STEEL POLES WITH POLYGONAL SECTIONS IN BENDING

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INTRODUCTION

Solid-wall steel poles are used as pylons to carry high voltage overhead power lines with heights up to 60 m as well as to support overhead wiring above railway tracks in much smaller versions. Due to their slender appearance, such poles offer an alternative design to lattice structures which are often less compact and therefore more space consuming. Steel poles are mainly subjected to bending which, in combination with the small wall thickness, may lead to stability problems on the compressed side near the clamped base or at changes of wall thickness. Consequently, the ultimate limit state of buckling is often the most critical consideration in choosing the necessary wall thickness which strongly influences the cost-effectiveness of these structures.
This paper investigates the current design procedures for steel poles with polygonal and cylindrical sections in bending and explores the present lack of consistency between the design rules for steel poles with polygonal sections containing a high number of corners, which are almost circular from a geometric point of view, to those design rules for steel poles with circular cross-sections. Also presented are the results of a large-scale bending test programme with different types of sections which form the basis for the development of a new numerical design concept for steel poles in bending which can be applied to structures with polygonal sections as well as to structures with circular sections. With this numerical design concept the lack of consistency between the different design rules can be overcome.

1 PLATE BUCKLING AND SHELL BUCKLING DESIGN

1.1 Steel pole design approaches

Polygonal sections are usually treated as a collection of individual vertical plate strips for the purpose of structural design. Consequently, the bending moment capacity of a pole with polygonal cross section is determined by the bearing load of the individual plate strips according to the EN 1993-1-5 [1] regulations for plate buckling. The bearing load of the compressed strips is thus determined according to the effective width concept and the sum of the contributing parts of the section in bending is expressed as an effective section modulus W_{eff} (*Fig. 1*).

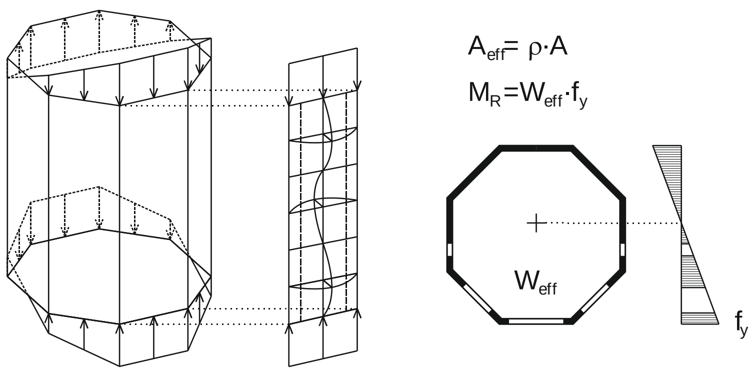

Fig. 1: left: idealisation of a polygonal section in bending as a stringing together of individual plate strips, right: bending moment capacity on the basis of the effective width concept acc. to [1]

This plate strip concept is the basis for the current European design rules for steel poles with polygonal sections in EN 50341-1 [2]. The key assumptions of this design concept are that the

corners of the polygonal section act as supports for the plate strips, remain straight and do not change position during loading. Furthermore the geometry of the corners is assumed to be perfect which implies that their initial geometric quality has no influence on the bending moment capacity. It is possible that the plate buckling design concept can be a satisfying approximation for some polygonal sections with a small number of corners in which the bending angles of the corners are large enough to provide the necessary bending stiffness to act as a rigid support for the plate strips. In these cases the collapse of the structure due to a bending load is characterised by a local buckling failure between two adjacent corners (*Fig. 2* left).
A contrasting interpretation suggests that the plate strip concept may not be valid for polygonal sections with an arbitrary high number of corners and corresponding small bending angles, as illustrated by the buckling failure under bending of a polygonal section with 24 corners where the buckles developed across several corners and flat sides (*Fig. 2* right). It may be seen that there is no local buckling between the corners but rather a global failure of the shell structure which is very similar to the failure mode of a cylindrical section with the same slenderness and thus may be described as a shell buckling failure. In this case a buckling assessment that considers each plate strip individually is no longer valid because it cannot describe the corresponding failure mode.

Fig. 2: left: polygonal section with 12 corners, local buckles between two adjacent corners; right: polygonal section with 24 corners, buckles throughout several corners

1.2 State of the art
Research on thin-walled tubular steel components with polygonal sections has often focused on the number of corners where the transition from a local buckling to a global buckling failure occurs. *Bulson* [3] found by experimental testing of very slender polygonal sections in compression that the scatter in the results strongly increased for sections with more than 18 corners which he interpreted as the transition to a shell bearing behaviour. *Migita* and *Fukumoto* [4] made numerical investigations with the same intention on simplified subsystems of polygonal sections consisting of two adjacent panels in compression and found that the transition from local to global buckling occurs at 22 corners. These varying conclusions suggest that there is a certain dependence of these limits on the methods of investigation.
Additionally, only very few studies [5, 6] have investigated components with polygonal sections in bending. The reason for this is that the failure of components with polygonal sections has usually been regarded as local and therefore the results from investigations in compression were assumed to be appropriate for the reason of design in bending too. A documentation of full-scale bending tests on polygonal steel poles can be found in *Cannon* and *LeMaster* [6]. Their results led to partially experimentally based design rules for polygonal poles in compression and bending which are included in the current American standard for the design of steel poles [7]. The various investigations led to several different design equations for polygonal sections with different numbers of corners which can be found in a comparative study by *Legeron* and *Godat* [8].

1.3 Design according to EN 50341-1
Current design codes for polygonal steel poles in bending limit the number of corners to 16 [7] and 18 [2] respectively, above which only the design rules for cylindrical sections can be applied. However, due to the existence of the previously described transition from plate buckling to shell buckling failure types, the design rules for steel poles with polygonal sections with a high number

of corners and the design rules for cylindrical sections cannot be transferred to one another. Indeed their application to geometrically very similar sections such as a polygonal section with 18 corners (N18) and a cylindrical section can lead to considerably different bending moment capacities, as illustrated in *Fig. 3* using the design rules of EN 50341-1 [2].

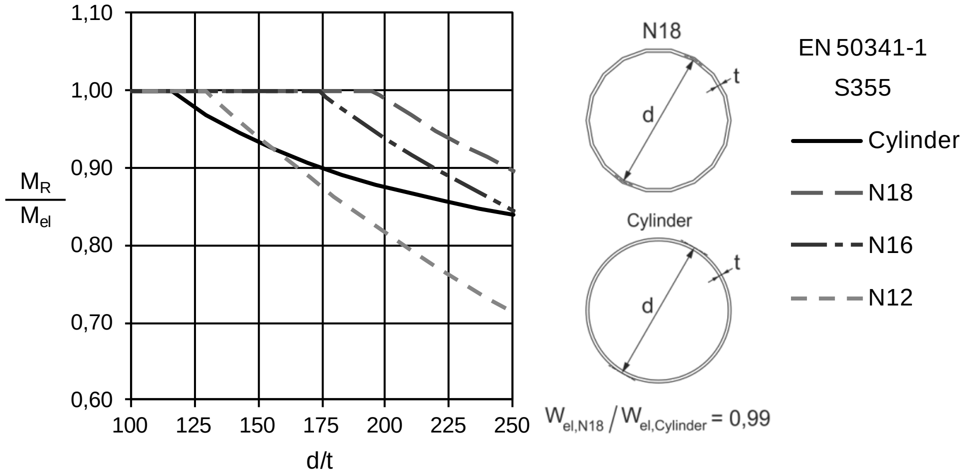

Fig. 3: normalised bending moment capacity M_R/M_{el} for polygonal and cylindrical sections acc. to EN 50341-1 [2]

The above diagram illustrates the bending moment capacity for polygonal sections with 12, 16 and 18 corners together with the bending moment capacity of cylindrical sections, expressed as normalized values M_R/M_{el} varying with the section slenderness d/t. While the capacity curves for the polygonal sections are based on the previously illustrated plate buckling plate strip concept (*Fig. 1*) the development of the design curve for the cylindrical section is based on experiments with tubes in bending [7, 9] which therefore treats the section as a cylindrical shell.

A direct comparison between N18 and the cylindrical section, both of which exhibit the same elastic section modulus, reveals that there can be a considerably different bending moment capacity due to the different design approaches. This is especially valid in the slenderness range of d/t = 150 to 225. As a result of these distinctly different design approaches, the capacity curves for N18 and the cylindrical section exhibit a different behavior in the presented slenderness range and varying slendernesses at which the full elastic bending moment capacity is reached (M_R/M_{el} = 1,0). Both design approaches do not consider partial plastification of the less slender sections before collapse in bending which means that the normalized bending moment capacity is limited to the first yield moment ($M_R/M_{el} \leq 1{,}0$).

A further important difference between the two design approaches concerns the assumption of tolerances related to the fabrication quality which may also be a reason for the different developments of the capacity curves. Due to their origin the capacity curves for the polygonal sections consider geometric imperfections of the individual plate strips only while the geometric quality of the shell structure such as out-of-roundness or the straightness of the corners is not taken into account. In contrast to this the capacity curve for the cylindrical sections considers geometric imperfections of the shell due to the experimental origin of the design rule even though they are not explicitly defined in [7].

2 EXPERIMENTAL INVESTIGATIONS

2.1 Test setup and specimens

A series of three-point bending tests with full-scale specimens was performed with the aim to identify the decisive influences on the bearing behavior of steel poles with different numbers of corners. *Fig. 4* shows the test setup and the dimensions of the tested sections with 6, 12 and 24 corners as well as a cylindrical section. The wall thickness t of the specimens was consistently chosen with t_{nom} = 3 mm leading to a slenderness of the sections of approximately $d_{max}/t \approx 280$ which represents the upper range of slenderness for the application as overhead power line poles. The high slenderness ratio d/t was chosen to emphasize the influence of geometric imperfections on the bearing behaviour. The specimens had a massive vertical load introduction plate located at midspan with two further plates at the ends to restrict ovalisation. The central plate divided the specimens into two halves, each of which represented a horizontal pole with a linearly distributed bending load and a clamped support at the centre of the test setup. The specimens were found to collapse due to buckling on the compressed side close to the central load introduction plate (*Fig. 2*).

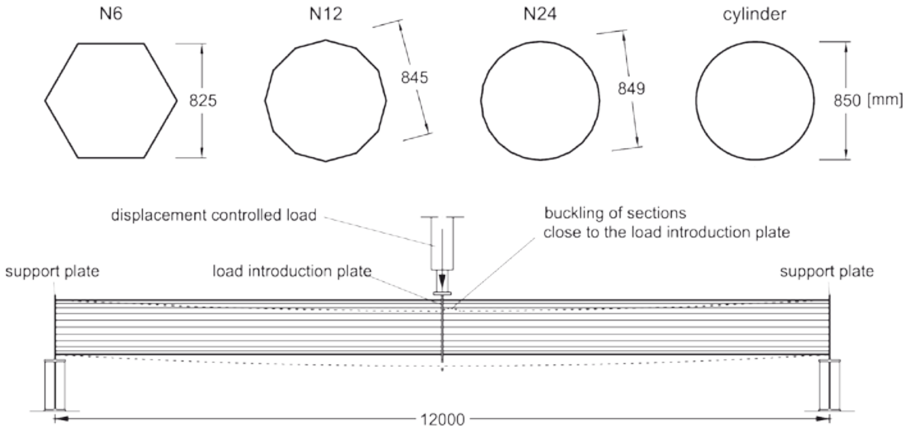

Fig. 4: test setup and section types

2.2 Test results

The results of the tests are presented in *Fig. 5* in the form of normalised bending moment capacities M_{test}/M_{el} as a function of the number of corners. The bending moment M_{test} was calculated from the magnitude of the applied midspan load using beam theory. The tests with polygonal sections were performed with two different orientations: bending with a corner in the compression zone (E) and bending with a flat side in the compression zone (P).

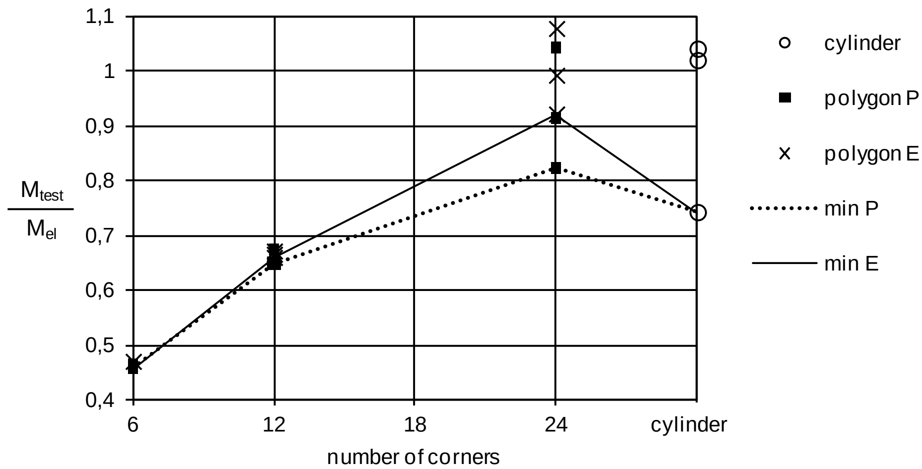

Fig. 5: normalised bending moment capacity M_{test}/M_{el} vs. number of corners

The dimensions of each of the specimens were chosen to maintain the same elastic section modulus $W_{el,}$ thus allowing the influence of the section type and number of corners on the effective bending moment capacity to be determined. The bending tests suggest that the bending moment capacity can be considerably increased by the introduction of additional corners (*Fig. 5*). Indeed, the highest bending moment capacities were achieved for polygonal sections with 24 corners and cylindrical sections, though these were also found to exhibit the largest scatter in the results. The cause of the large differences in the bending moment capacities for these sections may be attributed to the presence of initial imperfections which are known to considerably influence the bearing behaviour of thin shell structures, and which therefore must be carefully considered in design.

3 NUMERICAL INVESTIGATIONS

3.1 Substitute imperfection approach

The geometric imperfections of a structure are usually not known in advance and can be very different in type and appearance. Consequently, their influence on the bearing behaviour has to be estimated in FE-analyses by suitably chosen substitute imperfections which must be appropriately but not prohibitively deleterious to the buckling strength while remaining a realistic reflection of the manufacturing or handling process. It is therefore of interest to find the most damaging realistic imperfection rather than simply the most damaging possible imperfection [10]. For this purpose a number of the specimens were measured by means of a laser scanning device prior to testing. This laser survey provided a high resolution scan of the initial surface of the specimens which allowed the critical geometric imperfections to be identified and used for the development of a substitute imperfection suitable for numerical investigations.

Fig. 6: substitute 'impact' imperfection form

A set of investigative FE calculations using the measured surfaces revealed that the largest decrease in bending moment capacity may be achieved by assuming a geometric imperfection in the form of a deep local indentation, possibly caused by an impact (*Fig. 6*). This substitute imperfection may be thought of as being created by a small but wide object being pushed into the structure thus causing a permanent indentation. The simulation of this impact is performed as an initial numerical step prior to the actual calculation of the ultimate bending load, which has the advantage that it can be applied to sections with arbitrary numbers of corners thereby providing a comparable initial damage to all types of sections. This substitute imperfection has been found to be both more detrimental and realistic than more classical approaches such as the assumption of an eigenmode imperfection, particularly at higher imperfection amplitudes.

3.2 Comparative numerical calculations

A set of FE calculations were performed using the assumed 'impact' imperfection form applied to polygonal sections with a varying number of corners (*Fig. 7*) and cylindrical sections. The load was applied on pre-damaged sections as a constant bending moment. The amplitudes Δw of the pre-generated indentations were chosen in accordance with EN 1993-1-6 fabrication tolerance quality class B (FTQC B) [11]. The results indicate a gain in bending moment capacity due to the number of corners which can be very different at different slenderness ratios d/t. However, in contrast to the codified strengths (*Fig. 3*), the bending moment capacity of polygonal sections with a higher number of corners (e.g. N16 or N18) and cylindrical sections are very similar, as would be expected due to the geometrical similarity. A design approach based on numerical analysis that considers a uniform level of initial damage for all types of sections thus provides a considerably more accurate description of the bearing behaviour compared with an approach based on classical simplifying assumptions.

The FE study shows that polygonal sections with 16 corners or above behave similarly to cylindrical sections, and indeed a bending moment capacity similar to that of cylindrical sections may already be achieved for 12 corners for small slenderness ratios ($d/t < 150$). A comparison with codified results (*Fig. 3*) shows that the application of the current design concepts according to [2] can overestimate the bending moment capacity of sections with more than 16 corners.

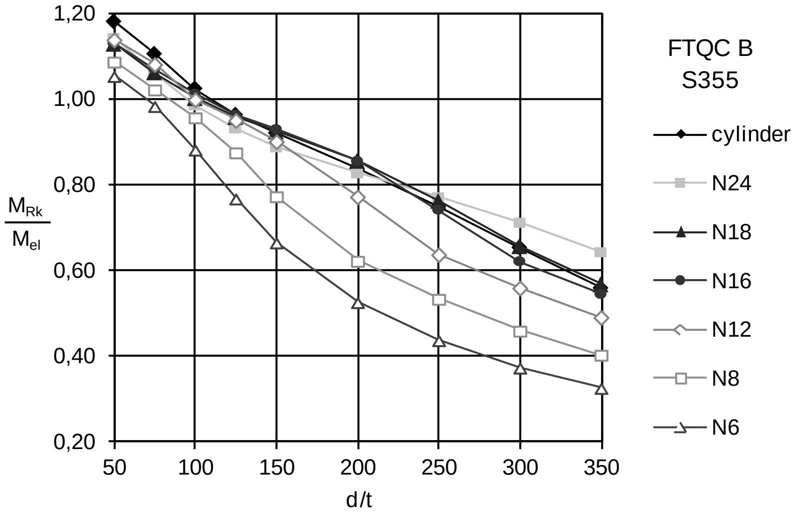

Fig. 7: computed normalised bending moment capacity M_R/M_{el} vs. slenderness d/t for steel poles with polygonal and cylindrical sections

4 SUMMARY

This paper illustrates the inconsistencies in the design rules relating to the transition from steel poles with polygonal sections with a high number of corners to cylindrical sections subjected to bending. An alternative numerical design concept with a substitute imperfection approach based on experimentally gained data is used to investigate the bending moment capacity of steel poles with sections with arbitrary numbers of corners. The presented experimental and numerical investigations show that the current design regulations for steel poles with polygonal sections in bending do not fully address all possible influences such as geometric imperfections and shell-like buckling behaviour.

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