Supports

12

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Contents

12.1 Introduction 826
12.2 Types of Supports 827
  12.2.1 Regarding Function in the Line 827
  12.2.2 Number of Circuits/Phase Arrangements/Tower Top Geometry 828
  12.2.3 Structural Types, Structural Modeling 828
  12.2.4 Formats, Aspects, Shapes 829
  12.2.5 Material Used 830
12.3 Design Loadings 831
  12.3.1 Design Philosophy 831
  12.3.2 Loadings 836
  12.3.3 Static and Dynamic Loads 838
12.4 Structural Modeling 839
  12.4.1 Structural systems 839
  12.4.2 Structural Modeling 842
  12.4.3 Structural Analysis 844
  12.4.4 Advanced Tools and Techniques 846
12.5 Calculation and Dimensioning 850
  12.5.1 Materials and Standards 850
  12.5.2 Lattice Towers 853
  12.5.3 Metallic Poles 867
  12.5.4 Concrete Poles 868
  12.5.5 Wooden Poles 871
12.6 Detailing Drawings and Fabrication Process 871
  12.6.1 Lattice Supports 871
  12.6.2 Metallic Poles 879

Originally published by Cigré, 2014, under the ISBN 978-2-85873-284-5. Republished by Springer International Publishing Switzerland with kind permission.

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K.O. Papailiou (ed.), *Overhead Lines*, CIGRE Green Books,
DOI 10.1007/978-3-319-31747-2_12

12.6.3 Concrete Poles ... 880
12.6.4 Wooden Poles ... 881
12.6.5 Fabrication Process ... 881
12.7 Prototype Tests ... 883
12.7.1 Objectives ... 883
12.7.2 Normal Tests ... 884
12.7.3 Destructive Tests ... 886
12.7.4 Acceptance Criteria ... 886
12.8 Special Structures ... 890
12.8.1 Guyed Supports ... 890
12.8.2 Guyed Structure Types ... 890
12.8.3 Supports for Direct Current Lines ... 895
12.8.4 Supports for Large Crossings ... 895
12.9 Environmental Concerns & Aesthetic Supports ... 899
12.9.1 Environmental Issues ... 899
12.9.2 Innovative Solutions ... 900
12.9.3 Landscape Towers ... 901
12.9.4 Overhead Line Supports into Artworks ... 910
12.9.5 Experiences around the World: Conclusions ... 912
12.10 Existing Lines & Tower Aging ... 917
12.10.1 Asset Management/Grid service ... 917
12.10.2 Assessment of Existing Supports ... 917
12.10.3 Inspection Philosophies ... 925
12.10.4 Types and Causes of Defects/Industry Repair Practices ... 926
12.11 Highlights ... 927
12.12 Future of the Supports ... 928
References ... 929

12.1 Introduction

This Chapter intends to cover the aspects related to the design, test and fabrication of the structures used as support for the Overhead Transmission Lines.

The supports are essential elements on the universe or the Overhead Lines. Their importance can be understood by the functions they exert on the transmission line system. Besides being responsible to support all the loads coming from the cables, they are also in charge of keeping the electrical clearances between the conductors and the grounding parts, as well as to maintain the phase-to-phase and phase-shield wire distances. They also need to be strong enough to absorb the conductor tension in the anchorage/terminal towers guarantying, thus, the existence of the angles and/or the end of the lines.

From the reliability point of view, it is important to highlight the role played by the supports on the reliability analysis of the Overhead Lines. According to IEC 60826 Standard (IEC 60826 Standard Design Criteria of Overhead Transmission Lines), the suspension tower should be designed as to be the "weakest link" of the system, being thus, the first element to fail. This consideration means, in other words, that the line reliability is directly linked with the mechanical strengths of the supports.

As far as the environmental impact of the Overhead Lines is concerned, again the supports are in evidence since they are the most visible elements in the landscape.

As a consequence, they need to be treated more and more as aesthetic element having their visual aspect improved.

The content of this chapter is organized in the following way: Section 12.2 will describe the different types of supports classified according to their function in the line, circuit dispositions, structural types, shapes, formats, etc. The acting loadings on the structures are treated in Section 12.3, while Sections 12.4 and 12.5 are dedicated to the support calculations.

Information related to the drawing detailing practices and fabrication processes are given in Section 12.6. Section 12.7 will treat issues regarding to the full scale prototype test practices and acceptance criteria currently adopted.

In Section 12.8, special structures are shown; The environmental aspects of the transmission lines supports are treated in Section 12.9, discussing their impacts in the landscape and describing the measures recently adopted around the world to mitigate them.

Closing the subject, Section 12.10 will cover aspects related to the life of the structures, their aging process, and techniques usually adopted for repairs.

12.2 Types of Supports

The Overhead Line Supports can be classified according to different characteristics, or criteria. Generally these classifications are: regarding to their function in the line, number of circuits, line voltages, structural characteristics or types, structural modeling, shapes, formats, used materials, etc.

12.2.1 Regarding Function in the Line

It is common that a line, reasonably long, is composed by the following types of supports:

- Suspension or tangent towers: those which main function is just to suspend or support the conductors in alignment or small line angles (e.g., 1°, 2°, 3°) (Figure 12.1). Lines relatively long (around 500 kms or more) use to have two or three different suspension towers classified according to their wind spans (e.g., 500 m, 600 m, 700 m) and weight span (e.g., 600 m, 700 m, 800 m).
- Angle towers: those which the main function is to keep the line angles. It is common the use of suspension towers for small angles (up to 5°), while for bigger ones (above 10°), is more practical the use of angle/tension towers.
- Anchorage or tension towers: those used to anchorage the conductors and ground wires on them, supporting their tensions. They are currently designed to support all the cable tensions and also the big line angles (e.g., 30°, 45°, 60°) (Figure 12.2).
- Dead end or terminal towers: as described by the name, these towers are used to segment the line in intermediate or final substations.

Figure 12.1 765 kV Suspension tower.

12.2.2 Number of Circuits/Phase Arrangements/Tower Top Geometry

As far as the number of circuits and the phase arrangements are concerned, the towers can be classified as single circuit horizontal (flat) configuration, delta configuration, inverted delta or double vertical circuit, double delta circuit (Danubio type) or even triple, quadruple circuits, etc. (see Figures 12.3 and 12.4).

12.2.3 Structural Types, Structural Modeling

According to the structural modeling, the towers can be classified as self-supported or guyed (Figure 12.5). They can also be from the latticed type, or formed by box sections like poles, as can be seen in Figure 12.6.

Figure 12.2 500 kV Anchorage Tower.

Figure 12.3 Single circuit Horizontal and Delta configuration supports.

12.2.4 Formats, Aspects, Shapes

Looking at the whole structures, they still can be named as pyramidal trunk (Christmas tree), Delta type, "Cat face", "Raquet", "Monopoles", Guyed V, Portal type, Cross-rope suspension (CRS) etc., according to their aspects or formats, basically as a result of the phase arrangements and/or circuits dispositions (see Figure 12.7).

Double Vertical circuits Double Delta circuits

Figure 12.4 Double circuits supports.

Figure 12.5 Self-Supported and Guyed towers.

12.2.5 Material Used

The Overhead Lines Supports can be fabricated by steel, concrete or wood. The use of one or other material type depends on the local availability of that material, market conditions and, also, on technical limitations. Usually for lower voltages levels, wood and concrete can be feasible and more economical, while for higher levels the steel is predominant (Figures 12.8 and 12.9). For ultra high voltage lines, the latticed steel supports are, nowadays, the unique solution adopted.

500 kV Lattice support 138 kV Box section pole

Figure 12.6 Latticed and Box Section Supports.

12.3 Design Loadings

12.3.1 Design Philosophy

During the 90's, there was a great change in the adopted philosophy for the overhead transmission lines designs. The criteria that had been currently used, changed from the so called "deterministic approach" to the "RBD - reliability based design" or "probabilistic based method". By that time, considerable amount of meteorological data like ambient air temperature, wind velocity, lightning, etc., had already been accumulated by many utilities, enough to sustain a probabilistic approach based on data bank.

The IEC 60826 Standard (IEC 60826 Standard Design Criteria of Overhead Transmission Lines) issued during this period, establishes interesting premises for the design of high voltage overhead lines. The design, as proposed, must fulfill reliability, security and safety requirements. Reliability requirements aim to ensure that lines can withstand the defined climatic limit loads, that can basically be treated by using probabilistic approaches, such as wind, ice, ice and wind and the loads derived from these events. It is usual to associate them to a return period T. Security requirements correspond to special loads and/or measures intended to reduce risk of

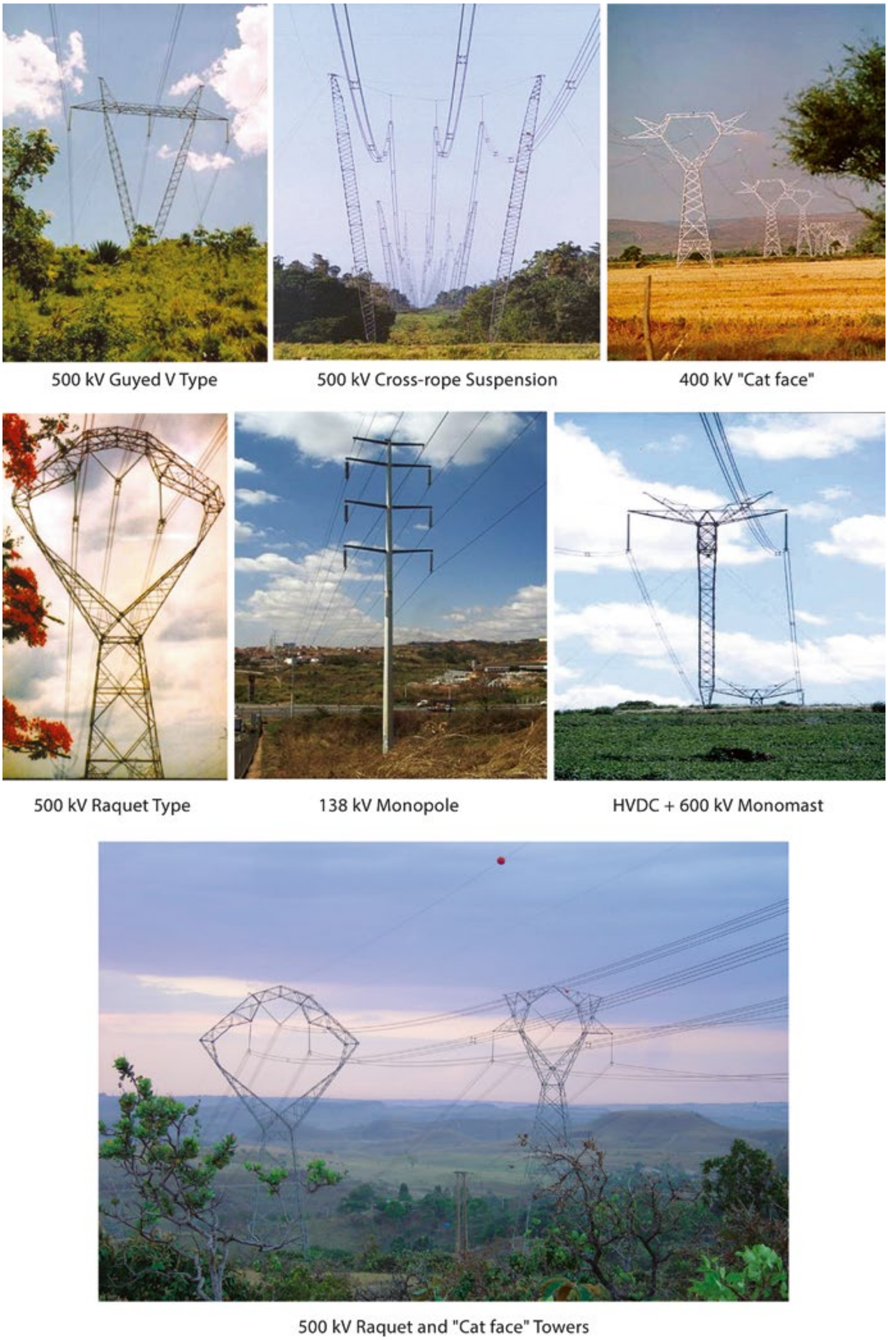


Figure 12.7 Different formats and shapes of supports.

Figure 12.8 Concrete Supports.

Figure 12.9 Wood OHL Supports.

uncontrollable progressive (or cascading) failures. Safety requirements consist of special loads for which line components have to be designed, to ensure that construction and maintenance operations do not pose additional safety hazards to people.

Among the reliability requirements, and with direct impact on the support designs, it deserves to be mentioned:

- The lines are classified according to their target reliability within the system. These reliability levels were linked to a wind velocity associated to an adopted return period (e.g., 50, 150, 500 years).
- Establishment of a "preferential sequence of failure" or "failure coordination", where is defined that the so called "suspension tower", the most numerous one in the line, should be designed as to be the weakest link of the transmission line system. The other towers, as well as the other line components (conductors, shield wires, insulators, foundations, etc), should be designed stronger enough to have their probability of failure lower. According to IEC 60826 (IEC 60826 Standard Design Criteria of Overhead Transmission Lines), so, the general strength equation is given by:

$$\gamma_U Q_T < \Phi_R R_C$$

where:

Q_T = Design loading referred to a returned period T (or/with specific load factors)
γ_u = Use factor coefficient
R_C = Characteristic strength of the structures, assessed by calculations and/or calibrated by loading test and/or professional experience
Φ_R = "Strength factor" to be used in the design and evaluated as function of the statistical distribution of support strength.

Simplifying the concept and the understanding, one can say that load effects must be increased according to their uncertainties and the support strengths should be reduced in accordance with their accuracy grades.

Taking the distribution curves for load effects and strengths and adopting, an "exclusion limit" of 10 %, as recommended by IEC 60826 (IEC 60826 Standard Design Criteria of Overhead Transmission Lines) for the design of the supports, the probabilistic design approach may be illustrated by Figure 12.10.

From the probabilistic perspective, an exclusion limit of 10 % is proposed to account for the uncertainty arising from the strength. Thus, one can right, $R_{10\%} = \Phi_R R_C$ and $\gamma_u Q_T \leq R_{10\%}$

For the correct application of the probabilistic based method, it is extremely important to know the "statistical distribution of support strengths", the mean value and the respective standard deviation. Aiming to improve the knowledge for a better definition of this function, Cigré has conducted many important researches during

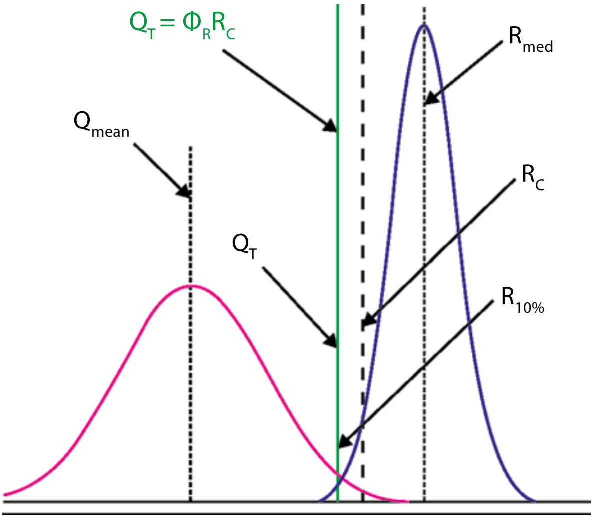


Figure 12.10 IEC 60826 reliability based design philosophy.

the last 20 years. Among them, it deserves to be mentioned the publications Paschen et al. (1988) and Riera et al. (1990), where mathematical/statistical treatments were made over sets of full scale prototype test results, leading to strength statistical distributions, well fitted by log-normal curves, and having in the case of article (Riera et al. 1990) a mean value of 104.6 % and a standard deviation of 8.51 % for lattice suspension towers. Taking into consideration the specified exclusion limit of 10 %, the "strength factor" value of $\Phi_R = 0.93$ may be calculated for this case. For other type of supports, like anchor, heavy angle, dead-end, considered as having greater dispersion in their design, values of 0.90 and/or even 0.85, have been deterministically adopted by the industry in some countries.

As above said, during the last years, Cigré has devoted many researches for better understanding and calibrating Φ_R values. The influence of the methods, tools and techniques used in the support designs, as well as the impact of the structural modeling adopted, the material property dispersions and the accuracy of the fabrication and erection procedures have been investigated and are published, as can be seen in (An experiment to measure the variation in lattice tower strength due to local design practice 1991; Variability of the mechanical properties of materials for transmission line steel towers 2000; Diaphragms in lattice steel towers 2001; Statistical analysis of structural data of transmission line steel towers 2005; On the failure load of transmission line steel towers considering uncertainties arising from manufacturing & erection processes; Influence of the hyperstatic modeling on the behavior of transmission line lattice structures 2009; The effect of fabrication and erection tolerances on the strength of lattice steel transmission towers 2010; Investigation on the structural interaction between transmission line towers and foundations 2009), as listed in Section 11.

12.3.2 Loadings

Loads acting on overhead line supports result from actions coming from the components (conductors, shield wires, insulator strings, guy wires), besides those acting directly to them like wind loads and dead-weights. To be easily treated in the structural analysis, these loads are decomposed in vertical and horizontal components, these ones, being transverse or longitudinal, as referred to the transmission line axes. From the design point of view, it is necessary to identify critical load combinations that can act during the supports lifetime.

As vertical loads, always acting, there are the cable tensions decomposed in vertical axis, the insulators weights, the dead load of the support and eventually the weights coming from snow and/or ice. It is also important to consider construction loads coming from the tower erection and conductor stringing works. They can also be representative in terms of load effects to some structural members, requiring special attachment points in some cases.

Transverse horizontal loads (orthogonal to the line direction) are basically due to wind action on the conductors, insulator strings and supports, as well as the cable tensions decomposed in transversal axis, which is very significant in case of angle towers.

As far as longitudinal loads are concerned, they result from cable tensions decomposed in longitudinal axis, arising from conductors and shield wires, acting on the anchor and/or dead end towers. Loads due to broken wires (longitudinal unbalance) also should be considered and are important specially for designing suspension supports. Loads arising from longitudinal or oblique winds should also be taken into account.

12.3.2.1 Loads Associated to Design Requirements

Such loads must be evaluated keeping in mind the premises and requirements above mentioned.

Therefore, to fulfill reliability requirements, loads which show a random nature (as wind velocity and ice accretion or a combination of them) are addressed in a probabilistic approach. Some of other loads acting simultaneously may be treated in a deterministic format due to their low variability (e.g.,: dead load).

To address requirements to the line security is necessary to carefully consider some loading conditions that may eventually trigger various types of failures. Loads involving storms of large footprints are likely to trigger multiple failures of weak components, while more localized consequences are expected when accidents happen in normal operational conditions. Typical loading cases aiming to fulfill such requirements are due to broken wires (conductors and shield wires). For those cases, semi-probabilistic approaches are appropriated. In traditional design practices, these loads on transmission lines have been represented by conductor failure (conductor breakage load or unbalanced residual static load (RSL)) occurring in normal operational conditions. Although conductor breakages or failures of conductor attachment hardware are rather uncommon in normal conditions, their likelihood is increased in severe weather conditions and their consequences will obviously differ. This highlights the importance of these considerations as they will influence the

Figure 12.11 Workers on the support during tower erection.

choice of security measures in the specific context of each utility. Other exceptional loads that are defined as climatic or natural loads in excess of design loads can also be specified using a reliability analysis approach.

From the safety requirement perspective, there are some operations during construction and maintenance works that a failure of a line component may cause injury or even losses of human lives. Therefore, such events cannot be treated under a probabilistic model, and are usually regulated by National Codes and/or practical regulations to fulfill minimum safety requirements, being the supports calculated under deterministic approaches. As IEC 60826 (IEC 60826 Standard Design Criteria of Overhead Transmission Lines) establishes, system stress under these loadings shall not exceed the damage limit, and the strength of the supports shall be verified either by testing or by reliable calculation methods. Typical loadings are due to erection of supports, construction stringing and sagging, longitudinal and vertical loads on temporary dead-end supports. As the consequences may involve injury or loss of life, special attention is also needed whether the loads are of static or dynamic nature, with or without the presence of workers on the support (Figure 12.11). All the following figures must be one number more.

12.3.2.2 Loading Cases

The structures are designed to withstand many different loading combinations (loadings cases or loading hypothesis) that can occur during their lifetime. It is a common practice, in the industry, to take into account at least the following loading hypothesis in the calculation of the supports:

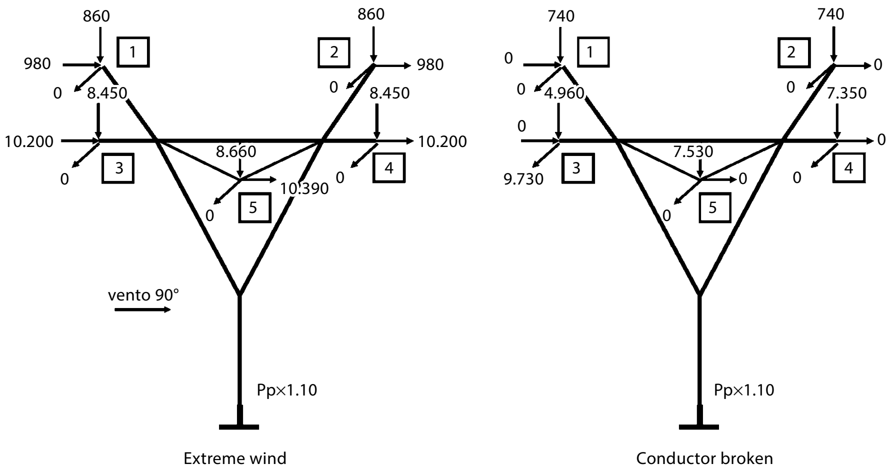


Figure 12.12 Loading cases.

- Extreme wind
- Ice and associated wind
- Only ice
- Shield wire broken
- Conductor broken (one conductor/phase at any point)
- Anti-cascading
- Construction and maintenance operations.

The loads are combined into "loading trees" like it is shown in Figure 12.12.

In the majority of the countries, loadings are calculated according to standards (or national codes) and, after considering the requirements above discussed, they are considered as "ultimate loads", not being necessary to apply any other additional safety factor. However, the concept of "partial safety factor" (Comparison of general industry practices for lattice tower design and detailing, Cigré 2009) is also applied especially in some European countries.

Usually, the most difficult loads to be evaluated, are the wind loads, not only for their correct characterization of intensities, but specially for their dynamic nature that may be dependent of the type of storm. Most of the codes have methods to appropriately consider synoptic winds but still do not have appropriate methodologies to care of other wind types (e.g. thunderstorms).

12.3.3 Static and Dynamic Loads

Loadings from wind and broken conductors on a transmission line support are clearly of dynamic nature. To be complete and consistent, the dynamic nature of such loadings has to be evaluated in both sides of the design equation: in the load effect side and in the member strength side.

There are many initiatives around the world aiming to model the dynamic effect of the wind on the towers, as well as, the impact of the broken wires. Menezes et al. (2012) and Leticia et al. (2009), have modeled the dynamic loading, such as wind and the breakage of wires, and numerically obtained the response of the supports in terms of member forces (Figure 12.13). McClure et al (Mechanical security of overhead lines- containing cascading failures and mitigating their effects 2012) has modeled the load due the breakage of wires and compared it to field measurements (Figure 12.14). From these references, what can be seen is that supports and cables work together for the absorption of those loads and their dynamic effects.

The industry, however, has addressed these cases considering them as static loadings. From the practical point of view, and for the purpose of tower calculations, wind loads as well as broken wires effects are treated as "equivalent static loadings".

Because the important part of the frequency spectrum of a broken conductor load is significantly higher than that of most transmission line tower natural frequencies, this load would only be felt on the tower as an impulsive load. After the transient part is damped, the tower would then be loaded with the residual load of the conductor tension (Figure 12.14).

By this way, aiming to take into account the dynamic impacts of occurrence of broken wires, what is normally done is to load the supports with the "residual static loads" (RSL), applying a dynamic impact factor to compensate the "peak dynamic load" effect.

Therefore, utilities have used two different approaches to address this type of loading event. For towers to resist the full impulse load a dynamic impact factor is applied to the conductor tension and this load is applied as a static load in the calculations. In a transmission line system the behaviour of an individual tower to a broken conductor event can be influenced by the unbroken conductor phases, the overhead ground-line, and the deflection of the adjacent towers.

An option against the detrimental effects of a broken conductor is to install controlled sliding clamps.

A controlled sliding clamp is made of a body in which rests the conductor and of a cover (see Figure 12.15). This cover is installed over the conductor by means of a system the pressure of which can be adjusted to make it possible to carry out the calibration of the load that causes the conductor to slide through the clamp. More details can be found in Chapter 5, page 44 ff.

12.4 Structural Modeling

12.4.1 Structural systems

As mentioned in Section 12.3, from the structural system point of view, the supports can be from self-supported or guyed types, in this last case, when they combine rigid bars with flexible elements needed to keep their equilibrium (Figure 12.5).

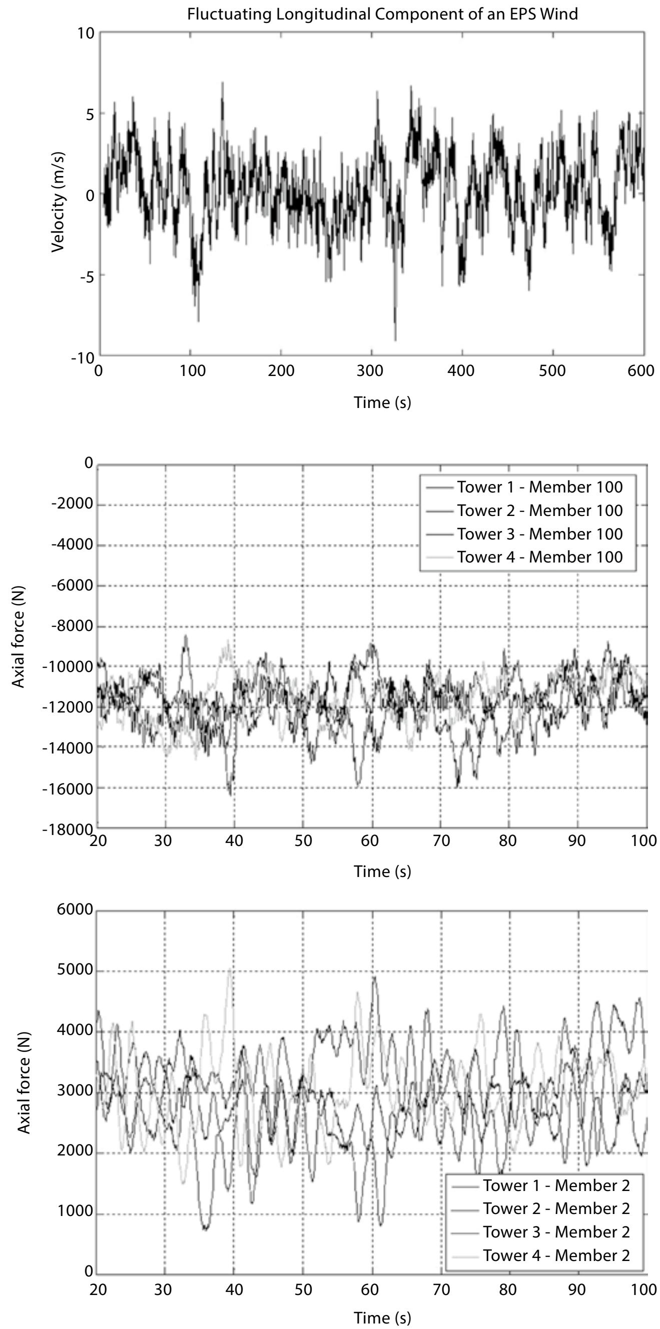
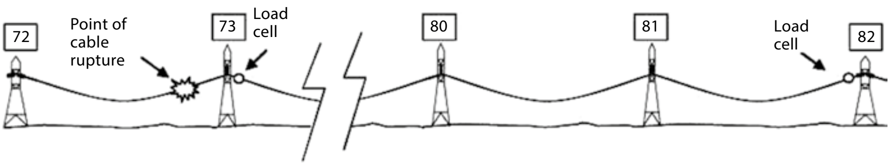
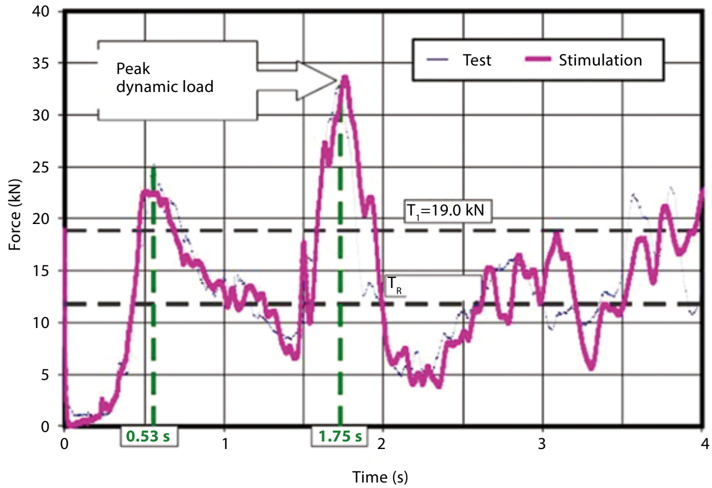


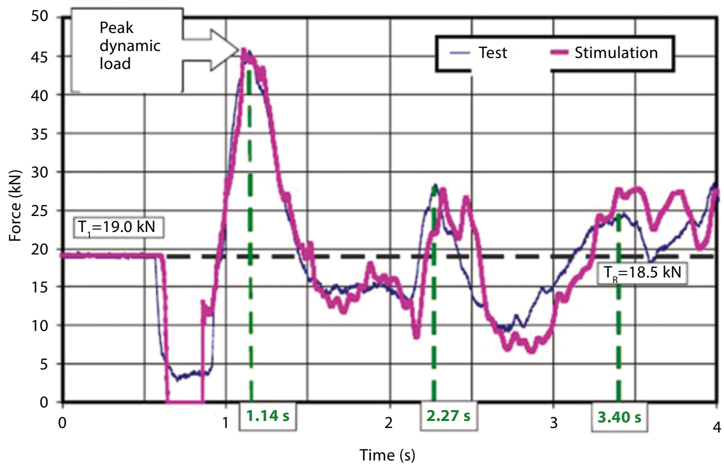
Figure 12.13 Simulated wind velocity and axial forces in main and diagonal members of a crossing tower GTS due to EPS wind load.



Broken donductor test on Saint-Luc-de-Vincennes 230 kV line
(Cigré Report B2-308, Vincent et al., 2004)



a) Conductor tension at Tower 73 (Initial 19.0 kN; Peak dynamic 33 kN; Residual 12.5 kN)



a) Conductor tension at Tower 82 (Initial 19.0 kN; Peak dynamic 45 kN; Residual 18.5 kN)
Measured vs. Predicted dynamic conductor tension on Saint-Luc-de-Vincennes
230 kV line (Cigré Paris Session 2004 Report B2-308, Vincent et al., 2004)

Figure 12.14 Modeled load due to a broken conductor and field measurements.

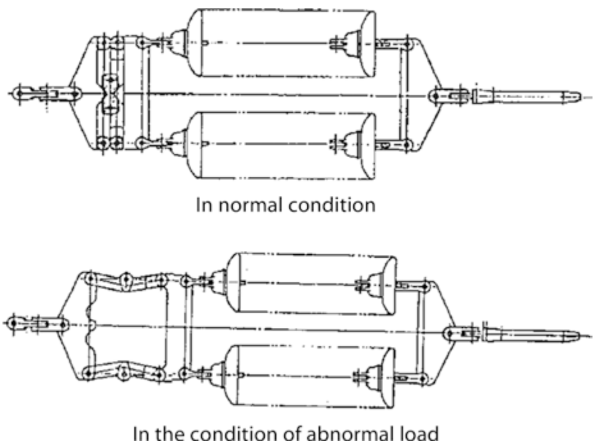


Figure 12.15 Elongating LCD type.

Usually, heavy-loaded supports (anchor, heavy angles, dead-end) of a line are of the self-supported type, being guyed towers more used as suspension structures. Section 8 gives more details of these structures.

In addition to that, the supports can also be shaped/formed by box sections, like poles, or constructed using, the concept of trusses (Figure 12.6).

In lower voltage level lines, the supports can still be in wood or concrete poles with gross cross sections. As the line voltage level increases, the use of lattice tower become predominant. This fact is basically due to the excellent strength capacity of the lattice structures when large structural dimensions are needed to accommodate the phase arrangements as well as the insulator string swings. It is also important to point out the logistics involved in transportation and erection of the supports, that, in the case of lattice supports, is tremendously simpler since they can be sent to site location disassembled.

12.4.2 Structural Modeling

The lattice solutions currently used in OHL structures are, are from the spatial truss type, here named as "incomplete" or "imperfect", since they are not formed by "spatial rigid nodes" like the edges of a tetrahedron as shown in Figure 12.16.

The three-dimensional truss normally used can have some "pseudo-unstable" nodes, that are carefully carefully studied to support loads that are very well defined in terms of application point, direction and plane of action (vertical, horizontal transverse or longitudinal). In other words, the structures are well proposed and designed to support "well defined" load systems (not generic loads). The structural model here used is from the "tension-compression" system type (or "composite truss"), forming lattice panels juxtaposed (or combined).

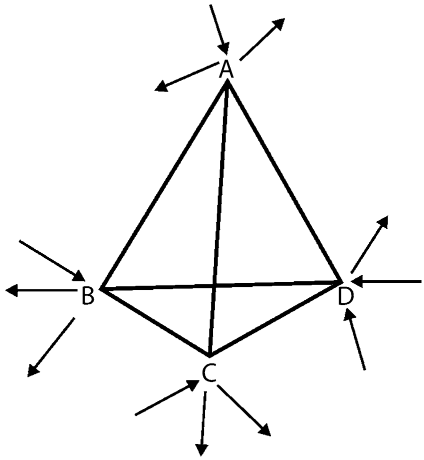


Figure 12.16 Spatial rigid nodes.

This system is made up of diagonal cross bracings, being shear forces equally. Shear is equally distributed between the two diagonals of each panel, one in compression and the other in tension. Both diagonals are designed for compression and tension in order to permit reversal of externally applied shear. The diagonal braces are connected at their cross points, since the shear effect is carried by both members and the critical length is approximately half that of a corresponding single web system. This system is used for large and small towers and can be economically adopted throughout the shaft except in the lower panels, where bracing portal systems can, sometimes, be more suitable.

Thus, at the end, the spatial truss obtained is exactly a three dimensional lattice structure formed by the combination of panels of individual plane trusses, supporting loads on their planes or in parallel (or quasi), as shown in Figure 12.18.

In order to guarantee a good structural performance and to obtain predictable results, it is recommended to pay attention on some particular aspects of those uncommon structures:

- All loads must be applied on their nodes. If for any reason, a new load is to be supported by the structure, a new node shall be created, modifying, thus, the original proposal.
- It is always recommended to use "horizontal struts" in nodes where a load exists. This will create a better distribution of shear loads on the diagonals of the panel (see the effect of load P_1 and P_2 in Figure 12.17).
- In some cross sections on those "imperfect spatial trusses", special diaphragms are necessary to guarantee the overall structural behavior, as reported by the proposed in the relevant Cigré TB 196 (Diaphragms in lattice steel towers 2001).
- All attention should be paid in order to confirm that, in each panel, tension and compressive diagonals receive the same load effect with opposite signals, giving to the tension diagonal capacity to brace the compressive one as diagonal as can be seen in Figure 12.19.

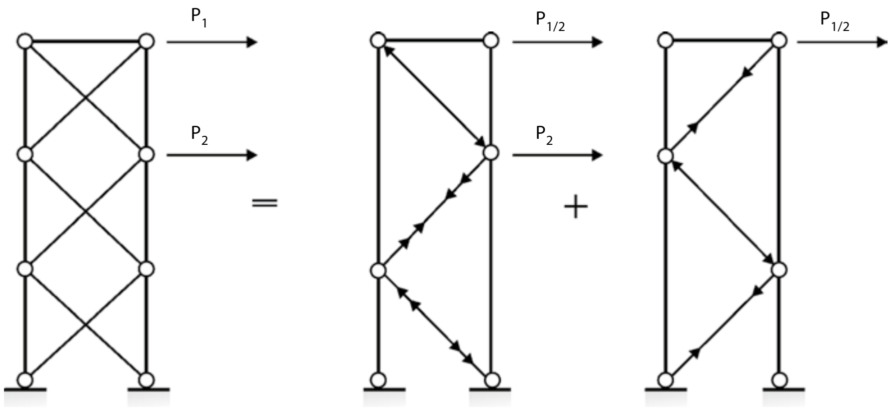
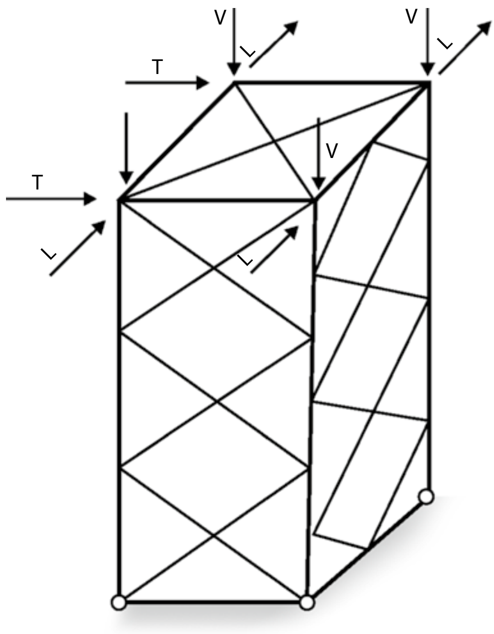


Figure 12.17 Tension-compression system. Load distribution on diagonals.

Figure 12.18 "Incomplete" 3-D lattice structure.



12.4.3 Structural Analysis

The method of analysis for transmission line towers has advanced quite a lot since the first towers have been designed. In the beginning the transmission line engineer used graphical analysis developed to a professional art for application to three dimensional space truss towers. With the introduction of the computer, mainframe computer analysis capabilities using structural matrix algorithms were developed.

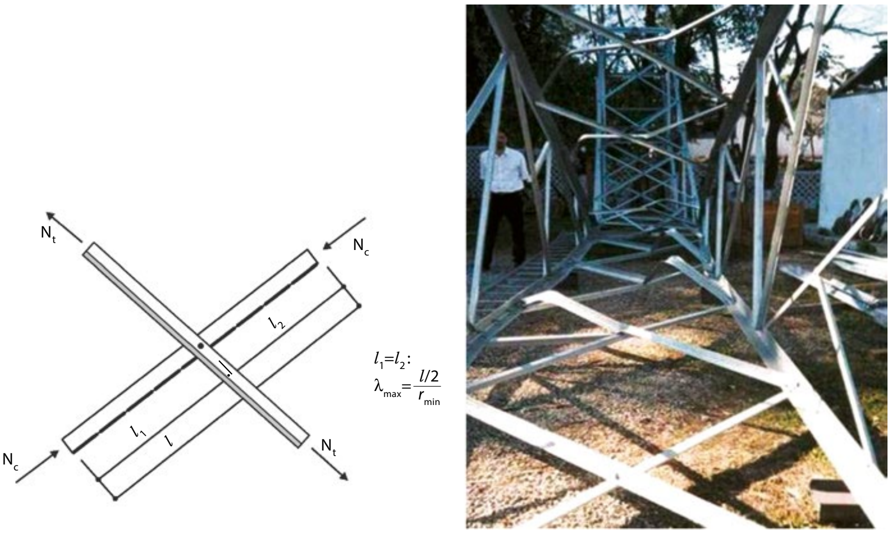


Figure 12.19 "Tension-compression" system behavior. Failure node during test.

As described in the last item, the standard of practice for analysis of lattice steel transmission line towers is a three-dimensional truss analysis. The structural computer model is based on the tension and compression behavior of the individual tower members, i.e. on a truss model. These tower analysis programs are based on linear elastic structural performance, whereby members are assumed to be axially loaded and to have pinned connections. The member forces determined from the computer model are compared to the allowable member capacities. In special cases, a three-dimensional frame model may be required to provide necessary information on the member forces, including bending moments.

In survey carried out by Cigré (An experiment to measure the variation in lattice tower strength due to local design practice 1991) and according to (ASCE 10–97 2000), the structural analyses model more used around the world is the 3-D truss analysis. Good results have been reported using linear analysis methodology for the self-supported lattice OHL supports.

For more deformable structures, like the guyed towers or poles, non linear analysis are currently required.

There are many well known softwares based on "Finite Element Method - FEM", available in the market appropriate for the design of the transmission line supports, such as TOWER, OPSTAR, SAPS, ANSYS, etc. It is extremely important that the software to be used have a friendly approach and the specialized for OHL Structures, facilitating the inputs and organizing accordingly the outputs. Otherwise design activities of OHL supports can become extremely complex in terms of pre and post processing and consequently become unviable for commercial purpose.

A good example that can be reported is the software named "TOWER", developed by Power Line Systems Inc. that works together with the PLS-CADD, the most used software around the world for towers spotting. The main characteristics of the "TOWER" software are:

- Specialized program for the analysis and design of steel latticed supports of overhead lines and substations;
- Easy information input through interactive menus;
- Linear and nonlinear finite element analysis options;
- Databases of steel angles, rounds, bolts, guys, etc.
- Automatic generation of joints and members by symmetries and interpolations
- Automated mast generation;
- Steel angles and rounds modeled either as truss, beam or tension-only elements
- Guys are modeled as exact cable elements;
- Automatic calculation of tower dead weight, ice, and wind loads as well as drag coefficients;
- Automatic design checks according to many codes;
- Design summaries printed for each group of members;
- Easy to interpret text, spreadsheet and graphics design summaries;
- Automatic determination of allowable wind and weight spans;
- Automatic determination of interaction diagrams between allowable wind and weight spans;
- Capability to batch run multiple tower configurations and consolidate the results;
- Automated optimum angle member size selection and bolt quantity determination;
- Tool for interactive angle member sizing and bolt quantity determination;
- Capability to model foundation displacements. Can optionally model foundation stiffness;
- Directly link to line design program PLS-CADD.

Figure 12.20 illustrates the input data, showing the generation of the here called "primary nodes" and "secondary nodes".

Due to the three dimensional characteristic of the overhead line structures, the inherent symmetry in their geometry, as well as to avoid complexity in calculations, it is important to create the concept of the so called "bar groups". Those are groups of structural elements that, for optimization purposes, should be designed using the same profile and connections (see Figure 12.21).

12.4.4 Advanced Tools and Techniques

With the maturity of the transmission tower engineering profession and the advances in computer analytical solution power, the transmission engineer can now push the margin of analysis beyond the simple truss model. The uses of the simple truss model is still the main analysis tool of the transmission engineer, but advanced tools for evaluating the performance of space truss towers, including effects other than simple truss action, are now being utilized. As an example, three advanced transmission tower analysis computer programs will be briefly described:

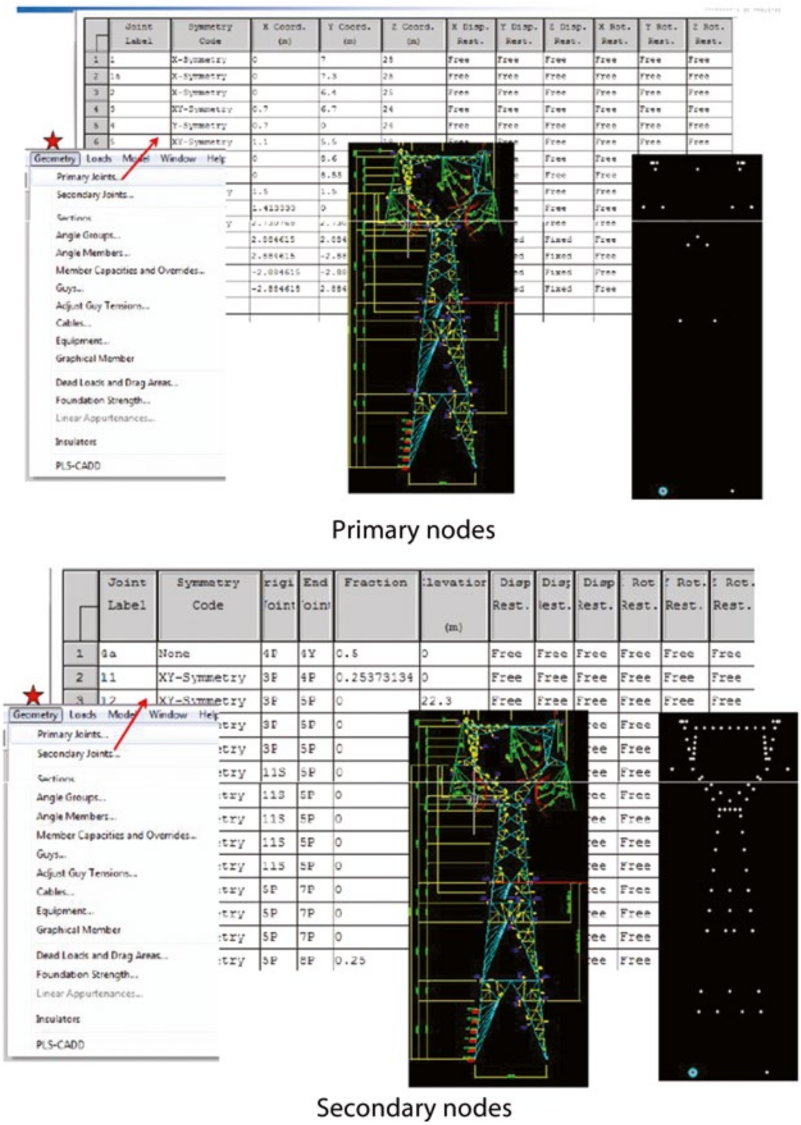
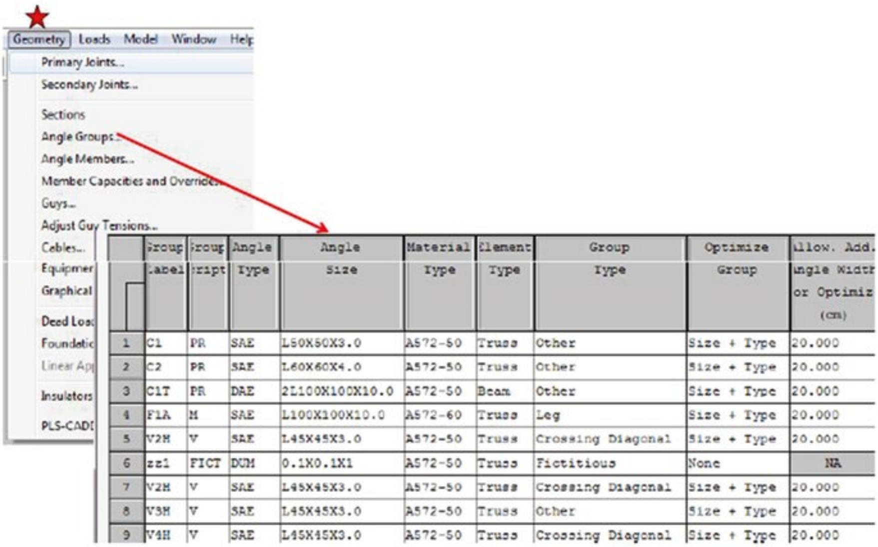


Figure 12.20 "Primary nodes" and "Secondary nodes" by software "Tower".

*AK TOWER Program: A*K TOWER is a finite element computer program that uses geometric and material nonlinear analysis to simulate the ultimate structural behavior of lattice transmission towers (Albermani 1992). The program has been calibrated with results from full-scale tower tests with good accuracy both in terms of failure load and failure mode. It is capable of accurately predicting tower capacity under static load cases by progressively detecting buckling and yielding in various parts of the structure until collapse. The software has been used by electricity



	Group Label	Group Script	Angle Type	Angle Size	Material Type	Element Type	Group Type	Optimize Group	Allow. Add. Angle Width or Optimiz (cm)
1	C1	PR	SAE	L50X50X3.0	A572-50	Truss	Other	Size + Type	20.000
2	C2	PR	SAE	L60X60X4.0	A572-50	Truss	Other	Size + Type	20.000
3	C1T	PR	DAE	2L100X100X10.0	A572-50	Beam	Other	Size + Type	20.000
4	F1A	M	SAE	L100X100X10.0	A572-60	Truss	Leg	Size + Type	20.000
5	V2H	V	SAE	L45X45X3.0	A572-50	Truss	Crossing Diagonal	Size + Type	20.000
6	zz1	FICT	DUM	0.1X0.1X1	A572-50	Truss	Fictitious	None	NA
7	V2H	V	SAE	L45X45X3.0	A572-50	Truss	Crossing Diagonal	Size + Type	20.000
8	V3H	V	SAE	L45X45X3.0	A572-50	Truss	Other	Size + Type	20.000
9	V4H	V	SAE	L45X45X3.0	A572-50	Truss	Crossing Diagonal	Size + Type	20.000

Figure 12.21 Group of bars.

utilities to verify new tower design and reduce or eliminate the need for full-scale tower testing. The structure is modeled as an assembly of general thin-walled beam-columns, trusses, and cable nonlinear elements.

MORENA Program: MOdular REliability aNAlysis – is a computer program that can be used to study the variability of the strength of transmission line towers. The program main modules are:

- Statistical analysis of data: perform standard statistical analysis including distribution fitting and correlation evaluation;
- Random samples generation: generate sets of random samples following established distribution, including correlated variables;
- Probability of failure calculations: estimate probability of failure for explicit or implicit limit state functions using advanced Monte Carlo Simulation procedures;
- Finite Element Analysis: in case of an implicit limit state function such as the response of a structure, perform classical finite element analysis (FEM) including standard linear or full nonlinear analysis (geometric and material nonlinearities) up to failure.

MORENA manages information which is stored and allows the user to call modules for specific purposes by means of commands written in the input file. The user has special commands to perform "loops", "jumps" and tests. It can be deduced, that within this concept, the FEM analysis is just a step to get a response for a limit state function in a reliability analysis. Therefore, the complexity of such a step is up to the user and, within a modular concept it can be improved as much as possible and desired. Laboratory testing of individual tower members (Menezes 1990), full-scale experimental tests of strength capacity of transmission line towers under lateral loads (Menezes 1988; Riera et al. 1990) and an extensive survey of the field performance of transmission lines in Southern and Central Brazil (Menezes 1988) provides data for strength variation evaluation of transmission line towers. This type of information permits: (a) Evaluation of the capacity of numerical procedures for nonlinear structural analysis to predict the carrying capacity of steel towers under lateral loading; (b) Evaluation of the capacity of reliability models to determine the failure probability of transmission lines towers and (c) comparison of the calculated probability of failure with field experience.

LIMIT Program: The Bonneville Power Administration (BPA) and Portland State University (PSU) developed a computer program called LIMIT. LIMIT performs a first-order non-linear analysis accounting for individual member post-buckling performance of lattice steel transmission towers. A direct iteration, Secant Method (Kempner 1990), procedure is used to account for non-linear member behavior. The analysis results can be used to determine the failure "collapse" load mechanism, and the member load flow at failure. The LIMIT analysis uses individual member post-buckling performance curves to track the member load after it reaches and exceeds its calculated compression capacity. Two options are used to model the post-buckling member performance: bilinear and normalized (empirical) curves. Normalized member performance curves were developed from actual tests of angle steel members (Bathon 1990, Huntley 1991). The LIMIT program has 29 normalized curves that can be used to represent the post-buckling member performance of steel angles (equal, unequal, and double angles). Both the bilinear and normalized curves assume linear behavior to the member compression capacity as shown in the Figure 12.22.

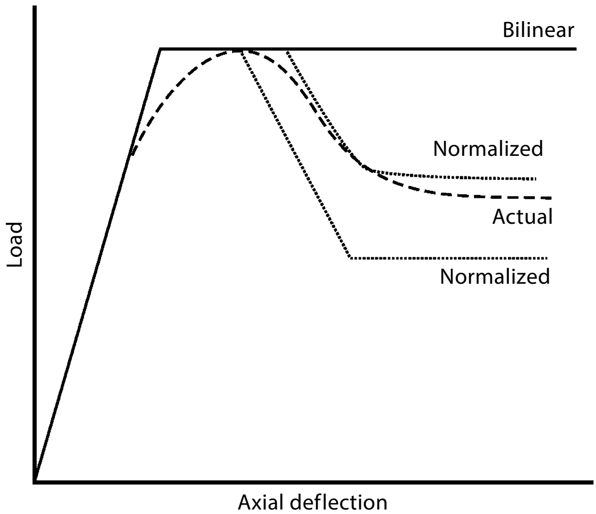


Figure 12.22 Member Performance Curves.

The LIMIT program provides three post-buckling analysis methods: Deterministic, Probabilistic, and Capacity Variations analysis. The deterministic analysis, referred to as LIMIT, provides a collapse load capacity based on individual member capacities determined from minimum yield strength, actual member yield strength, and/or observed member behavior. Probability based analysis can provide a design tool that accounts for variation in parameters which can affect member capacities, such as yield strength, connection eccentricity, engineer judgment, etc. A probabilistic analysis generates a set of randomly distributed member capacity. LIMIT provides two simplified probabilistic analysis options: LIMIT/PBA and LIMIT/CVA. The LIMIT/PBA, Probability-Based-Analysis, option varies member capacities based on selected yield strength distributions. The LIMIT/CVA - *Capacity Variation Analysis* option varies member capacities based on user selected +/− percentage variation about a calculated "base" member capacity.

Both PBA and CVA use the Monte Carlo technique to randomly vary member capacities. The first order non-linear post-buckling analysis is repeated using the random member capacity variation to obtain the tower collapse load distribution.

12.5 Calculation and Dimensioning

12.5.1 Materials and Standards

Supports for transmission lines are designed and fabricated according to different standards and practices. National or international standards are currently used, as well as local codes, guidelines or simply utilities' specifications. Cigré TB "Comparison of General Industry Practices for Lattice Tower Design and Detailing" (2009) gives a clear picture on that. According to document (An experiment to measure the variation in lattice tower strength due to local design practice 1991), ASCE 10-97(ASCE 10–97 2000) is the most used standard around the world for design of OHL supports.

12.5.1.1 Profiles

Profiles to be used in the fabrication of overhead lines lattice supports (mostly angles and plates) shall be shaped by means of hot-rolling or cold-forming processes, as per ASCE 10-97(ASCE 10–97: Design of lattice steel transmission structures 2000) or EN 50341–1:2012 – European CENELEC Standard (7.2.3) (CENELEC – EN 50341–1 2012) and EN 10025 European CENELEC Standard (7.2.3) (CENELEC EN 10025) (Figure 12.23).

The steels specified for the fabrication of those profiles shall be of the low carbon steel types, which present good resistance and ductility to guarantee strength and workability (ASCE 10–97 2000). As examples, as per ASTM standard (ASTM American Society for testing and materials):

- Structural steel: ASTM A36.

Figure 12.23 Raw material – Steel angles stock.

- High-strength low-alloy structural steel: ASTM A242.
- Structural steel with 42.000 psi (290 N/mm^2) minimum yield point: A529.
- Hot-rolled carbon steel sheet and strip, structural quality: ASTM A570.
- High-strength low-alloy structural Columbium-Vanadium steels of structural quality: A572.
- High-strength low-alloy structural steel with 50,000 psi (345 N/mm^2) minimum yield point to 4-in. thick: A588.
- Steel sheet and strip, hot-rolled and cold-rolled, high-strength low-alloy with improved corrosion resistance: ASTM A606.
- Steel sheet and strip, hot-rolled and cold-rolled high-strength low-alloy, Columbium or Vanadium, or both, hot-rolled and cold-rolled: ASTM A607.

12.5.1.2 Bolts

There are many standards and specifications for bolts around the world. According to Cigré TB (2009), however, the most used types types in the transmission line industry are ASTM A394 (ASTM-A394 2000), ISO 898–1 (ISO 898 1 Internacional Standard Organization) and CENELEC 5.8.1 (CENELEC – EN 50341–1 2012).

According to ASCE 10–97 Standard (ASCE 10–97 2000), bolts should be as per ASTM A394:

Type “0”

Low or medium carbon steel, zinc-coated (hot dip):

$$f_u = 510.0 \ \mathrm{N/mm^2}$$
$$f_v = 380.5 \ \mathrm{N/mm^2} \left(\text{for the thread}\right)$$
$$f_v = 316.5 \ \mathrm{N/mm^2} \left(\text{for the body}\right)$$

Type “1”

Medium carbon steel, with heat treatment, zinc-coated (hot dip):

$$f_u = 827.5 \ \mathrm{N/mm^2}$$
$$f_v = 513.0 \ \mathrm{N/mm^2} \ (\text{thread and body})$$

Type "2"
Low carbon martensilic steel, zinc-coated (hot dip):

$$f_u = 827.5 \ \mathrm{N/mm^2}$$
$$f_v = 513.0 \ \mathrm{N/mm^2} \ (\text{thread and body})$$

Type "3"
Corrosion resistant steel, with heat treatment:

$$f_u = 827.5 \ \mathrm{N/mm^2}$$
$$f_v = 513.0 \ \mathrm{N/mm^2} \ (\text{thread and body})$$

where:

f_u = ultimate tensile strength of bolt
f_v = allowable shear stress

According to ISO 898–1 (ISO 898 1 Internacional Standard Organization):
Class 5.8 (low or medium carbon steel, zinc-coated):

$$f_u = 520 \ \mathrm{N/mm^2}$$
$$f_v = 381 \ \mathrm{N/mm^2} \ (\text{for the thread})$$
$$f_v = 322 \ \mathrm{N/mm^2} \ (\text{for the body})$$

Class 8.8 (medium carbon steel, with heat treatment, zinc-coated):

$$f_u = 800 \ \mathrm{N/mm^2} \ (\text{diameter} \leq 16\mathrm{mm})$$
$$f_u = 830 \ \mathrm{N/mm^2} \ (\text{diameter} > 16\mathrm{mm})$$
$$f_v = 496 \ \mathrm{N/mm^2} \ (\text{thread and body, diameter} \leq 16\mathrm{mm})$$
$$f_v = 515 \ \mathrm{N/mm^2} \ (\text{thread and body, diameter} > 16\mathrm{mm})$$

12.5.1.3 Metallic Poles

The steel used in the fabrication of poles are currently from the high strength low alloy carbon types. Yield stress (Fy) and ultimate tensile stress (Fv) are around 3.500 kgf/cm^2 (355 N/mm^2) and 4.500 kgf/cm^2 (490 … 630 N/mm^2) respectively, values that can vary according to the thickness of the plates.

12.5.1.4 Concrete and Wood Poles

Standards for concrete and/or wood poles vary quite a lot depending on utility specification, national standards or local conditions such as materials, labor work or wood quantities.

12.5.2 Lattice Towers

Once the external loads acting on the support are determined and the structural analysis has been carried out, one proceeds with an analysis of the forces in all the members with a view to fixing up their sizes. Since axial force is the only force for a truss element, the member has to be designed for either compression or tension. When there are multiple load conditions, certain members may be subjected to both compressive and tension forces under different loading conditions. Reversal of loads may also induce alternate nature of forces; hence these members are to be designed for both compression and tension. The total force acting on any individual member under the normal condition and also under the broken-wire condition is considered as "ultimate force" and the dimensioning is done aiming to ensure that the values are within the permissible ultimate strength of the particular steel used. In some countries, the concept of "partial safety factor" also needs to be taken into account.

It shall be verified that bracing systems have adequate stiffness to prevent local instability of any parts. Bending moments due to normal eccentricities are treated in item 12.5.2.2 of buckling cases. If the continuity of a member is considered, the consequent secondary bending stresses may generally be neglected.

According to Cigré TB (2009), the most used standards around the world for design and calculation of lattice steel supports for overhead lines are ASCE 10–97 (ASCE 10–97 2000) and European CENELECs EN 50341–1:2001 (CENELEC – EN 50341–1 2012), EN 50341–3:2001 (CENELEC - EN 50341–3 2001) and EN 1993–1:2003 (EN 1993). In this section, emphasis will be given mainly to ASCE 10–97.

12.5.2.1 Design of Tension Members

According to ASCE 10–97 Standard (ASCE 10–97 2000), the tensile stress on a tension member is calculated as follows (Figure 12.24):

s - distance between holes in the direction parallel to the direction of the force
g - distance between holes in the direction perpendicular to the direction of the force.

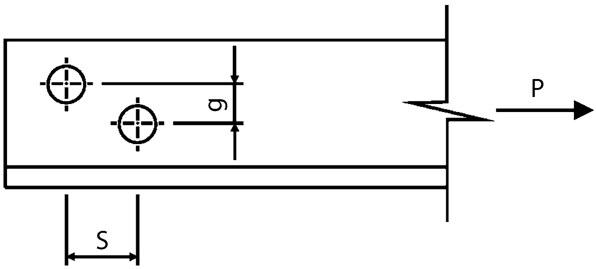


Figure 12.24 Tension members design.

Concentric Loads

$$f_t = P / A_n \leq f_y$$

f_t - tensile stress on net area
f_y - yield strength of the material
P - force transmitted by the bolt

$$A_n = A_g - n\ d_0\ t + [\Sigma(s^2/4g)]\ t$$

s - distance between holes in the direction parallel to the axial force
g - distance between holes in the direction perpendicular to the axial force
A_n - net cross section area
A_g - gross cross section area
n - number of holes
t - plate thickness
d_0 - hole diameter.

Eccentric Loads: Angle Members Connected by One Leg

$$f_t = P / A_e \leq f_y$$
$$A_e = 0.9\ A_n$$

A_e - effective cross section area

PS: For unequal leg angles, connected by the shorter leg, the free leg shall be considered as having the same width as the shorter leg.

Eccentric Loads: Other Sections
Members shall be proportioned for axial tension and bending.

Threaded rods:

$$f_t = P / A_s \leq f_y$$

A_s - core section at the thread.

12.5.2.2 Design of Compression Members

According to ASCE 10–97 Standard (ASCE 10–97 2000), the compression stress is calculated as follows:

Allowable Compression Stress on the Gross Cross-Sectional Area (N/mm2):

$$f_a = \left[1 - 0.5\ \left(\left(kL/i\right)/C_c\right)^2\right] f_y \quad \text{if}\left(kL/i\right) \le C_c$$

$$f_a = \pi^2 E/\left(kL/i\right)^2 \quad \text{if}\left(kL/i\right) > C_c$$

$$\text{with } C_c = \left[\left(2\pi^2 E\right)/f_y\right]^{1/2}$$

where:

f_y - yield strength (N/mm^2)
E - modulus of elasticity (N/mm^2)
L - unbraced length
i - radius of gyration corresponding to the buckling axis
k - effective buckling length coefficient (See Clauses 11.1 and 11.2).

Local and Torsional Buckling
For hot rolled steel members with the types of cross-section commonly used for compression members, the relevant buckling mode is generally *flexural* buckling. In some cases *torsional*, *flexuraltorsional* or *local buckling modes may govern*. Local bucking and purely torsional bucking are identical if the angle has equal legs and is simply supported and free to warp at each end; furthermore, the critical stress for torsional-flexural bucking is only slightly smaller than the critical stress for purely flexural buckling, and for this reason such members have been customarily checked only for flexural and local buckling (Figure 12.25).

- For ASCE 10-97(2000) f_y shall be replaced with f_{cr}.
- For EN 50341-1(CENELEC – EN 50341–1 2012) b and A shall be replaced with b_{eff} and A_{eff}.
- For ECCS 39 (Recommendations for angles in lattice transmission towers 1985) f_y shall be replaced with f_{cr}.

$$\left(w/t\right)_{lim} = 670.8/\left(f_y\right)^{1/2}$$

If $(w/t) < (w/t)_{lim}$ then the previous formulas for f_a apply without any change (Figure 12.26).

$$\text{If } 670.8/\left(f_y\right)^{1/2} \le \left(w/t\right) \le 1207.4/\left(f_y\right)^{1/2}$$

Then the allowable compression stress f_a shall be the value given by the previous expressions with f_y replaced by f_{cr} given by:

$$f_{cr} = \{[1.677 - 0.677\ \left[\left(w/t\right)/\left(w/t\right)_{lim}\right]\}\ f_y$$

$$\text{If} \quad \left(w/t\right) > 1207.4/\left(f_y\right)^{1/2}$$

$$f_{cr} = 668086/\left(w/t\right)^2$$

where (Figure 12.26):

w - flat width of the member
t - thickness of the member
b - nominal width of the member
R - rolling radius

Figure 12.25 Diagonals buckling during test. (See the bracing provided by the tension diagonals)

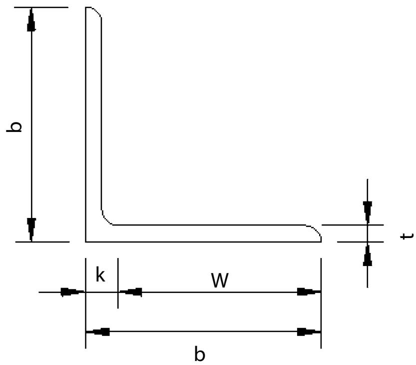


Figure 12.26 Determination of w/t ratios.

$$k = R + t$$
$$w = b - k$$

Note: The provisions of this section are only applicable for 90° angles.

Buckling Lengths

According to ASCE 10–97 Standard (2000), Effective buckling lengths for end restraints and eccentricities are modelled as following:

PRIMARY BRACING MEMBERS (EXCLUDING MAIN LEGS, CHORDS AND REDUNDANT MEMBERS)

Curve 1 - members with concentric load at both ends of the unsupported panel

$$kL / i = L / i \quad 0 \le L / i \le 120$$
$$(\text{according to ASCE Eq.3.7} \quad 5)$$

Curve 2 - members with concentric load at one end and normal framing eccentricity at the other end of the unsupported panel

$$kL / i = 30 + 0.75\ (L / i)$$
$$0 \le L / i \le 120\ (\text{according to ASCE Eq.3.7} \quad 6)$$

(*) To qualify a member for partial joint restraint the following rules are recommended (BPA criteria):

a - the member shall be connected with at least two bolts
b - the connection shall be detailed to minimize eccentricity
c - the relative stiffness (I/L) of the member must be appreciably less than one or more other members attached to the same joint. This is required for each plane of buckling being considered. To provide restraint the joint itself must be relatively stiffer than the member to which it is providing restraint. The stiffness of the joint is a function of the stiffness of the other members attached to the joint.

Curve 3 - members with normal framing eccentricities at both ends of the unsupported panel

$$kL / i = 60 + 0.5\ (L / i)$$
$$0 \le L / i \le 120\ (\text{according to ASCE Eq.3.7} \quad 7)$$

Curve 4 - members unrestrained against rotation at both ends of the unsupported panel

$$kL / i = L / i \quad 120 \le L / i \le 200$$
$$(\text{according to ASCE Eq.3.7} \quad 8)$$

Curve 5 - members partially restrained against rotation at one end of the unsupported panel

$$kL/i = 28.6 + 0.762(L/i)$$
$$120 \le L/i \le 225(*)(\text{according to ASCE Eq.3.7 } 9)$$

Curve 6 - members partially restrained against rotation at both ends of the unsupported panel

$$kL/i = 46.2 + 0.615(L/i)$$
$$120 \le L/i \le 250(*)(\text{according to ASCE Eq.3.7 } 10)$$

Redundant Members

$$kL/i = L/i \quad 0 \quad \le L/i \le 120$$
$$(\text{according to ASCE Eq.3.7 } 11)$$

Curve 4 - members unrestrained against rotation at both ends of the unsupported panel

$$kL/i = L/i \quad 120 \le L/i \le 250$$
$$(\text{according to ASCE Eq.3.7 } 12)$$

Curve 5 - members partially restrained against rotation at one end of the unsupported panel

$$kL/i = 28.6 + 0.762(L/i) \quad 120 \le L/i \le 290$$
$$(\text{according to ASCE Eq.3.7 } 13)$$

Curve 6 - members partially restrained against rotation at both ends of the unsupported panel

$$kL/i = 46.2 + 0.615(L/i)$$
$$120 \le L/i \le 330 \ (\text{according to ASCE Eq.3.7 } 14)$$

Buckling Length for Different Bracing Types

Main Members Bolted in both Faces at Connections:

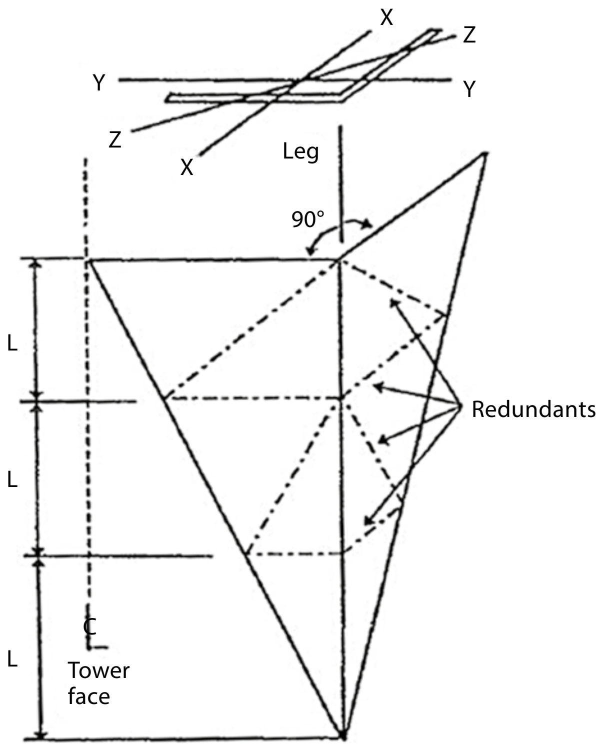
As per ASCE 10–97 Standard (2000),

$$kL/i = L/i \quad 0 \le L/i \le 150$$
$$(\text{according to ASCE Eq.3.7 } 4)$$

For main members of equal angles, having no change in member force between panels, designed with staggered bracing, the controlling L/i ratio shall be as shown in the Figure 12.28, 12.29 and 12.30 here below:

- Leg Members with Symmetrical Bracing (Figure 12.27)
- Leg Members Controlled by (2 L/3)/i_{zz} (Figure 12.28)
- Leg Members Controlled by (1.2 L)/i_{xx} (Figure 12.29)
- Leg Members Controlled by (1.2 L)/i_{xx} (Figure 12.30).

Figure 12.27 Leg Members with Symmetrical Bracing.



Diagonals and Bracing Members

ASCE10-97(Annex B) (ASCE 10–97 2000) and EUROPEAN CENELEC Standard (J.62 J.6.3) (EN 1993) provide various slenderness ratios $\lambda = kL/i$ applicable to all types of bracings (single, cross and multiple bracing with or without redundant members, k-bracing etc), as well as the appropriate bucking length and bucking axis.

Slenderness Ratio Limits

According to ASCE 10–97 Standard (2000),

Leg members and chords	$L/i \leq 150$
Primary bracing members	$L/i \leq 200$
Redundant members	$L/i \leq 250$
Tension-hanger members	$L/i \leq 375$
Tension-only members	$300 \leq L/i \leq 500$
Web-member – multiple lattices	Not specified
Horizontal edge members	Not specified

Cross bracing diagonals where the loads are equally or almost equally split into tension-compression system shall be dimensioned considering the centre of the

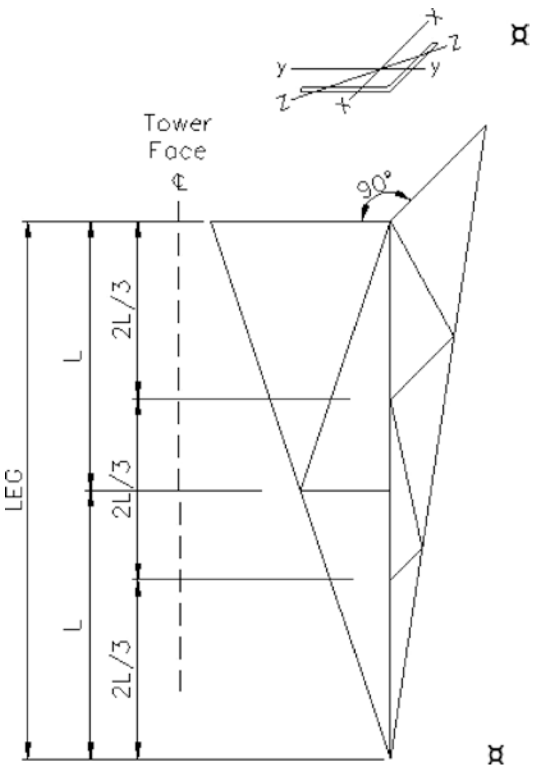


Figure 12.28 Leg members effective buckling lengths (a). Leg members shall be supported in both faces at the same elevation level every four panels.

cross as a point of restraint for both transverse to and in the plane of the bracing (Figure 12.31). Furthermore, the bracings shall have an effective maximum slenderness of 250 when considering the whole length.

Design of Redundant Members

Redundant (or secondary) members are installed on the overhead line lattice supports, basically for the function of bracing the loaded bars (leg members, chords, primary members). Even not having calculated loads, they must be rigid enough to guarantee efficient support against buckling. For this reason, it is recommended to attribute to the redundant members, hypothetical loads which magnitude are, usually, percentages of the loads of the supported members.

As per ASCE 10–97 Standard (2000),

The magnitude of the load in the redundant member can vary from 1.0 to 2.5 % of the load in the supported member.

Design of Members Acting only under Tension

It is a current practice that some very long members are installed only under tension to provide better assembly and overall rigidity.

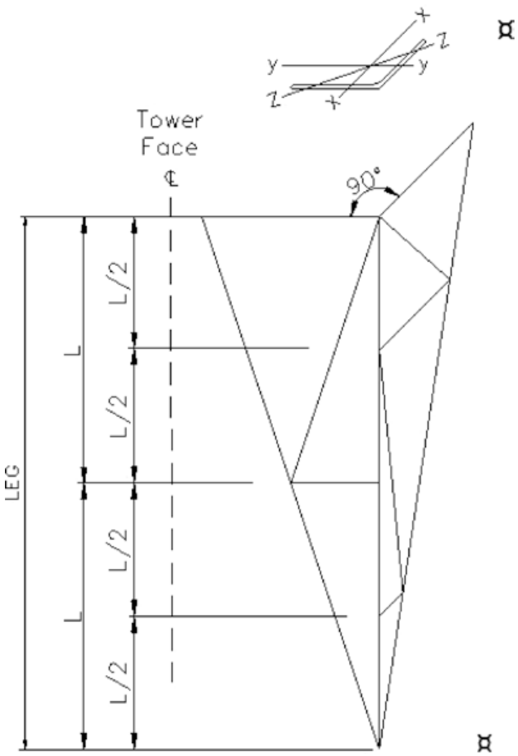


Figure 12.29 Leg members effective buckling lengths (b). Leg members shall be supported in both faces at the same elevation level every four panels.

According to ASCE 10–97 Standard (2000),

Connections should be detailed with at least two bolts to make assembling easier.

Reductions in length shall be as specified per Table 12.1.

Design of Members Subjected to Bending and Axial Force

As per ASCE 10–97 Standard (2000),

Members subject to both bending and axial tension shall satisfy the following formula:

$$(P / P_a) + (M_x / M_{ax}) + (M_y / M_{ay}) \leq 1$$

where:

P - axial tension

P_a - allowable axial tension

M_x, M_y - moments about x- and y-axis, respectively

M_{ax}, M_{ay} - allowable moments about the x- and y-axis, respectively, as defined in previous section.

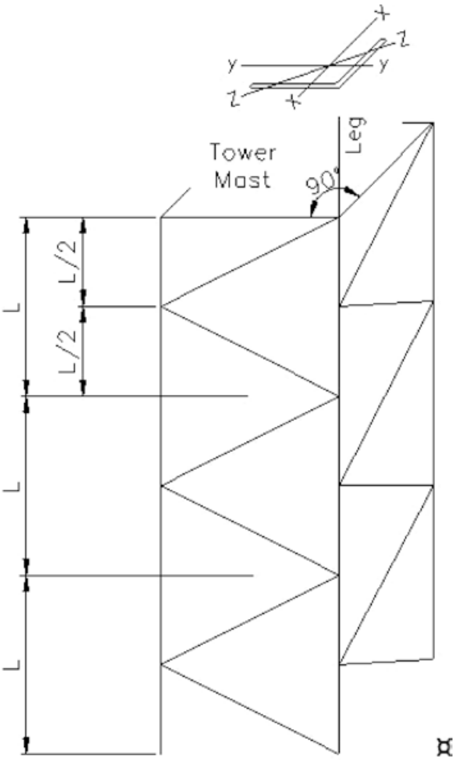


Figure 12.30 Leg of tower mast effective buckling lengths. For these configurations some rolling of the leg will occur. Eccentricities at leg splices shall be minimised. The thicker leg sections shall be properly butt spliced. The controlling L/r values shown shall be used with k =1.

Figure 12.31 Diagonals buckling.

$$M_{ax} = W_x \ f_y$$
$$M_{ay} = W_x \ f_y$$

where:

W_x, W_y - x and y-axis section modulus, respectively
f_y - yield strength

Table 12.1 ASCE 10–97
Tension members detailing

Standard ASCE 10-97	
Length (mm)	Reduction (mm)
$L \leq 4600$	3.2
$L > 4600$	$3.2 + 1.6$ (*)

(*) for each additional of 3100 mm or fraction

Members subjected to bending and axial compression shall be designed to satisfy the following equations:

$$(P/P_a) + C_m (M_x / M_{ax})[1/(1 - P/P_{ex})] + C_m (M_y / M_{ay})[1/(1 - P/P_{ex})] \leq 1$$

$$(P/P_a) + (M_x / M_{ax}) + (M_y / M_{ay}) \leq 1$$

where:

P - Axial compression
P_a - Allowable axial compression according to item 10

$$P_{ex} = \pi^2 E\ I_x / (k_x\ L_k)^2$$

$$P_{ey} = \pi^2 E\ I_y / (k_y\ L_y)^2$$

where:

I_x - moment of inertia about x-axis
I_y - moment of inertia about y-axis
$k_x\ L_x$, $k_y\ L_y$ - the effective lengths in the corresponding planes of bending
M_x, M_y - the moment about the x- and y-axes respectively, see Notes below;
M_{ax}, M_{ay} - the allowable moments about the x- and y-axis respectively, see explanation below

$$C_m = 0.6 - 0.4\ M_1 / M_2$$

The last equation is for restrained members with no lateral displacements of one end relative to the other, and with no transverse loads in the plane of bending (linear diagram of moments), being M_1/M_2 is the ratio between the smaller and the larger end moments in plane of bending. M_1/M_2 is positive when bending is in reverse curvature and negative when it is in single curvature (Figure 12.32).

$C_m = 1.0$ for members with unrestrained ends, and with no transverse loads between supports

$C_m = 0.85$ if the ends are restrained and there are transverse loads between supports

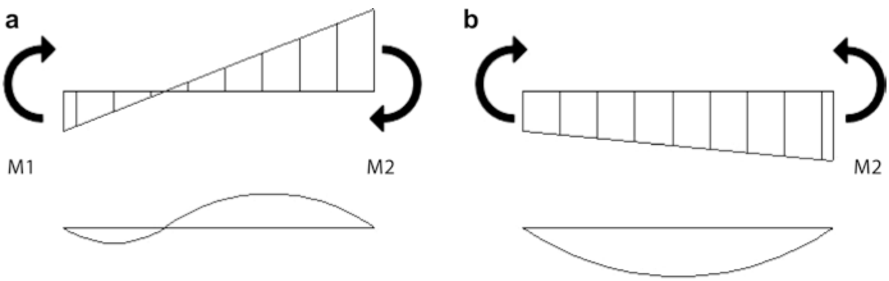


Figure 12.32 Lateral buckling moments.

For Laterally Supported Beams:

$$M_{ax} = W_x \ f_y$$
$$M_{ay} = W_x \ f_y$$

where:

W_x, W_y - x and y-axis section modulus respectively based on the gross section or on the reduced section (when applicable);
f_y - yield strength

For Laterally Unsupported Beams:

Verify lateral buckling according to paragraphs 4.14.4 and 4.14.8 of the ASCE Manual 52 – 1989 or 3.14.4 and 3.14.8 of Standard ASCE 10–97.

Notes:

M_x and M_y are determined as below:

a) If there are transverse loads between points of support (in the plane of bending):

M_x and M_y in the first equation above are the maximum moments between these points, which in the second are the larger of moments at these points;

b) If there are no transverse loads between points of support (in the plane of bending):

M_x and M_y in both equations above are the larger of the values of M_x and M_y at these points.

12.5.2.3 Design of Bolted Connections

The minimum distances bolt to bolt and bolt to the edges of a member have an impact on the capacity of the connection and on the ease of assembly (Figure 12.33). As example, according to ASCE 10–97 Standard (2000), distances vary according to the allowable shear and bearing stresses adopted and can be reduced when those stresses are reduced (see Section 6, item 6.1.4).

As per EN 50341–1:2001 - European CENELEC Standard (J.11) (2012), distances vary according to the allowable shear and bearing stresses. Distances are not specified for inclined directions.

Figure 12.33 Leg members connections.

Shear
According to ASCE 10–97 (2000) and ASTM-A394 (2000),

Type "0"

Low or medium carbon steel, zinc-coated (hot dip):

$$f_v = 380.5\ \text{N/mm}^2\ (\text{for the thread})$$
$$f_v = 316.5\ \text{N/mm}^2\ (\text{for the body})$$

Type "1", "2" and "3"

$$f_v = 513\ \text{N/mm}^2\ (\text{thread and body})$$

fv - allowable shear stress

Bolts that have no specified shear strength:

$$f_v = 0.62\ f_u\ (\text{thread or body})$$

f_u - ultimate tensile strength of the bolt

The minimum shear force shall be evaluated multiplying the effective area (root cross-section area at the thread or gross cross-section area at the body) by the corresponding allowable shear stress.

The cross-section area at the root thread is based on the core diameter (ANSI)

According to EN 50341–1:2001 - European CENELEC Standard (J.11.1), If the shear plane passes through the unthreaded portion of the bolt:

$$F_{v,Rd} = 0.6\quad f_u\ \ A / \gamma_{Mb}$$

If the shear plane passes through the threaded portion of the bolt for classes 4.6, 5.6, 6.6, 8.8:

$$F_{v,Rd} = 0.6 \quad f_u \quad A_s / \gamma_{Mb}$$

If the shear plane passes through the threaded portion of the bolt for classes 4.8, 5.8, 6.8, 10.9:

$$F_{v,Rd} = 0.5 \quad f_u \quad A_s \; / \; \gamma_{Mb}$$

A - cross section area of the bolt
A_s - tensile stress area of the bolt
$F_{v,Rd}$ - shear resistance per shear plane
γ_{Mb} - partial factor for resistance of bolted connections
f_u - ultimate tensile strength of the bolt

Tension
According to ASCE 10–97 (2000) and ASTM-A394 (2000):

Type “0”
Low or medium carbon steel, zinc-coated (hot dip):

$$f_v = 316.5 \;\; N / mm^2 \left(\text{for the body}\right)$$

Type “1”, “2” and “3”

$$f_v = 513 \;\; N / mm^2 \left(\text{thread and body}\right)$$

f_v - allowable shear stress

Bolts that have no specified proof-load stress

$$f_t = 0.6 \;\; f_u \quad \text{over the net area } \left(A_s\right) \text{ of the bolt}$$

f_u - ultimate tensile strength of the bolt

Net stress areas for bolts in tension (Table 12.2):
Bolts in inches: $A_s = (\pi/4)\;[d - (0.974/n_t)]^2$

Table 12.2 Net areas for tension bolts

D	n_t	A_s (cm^2)	d	p (mm)	A_s (cm^2)
1/2″	13	0.915	M12	1.75	0.843
5/8″	11	1.450	M14	2.00	1.154
3/4″	10	2.155	M16	2.00	1,567
7/8″	9	2.979	M20	2.50	2.448
1″	8	3.908	M24	3.00	3.525

Metric bolts: $A_s = (\pi/4)\ [d - 0.9382\ p]^2$
where:

d - nominal diameter of the bolt
n_t - number of threads per inch
p - pitch of thread

Combined Shear and Tension
As per ASCE 10–97 Standard (2000),

$$f_t(v) = f_t\ \left[1 - \left(f_{cv} / f_v\right)^2\right]^{1/2}$$

where:

f_t - design tensile stress under tension only (item 8.2.2)
f_v - design shear stress under shear only (item 8.2.1)
f_{cv} - computed shear stress on effective area (thread or body)
$f_t(v)$ - design tensile stress when bolts are subject to combined shear and tension. The combined tensile and shear stresses shall be taken at the same cross section in the bolt, either in the threaded or the unthreaded portion.

Bearing
According to ASCE 10–97 Standard (2000),

$$f_P \le 1.5\ f_u$$

f_u - ultimate tensile strength of plate or bolt
f_p - ultimate bearing stress.

12.5.3 Metallic Poles

The most attractive solution for urban or suburban OHL towers has been the monopole supports. They have been extensively used in all over the world, having adaptations and characteristics according to local necessities.

There are many reasons for the extensive use of monopoles as the main aesthetic solution. Among them it can be noteworthy: the simplicity, the slenderness, the low visual impact, the elegance, the beauty and the reduced area for settlement. As a summary, they are attractive solutions having appropriate painting system to fit them into special environmental circumstances (Figure 12.34). Section 12.9 gives more details about the use of aesthetic solutions for overhead line supports.

From the structural analysis point of view, it is important to observe that the monopoles are very flexible structures with high level of elastic deformation (up to 5 % of the pole height or even more) especially when compared with similar latticed towers. For this reason, it is recommended that the calculations should be carried out through

Monopole Support, Brazil — Aesthetic Pole, Japan — Compact Plus - Top insulated Support, Sweden

Figure 12.34 Monopole Solutions.

"physical and geometric non linear" analysis. Second order effects, may be of great relevance in the case of structural analysis for monopoles. To limit pole top deflection, however, can be very expensive. Therefore, aiming to reach both aesthetic and economic targets, it is suggested to verify the pole top deformation at the following stages:

- at EDS condition: maximum top deformation equal or smaller than 1.5 to 2 % of the pole height,
- at ultimate stage: 4 to 5 % of the pole height.

Figure 12.35 illustrates how deformable are the monopoles during prototype tests.

12.5.4 Concrete Poles

12.5.4.1 Design Methods

Concrete poles were initially developed to meet the expanding need of a supporting structure on streets, highways and area lighting, stadium lighting, traffic signals etc, and for overhead power transmission and distribution (Figures 12.36 and 12.37).

Later on, the quality of concrete was improved being possible to extend the utilization of concrete towers as support for high voltage transmission lines or wind turbine.

Generally, the lifetime of a concrete pole is in range of 50 years. This life can be increased using special protections of surfaces, or special reinforcing material, like high-corrosion resistant types, or free-corrosion or composite types.

The design methods cover pole reinforcing as material and forms, concrete, or special concrete material and pole as technical and functional requirements.

Figure 12.35 Pole deflections during Tests.

Figure 12.36 110 kV Concrete Pole.

The selection of materials and design methods is subject of optimization, thus it is directly reflected in the costs. This part depends on marketing and is solved separately. The other aspects such condition as the controlling of longitudinal cracking and the limitation of pole deflection during bending are considered generally requirements of production and thus solved by factory procedures. Interfaces with metallic materials for crossarms shall be designed based on steel lattice tower design methodologies.

Figure 12.37 Concrete Pole.

Figure 12.38 Pole structure for 110 kV line.

Depending on the requirements, concrete poles can be installed as self-supported, Figure 12.38, or guyed structure.

12.5.4.2 Standard and Practices

The basic design requirements can follow "safety factors method" or "ultimate design state" method. Both methods are standardized in many countries, based on local experiences and type tests. The full scale test shall be carried out to determine the load resistance, deflections and width of cracks.

12.5.5 Wooden Poles

12.5.5.1 Design Methods

Wood poles are composed of a naturally grown biological material which exhibits inconsistent material properties throughout the length of the pole. These inconsistencies, which have a direct impact on strength, are knots, checks, shakes and splits.

Wood poles are susceptible to rot and decay over the design life of the structure. Wood poles normally have less strength at the end of their service lives than when they were originally placed in service that required specific safety factors or partial material factors.

Moreover, insects and animal attacks can significantly decrease the load carrying capacity of the wood pole well before the end of the anticipated service life. To keep the safety level, the standards specify higher safety factors when compared to concrete or steel poles. As an example, NESC requires the wood pole to have a strength that is 60 % higher than the prestressed concrete pole; EN recommends that design should take into account the very probable loss of strength that will occur over the service life of the pole.

12.5.5.2 Standard and Practices

The basic design requirements can follow "safety factors method" or "ultimate design state" method.

In case of "ultimate design state" method, the internal forces and moments in any transverse section of the structure shall be determined using linear elastic global analysis.

12.6 Detailing Drawings and Fabrication Process

The works with transmission line lattice supports are unique when compared with those employed in other metallic construction types, as bridges, roofs or buildings. At least two characteristics make them different and specialized: the extensive use of bolted connections on small web angle profiles and the great number of equal pieces to be produced (sometimes reaching millions of units), which requires series production in "numeric controlled machines" (CNC). This means, in other words, that lattice supports fabrication is a specific business, requiring its own design and detailing professionals, as well as expert production working teams.

Metallic, concrete and wood poles also demand specialized construction works. This is particularly true when the line voltage increases due to the high level of loads involved and the required support heights.

12.6.1 Lattice Supports

12.6.1.1 Detailing Drawings

As mentioned above, the preparation of latticed tower detailing drawings is not a simple task. It requires expertise and qualification of the involved personnel. Profiles are almost a hundred percent angles having "reduced legs" to accommodate one or

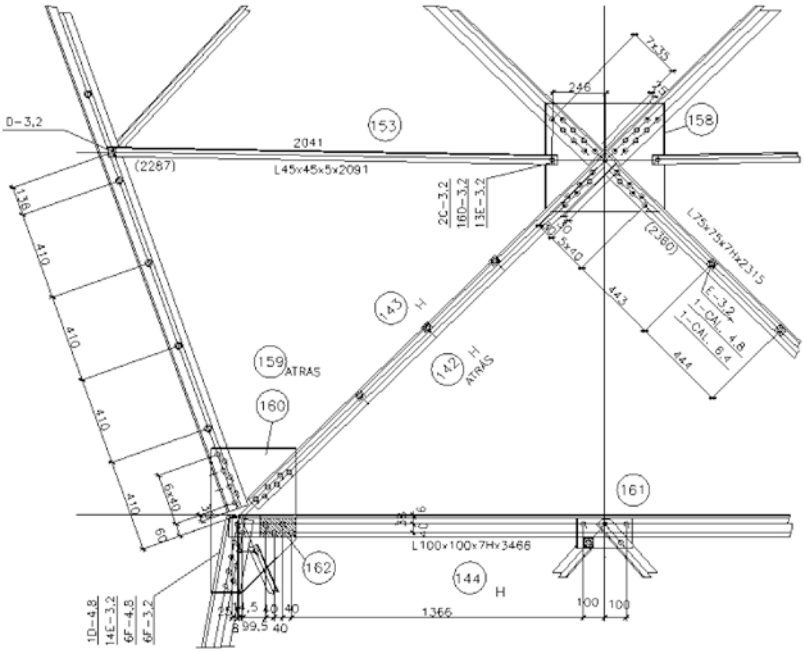


Figure 12.39 Typical lattice tower drawing.

more bolts. Furthermore, the most important premise to be observed in the detailing practices is trying to guarantee the "original structural figure", creating the nodes through the crossing of bar axis at the same point. Figures 12.39 and 12.40 are typical OHL lattice support detail drawings.

As a design premise, structural bars must be only submitted to axial forces (tension and/or compression). To assure this, joints must be designed and detailed as much as possible without eccentricities. Figures 12.41 and 12.42 show correct and incorrect detailing practices.

12.6.1.2 Bolts

Lattice OHL supports are normally modelled and calculated as "pin-jointed truss structures", being the bolt the key structural connection element. In the majority of the cases, tower bolts are submitted to shear forces. To cope with this function, those bolts are normally detailed having a "not threaded part" on the length, to avoid the coincidence of shear plane with the threaded section.

As shown in Figure 12.43, the unthreaded length of the bolts must be long enough to accommodate all the pieces to be connected (length equal to the sum of bar thicknesses). On the other hand, the threaded part is defined according to the bolt diameter, having lengths enough to permit the installation and tightening of the washers, nuts and eventually "locking devices" when specified.

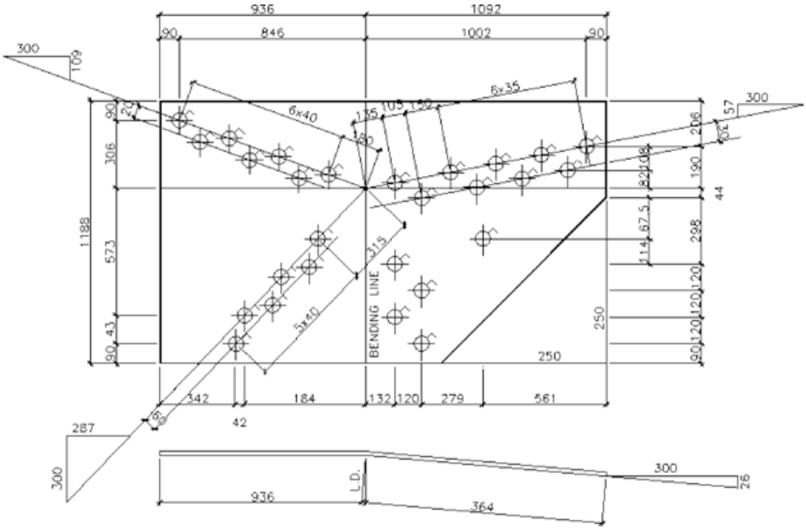
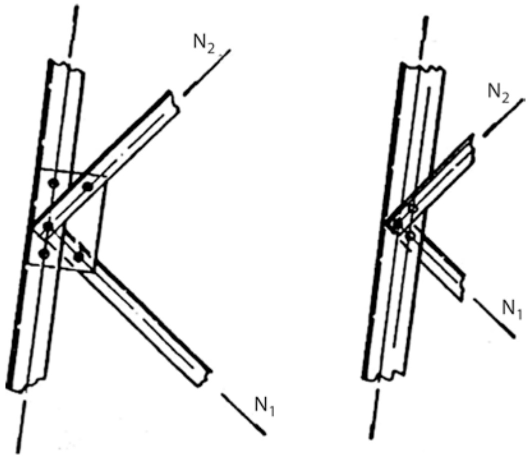


Figure 12.40 Typical gusset plate detailing.

Figure 12.41 Correct Detailing.



12.6.1.3 Nuts, Washers and Locking Devices

Nuts are essential elements on the bolted connections, being responsible for the tightening procedure itself. As per Cigré TB (2009), nut specifications (materials) are according to ASTM A 563 (ASCE 10–97) (ASTM-A563 2000), EUROCODE 3 EN 1993-1-1 (EN 1993) and EN 10025 (EN50341-1) (CENELEC – EN 50341–1 2012) or ISO 898–2 (ISO 898 1 Internacional Standard Organization) (Figure 12.44).

Plane washers are currently used in the lattice tower bolted connections, aiming to protect the galvanized surfaces during the tightening operation, and for better forces distribution on the pieces. According to the above mentioned Cigré TB (2009), washers are normally circular and specified by ASTM A 283 (ASTM-A283

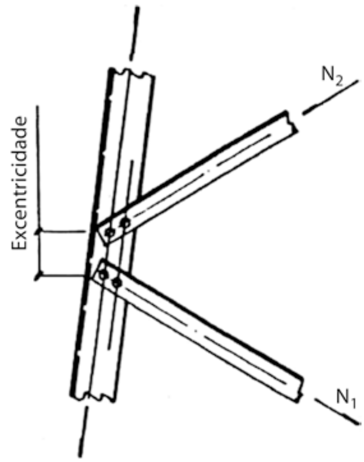


Figure 12.42 Detailing with eccentricities.

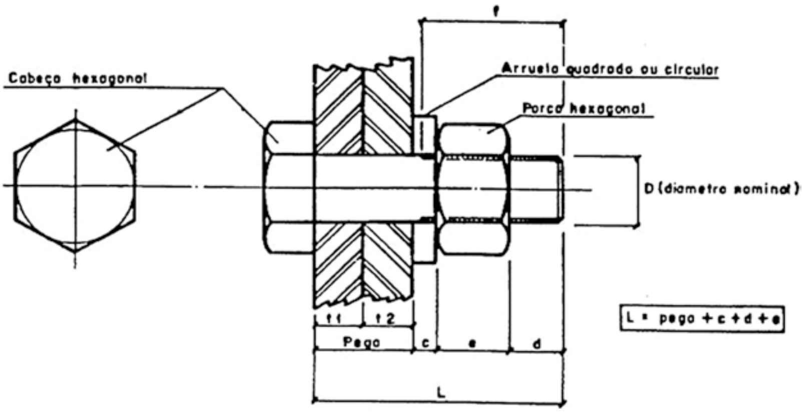


Figure 12.43 Tower bolts detailing.

Figure 12.44 Typical galvanized tower nuts.

2000), EUROCODE 3, EN 1993-1-1 (EN 1993) and EN 10025 and EN 10113 (EN 50341–1) (CENELEC – EN 50341–1 2012).

Locking devices are used aiming to prevent nuts from loosing tightening due to dynamic or thermal effects and, in extreme circumstances, tampering by vandals. Locking methods can be done by means of adequating tightening of the nuts (controlled assembly torques), deformation of the threads, application of thread locking material, spring washers, "palnuts" installation, tamper proof nuts, swaged nuts, welding procedure, etc.

12.6.1.4 Minimum Bolt Distances

As just said before, bolted connections in OHL latticed supports currently need to be designed and detailed considering the use of reduced spaces due to the small leg dimensions of the angle profiles. The minimum bolt distances are, therefore, important premises to be taken into account in the preparation of the detailing drawings. The minimum bolt to bolt and bolt to member edge distances may have impact on the connection capacity and on the assembly easiness. In fact, distances vary according to the allowable shear and bearing stresses adopted in the calculations and can be reduced when those stresses are reduced.

Document Cigré TB 384 (Cigré 2009) shows how national and international standards as well as industry practices treat the subject.

Distances between Holes

As per ASCE 10–97 standard (ASCE 10–97 2000) (Figure 12.45):

$$s \geq \left(1.2\ P / f_u\ t\right) + 0.6\ d$$

s - distance between holes
P - force transmitted by the bolt
f_u - ultimate tensile strength of plate or bolt
t - plate thickness
d - nominal diameter of the bolt
s ≥ nut diameter + 3/8" (recommendation for assembly)

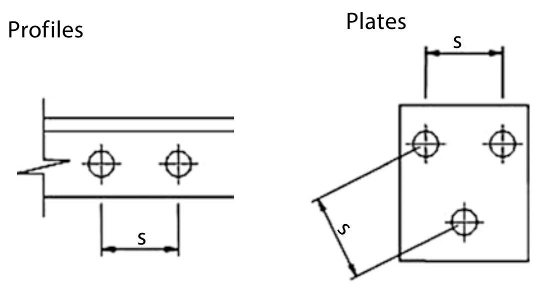


Figure 12.45 Distances between holes.

Table 12.3 Maximum nut diameter

ANSI B18.2.2/81	ANSI B18.2.4.1 M/79
1/2″ - 22.0	M12 - 20.8
5/8″ - 27.5	M14 - 24.3
3/4″ - 33.0	M16 - 27.7
7/8″ - 38.5	M20 - 34.6
1″ - 44.0	M24 - 41.6

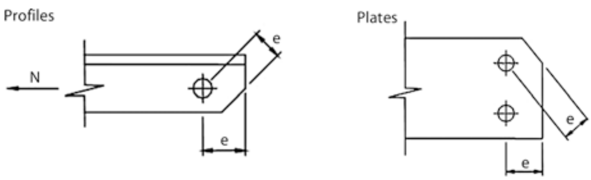


Figure 12.46 Hole distances to cut edge.

Maximum nut diameter (mm) in accordance to Table 12.3:

According to EN 50341–1:2001 – European CENELEC Standard (CENELEC – EN 50341–1 2012):

$$s = \left[\left(P\ \gamma_{M2} / 0.96\ f_u\ \ d\ \ t \right) + 0.5 \right]\ d_0$$

s - distance between holes
P - force transmitted by the bolt
f_u - ultimate tensile strength of plate or bolts
t - plate thickness
d_0 - hole diameter
γ_{M2} - partial factor for resistance of net cross section at bolt holes
d - nominal diameter of the bolt

$$\gamma_{M2} = 1.25\ \left(\text{Claues}\ \ 7.3.5.1.1\ \ \text{of}\ \ \text{EN}\ \ 50341\ \ 1 \right)$$

γ_{M2} may be amended in the National Normative Aspects or the Project Specification.

Distance between Hole and the Bar End

As specified by ASCE 10–97 Standard (2000) (Figure 12.46),

$$e \geq 1.2\ \ P / f_u \cdot t$$

$$e \geq 1.3\ \ d$$

$$e \geq t + d / 2 \quad \left(\text{for}\ \ \text{punched}\ \ \text{holes} \right)$$

e - distance of the hole to the end or the cut edge of the profile
f_u - ultimate tensile strength of the connected part
t - thickness of the most slender plate
d - nominal diameter of the bolt
P - force transmitted by the bolt

Redundant members:

$$E \geq 1.2\ d$$
$$e \geq t + d/2 \quad (\text{for punched holes})$$

Note: Maximum bearing stress f_p to implicitly check out minimum distances:

$$P_{max} = f_p\ d\ t$$
$$e \geq (1.2\ P_{max})/(f_u\ t) = (1.2\ f_p d\ t)/$$
$$(f_u\ t) = (1.2\ f_p/f_u)d$$

and because $e \geq 1.3d$

$$1.2\ f_p/f_u < 1.3 \text{ or}$$
$$f_p \leq 1.3/1.2\ f_u = 1.0833\ f_u$$

As per EN 50341–1:2001 - European CENELEC Standard (J.11.2) (CENELEC – EN 50341–1 2012),

The biggest of:

$$e \geq (P\ \gamma_{M2} d_0)/(1.2\ f_u d\ t)$$
$$e \geq [(P\ \gamma_{m2})/(1.85\ f_u d\ t) + 0.5]\ d_0$$

e - distance of the hole to the end of the profile
f_u - ultimate tensile strength of plate or bolt
t - plate thickness
d_0 - hole diameter
γ_{M2} - partial factor for resistance of net cross section at bolt holes
d - nominal diameter of the bolt
P - force transmitted by the bolt.

Distance between Hole and Rolled Edge

As quoted by ASCE 10–97 Standard (2000) (Figures 12.47 and 12.48),

$$f > 0.85\ e$$

with:

e - distance between the hole and the end
f - distance between the hole and the edge

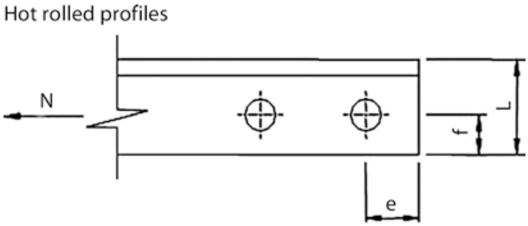


Figure 12.47 Distance between hole and edge.

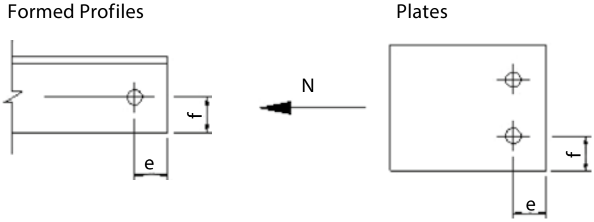


Figure 12.48 Hole distance to clipped edge.

According to EN 50341–1:2001 - European CENELEC Standard (J.11.2) (CENELEC – EN 50341–1 2012),

$$f = \left\{ \left[\left(P \ \gamma_{M2} \right) / \left(2.3 \ f_u d \ \ t \right) \right] + 0.5 \right\} \ d_0$$

f - distance between hole and edge
f_u - ultimate tensile strength of plate or bolt
t - plate thickness
d_0 - hole diameter
γ_{M2} - partial factor for resistance of net cross section at bolt holes (see Item 8.1.2)
d - nominal diameter of the bolt
P- force transmitted by the bolt.

12.6.1.5 Clearances in Holes

Usually the hole diameters are larger than the bolt shank to allow easy assembly. The increase in the hole size must allow some tolerance on fabrication and free zinc remaining in the hole after galvanizing process. Document Cigré TB 384 (Cigré 2009) indicates such clearances as per standards and practices around the world: 1/16” (1.6 mm or 1.5 mm) for bolts M12, M16, M20 and M24 or 2.0 m for bolts M27 and M30.

12.6.1.6 Other Detailing Assumptions

Cigré TB 384 (Cigré 2009), contains many other detailing decisions and/or assumptions that need to be taken into account in the preparation of shop drawings of lattice steel supports. As examples, it deserves to be mentioned, the “maximum permitted length” and the “minimum thickness limit for members”.

Maximum Permitted Length for Members
The maximum physical length of an individual member is generally controlled by restrictions of manufacture, transport and erection. Particular practical limitations are:

- The length of raw material;
- The ability to handle and maintain straightness;
- The size of the hot dip galvanizing bath;
- Transportation limits;
- In rare situations the maximum weight and handling.

As these are non-technical limitations and often based on economics, these limitations are indicated by the Industry or Guidelines, but not in the normative Standards. Industry practices around the world (Cigré TB 384 2009), suggest 9 m as the most accepted value.

Minimum Thickness Limit of Members
The minimum thickness limit of a member affects:

- The life of the member as it affects the thickness of applied zinc;
- The local buckling of sections;
- The vulnerability to damage from manufacture, transport and maintenance;
- The minimum bearing and pullout capacity in a connection.

According to Cigré TB 384 (2009) the more used values as minimum thickness limit of members are 3 or 4 mm.

12.6.2 Metallic Poles

Currently, the steel poles are made from carbon steel, mainly from the high strength low alloy quality. They are normally shaped in modulus from 9 up to 12 m length, with continuous variable polygon cross sections. Depending on the dimensions involved (height and pole base width), the cross section used can be from the square or rectangular (seldom used), to the dodecagonal or circular types (Figure 12.49). The most common joints used are from the “overlapping splices” type, more suitable for suspension or light angle poles. The flanged joints are currently used for the heavy angle or dead-end poles (Figure 12.49).

The finishing types are specified basically to meet two needs: the corrosion protection and the aesthetic. For this purpose, the poles can be only externally painted with

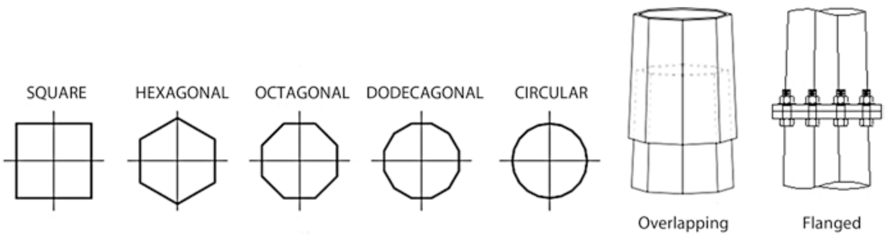


Figure 12.49 Typical Pole Cross-Sections & Joints.

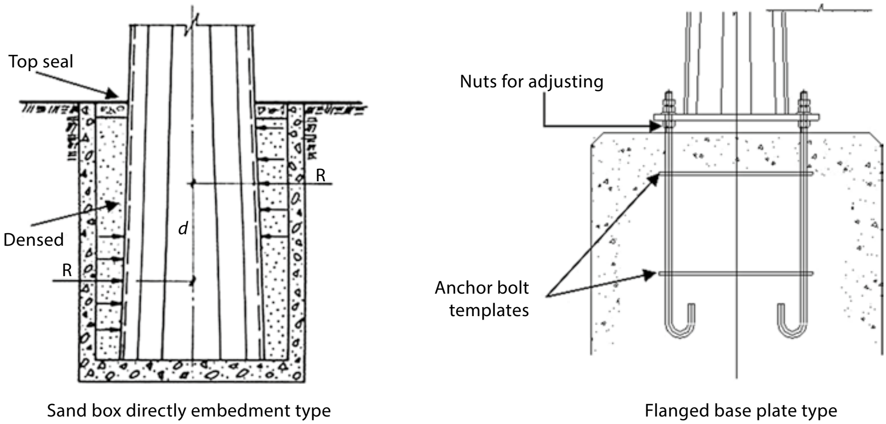


Figure 12.50 Typical PoleFoundation Details.

complete sealed joints, or hot-dip galvanized. The first solution provides an excellent aesthetic finishing, while the second one excellent protection. Therefore, some clients use to specify both finishing procedures, hot-dip galvanizing plus painting to obtain both advantages. For this, it is mandatory to use an appropriate "shop primer" over the galvanized coating in order to create a sufficient/necessary anchorage surface for painting.

As far as the pole foundations are concerned, they have been constructed using two different approaches: the "direct embedment" or the "flanged base plate with anchor bolts". The directly embedment on densed sand box, or concrete, is a very economic solution mainly when designed for light suspension poles. The flanged base plate proposal is specially recommended for angle/dead-end poles, since the "two nuts adjustable system" helps to adjust the pole top deflection (Figure 12.50).

12.6.3 Concrete Poles

Due to its quantity in the grid, the concrete poles are standardized by length and working load. This load is specified to a conventional distance "d" from the top of the pole, generally equal to 0.25 m.

Figure 12.51 CuNap-treated poles at PWP's yard in Sheridan.

The value of load is such that, its effect in terms of moment at the base of pole, is equivalent to the effect of the design live loads. The sections from top to the base increase with a certain ratio to obtain finally an economic structure with quite the same safety along the pole height. The concrete poles are manufactured in special forms, suitable to be centrifuged of vibrated.

The higher structures may be manufactured in modules, attached by bolts or telescoped.

12.6.4 Wooden Poles

The wooden poles are mainly of southern pine, Douglas-fir and Western red cedar. To increase their life, the wooden poles are treated with preservative such as pentachlorophenol (penta), CCA, creosote, copper naphthenate and ammoniacal copper arsenate or ammoniacal copper zinc arsenate.

All wood preservatives typically used by the Utilities in the poles are robust, with many decades of data supporting effectiveness. The Utilities usually need big quantities of wooden poles, both for new overhead lines, or to replace poles from existing ones which can reach the range of hundreds millions. To fulfill these requirements the wooden pole factories are located on large yards (Figure 12.51) and special protection environment measures are needed (Figure 12.52).

12.6.5 Fabrication Process

Due to the large number of pieces to be produced, usually millions, besides the repeatability and the required high speed in the operations of cutting and drilling, the towers must be produced in specialized factories.

Currently, the cutting, drilling and marking operations on angles and plates are made by automatic numeric controlled machines (CNC), which boost the production (Figure 12.53).

Figure 12.52 160′-long treating cylinders.

Figure 12.53 Transmission Line Supports Factory.

Hot dip galvanizing lines give the pieces the required finishing against corrosion, normally enough to withstand for typically longer than 40 years rural atmosphere, practically without any maintenance working.

One relevant aspect to be observed in the manufacture of TL structures is the accuracy to be assured by the machines during the cutting and drilling operations. As per standards and guidelines for design/detailing of towers, the gaps in holes are about 0.8 mm in relation to the bolt diameters. According to Cigré TB 384 (2009), the fabrication tolerances result in a total hole clearance of 1.6 mm (1/16”) over the bolt diameter as a consequence of the “punching procedures”. Anyway, to ensure good structural performance, and perfect mountability, these design and fabrication tolerances must be compatible with the construction and erection ones and be followed during all the process (Cigré TB 2009).

In accordance with Cigré TB (2010), positions of holes are punched or drilled within 1 mm of tolerance over the nominal value. During the fabrication process, the oversized holes result in a tolerance of ±1 mm on the member lengths. Still in accordance with Cigré TB (2010), for bracing members, length tolerances are about 0.15 % of the member length. Dimensions of the tower width are defined by erection and construction tolerances of about 0.1 % of the horizontal dimension of the tower base (Cigré TB 2009).

The erection of the supports can be done manually, piece by piece using auxiliary masts, or in a more automatic way through horizontal pre-assembly and lifting by cranes (see Chapter 15).

In any case, it must always be taken into account the large number of structures to be assembled (with millions of bars and bolts) and the difficulty of logistics that can be found in the field. Towers can be erected in locations with absolute lack of infrastructure, such as roads, any kind of access, electricity, water, concrete for foundations, etc.

12.7 Prototype Tests

It has been a current practice, that new OHL support designs are validated by prototype full scale tests. Clients, designers and tower manufacturers meet in a test facility area (test station) for simulation of all the extreme operational conditions to be supported by the structure during its expected lifetime (Figure 12.54).

In order to make the different interests of all participants compatible and to give a guide for such tests, the Standard IEC 60652 (IEC 60652 2002) was published in 1979 and reviewed in 2002. This document normally is the base to perform all the full scale prototype tests around the world.

12.7.1 Objectives

Full scale tests are normally carried out on prototype supports to verify the design method (and inherent assumptions), the detailing process, the quality of the materials and fabrication procedures. This way, full scale tests are currently performed on the following circumstances:

- to verify compliance of the support design with the specifications (known as “type tests”).
- to validate fabrication processes.
- as part of a research or development of an innovative support.

As general test criteria, the material and the manufacturing processes used in the fabrication of the prototype support shall be from the same specifications to be used during the fabrication of the supports. These specifications shall include the

Figure 12.54 Full scale test facility.

member sectional properties, connection details, bolt sizes, material grades and fabrication processes. In other words, the materials used for the fabrication of a prototype support shall be representative of the materials used in the production of the structures. Aiming this objective, prior to or during the series fabrication, sample tests are required to check the quality of the materials being used. The prototype support to be tested shall be fabricated using material taken at random from the manufacturer stock.

Unless otherwise specified, prototypes shall be galvanized prior to the test procedures, since there is no "black support" in the line (unless they were designed to be installed without galvanization).

12.7.2 Normal Tests

The tests to be performed can be from the "normal" or "destructive" types. They are considered "normal tests" when they are carried only to the specified design loadings (100 %). As a general rule, all loading cases that are critical for any support member should be simulated during the normal tests. As test procedure, according to IEC 60652(IEC 60652 2002), loads shall be applied in increments to 50 %, 75 %, 90 %, 95 % and 100 % of the specified loads (Figure 12.55).

Figure 12.55 Normal test - 500 kV Guyed Tower.

Even if only the 100 % step the only one important for the tower acceptance, intermediate steps are perceived to be useful for the following reasons:

- For balancing the loads prior to the 100 % step.
- For comparing measured displacements and stresses to theoretical values, and possibly, for rapidly identifying any abnormal structural behavior.
- They can be essential to ensure proper rigging settings (load orientation and rigging interference).
- To prevent a premature collapse of the whole tower.

Once the final 100 % load level is reached, the loads shall be maintained for a period of 5 minutes (minimum 1 minute as per IEC 60652). As an acceptance criteria, during this holding period, no failure of any component can occur, mainly near below 100 % step. As in the majority of the cased, the loading hypotheses are “ultimate loads”, it is common that during “normal tests” some failure occurs. In these cases, designs are checked, sometimes members are reinforced and the tests continue until reach the 100 % level (Figure 12.56).

Figure 12.56 Failure at 100 % loading step.

12.7.3 Destructive Tests

If required by the client, and upon agreement with the designer and/or the fabricator, "destructive test" can be performed. If a destructive test has to be carried out, it is a common practice to do it using one of "exceptional loading cases" (e.g., extreme transverse wind), by increasing both transverse and the vertical loads (or even longitudinal in case of "anti-cascade" hypothesis or dead-end tower), in steps of 5 % until the support failure. This procedure permits to gain information on actual versus predicted behavior or, in the case of suspension towers, the failure load can be related to an increase in span utilization. In cases where there is no ice involved, it is a current practice to increase only the transverse (or longitudinal) loads (Figures 12.57 and 12.58).

12.7.4 Acceptance Criteria

As full scale prototype test acceptance criteria, IEC 60652 (IEC 2002) quotes that "the performance of the support shall be considered acceptable if it resists the specified design loads (at 100 % of each load case) for minimum 1 minute without

Figure 12.57 Destructive test – 230 kV Double Circuit tower.

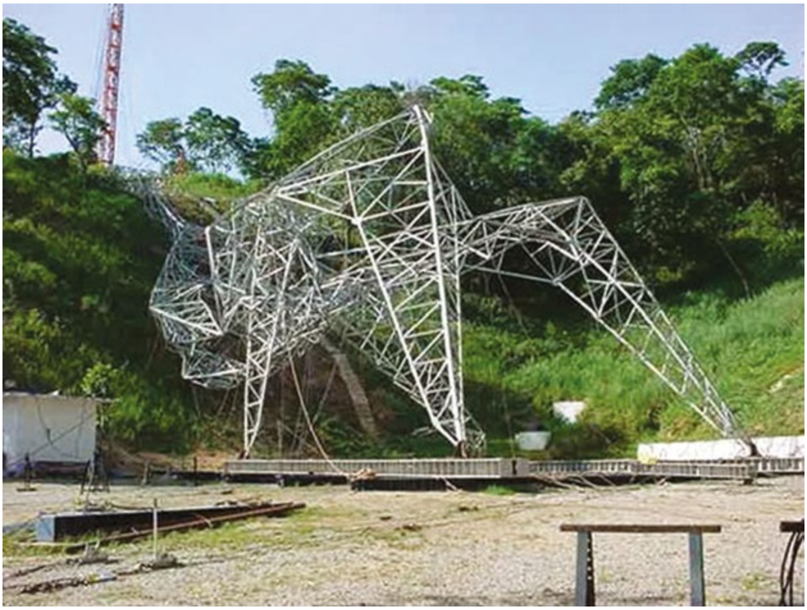


Figure 12.58 Destructive test – 500 kV TL Tower.

failure of any components or assemblies even though a longer holding period may have been specified (normally 5 minutes)". This is a "deterministic approach" and, even being practical, it is not perfectly coherent with the probabilistic based design method. As per IEC 60826 Standard (Design Criteria of Overhead Transmission Lines), concepts such as statistical distribution of support strengths, strength factors ($Ø_R$), exclusion limits etc, need to be taken into account due to their importance for the support designs and consequently, for the prototype test result interpretations.

As per Cigré TB 399 (Improvement on the tower testing methodology 2009), applying IEC 60826 (Design Criteria of Overhead Transmission Lines) as basic philosophy for the design of overhead lines, means in terms of structures, to adopt the following general equations:

$$\gamma_u Q_T \leq R_{10\%}$$

Where:

Q_T = Design loading referred to a returned period T (or/with specific load factors);
$R_{10\%}$ = Design strength with a 10 % exclusion limit
γ_u = Use factor coefficient.

On the other side, the 10 % exclusion limit strength can be obtained by:

$$R_{10\%} = \phi_R R_C$$

Where:

R_C = Characteristic strength assessed by calculations and calibrated by loading tests (or professional experience);
ϕ_R = Strength factor to be used in the design, and evaluated as function of the statistical distribution of towers strengths.

Therefore, as far as the OHL support designs are concerned, to use IEC 60826, enables the designer to estimate the characteristic strengths of the towers applying realistic known "Strength factors". Both, the characteristic strength and the strength factor, are concepts, parts of the same issue involving design and loading tests.

As mentioned in Section 3, the statistical distribution of tower strength has been firstly studied by, Paschen et al. (1988), and afterwards by Riera et al. (1990). Both studies have similar conclusions indicating log-normal distribution as the best fitted curve to represent the population of support strengths, with mean values "little above" 100 % and coefficient of variations below 10 %.

Taking the Riera et al. (1990) study as example, the design curve showed on Figure 12.59 can be established:

Where Cv = Coefficient of Variation, Rm = Mean Strength, Rc = Design or characteristic Strength, R_{10} = 10 % Exclusion Limit Strength;

From that study, and making the calculations accordingly, the 10 % Exclusion limit Strength and the "Strength Factor" "ϕ_R = 0.93" can be obtained.

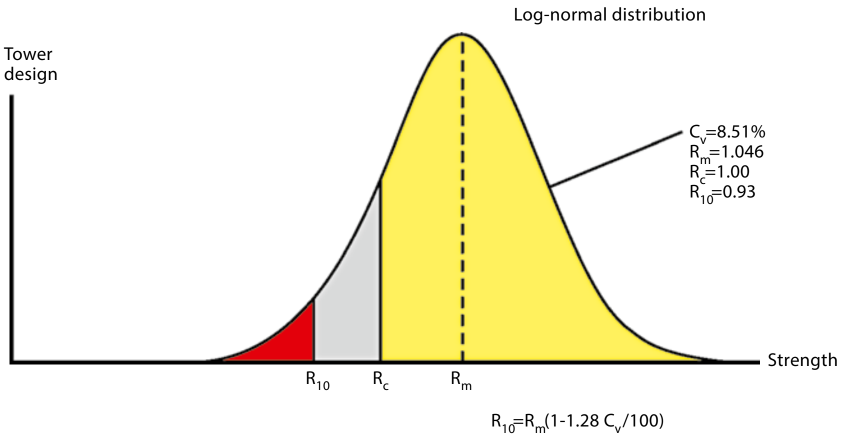


Figure 12.59 Support design strength statistical distribution.

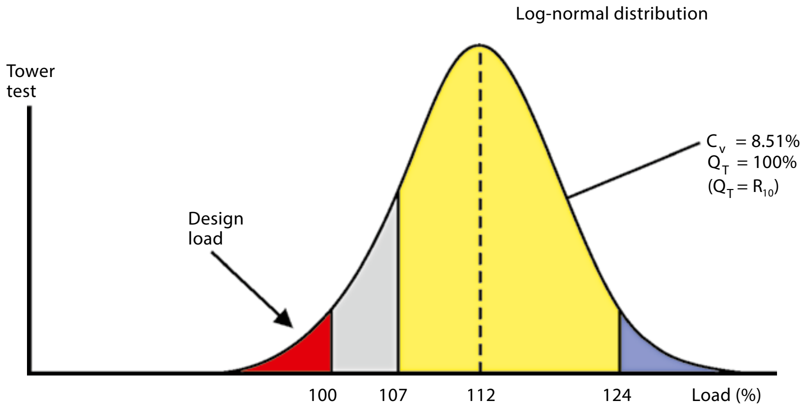


Figure 12.60 Tower test strength statistical distribution.

A corresponding loading test on a probabilistic based approach should also have as targets, to reach statistical distribution curves such as, for example, the herebelow shown on Figure 12.60, where Q_T = Design Load.

So, the results of tower loading tests on a probabilistic based philosophy, should have the objective to confirm and/or calibrate both curves in terms of "Loading and Strength".

As conclusion, the current practice adopted by the industry on testing OHL supports is correct and valid, but it seems that the test objectives should be improved aims should be to check the loading supportability and the design adequacy.

As well as desigh premises specially for long new lines, at least the light suspension(s) tower(s) should be tested, preferably up to the destruction. It is always important to remark that, according to IEC 60826 (IEC 60826 Standard Design Criteria of Overhead Transmission Lines), those towers should be designed as the "weakest link" of the transmission line system, being, therefore, its risky element.

When, for any reason, it is not possible to test at least the most numerous suspension tower, the desirable reliability level should be evaluated and adequate "Strength factors $Ø_R$" should be adopted aiming to reach that proposed target.

Finally, if the "Probabilistic Based Approach" is used as the main design philosophy, it is recommended to perform loading tower tests coherent with that philosophy. Using this concept, those tests should be carried out aiming to get from the supports the following responses:

- To support expected "loading cases" as minimum;
- To behave as estimated by the structural calculations, having a "failure strength" compatible with the strength factor ($Ø_R$) adopted.

Therefore, as better explained previously, the interpretation and approval of the test results should be based in two premises: the "loading support capability" and the "expected strength/behavior".

12.8 Special Structures

12.8.1 Guyed Supports

As mentioned in Section 2, guyed supports are those that combine rigid elements (mainly lattice beams, masts, frames or even poles) with prestressed guy wires resulting in stable economic and structural systems (Figure 12.61).

They are very much used in high voltage overhead transmission systems, especially for long lines where the servitude in not a so critical issue. Currently, guyed structures are used, or in distribution lines (voltage level below 50 kV), or extra high voltage lines (above 300 kV). For intermediate voltage levels, due to the heights and loadings involved, the guyed supports, in the majority of the cases, use to be not economical.

Normally, guyed structures are used as suspension supports, while self-supported towers are applied to the other support functions in the line. Therefore, according to IEC 60826 (IEC 60826 Standard Design Criteria of Overhead Transmission Lines), the suspension guyed structures are designed to be the weakest links in the transmission line systems.

12.8.2 Guyed Structure Types

Depending on the arrangements proposed among the rigid elements and the wires, the guyed supports can be of the following types:

12.8.2.1 V-Guyed Type

The V-guyed type support is very successful as suspension tower, and the most used type of guyed structure for high voltage overhead lines around the world so far (Figure 12.62).

Their main advantages are:

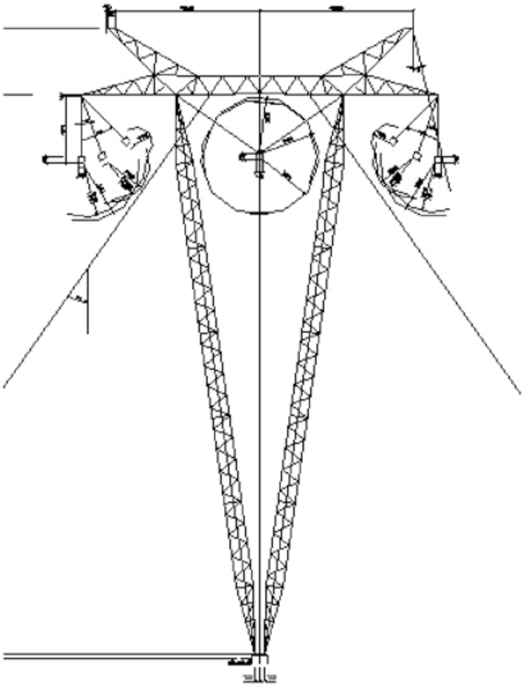


Figure 12.61 Guyed OHL support.

Figure 12.62 V-Guyed Support.

Figure 12.63 Portal Guyed Support.

- Narrow corridors required as compared with other guyed types (≤60 meters for 500 kV).
- Lower weights in comparison with equivalent self-supported towers (about 40 % less).
- Only 5 small foundations (4 of them having only uplift loads)
- very easy erection and adjustments.
- Only a single point in the terrain required for installation.

12.8.2.2 Portal Guyed Type

Portal guyed type (or H-type) is also a well known and very used type of OHL suspension support in the world grid (Figure 12.63).

Their main characteristics and advantages are:

- Possible reduced servitude as compared with the V-type.
- Only 4 foundations required (2 only in uplift).
- Normally a little heavier than the V-guyed.
- Two location points in the terrain for settlement
- Easy to assemble and erect.

12.8.2.3 Cross-Rope Suspension (CRS) Type

The Cross-rope suspension towers are an evolution of former "chainette" concept and started to be installed during the 90's. The evolution done on the chainette type, was basically a simplification on the suspension arrangements of the conductors, using, in this case, a single rope crossing through the top of the masts. This single modifications enables to reduce the number of joints (compression terminals or pre-formed strips) and simplifies the installation (Figure 12.64).

Thousands of kilometers of extra high voltage overhead transmission lines have been successfully installed around the world using cross-rope suspension supports. When applicable, excellent results have been reported with the use of this type of support.

Figure 12.64 Cross Rope Suspension Support.

The main characteristics and/or advantages of the CRS supports are:

- Larger “right of way” required (normally above 75 m for 500 kV).
- Very cost effective.
- 6 small foundations necessary (4 only in uplift loading).
- Due to the open space on the tower top geometry, it is the best structural solution for the use of “phase compaction” or “expanded bundle”(HSIL) techniques.
- More feasible and specially recommended for very long lines crossing inhabited areas, or on other cases where the servitude width is not a so critical issue.

12.8.2.4 Lattice Guyed Monomast

The “lattice guyed monomast” support is not a new solution on the transmission line grid, but its use has been recently increased quite a lot, due to the great number of long EHV line projects under construction in the world (Figure 12.65).

The main characteristic of these structures is that they have only one mast. Their advantages normally are:

- Very cost effective, similar to CRS type.
- Reduced servitude when compared to all the other guyed support types (50 m can be adequated for 500 kV OHL’s).
- 5 foundations, similar to V-guyed type.

Figure 12.65 500kV Lattice Guyed Monomast Support.

12.8.2.5 Structure Characteristics

All the types of guyed supports are a combination of masts beams or frames, basically latticed elements, with guy wires. The joints between those elements are from the pin-joint type with bolts, while the mast base support is from the "universal" hinge type. This way, it is possible to design masts and beams as modules, calculating them according to their critical loading cases (beams very sensitive to insulator string "swing angles" and vertical loads while masts to their heights and wind loads) and combine them accordingly forming interesting and economic modular supports.

Aiming to reduce the support elastic deformation, the guy wires to be specified must be of the "pre-stressed" type. Their initial installation stresses are, currently, of about 10 % of the "ultimate tensile stress"(UTS), while the maximum calculated guy wire loads shall not exceed 75 % of their UTS.

12.8.2.6 Structural Analysis

As quoted by ASCE 10-97 (2000), *guyed structures normally require a second-order analysis. Guyed structures and latticed H-frames may include masts built-up with angles at the corners and lacing in the faces. The overall cross-section of the mast is either square, rectangular, or triangular. Latticed masts typically include a very large number of members and are relatively slender, that is, may be susceptible to second-order stresses. One alternative to modeling a mast as a three-dimensional truss system is to represent it by a model made up of one or several equivalent beams. The properties of equivalent beam that deflects under shear and moment can be worked out from structure analysis principles. The beams are connected to form a three-dimensional model of the mast or entire structure. That model may be analyzed with any three-dimensional finite element computer program. If large deflections are expected, a second order (geometrically nonlinear) analysis should be used. Once the axial loads, shears, and moments are determined in each equivalent beam, they can be converted into axial loads in the members that make up the masts*.

The guyed supports are, in general, very flexible and elastically deformable structures when compared to the self-supported ones. They adapt much better to the flexibility and elasticity of the cables, making the transmission line itself to behave more similar to a homogenous system. This way, in many circumstances they may act as a kind of "transverse or longitudinal virtual line dampers" dissipating energy through their deformations and helping the system to absorb for example, breakage of conductors, impacts, cascade effects, etc. This can be seen when segments when a segment of line containing some spans and structures are entirely modeled like a system.

12.8.3 Supports for Direct Current Lines

The DC transmission systems have has had a great demand by the electricity market lately. This recent tendency is basically due to:

- New technology development for the DC equipments (valves, converter stations, etc) allowing considerable cost reduction.
- Conversion of AC to DC lines, aiming to optimize power transfer capability in existing corridors.
- Very big blocks of electrical energy to be transported (up to 6000 MW per bipole) in some regions.
- Very long transmission lines (above 1500 kms) being installed.
- Reduction in the line losses especially on those "supergrids".

As a consequence, more and more DC line supports have been installed recently.

12.8.3.1 Main Characteristics

From the supports point of view, DC lines are very similar to the AC ones. When applicable, suspension towers are normally from the monomast guyed type, while self-supported structures are used for the other support functions in the line.

Their format is always from the "pyramidal" type, having only two cross arms (one for each pole) and two ground wires peaks (Figure 12.66 and 12.67).

12.8.3.2 Structural Analysis

In the majority of the cases, the structural element used is the so called three dimensional truss, formed by lattice planes of "tension-compression" systems as described in Section 4.

As far as the structural analysis is concerned, the DC supports are generally modeled as 3-D truss linear analysis. For the guyed-monomasts, however, "non-linear analysis" is always required.

12.8.4 Supports for Large Crossings

Finding new routes for high voltage overhead lines may require designs that address obstacles such as valleys, wide rivers and arms of seas. Large overhead line crossings are currently designs at the limit of the "state-of-the-art", as they can demand

Figure 12.66 DC Guyed Monomast Support.

very long spans and/or extra high supports. Standards generally do not cover all the necessary load assumptions and design approaches for such projects. This way, information about crossing projects already constructed can be essential to assist designers around the world to make decisions in the absence of relevant standards. Aiming to contribute to this demand, Cigré has published TB entitled (Large Overhead Line Crossings 2009). An interesting data bank was created containing valuable information, such as used conductor types used, tension applied and vibration control devices, employed phase spacings, spans, sags, insulator strings, tower heights, tower weights, etc.

For the purpose of the Cigré study, a large crossing was defined as a project having a wind span of 1000 meters or above, and/or a tower with height of 100 meter or more (Figures 12.68, 12.69, and 12.70).

12.8.4.1 Main Characteristics

Currently, supports for big crossings are uncommon and unique projects, usually having very high structures and supporting big loads as result of large spans.

They may demand the fabrication of "out of standard and special profiles", having double or quadruple sections, or even latticed elements as diagonals or main members. Many welded joints are normally used to make parts or structural components.

Figure 12.67 DC Self-Supported Structure.

Figure 12.68 132 kV Ameralik Fjord Crossing, Greenland (5.37 km crossing span).

Figure 12.69 4 × 380 kV - Elbe River Crossing, Germany.

Figure 12.70 500kV Jiangyin Yangtze River Crossing, China (Suspension tower with 346.5 m height).

Due to the support heights and the inherent elastic deformations, non linear analysis models are always required. In the majority of the cases, a structural modeling covering the entire crossing is recommended to take into account dynamic effects.

Supports fabrication demands special machines and devices, as well as strict quality control system for the welding procedures. In many cases thermal treatments are required on welded components for stress relieving purposes.

Pre-assembling of parts at the factory yard is well recommended and erection at site location currently demands special equipments and methods.

12.9 Environmental Concerns & Aesthetic Supports

12.9.1 Environmental Issues

Years ago, aesthetics was not a value taken into account in the design of supports for new transmission lines. Towers were not judged as "pretty" or "ugly"; they were just considered essential elements for transmission of electricity. Many thousands of kilometers of transmission lines were, thus, built in all continents justified by the benefit of the electricity, considered a privilege of modern societies.

This reality started to change mainly after the 60's, when the environmental aspects of the lines and the aesthetics of the supports, began to be more and more questioned in the implementation of new projects. These changes can be understood as a result of many factors, such as:

- The existence of thousands of kilometers of lines already built in some countries and/or some regions.
- The evolution of the benefit of electricity, from "a privilege of few societies" to an "acquired right" of the citizens of the twentieth century. Electricity subtly became a "social right", and an obligation of the governments to provide it.
- The increasing presence of transmission lines in inhabited areas, in such a way, that the towers became familiar elements in the cities.
- The difficulty of obtaining new urban corridors for bringing more power to central regions of cities especially those with high vertical growth.
- A greater environmental conscience motivated by the various aggressions to the environment in different regions of the world.

All this together made that the environmental aspects had become one of the most important premises in the studies for the implementation of new transmission lines. Nowadays, transmission line projects have to start with an environmental impact assessment, where environmental auditors identify and analyze the impacts on the nature and human environment. New line routes have to find a balance between the need of electricity transmission and the environmental perspectives. As part of these studies, the visual appearance and the aesthetic of the towers began to play an important role in the analysis, once they are the most visible elements on the landscape.

12.9.2 Innovative Solutions

12.9.2.1 First OHL Tower Aesthetic Studies

The subject of the aesthetic of transmission line towers is not a new issue. During the 60's, designers and power lines engineers had already started to study improvements on their aesthetics.

One of the first remarkable initiatives was the studies reported by H. Dreyfuss & Associates, through a publication of Edson Electric Institute in 1968 (Electric transmission structures – a design research program 1968). The document contains 47 innovative proposals for Overhead Line Supports with different conductor configurations, on single or double circuits, and using different materials such as steel, concrete or wood. Outstanding aesthetic solutions were suggested, perhaps a little advanced for their time, but with a clear vision of future (Figure 12.71 and 12.72).

The Dreyfuss' studies were carried out in times when more and more power lines began to cohabitate with citizens and cars on the cities, disputing their urban space. Transmission lines had to bring more power to the downtown of big cities growing vertically, and/or cities expanding horizontally reaching existing servitudes of lines already constructed. For these reasons, in different parts of the world, utilities have seriously started to think more and more seriously on the aesthetic of the overhead line supports.

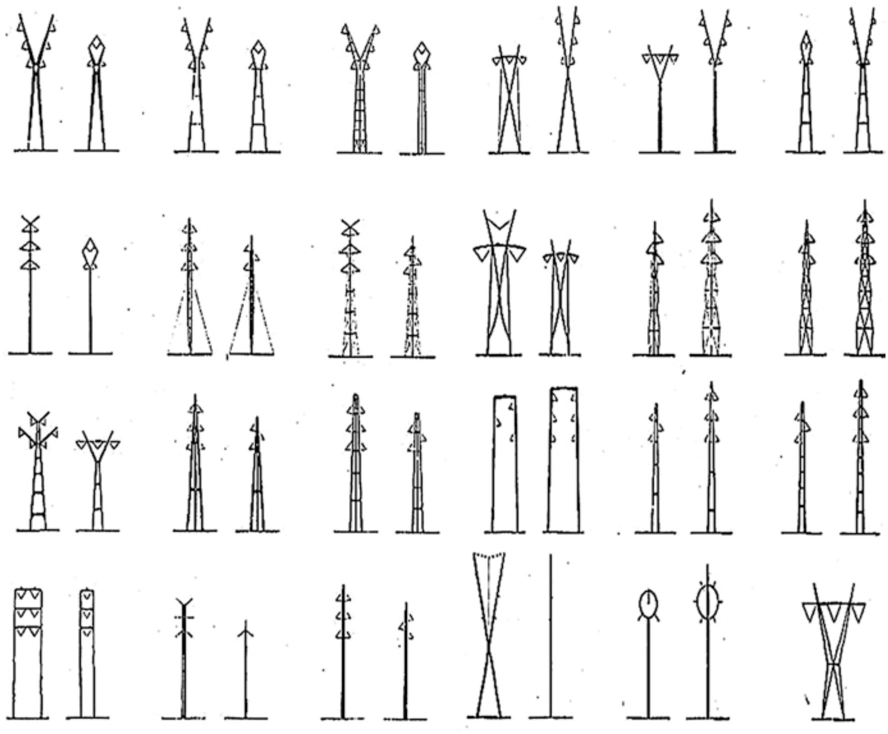


Figure 12.71 H. Dreyfuss Pictorial Index.

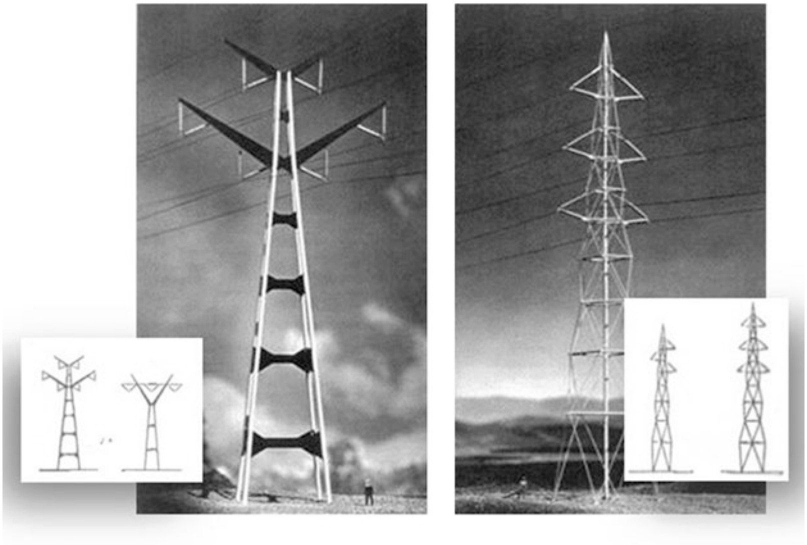


Figure 12.72 H. Dreyfuss Proposal's.

12.9.2.2 First Initiatives

The first practical initiatives in terms of "aesthetic towers" were not so ambitious and, basically, oriented by the following principles:

- To compact the lines and the supports as much as possible;
- To reduce the number of structural elements on the towers;
- To try to put them invisible or camouflaged at the landscape.

The compact solutions like the monopoles, the portal and V guyed, the chainette and the "cross-rope suspension - CRS", were solutions that have fulfilled those objectives (Figure 12.73).

Thousands of kilometers of lines were constructed around the world using these solutions, which aesthetic principles were based on simplicity, slenderness, symmetry, invisibility, reduced number of structural elements, transparence.

12.9.3 Landscape Towers

During the 90's, the aesthetic of the OHL towers became a real issue in some regions, and the first "landscape towers" were installed. New approaches and techniques were applied envisaging a better public acceptance. Aiming to collect all those new ideas, Cigré carried out a survey publishing a document entitled (Innovative Solutions for Overhead Line Supports 2010).

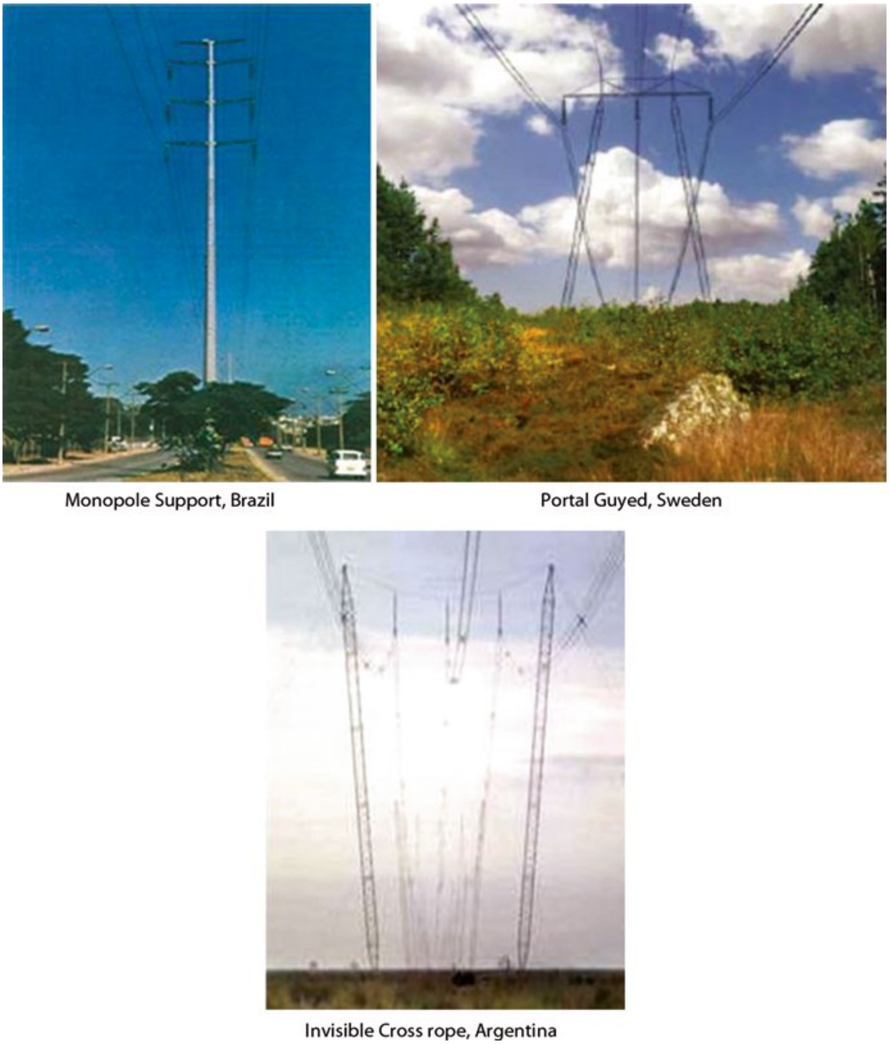


Figure 12.73 First Aesthetic Solutions.

An interesting databank was created showing the great variety of aesthetic solutions adopted in different parts of the world.

Analyzing the solutions reported, it could be identified that the so called “aesthetic proposals” already adopted by the Utilities, follow three basic principles: to design aesthetic solutions for unique places, for a single line, or to develop standard aesthetic solutions. Examples of these trends can be found, for example, in Finland, in Denmark and in France, as described in the following items.

12.9.3.1 Solutions for Unique Places: The Finnish Experience

In Finland, there are good examples of unique tower solutions for specific places. The first landscape towers were constructed in the early 90’s by Fingrid Plc, the

Figure 12.74 The "Yellow beak" - Turku, A. Nurmesniemi.

national grid operator in Finland. The company wanted to get better public acceptance for its lines and, in some cases, to use them as landmarks in public places.

First landscape towers were installed in 1994, in the Southwest of Finland coast in the city of Turku. It was a series of six towers, the design of which was matched with the gabled one-family houses of the residential area nearby. Colour schemes were inspired by the surroundings (Figure 12.74).

Few years later, a multi-level junction in Espoo was provided with an unique landmark and piece of environmental art: a series of three 400 kilovolt towers, referred to as "Espoon sinikurjet" (Blue cranes of Espoo) on account of their blue colour, (Figure 12.75).

After this, towers adapted to the surroundings were erected at the cities of Virkkala (Figure 12.76), Tuusula (Figure 12.77), Jyvaskylan (Figure 12.78), Hameenlinna (Figure 12.79), Porvoo (Figure 12.80), Vantaa (Figure 12.81) and Oulu (Figure 12.82) (Pettersson et al. 2008; Exhibition "Suuret Linja" 2003).

12.9.3.2 Solutions for Specific Lines

The most common approach for reaching environmental friendly power lines, is to propose an aesthetic solution for a specific line (or just a line segment in some cases), which crosses a sensitive region.

A good example of that, is the 400 kV connection line between the cities of Aarhus and Aalborg in Denmark (Öbro et al. 2004). Another remarkable case is the transmission line Salmisaari-Meilahti in Helsinki, Finland, shown in Figures 12.83 and 12.84.

Figure 12.75 "Blue Cranes", Espoo, Studio Nurmesniemi.

Figure 12.76 Petäjävesi, Virkkala, B. Selenius.

Figure 12.77 Tuusula, IVO Power Engineering.

Figure 12.78 Jyvaskylan, J. Valkama.

Figure 12.79 "Antinportti", Hämeenlinna, Studio Nurmesniemi.

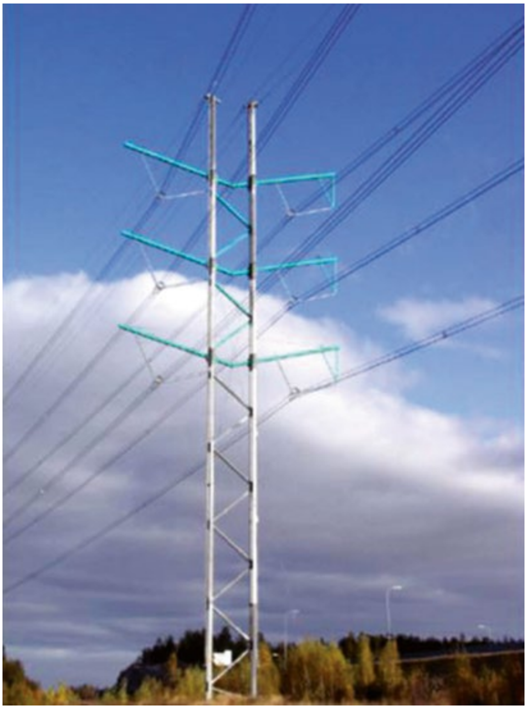


Figure 12.80 Ilola, Porvoo, Studio Nurmesniemi.

Figure 12.81 Rekola, Vantaa, J. Valkama.

Figure 12.82 2003: Oulu, Kuivasjarvi.

Figure 12.83 400kV TL Aarhus/Aalborg - Denmark.

Figure 12.84 TL Salmisaari/Meilahti - Finland.

12.9.3.3 Standard Aesthetic Solutions

To develop "standard aesthetic tower solutions" is one of the approaches used by RTE, the French Transmission Grid Operator, for the integration of Overhead Lines

Figure 12.85 The "ROSEAU" and The "FOUGERE" Towers.

into the environment. Aiming to reach this objective, RTE has promoted two experiences for development of innovative supports: the architects and the tower manufacturers design competitions.

The first architects design competition was carried out in 1994 and had, as main target, to develop standard aesthetic solutions for 400 kV Overhead Line Towers, to be used when and where it would be necessary. As per (EDF Brochure 1995), the competition process lead to the definition of two standard aesthetic tower families: the "Roseau (reed)" and the "Fougère (fern)".

The "Roseau" is a slender structure, exploring the verticality of the support element. An original technology was used based on open-work modules of casting material for the lower part of the tower (Figure 12.85).

The "Fougère" type support consists of a tubular tower whose originality lies on the distribution of conductors in a horizontal position spread over two independent structures in the shape of an "f". In these solutions the architects were looking very pure forms (Figure 12.85).

The second experience was performed along 2004/2005 and proposed only among support manufacturers. Differently from the previous architects' competition, when as much freedom as possible was given to the proponents, the advantages of this new procedure were basically the use of tested/existing solutions, with little industrialization difficulties, reduced development times and at reasonable costs. The main disadvantage was that the creativity was reduced resulting on more traditional shapes and formats. With this procedure, a new wood support was developed for using in the 225 kV OHL's, named as "The Arverne". (Figure 12.86).

Figure 12.86 The "ARVERNE" Tower - Transel, Linuhonnun.

12.9.4 Overhead Line Supports into Artworks

The various experiences carried out in the world with aesthetic towers were, generally, very successful in terms of public acceptance. Those initiatives motivated the Utilities that had lines in sensitive areas, to expand the concepts and the use of "landscape towers": Since the 90's, slowly, the towers were evolving from OHL Supports to "Urban Electrical Sculptures".

In Finland, after the well succeeded first experiences, landscape towers have continued to be designed and constructed. As examples, Figures 12.87 and 12.88, show new solutions in the cities of Lempäälä and Vihti (Pettersson et al. 2008), that are really sophisticated sculptures used as towers to support conductors.

In France a new technique was utilized for improving the aesthetic of OHL transmission Lines: The artistic treatments of lattice towers designed by Elena Paroucheva (2007).

Her works aim at emphasizing the above ground networks such as, energy transmission and distribution supports. Instead of trying to hide them in the landscape, they are transformed into "artworks" (Paroucheva 2007). Generally, two kinds of techniques are applied: "Art Installations" and "Sculptures". The "Art Installations" solutions treat the transformation of existing infrastructures elements in the

Figure 12.87 Lempäälä, Konehuone/J. Valkama.

Figure 12.88 Nummela, Vihti, Konehuone/J. Valkama.

environment. They are investigated according to their installation areas and allow the modification of their visual aspect into artistic works (Figures 12.89 and 12.90).

The "Sculptures" explore new forms of towers to be implanted in the landscape, in both urban and rural areas (Figure 12.91).

Figure 12.89 Art Installations on Overhead Line Supports.

12.9.5 Experiences around the World: Conclusions

Analyzing the aesthetic solutions collected, and the arguments justifying them, interesting aspects can be reported. Firstly, it can be observed an increasing

Figure 12.90 Paroucheva's art installation works at Amnéville-les-thermes.

environmental concern resulting in a great discussion for approval of almost all new OHL projects around the world. This is valid for short or long lines, and for both, urban and rural landscapes. There are, however, different policies regarding the OHL Lines and the environment.

In the case of very long lines, normally crossing rural areas, cost is an absolute relevant issue which targets of economy cannot normally be reached with aesthetic towers. In these cases, premises adopted for environmental friendly supports are still the same as already mentioned before: invisibility, transparence, slenderness, compaction, camouflage, all together driven by cost (Figure 12.92).

Figure 12.91 Paroucheva's Studies: Tower Sculptures.

In urban areas (or even rural sometimes), aesthetic solutions have been more and more used in different parts of the world, aiming to reach public acceptance. As seen previously, to achieve this, different policies have been implemented such as to design "unique landscape towers" for specific places, "for a specific line", and even to design "standard aesthetic solutions" (Figures 12.93, 12.94, and 12.95).

Figure 12.92 Tower solutions for long OHL's around the world.

Figure 12.93 1992 Seville Expo Tower - Spain.

Figure 12.94 Vaasa, Palosaari, Konehuone/J. Valkama.

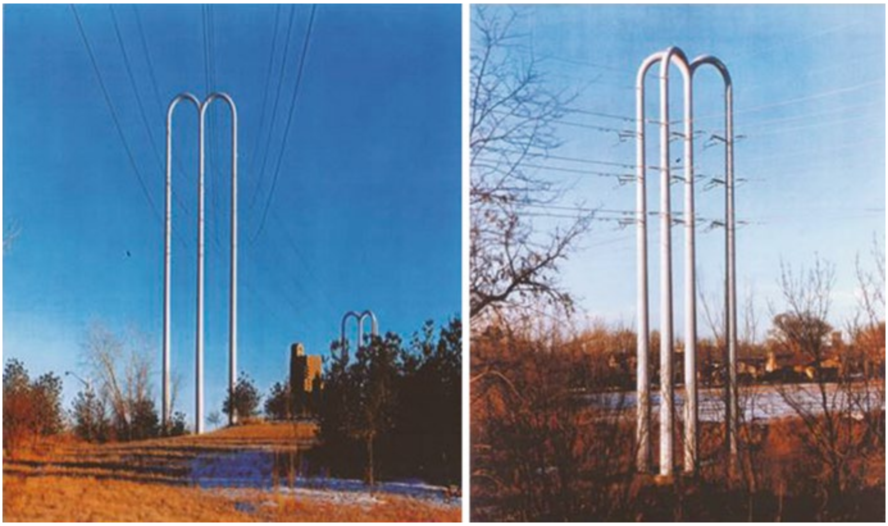


Figure 12.95 Double circuit and triple arches - USA.

12.10 Existing Lines & Tower Aging

12.10.1 Asset Management/Grid service

Asset management models, currently assign distinct roles to the asset owners, asset managers and service providers. Under this approach the asset owner prepares the strategy for the high-voltage grid, inclusive accompanying frameworks and targets. Within these prescribed risk confines, the asset manager formulates proposals for construction and maintenance of the grid. The asset manager subsequently instructs one of the service providers to carry out the work, manage services and perform maintenance.

The employees of the Grid Service performance unit take care of the infrastructure (stations, lines and cables) used by the market parties to transmit electricity. This performance unit manages and maintains the grid, takes care of grid planning and provides advice on possible new installations to be constructed. Transmission Operations also continually monitors whether the grid needs to be altered.

This chapter shall focus on asset management of supports. More details can be found in Chapter 17, which treats the subject in a broader approach. Furthermore, it shall be pointed out the activities of Cigré Study Committee C1, which deals with general asset management strategies and practices, risk and reliability assessment, new approaches and system planning criteria.

12.10.2 Assessment of Existing Supports

Aiming to assess information about the aging process of the existing OHL supports, Cigré carried out an international practices survey regarding assessment of existing supports and the consequences for maintenance, refurbishment and upgrading (TB: Assessment of existing overhead line supports 2003). The following keywords describe the subject:

- Inspection tools and methods
- Inspection reports
- Assessment of inspection data
- Type and cause of defects
- Inspection philosophies
- Criteria for management decisions
- Experiences and solutions.

The questionnaire was replied by 61 company representatives from 29 countries. The majority of answers came from Europe, only a few replies came from America, Asia and Africa. As the questionnaire was split between overhead transmission and distribution lines, 104 filled in forms were received. The percentages in Figures 12.96, 12.97, 12.98, 12.99, 12.100, 12.101, 12.102, and 12.103 refer to these responses and reflect the “yes”-answers only.

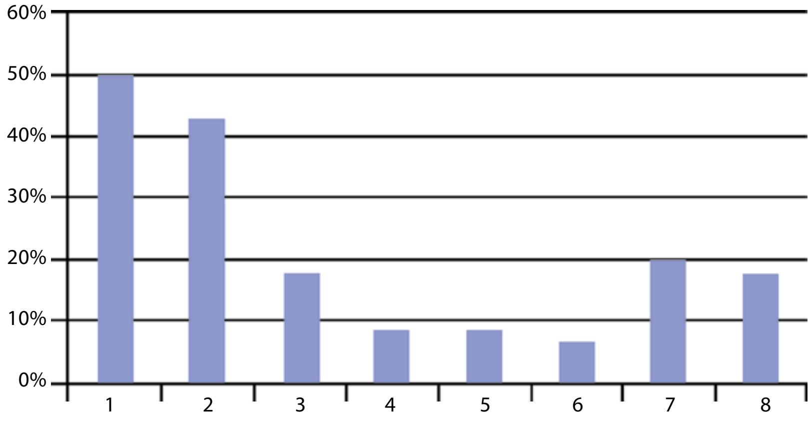


Figure 12.96 Used inspection tools.

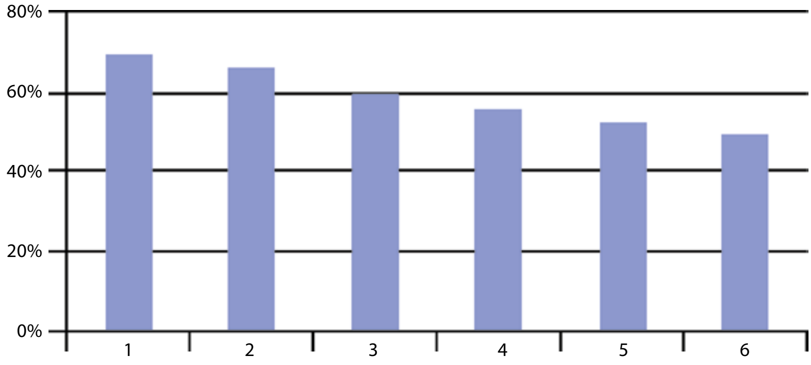


Figure 12.97 Importance of information regarding support member deformation/damage.

The chapter shall support companies when establishing or benchmarking a support management system.

For a better understanding, important terms are defined below:

- Maintenance - routine conservation and small/local repair
- Refurbishment - extensive renovation or repair to restore their intended design strength
- Upgrading - increasing the existing strength which may resist increasing loads.

12.10.2.1 Inspection Methods and Tools

The questionnaire asked for the methods of support inspections and the tools used. It was differed between steel, concrete and wooden supports. There were also questions regarding laboratory examination on materials.

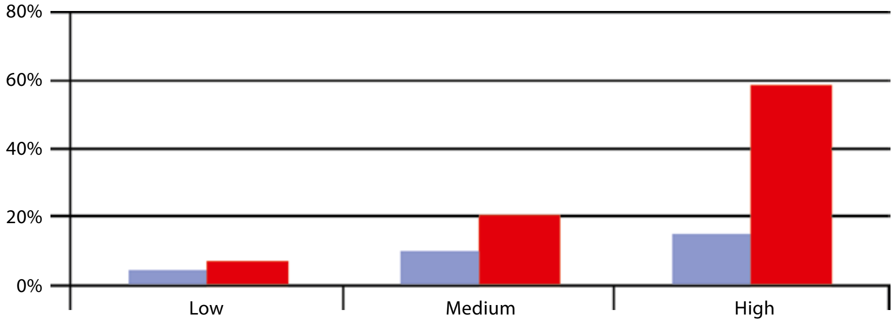


Figure 12.98 Different classification of corrosion extend (percentage of surface attacked).

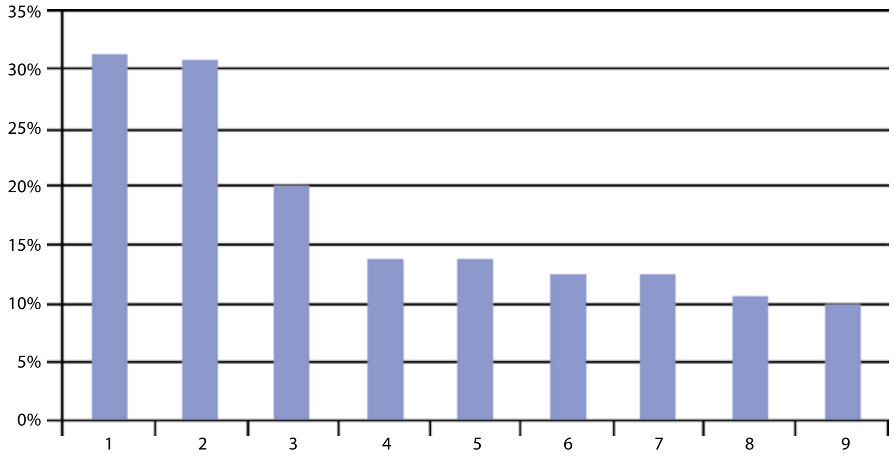


Figure 12.99 Typical identified faults.

Generally, it can be said that inspection is mostly limited to visual inspections but the majority of the companies use also special tools. Destructive tests are seldom applied on existing supports.

Nearly 50 % of the companies stated to perform laboratory tests on support material (metallurgical, chemical and/or mechanical analysis). This special laboratory examination is not done systematically but rather rarely, mostly after failure. Many companies mentioned that laboratory tests are useful only before manufacturing or erection, so it is not clear whether the 50 % refers to existing structures or both new and existing ones. On the other hand, recent findings of possibly steel aging (hydrogen brittlement) have released a systematically laboratory analysis of transmission line tower steel produced in the sixties in Germany.

It shall be pointed out to a German pre-standard VDE V 0109–2 (DIN V VDE V 0109 2 Maintenance of buildings and plants in electrical networks Part 2 Diagnosis

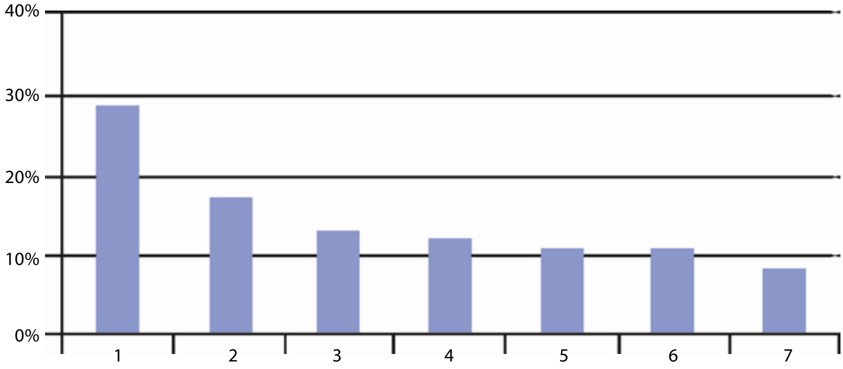


Figure 12.100 Reasons of support failures.

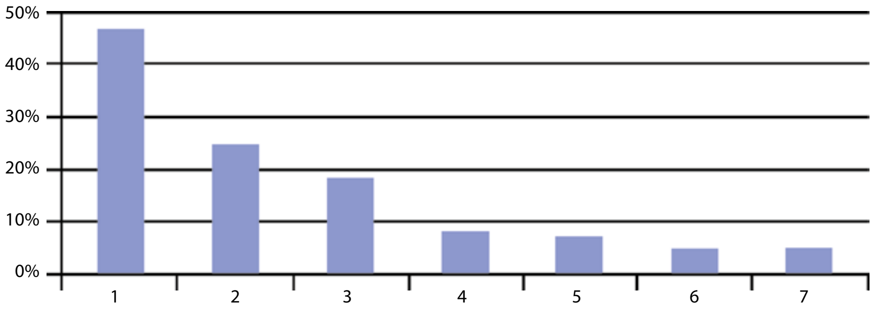


Figure 12.101 Reasons of corrosion.

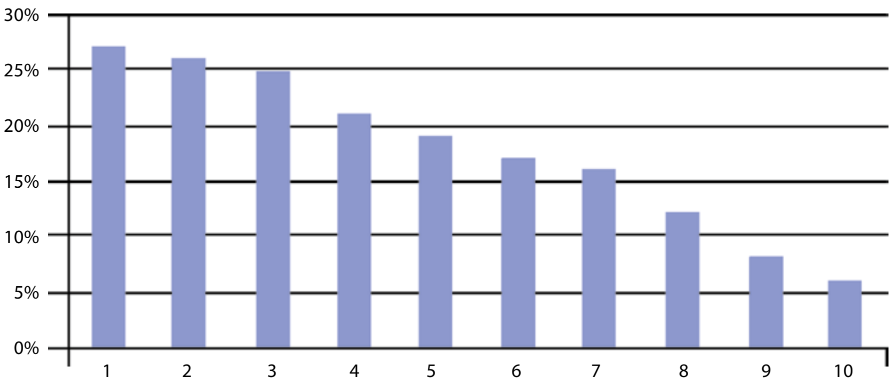


Figure 12.102 Reasons of corrosion.

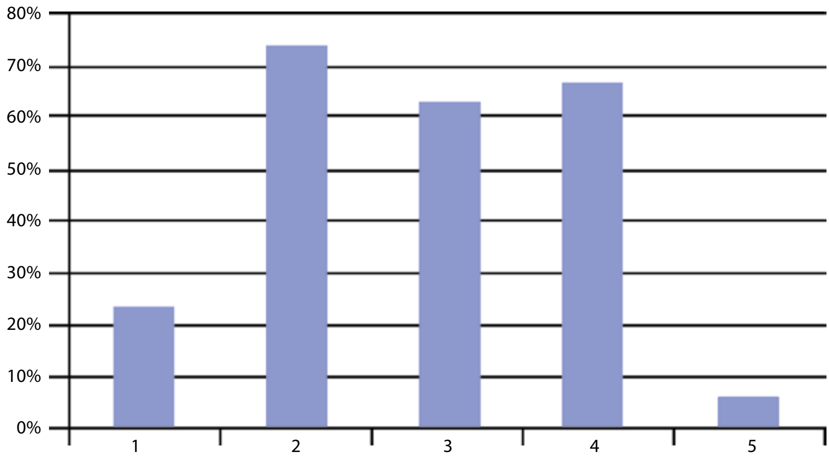


Figure 12.103 Kind of support inspections.

of conditions of buildings and plants) which deals with the diagnosis of conditions of electrical buildings and plants, in order to guarantee security of human, operation, environment and function. Inspection items for supports (steel tower, concrete and wood poles, bolts, anchors) and methods are defined (mostly visually but with measurements of mechanical strength in case of visible defects).

The mostly used tools are measuring equipments for galvanization and painting thicknesses of steel supports. The application of other tools was also questioned and responded as following (Figure 12.96):

- Galvanization thickness meter (electromagnetic gauge) for steel supports
- Paint thickness meter (electromagnetic gauge) for steel supports
- Deflection of supports (e.g., with theodolite)
- Steel corrosion metrology
- Surface carbonation (chemical) for concrete poles
- Concrete impact drive (Schmidt hammer)
- Drilled core for wood poles
- Hammer test for wood poles.

12.10.2.2 Inspection Reports

The questionnaire also aimed to document the inspection results, asking for the practices regarding checklists and records, which shall be taken during support inspections, e.g., regarding displacements, deformations and corrosion attacks.

74 % of the companies use formatted checklists for support inspections. Some companies record the inspection data in a special data base for statistical evaluation. The following gives a ranking of the importance of the information (Figures 12.97 and 12.98):

- Location of deformed/damaged members
- Number of deformed/damaged members
- Kind of deformation - local
- Kind of deformation - bending
- Kind of deformation - cracking
- Kind of deformation - buckling.

Other kinds of records were noted like presence of danger of number plates, anti-climbing devices and deformations at base level due to animals and tractors.

Many companies (75 %) differentiate the corrosion attack according to the surface extent, location and depth. The extent of corroded surfaces is assessed in a wide range. While some companies classify a corrosion attack as medium when 3-10 % of the surface is corroded, others allow up to 20 %.

As a result of the inspections, many companies (70 %) categorize the urgency of repairs, mostly in two or three categories in their reports (Figure 12.98):

- Good - no repair required
- Not good - actions required but not with urgency
- Critical - repair with urgency.

12.10.2.3 Assessment of Inspection Data

The assessment of the inspection data is a comparison of the inspection findings with the transmission line documentation. The documentation of line and support data is essential therefore. 88 % of the questioned companies have complete documentations regarding:

- Support lists
- Site maps/longitudinal profiles
- Workshop drawings of supports
- Input data for structural analysis (support geometry, load trees)
- Results of structural analysis (steel quality, profile and bolt data)
- Computerized data basis with reference to geographical information systems.

The findings during inspections are mostly reductions of support strength (locally or generally) due to damages, corrosion, deflection or theft of components. The comparison with the support documentation enables the line operator:

- To replace damaged members by using workshop drawings for re-manufacturing
- To re-calculate the support considering actual conditions (support deflections, reduced member sizes due to corrosion, altered material properties).

Good and complete support documentations also allow statements about refurbishment or upgrading. Support re-calculations can be performed easily taking into account new load trees or updated design standards. Only 37 % of the companies verify load carrying capacity of existing supports by tests.

The evaluation of inspection results and the comparison with the line and support documentation is done by experienced technical personnel in 97 % of the questioned companies. The following comments were received in addition:

- Damage or corrosion of members is usually obvious and replacement or refurbishment goes without saying
- Training to assess degree of corrosion is required
- Evaluation of the personnel's experience is done
- Assessment of inspection results is based on technical audits.

Furthermore, 55 % of the companies have defined parameters to support management decisions about repair, refurbishment or upgrading. These parameters are:

- Importance of the line
- Public and worker safety
- Weather conditions
- Existing damage classification
- Comparison with actual standards in use.

The following methods can be used if no documentation is available but assessment of inspection findings is necessary:

- Field measurements of sizes and model support for re-calculation
- Mechanical tests of support components in order to ascertain material properties.

12.10.2.4 Type and Causes of Defects

In order to focus the support inspection program on the essential items, the questionaire asked then for typical identified faults, the main cause of collapse of supports, the type and reason of corrosion as well as the most affected components, and the type of crossarm deformations or failures.

The most typical defects in supports are related to corrosion and painting problems.

Loose or missing bolts as well as deformation of support elements are other typical types of defects. For many companies, the corrosion problems occur at or below ground level, where steel is in contact with soil. Wood and concrete deterioration are very important defects for distribution lines. Reduced tensions in stay wires or deformed stays are major defects for guyed supports. The Figure 12.99 gives a ranking of typical identified faults.

- Fault of structural steel corrosion
- Fault of protection painting
- Loose or missing bolts, nuts, washers
- Foundation connection
- Concrete deterioration
- Deformation of support members

- Missing or deformed stays
- Deformation of crossarms
- Reduced tension in stay wires.

The main causes of support collapse are wind loading. It is followed by combined wind and ice loading and ice loading only. Vandalism and material defects are also reasons for support collapse. The Figure 12.100 gives a ranking of main reasons for support failures.

- Wind loading
- Wind and ice loading
- Vandalism
- Ice loading
- Cascade
- Material defect
- Erection/construction faults.

Many other reasons of support collapse are mentioned in the response to the questionnaire but with less general importance: motor vehicle collision, landslide, avalanches, tornados, foundation failures.

As corrosion is the most frequent support fault, more details were queried in this regard. The Figure 12.101 shows the main reasons of corrosion.

- Normal weathering
- Industrial pollution
- Salt (maritime) corrosion
- Gap corrosion
- Heavy vegetation growth in temperate zones
- High humidity in temperate zones
- Inter-crystalline corrosion of material.

Other typical causes of corrosion are steel in contact with soil, grillage footing below ground level and temporary accumulation of rain water. The reasons for corrosion problems are various, but the following are typical:

- No galvanization
- No painting or re-painting
- Delayed maintenance
- Weathering steel (Corten)
- Inadequate detailing.

Referring to corrosion, another question should clarify which support components are mostly affected by corrosion problems. The responses revealed that bolts, washers and nuts are often corroded. For the rest of the supports, secondary members are affected followed by main members and their connections (Figure 12.102).

- No galvanization
- Nuts of bolts
- Secondary members of lattice steel towers
- Complete supports
- Main members of lattice steel supports
- Shafts and washers of bolts
- Connections between members
- Gusset plates
- Stays
- Welding seams.

The last item of the questionnaire concerning type and causes of defects deals with crossarm deformations and failures. Deformation of crossarms is considered as a minor problem.

Rotation or torsion of crossarms around its longitudinal axis is more problematic than deformation of insulator string attachment points or local deformation or bending.

Thirteen companies (12 %) mentioned problems with crossarm hangers resulting fatigue failures due to aeolian vibration of conductors. To avoid resonance between conductor and member frequency, it is suggested to reduce the slenderness ratio of less than 300 for such members. Steel ductility problems are not reported but local vibration cracks of bolt holes due to stress concentration are established.

12.10.3 Inspection Philosophies

Inspection philosophies differ on the period of time between inspections, the qualification of inspectors, the available budget and the organization of inspection, maintenance or refurbishment.

Each of the companies responded to the questionnaire they perform regular inspections on supports. Inspections by helicopter are becoming the most common method while inspection from car or on foot is becoming less popular. Mostly there are visual inspections (Figure 12.103).

Nearly 40 % of the respondents confirmed to inspect in more detailed way by diagnostics.

However, it seems that those diagnostics are not related to supports only but to conductors, insulators and foundations too.

It shall be referred to a German VDE - Application Rule VDE-AR-N 4210–4 (VDE AR N 4210 4 Requirements for the reliability of existing supports of overhead lines) which guides a network operator in providing evidence that the technical security of his transmission lines is guaranteed. Supports are analyzed regarding their endangering of third parties, categorized in five reliability classes and the consequences of support collapses are assessed.

- Car
- Ground

- Climbing
- Helicopter
- Other.

The mean period of time between two successive inspections is nearly 1.5 years from helicopter, 1.4 years from ground and 4.2 years by climbing.

Besides the regular inspections, some companies stated to perform extra inspections, e.g., after damage event, after special meteorological circumstances.

Line inspectors have line experience in the field in 84 % of the questioned companies. The inspectors completed periodic practical or specialized trainings. Many of them have experience in tower erections and are former linesmen. Special training programs are foreseen in 47 % of the companies. There are special safety procedures, tests, and new skills developed on the job.

Half of the companies have a maintenance and refurbishment budget of more than 15 % of the overall budget expended on supports. The mean budget amounts to 28 % as per the results of the questionnaire.

12.10.4 Types and Causes of Defects/Industry Repair Practices

The most typical defects in supports are related to corrosion and painting problems (Figure 12.104).

Corrosion caused by normal weathering is more frequent than by industrial pollution. Salt corrosion and heavy vegetation growth are circumstances favouring the corrosion process. The reasons for corrosion problems are various and none of them is preponderant except no galvanizing and no-repainting. Low or delayed maintenance are also recorded as possible causes.

Here, typical refurbishment measures are applied: Removing pollution, rust and old paint by hand cleaning or sand blasting, repainting, replacement of single support members and bolt connections.

Wind loading remains, by far, the most important causes of collapse of supports. It is followed by combined wind and ice loading and ice loading only. Such higher

Figure 12.104 Corroded members.

Figure 12.105 Replacement of main members.

loads (e.g., due to climate changes) require an upgrading of the supports. The need of upgrading can also be caused by increasing of conductor sizes for reaching higher transmission capacities.

Upgrading studies and corresponding works, may require replacements on the main members (Figure 12.105), or just reinforcement by adding an additional angle profile (Figure 12.106).

12.11 Highlights

This chapter has reported different aspects regarding OHL Supports, from the conceptual design drawings to the complete design, calculations, testing and fabrication process.

Emphasis was given to the great variety of solutions, available for the transmission line designers, to face the increasing challenges for the installation of new projects. It was shown that towers, even looking similar, are not all equal and, certainly, there is one single solution that fits better for each new specific line design.

The new design philosophy based on probabilistic assumptions was emphasized as the best proposal for reaching both required targets for new projects, reliability and economy. New advanced softwares and techniques for structural analysis were described, which enables tower designers to improve their accuracy and predictability of results.

Figure 12.106 Main member reinforcement by bolting additional angle profile.

The aging process of the existing supports was also addressed showing its critical mechanisms (e.g., corrosion) and utilities' great concern. In this context, the inspection tools and methods, the diagnostic and assessment procedures, and experiences and solutions adopted for repairs in the industry were shown.

12.12 Future of Overhead Line Supports

The future indicates that the consumption of electricity in the world will keep on growing, giving the market the need for even more transmission line projects, but also indicates great challenges regarding to increasingly demanding societies in terms of environmental friendly solutions.

The first solution for that challenge can be expected as ultra high voltage "super transmition grids", in both AC and DC technologies, can be already foreseen. Those super long lines will carry much higher quantities of energy, making the servitude strip more efficient, but also requiring great quantities of supports which designs will face new levels of challenges in terms of reliability and cost, as well as environmental requirements and public acceptance.

Regarding to downtown areas of big cities, it is expected to keep on growing vertically, needing more and more power to keep it running. In this important areas, underground cables and landscape towers overhead lines may grow in application, competing each other in terms of cost benefit ratio and public acceptance.

The last predictable overhead line project trend has to do with new materials being developed (such as carbon fiber, polymers, super conductors, composite materials etc). Soon, new material technologies will be available at the market influencing the design of conductors and supports, enabling reduction in tower heights and weights and facilitating the fabrication, erection and maintenance procedures.

Finally, thinking about the operation and maintenance point of view, it is expected the population of existing supports spread around the world to increase dramatically: the aging process of these structures will be a matter of great concern by the utility companies. As a result, more efficient diagnostics, inspection tools and methods will be a great demand on a near future.

Those utility companies will, thus, face the challenge of conserving their older and older tower population in a market of great cost concerns.

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