**Zeitschrift:** IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

**Band:** 13 (1988)

**Artikel:** Design of transmission towers: challenge for the practicing engineer

**Autor:** Ville de Goyet, Vincent de / Maquoi, René

**DOI:** https://doi.org/10.5169/seals-13086

# Design of Transmission Towers: Challenge for the Practicing Engineer

Calcul des pylônes de lignes électriques: défi à l'ingénieur

Bemessung von Hochspannungsmasten: Herausforderung für den praktischen Ingenieur

**Vincent de VILLE de GOYET**
Senior Res. Assist.
University of Liège
Liège, Belgium

Born in 1956, he received his civil engineering degree at the University of Liège in 1979. He devotes his scientific activity to the development of finite element programmes and non-linear analysis.

**René MAQUOI**
Professor
University of Liège
Liège, Belgium

Born in 1942, he got his civil engineering degree and Ph.D. degree at the University of Liège in 1965 and 1973 respectively. He is mainly involved in stability problems and steel construction.

**SUMMARY**
Transmission towers are among the most complex civil engineering structures. Though a feasible preliminary design can be based on rather simple computer calculations, the final check often requires more sophisticated computer programmes. The paper is aimed at stressing the fundamentals of both approaches and at presenting a comparison between the corresponding results.

**RÉSUMÉ**
Les pylônes de lignes électriques comptent parmi les structures les plus complexes du génie civil. Un dimensionnement préliminaire peut être conduit à partir de calculs relativement élémentaires. Une vérification finale à l'aide de programmes de calcul plus élaborés reste néanmoins nécessaire, ainsi que le montre l'analyse comparative des résultats selon les deux approches mentionnées.

**ZUSAMMENFASSUNG**
Hochspannungsmasten sind sehr komplexe Ingenieur-Tragwerke. Obwohl eine sinnvolle Vorbemessung mit einfachen Computer-Berechnungen möglich ist, erfordert die endgültige Bemessung oft wesentlich verfeinerte Rechenprogramme. Dieser Beitrag beschreibt die wesentlichen Punkte der beiden Methoden und vergleicht deren Resultate.

## 1. INTRODUCTION

Building transmission towers made with steel angles is a common and widely used practice. Such structures are nevertheless amongst the more complex ones in the field of civil engineering, for reasons which are commented briefly here below.

Though a simple section at first glance, the angle is normally subject to combined axial force, bending moment, shear and torsion, as a result on the one hand, of the non-coincident centroïd and shear center and, on the other hand, of the usual practice to make end connections on one angle leg only. Transmission towers are truss-type structures and are thus likely to produce kinematic mechanisms ; with a view to avoid these latter, an adequate geometry is necessary. Secondary bars, called redundants, are often used to reduce the buckling length of compressed chords and legs, with the result of zero axial force in these redundants, as far as it is referred to a first order analysis. A proper design must however account for unavoïdable geometrical imperfections (especially lack of bar straightness) and erection misalignments ; the redundants have thus to be designed to resist the associated destabilising forces. It is also of common practice, though unsafe, to assume pin-ended bars and to simplify the design by partitioning the tower and analysing it as an assemblage of non interactive plane trusses ; consequently load eccentricities and spatial behaviour are mostly neglected.

A lot of transmission towers collapsed in the past and lessons must be drawn from these accidents. The loading conditions should be revised with the result of a more accurate assessment of wind loads, differential bearing settlements, loads resulting from conductors failure,... Also it is presently attempted to improve the design procedures and allow the designer for more elaborated approaches. In this respect, it is while mentioning the publication, by ECCS, of Design Recommendations [1] and, more recently, of computer programmes and comments to which the junior author contributed much [2] ; these programmes enable the interactive preliminary design of several parts of a transmission tower.

Because the transmission tower is probably the sole huge reproducable structure in the field of civil engineering, a special care should be brought to its design with a view to optimization and to cost and material savings. Therefore the authors are of the opinion that refined calculations should be made finally, on base of the aforementioned preliminary design, by using non linear finite element computer programmes.

## 2. A STEP TOWARDS SPECIFIC DESIGN RULES

In 1985, ECCS produced recommendations for the use of angles in lattice transmission towers [1]. These recommendations were derived from a first draft issued in the Introductory Report to the Liège Colloquium of 1977 [4]. They are mainly aimed at giving rather simple calculation rules when designing the main structural components of transmission towers made with angles. The design buckling stresses are usually higher than those adopted for other steel structures, because they were drawn from a lot of test results of structures loaded up to failure, which make the transmission towers the best known steel structures.

The basic buckling curve adopted for angles in transmission towers is valid for equal and unequal leg rolled or cold-formed angles, which buckle in the plane of minimum stiffness without local failure due to plate instability. The European column buckling curve $a_0$ is adopted as this basic curve. The influence of local and torsional buckling on the overall buckling strength of rolled and cold-formed angles is accounted for in a simple and practical way by using a reduced yield stress; the latter is deduced from the yield stress with regard to the manufacturing process and the b/t ratio of the walls.

The buckling length and the radius of gyration to be used of legs or chords depend on the type of bracing, either symmetrical or staggered in two normal planes. Provided the slenderness ratios be computed accordingly, the engineer is allowed to disregard the secondary unfavourable effects due to the eccentricity of the loads from the web members.

The end connections of web-members in compression usually produce eccentricities and restraints (when 2 or more bolts), both of which affect the carrying capacity. It is assume that both effects cancel each other at a non-dimensional slenderness close to $\Lambda = \sqrt{2}$. For lower slenderness ratios, there is a resulting detrimental influence on the carrying capacity ; a modified increased slenderness ratio $\bar{\Lambda}$ (> $\Lambda$) is defined according to the type of eccentricity (at one or both ends) and the buckling axis (minimum axis or rectangular ones). For higher slenderness ratios, the beneficial effect of the end restraint exceeds the detrimental one due to connection eccentricity. A modified reduced slenderness ratio $\bar{\Lambda}$ (< $\Lambda$) is recommended, which depends on the type of member (continuous or not) and of connection (one or more bolts). When end restraints and member discontinuity cannot be clearly defined, simplified rules are suggested, where the slenderness ratio $\bar{\Lambda}$ depends on the buckling axis only.

Some general guidelines are also given for what regards the lengths to be used as buckling length of web member types and bracings.

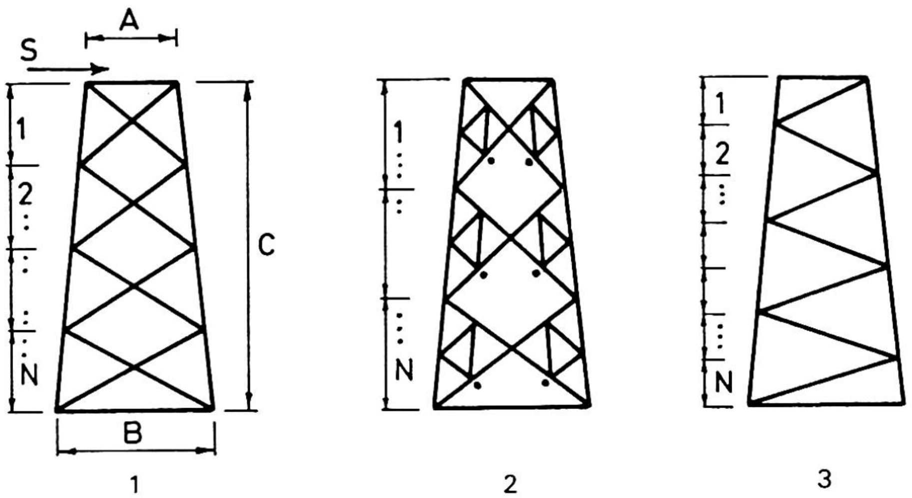
The ECCS Recommendations can thus be used only when the forces in the several bars are known. It can also be observed that each structural element is designed with nearly no account for the others. As the ECCS design rules are derived from failure test results, they could be expected to be safe. The systematic use of the ECCS Recommendations in the frame of a preliminary design and the assessment of their actual safety are two matters which are discussed in the following sections.

## 3. AN ATTEMPT TO USER'S GUIDANCE AND OPTIMIZATION

ECCS recently issued [2] a suite of programmes which were intended to assist the designer in arriving at the final stage of design by quickly looking at various alternatives and assessing the influence of changing some of the design parameters. The package consists of : i) a manual and a floppy disc containing four design programmes suitable for use on an IBM P.C. and ii) a file of angle member properties. The design rules are in accordance with the ECCS Recommendations [1] and the BSI Code of Practice [3].

As latticed transmission towers are generally statically indeterminate, advantage is taken of symmetry conditions to separate the four faces of the tower body and treat each face individually as a plane statically determinate problem. The loads applied to the tower generate a horizontal shear force S applicable to the panel under consideration, wherefrom the member forces of the bracing can be derived.

The programme REDCROSS is aimed to design these bracing members ; it is carried out interactively and allows for an optimum solution, that is characterized by the least weight at the most uniform buckling security within the individual bars. The input comprises four sections : i) the general geometry of the panel: widths at top and bottom and height ; ii) the applied residual shear force S at the top of the panel ; iii) the yield strength of the bracing members, and iv) the type of bracing. Four types of bracing are possible (fig. 2). The programme is not intended at all to replace a full analysis of the tower; it is just aimed at guiding the designer in coming to an acceptable preliminary design, that can be fully checked later. The user can takes advantage of the dialogue with the computer to reach a quasi-optimal solution for a specified type of bracing and eventually choose another type of bracing.




Note:Type 2 is with or without stayed across lower corners

Figure 1 - Types of bracing.

The second programme, called REDLOAD, facilitates the design of K-panels in latticed transmission steel towers, and more especially of redundants and secondary bars. Several patterns of redundants are possible. From the basic geometry information, the member of subpanels over the height of the frame, the leg load and a code number that denotes the minimum angle size it is desired to use, the programme gives automatically design loads and length of bars and then proceeds to design the sections, specify the bolts and gives reserve factors for both bars and bolts. It finally gives the weight of redundants in the panel.

Programmes REDBRACE and REDHIP are intended at designing automatically K-brace panels ; the components of the design are : i) the leg, ii) the main horizontal and iii) the main diagonal members.

## 4. A WAY TOWARDS THE ACTUAL BEHAVIOUR.

The actual behaviour of transmission towers cannot obviously be reflected by the previous design rules, but only either by a full scale test of the structure or by a numerical simulation on computer. The latter approach is quite suitable for parametric studies and is in addition not destructive. In this respect the junior author contributed much to the nonlinear finite element programme FINELG [5], that is able to compute the critical loads and the associated buckling modes as well as the whole behaviour up to and even beyond the ultimate load, account taken of geometrical and material non linearities. He developped more especially a very efficient spatial beam finite element [6] , the main pecularities of which are : account of warping, open or closed shape of cross-section made of thin walls, asymmetric cross section, eccentricities and semi-rigidity of end connections, residual stresses, elastoplastic material constitutive law, large displacements,... The reader who is interested in more details concerning this F.E. is begged to refer to [6].

With such a marvellous tool in hands, it was attractive to simulate the actual behaviour of a transmission tower that was tested in Great-Britain [7]. With a view to limit the duration of the computations, only a part of structure was studied; this part is composed of the experimentally collapse region and the adjacent space panels. The computations were carried out with account taken of initial bar out-of-straightness, rolling residual stresses, plasticity,dead load and eccentricities of connections. It is while mentioning that the F.E. used allows for a progression of yielding across the section and over the length of this element, so that a refined discretization is not at all necessary. An excellent agreement between the numerical and experimental results was found. On

the one hand, the collapse modes are identical and correspond to instability (fig. 2) with excess of plastic deformations in two same compression web members. On the other hand, the computed collapse load exceeds the experimental one by less than 6 %, what can be regarded as very successful account taken of the complexity of the problem. Another simulation was conducted by simply changing the orientation of the web members ; that means that the non-connected leg of the angles is oriented inwards instead of outwards previously, for compression members, and vice-versa, for tensile members. This change results in an increase of the computed collapse load, which now exceeds the experimental one by more than 13%. A non negligible gain of ultimate load can thus be obtained by connecting the compression members in such a way that their free leg be oriented inwards the tower. The bar out-of-straightness is another parameter which was also investigated, as well as the effect of the remaining part of the tower.
For the above computation, the initial geometrical imperfection was choosen similar to the first instability mode. The reference amplitude was equal to 1/1000 of the length of the bar, which was found the weakest by a bifurcation analysis; its sign was associated with the displacement of the same bar obtained by a first order computation. A change of that sign for all the bars yields a slight difference (< 1 %) in ultimate load and is thus not significative. Allowance was made for axial rigidity of the chords which connect the studied part to the foundation by introducing elastic spring supports (instead of fully restrained ones as considered in the previous computations); here too, only a slight drop (1,5 %) of the ultimate load is observed.

A step further [8] was to compare the results of some design rules (REDCROSS) for the chords, legs and bracing members with those drawn from the study, by means of FINELG, of the weakest plane panel extracted from the spatial tower part examined above. In other words, it was aimed to investigate whether the collapse locations coincide when either analysing successively the different bars of the structures or considering a plane panel. From this comparison, following conclusions are while being emphasised :

- The identification of the weakest panel, according to calculation of the axial forces in the bars based on the assumption of pin-ended bars, may be wrong. As a consequence, bracing members designed accordingly could be undersized.
- A more realistic value of the ultimate load would require that the spatial behaviour of the structure - i.e. the restraint provided by adjacent panels - be accounted for in the design. However it is still 20 % higher than the one found by a complete spatial analysis.
- It is compulsory to take account of connection eccentricities, otherwise the ultimate load can be overestimated by more than 80 %.

## 5. CONCLUSIONS

Though valuable design rules for the members and bracings of transmission towers are presently available, they must be reserved to preliminary design. It appears indeed that in spite of the fact that they were deduced from experimental evidence, they are not able to account for all the peculiar parameters which are likely to influence the ultimate load of such structures. A refined nonlinear analysis is the sole able to warrant for a specified safety. The old adage "You only get for what you pay for" has here an obvious relevance.

## REFERENCES

1. E.C.C.S., Recommendations for Angles in Lattice Transmission Towers. Publ. n° 39, 1985.
2. E.C.C.S., Computer Aided Design Programmes for Latticed Towers, 1987.

3. Draft for Development Code of Practice for Strength Assessment of Members of Lattice Towers and Masts, British Standard Institution,DD133,London, 1986.

4. E.C.C.S., 2d Int. Coll. on Stability of Steel Structures. Introd. Rpt., Liège, 1976.

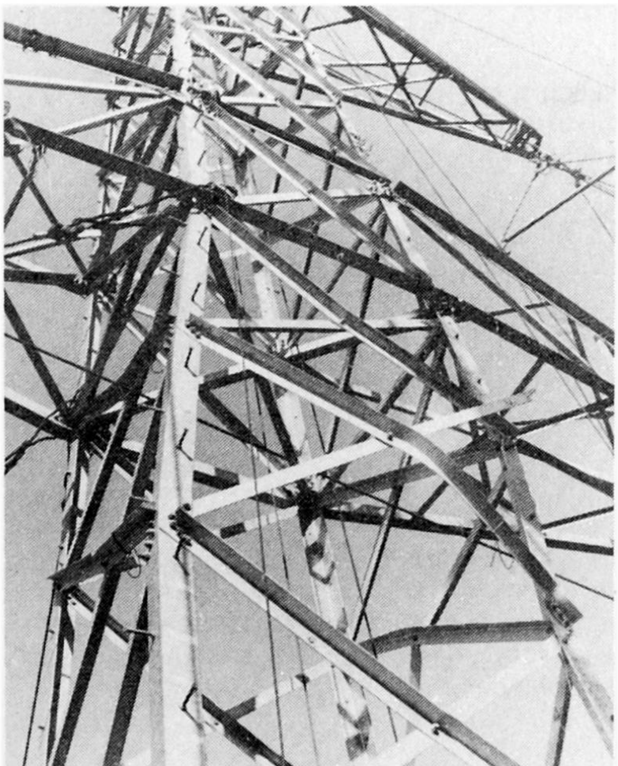
5. FREY, F. et al, FINELG nonlinear finite element analysis program. Rapport interne, M.S.M., Université de Liège, IREM, Ecole Polytechnique Fédérale de Lausanne, N° 86/6, Jul. 1986.

6. de VILLE de GOYET, V., L'analyse statique non linéaire par la méthode des éléments finis des structures spatiales composées de barres à section non symétrique . Ph.D. Thesis, Université de Liège (to be published).

7. C.E.G.B., A test report on a 400 kv double circuit tower type L12 DE6. Rpt. N° 183, Réf. T710/169, England, 1979.

8. THUNUS, R., Etudes paramétriques de certaines sous-structures de pylones à haute tension. Terminal work, University of Liège, Jul. 1987.

9. de VILLE de GOYET, V., Etude du comportement à la ruine de structures spatiales composées de poutres à sections non symétriques. Construction Métallique N° 4, 1987.



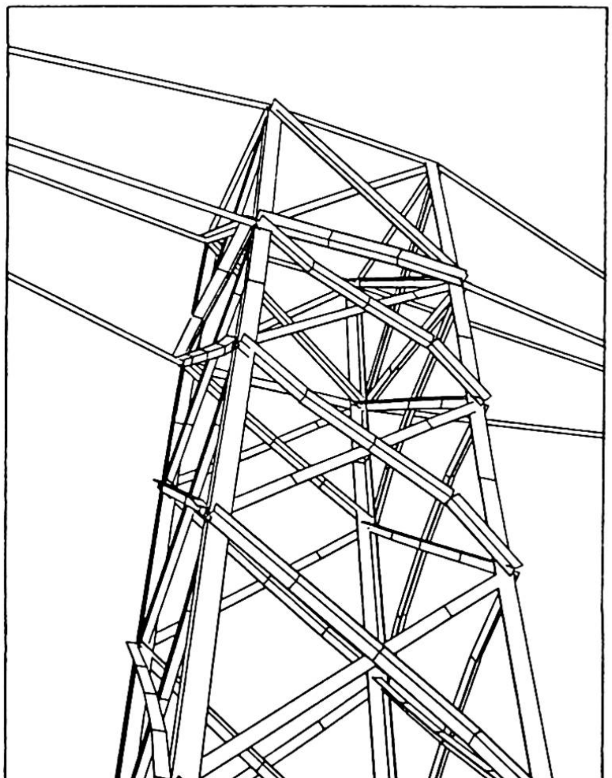


Figure 2