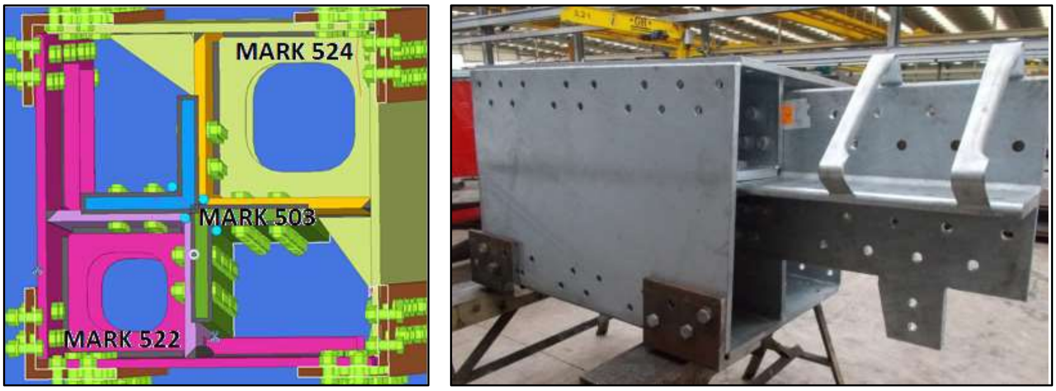

Figure 9 : Transition piece in steel manufacture

Foundations

The foundations are a combination of concrete piles of type Fundex and horizontal concrete beams, applicable for the lateral displacements (Fig.10). In total, 76 Fundex piles of diameter Ø44/56 cm and 4 steel tubes piles of diameter Ø 324x5 mm, with a length above 20 m have been used. The total concrete volume for the foundations amounts to about 700 m³ with steel reinforcement of about 13 tons.

Figure 10 : Foundations (a) main leg (b) connection beams

3.2. Montage

The assembly of the tower took place at the erection position. In terms of quality check, all segments were pre-assembled at the area of the manufacturing company (Fig.11). Therefore, the quality and the feasibility of mounting were checked in advance and some defects were repaired before the materials being delivered on site. The lower part was pre-assembled laid-on in L-form because a complete assembly was not possible. The crane supported the mounting of all parts.

Figure 11 : Pre-assembly at fabrication site

Each pre-assembly was inspected by a team member of TenneT, who gave the permission for the delivery. The assembly on site occurred also per segment. The installation of the conductors happened with the help of helicopter (Fig.12).

Figure 12 : Segmental mounting and mounting of conductors

The mounting of the transition piece was a big challenge for the team. The combination of bolting and welding under strict tolerances, disabled the work, was time-consuming and needed high accuracy. To facilitate the mounting team and to avoid irreversible mistakes on-site, it was decided that the transition piece should be delivered on-site preassembled. The implementation of fitting bolts M30 needed a special concept for the construction and mounting. Once the position of the holes was marked on the profiles, the drilling occurred, and the profiles were firstly fixed with normal M27 bolts. After that, the welding parts of the left side were removed, while the welding parts of the right side remained fixed to the longer parts for stabilisation. The holes for M27 were reamed to the diameter for the fitting bolts M30 – tolerances H11 [8]. To ensure that both holes are concentric and not losing the centre of the holes, an auxiliary guidance was used to position the drilling machine in all holes (Fig.13).

Figure 13 : Pre-assembly of the transition piece with help of auxiliary guidance

Once finishing all holes, the joint was completely re-assembled to rotate the part with the crane. After rotating the joint, the same process took place the other way around. In the end all the parts were disassembled to proof the condition of the holes and the zinc layer. Where necessary, the surface was re-covered with zinc paint (Fig.14).

Figure 14 : Transition piece (a) internal side (b) holes after reaming

4. Discussion

Although there are many high transmission towers worldwide, most of them are suspension towers. The particularity of Tower 19 is that it bears heavy loads due to its tension and dead-end function and it is positioned in relatively small land easement of 20 m. Without the inter-reinforced legs of the lower part, the land easement would exceed the 26 m. Moreover, the table 1 below enables the reader to get an idea of the axial forces at the base between a tension and suspension tower of a similar height. It is obvious that the loading of tension is about triple as that of suspension tower.

Table 1 : Comparison between tension and suspension tower – design loads

	$F_{d,compression}$	$F_{d,tension}$	M_d (in the centre)
Tension tower (h=99.5 m)	15,020 kN	11,605 kN	458,719 kNm
Suspension tower (h=97.5 m)	4,705 kN	3,543 kN	109,865 kNm

Another example, as reference for high construction, is the tower crossing the Amazon River in the northern region of Brazil [2]. As a suspension tower, the loads $F_{d,compresion}$=7,100 kN and $F_{d,tension}$=2,756 kN are significantly lower than Tower 19, but due to the extreme height of 293 m, it needs to cover a land easement of about 50 m, in order to guarantee its stability. The concept of Tower 19 could be implemented in such a case, to reduce the weight and the base width.

At an international level, the idea of a reinforced lower part with common equal-leg angles could be useful for projects with heavy loads and significant land restrictions [9]. This paper can be guidance both in terms of design and construction of such towers.

5. Conclusions

Tower 19 is a tension and dead-end transmission tower developed for the upgrade of corridor Stade-Landesbergen to 380 kV. Due to its height and heavy loads, resulting from the deviation angle, a special design with a sophisticated analysis has been implemented. Moreover, a special mounting concept to prevent irreversible mistakes was developed by the engineering team.

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