Wind induced stresses in the spire of Brussels Town Hall Tower

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Contraintes dues au vent dans la flèche de la Tour de l'Hôtel de Ville de Bruxelles

Windinduzierte Spannungen im Turmhelm des Brüsseler Rathauses

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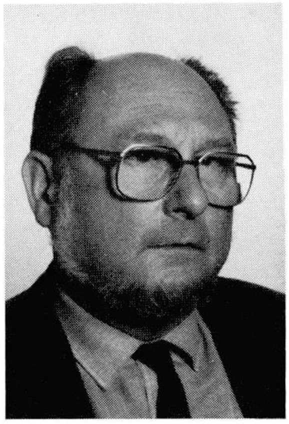


SUMMARY

The tower of the Brussels Town Hall (1454) is a 96m high gothic structure presently undergoing a major restoration. The results of a finite element analysis of its spire are presented. Small tensile stresses appear under wind loading at the top of the spire and also in its skeletal part, where the ribs are subjected to bending. This preliminary study, which assumes a linear elastic behaviour, allows the assessment of restoration options. For instance, it shows that the corroded metallic reinforcement has no structural function.

RÉSUMÉ

La tour de l'hôtel de ville de Bruxelles (1454) est une construction gothique haute de 96m qui est à présent l'objet d'une restauration majeure. Les résultats d'une étude par éléments finis de la flèche sont présentés. Des contraintes de traction faibles apparaissent sous l'effet du vent au sommet de la flèche et dans sa partie ajourée où les côtes sont soumises à flexion. Cette étude, quoiqu'élastique, permet l'évaluation d'options de restauration. Par exemple, elle montre que les renforcements métalliques, corrodés, n'ont pas de fonction structurelle.

ZUSAMMENFASSUNG

Der 96m hohe, gotische Turm des Brüsseler Rathauses (1454) wird gegenwärtig gründlich restauriert. Es wird über die Ergebnisse einer Finiten-Elemente-Analyse des Turmhelms berichtet. Schwache Zugspannungen treten bei Wind an der Helmspitze und im Masswerk auf, wo die Rippen einer gewissen Biegung ausgesetzt sind. Aufgrund dieser vorläufigen Analyse, die nur ein linear-elastischen Verhalten annimmt, können Restaurationsmöglichkeiten bewertet werden. Sie zeigt beispielsweise, dass die korrodierten Metallverstärkungen für das Tragwerk ohne Bedeutung sind.

1. INTRODUCTION

The main tower of the Brussels town hall (Fig.1) is a masterpiece of Gothic architecture dating back to the middle of the 15th Century (1449-1454). The tower suffers now from severe disorders which may be attributed to sequels of the bombing by the armies of Louis XIV and to unwise previous restorations with unsuitable materials and reinforcements like iron rods that rust and induce fracturing of the stones. A major restoration is therefore needed and is presently taking place.

The most damaged part of the tower is its spire. The spire looks like a pyramid with octagonal cross section. The height of the spire is 20m. Its lower, or skeletal part (11m), consists of 8 slender inclined columns at the 8 edges of the pyramidal volume; the architects called these inclined columns ribs ("côtes"). The upper part (9m) is a solid (or assumed so) pyramidal shaped volume supporting a 3.1m height statue of archangel St Michael culminating at 96m above ground level.

The ribs are connected together at their mid height by a fine stone tracery ("remplage"); the initial function of this tracery was to provide an intermediate support for the ribs during their erection process. Since the ribs exhibit a slight inward curvature, it has sometimes been suggested that the stiffness of the tracery was too weak to provide an effective support of the ribs. The ribs are furthermore connected at five different levels by two hoops of metallic reinforcement. The inner series of hoop reinforcement may be original, but the outer series is probably a late addition.

Preliminary studies have shown that wind is the main action inducing bending and therefore possibly tensile stresses in such a high building. These tensile stresses, if they appear, would be localized in the spire, which justifies that we focus our detailed analysis on this part. From the structural point of view, the following questions have been raised :
(1) what is the actual structural safety against failure under exceptional wind action ?
(2) what is the role of the metallic reinforcement ? Do we have to replace all the non original iron rods by stainless ones or only some of them ?
(3) what is the structural effectiveness of the tracery in providing support for the ribs and wind bracing for the spire? It has been suggested, for instance, that a rigid connection between the ribs should be added under the form of a solid stone disk behind the tracery.

Mortar and lead joints cannot sustain tensile stresses and the existence of tensile zones implies nonlinear material behaviour. The precise answer to the previous questions requires therefore sophisticated numerical modeling. But the first step before proceeding with aerodynamic tunnel experiments and modeling the nonlinear material behaviour is to apply code provisions for wind effects with a linear elastic structural model. This paper presents and comments the results of this analysis.

Fig. 1 The tower of the Brussels town hall (96m)

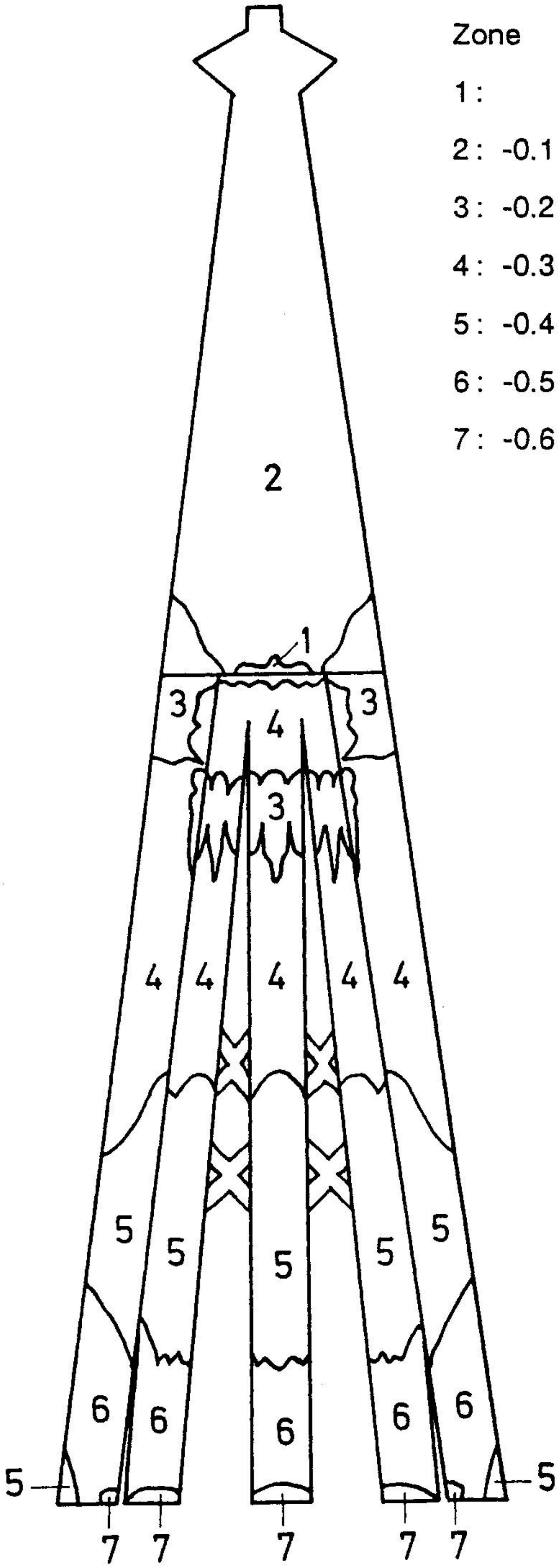
2. NUMERICAL MODEL

Thanks to symmetry, only one half of the spire needs to be considered. We built a very large three dimensional finite element model of the half spire : 4000 elements and 63000 degrees of freedom. All the geometrical dimensions are known from a recent survey. The mesh of the stone structure consists entirely of quadratic isoparametric volumic finite elements. Each rib is modeled by 37 layers of 10 elements. The area of the cross section of one rib is 3300 cm^2. We assumed for the stone a Poisson's ratio equal to 0.15 and Young's modulus equal to 50 GPa, which is the mean value for the "Gobertange" stone used during the 19th Century restoration and predominantly visible today. But we tested the sensitivity of the results to the value of the Young's modulus for the stone by introducing in one of the analyses a lower value (20GPa) corresponding to the original sandstone, which may still be present to a certain extent inside the structure. The total computed weight (with a specific weight equal to 24 kN/m^3) of the half spire is 569 kN. The cross section of the reinforcing rods varies between 9 and 25 cm^2, depending on their location in the spire. Four different structural models were considered : (1) present state, with hoop reinforcement; (2) without hoop reinforcement; (3) without hoop reinforcement, but with addition of a solid stone disk behind the tracery; (4) without hoop reinforcement and without tracery.

The wind forces and pressures were computed with the Belgian code on wind actions NBN B03-002-1 (1988), which is in agreement with the latest international recommendations in the field. At the mean level of the spire, the characteristic value of the peak wind speed is 154 km/h for a period of return of 10 years and 196 km/h for a period of return of 500 years. The corresponding dynamic pressures are 1,12 kN/m^2 and 1,84 kN/m^2 respectively. In fact, we subdivided the spire in five levels, and assumed for each zone a uniform value for the dynamic pressure, equal to its value at the mean height of the zone. The wind pressure acting on the surface of the model is then computed as the product of the dynamic pressure by a pressure coefficient c_p. The value of the pressure coefficients in each zone depends on the form of the cross section of the spire at the mean level of the zone, and of the orientation of the element of surface under consideration with respect to the wind direction. The value of the external pressure coefficients varies between -0,5 (suction) and 1 (pressure); the order of magnitude of the internal pressure coefficients (for the surface of the ribs inside the hollow part of the spire) is 0,1. Finally, wind forces are applied to all finite element sides belonging to the surface of the spire; the computed value of the wind resultant on the half spire is 62 kN for a period of return of 500 years. The wind force acting on the statue (9.9kN) contributes significantly to the wind action on the spire.

3. NUMERICAL RESULTS

The slenderness of the spire allows us to concentrate ourselves on the vertical component of the stress tensor referred to as "stress" below. Fig. 2 is a schematic section in a plane of symmetry representing the distribution of vertical stresses under self weight in the model without hoop reinforcement. There are



Zone	Stress value				
1 :	0.0	<	f	<	0.1 MPa
2 :	-0.1 MPa	<	f	<	0.0 MPa
3 :	-0.2 MPa	<	f	<	-0.1 MPa
4 :	-0.3 MPa	<	f	<	-0.2 MPa
5 :	-0.4 MPa	<	f	<	-0.3 MPa
6 :	-0.5 MPa	<	f	<	-0.4 MPa
7 :	-0.6 MPa	<	f	<	-0.5 MPa

Fig. 2 Vertical stress distribution in the spire under self weight

practically no differences in this distribution when hoop reinforcement is added. Fig.2 exhibits a very limited zone of small tensile stresses (< 0.1 MPa) along the axis of symmetry at the connection between the ribs and the solid part of the spire, and compressive stresses elsewhere. The ribs are mainly subjected to normal force. The slope of the ribs with respect to the axis of gravity is small (8.5°), but nevertheless sufficient to induce a slight bending behaviour of the ribs which may be seen as columns built in at both ends and supported at mid-height by the tracery. If the tracery is omitted, the span of the ribs doubles and the bending behaviour is fairly more noticeable with tensile stresses (≈ 0.2 MPa) appearing on the outer arrises at both ends of the ribs. The introduction of a solid stone disk behind the tracery has no influence on the stress distribution above the tracery and induces a slightly less favorable bending behaviour of the ribs between their springing and the tracery. We may therefore conclude that the tracery provides an effective support and that the introduction of a stiffer support behind the tracery seems neither necessary nor desirable.

Fig.3 represents the distribution of vertical stresses under combined action of self weight and a wind with a period of return of 500 years. The section shown lies in the direction of the wind and the windward side is on the left. Once again, we hardly notice any difference when the hoop reinforcement is added: some rods are in tension, other in compression, and the largest computed force in one rod is 1.8 kN when E = 50 GPa for the stone. The stress distribution in the masonry remains practically unchanged when E = 20 GPa for the stone, while the mean force increase in the rods is 33%. The upper, solid part of the spire behaves like a tapered short column and is typically submitted to bending combined with compressive normal force. Nearly all the windward side exhibits tensile behaviour, with values around 1 MPa in a 2m height below the statue and 0.1 MPa at the base of the solid part of the spire. These values are roughly halved for a wind with a period of return of 10 years. In the skeletal part of the spire, the windward rib exhibits a definite bending behaviour with relatively large tensile zones and a peak tensile stress of ≈ 0.6 MPa at the springing of the windward arris. The largest compressive stress (≈ -1.2 MPa) is observed at the springing of the outer arris of the leeward rib and remains a very low working stress for this kind of masonry.

4. DISCUSSION OF NUMERICAL RESULTS

The fact that nearly all the windward side of the upper part of the spire is submitted to tensile stresses is not astonishing and may be shown by application of elementary concepts of strength of materials. Heyman [1] furthermore recalls that the top of a solid slender spire is always subject to instability under wind. Heyman compares the overturning (wind) and stabilizing (self weight) moments computed with respect to the leeward arris. If we add the stabilizing effect of the weight of the statue (≈ 4kN) and the overturning effect of the action of the wind on the statue to the application of Heyman's analysis of equilibrium, we find that the upper half of the solid part of the spire is subject to overturn by wind. It has never been reported for this spire, and we must therefore infer that the anchorage of the statue (whose state is unknown and which has not been accounted for in the finite element analysis) contributes to the bending resistance of the upper

Zone	Stress value
1:	$f > 0.2$ MPa
2:	0.0 MPa $< f < 0.2$ MPa
3:	-0.4 MPa $< f < 0.0$ MPa
4:	-0.8 MPa $< f < -0.4$ MPa
5:	$< f < -0.8$ MPa

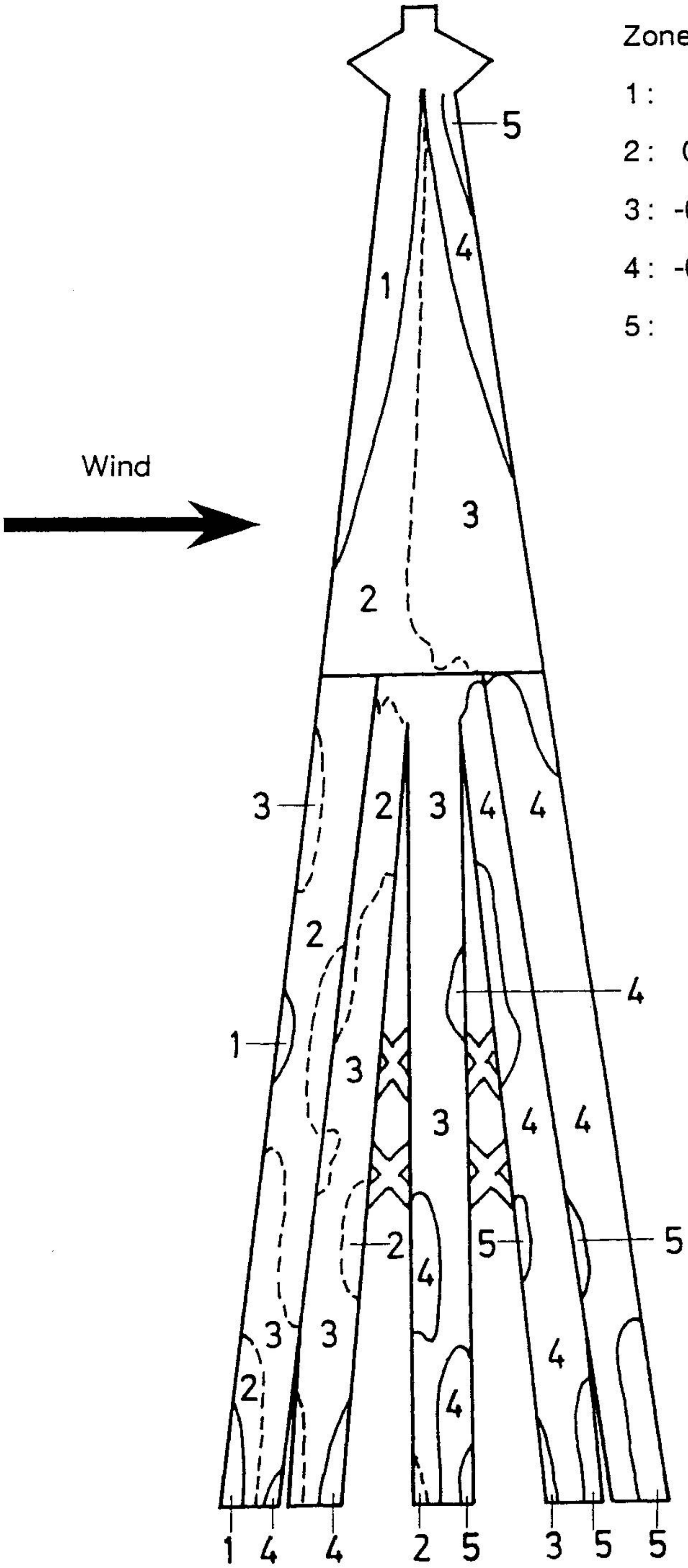


Fig. 3 Vertical stress distribution in the spire under self weight and wind loading

part of the spire. This favorable element could easily be estimated during the restoration and the theoretical existence of tensile stresses in the solid part of the spire is not worrying. More concerning is the magnitude of the tensile stresses at the springing of a windward rib. We cannot conclude from the present elastic analysis that such stresses are high enough to jeopardize the stability of a rib, or even the durability of joints which would open under strong wind and remain close under self weight and moderate wind. The answer to this question justifies further studies -more precise evaluation of wind action in an aerodynamic tunnel and modeling the nonlinear behaviour of a unilateral material like masonry- which are presently under progress.

5. CONCLUSIONS

The present study is therefore only one step in the process of determination of the restoration options, but it helped us to understand finely the structural behaviour of the spire and to reach some practical results:
1) the structural role of the metallic reinforcement, which could not have been tackled by simple analysis based on equilibrium concepts, appears very limited and not essential to stability. We may now propose to simply remove them or, if the archaeologists want to keep them, to treat the rods against further corrosion and disconnect them structurally from the stone. Their expensive replacement by stainless reinforcement is not structurally justified.
2) the tracery provides enough stiffness to sustain the ribs and there is no need for additional bracing at this level.
3) the anchorage of the statue should be unveiled during the restoration and carefully assessed because it probably plays an essential role in the stability of the upper part of the solid part of the spire.
4) further studies are necessary to determine whether the springing of the ribs needs strengthening to be able to withstand the tensile stresses occurring, if any, under the most severe wind loading.

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REFERENCES

[1] HEYMAN, J., "Hemingbrough Spire", Proc. of the 2nd Int. Conf. Structural Repair and Maintenance of Historical Buildings, ed. by C.A. BREBBIA et al., Vol. 1, pp.13-22, Computational Mechanics Publications, 1991.