EFFECTIVE BRACING OF TRUSSED TOWERS AGAINST SECONDARY MOMENTS

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ABSTRACT

A forensic study into the failure of six transmission towers revealed high values of secondary moments in some regions which were not accounted for in the design. These bending moments caused overstressing which led to the cascading failure of the towers. The study showed that improper bracing configuration of the main and/or secondary braces induced high secondary moments at these locations. High moments are also induced at the locations where the changes, however small, occur in the main leg alignment. It is possible to rearrange bracing scheme to modify the load path so that the secondary moments are effectively reduced. The study recommends consideration of secondary moments in both the analysis and design procedures.

INTRODUCTION

It is a common practice to model transmission towers as two-dimensional truss systems in the transverse and longitudinal directions. In three-dimensional modeling kinematic instability is encountered at the joints where the members lie in one plane. This problem is circumvented by introducing spring supports at such joints placed normal to the plane [1] or by modeling the main legs as continuous frame members [2]. When the analysis results in significant forces in the spring supports or moments in the main leg frame members, whichever is employed in the modeling, the suitability of bracing configuration should be investigated in order to effectively reduce these forces. It is advisable to proportion the main leg frame members as beam-column elements, and fabricate and splice them accordingly.

THE CASE STUDY

The authors were commissioned to investigate the failure of six 132 kV tangent transmission towers in Al-Qassim region of Saudi Arabia. The first and the last towers failed by bending of the top cross-arms, the three of the intermediate ones at their bases and the sixth above its first body. The towers had height of 45.45 m (Fig. 1) and were spaced over a span of 400 m. They had square planar configuration and were assembled from single angle continuous main legs and cross bracing. The bracing in adjacent faces was staggered.

A meteorological report on the day of failure recorded at a nearby station did not show any sign of high wind nor the people living in the surrounding regions reported unusual wind. However, the designer in his post failure assessment attributed the failure to un-foreseeable wind gust above the design speed of 170 km/hr (106 mph) [3]. The authors visited the site after the removal and restoration of the said towers. They examined the debris for possible clues. Laboratory test of the specimens from the material employed showed tensile strength in excess of the prescribed value of 345 MPa (50 ksi). The specimens also showed adequate ductility.

The governing design specifications [4] prescribed a design wind velocity of 170 km/hr (106 mph) and a gust speed of 220 km/hr (138 mph) in the region. The specified value is a good estimate of 50-year basic design wind speed recommended by an independent study [5]. The specifications require that a tower be analyzed for fourteen load cases covering full-loading, single circuit usage, broken conductors and under construction situations. They refer to ASCE Manual No. 52 [2] for design regulations.

ANALYSIS OF TOWERS

The towers were analyzed as three-dimensional structures with the main legs as frame members. This assumption is the closest to the actual detailing of the main legs. The applied loads when evaluated in accordance with the prescribed conditions were found to be adequate. The analysis was implemented on SAP 90 finite element package [6]. Presumably the designer employed two-dimensional truss model for his analysis.

The axial forces in most of the members, from the two analyses, were in agreement. However, the deviations between the two analyses, in some of the members, is attributable to the difference in the methods of modeling and analysis [2].

The most noteworthy results of the analysis are the bending moments in the main legs which are significantly high compared to the meager bending resistance of the legs. These bending moments are neglected in the original design calculations. It is also noteworthy to mention that the maximum drift of 570 mm (22.5 in.) was encountered under the severe most loading condition.

EFFECTIVE BRACING OF TOWERS

Some of the generalized observations on the bending moments are that they are induced only when the main legs are modeled as frame members, that they do not buildup over the tower height, are local in nature and occur where reversal of moments takes place, and that they are small in value but significantly large compared to the bending capacity of the leg members. It was noticed that bracing configuration significantly affected these moments. Three major configuration faults were identified and are discussed below.

Single-End Bracing Joint

Secondary bracing members are usually assumed to have zero force in the pre-buckled state. This assumption is not true when the main members are modelled as frame members. At a single-end bracing joint with the main leg, the axial force in the brace is equilibrated only by the shear resistance of the leg which in turn induces significant value of bending moment in the main leg at the joint. Fig 2a shows a two-dimensional part model of the tower to illustrate this point. This situation can be avoided by either eliminating single-end bracing joints wherever possible or by adding a member to redirect the force away from the main leg. Fig 2b shows such a solution which reduces the maximum moments of 115, 151, 148 and 168 kN-mm at four joints to 21, 27, 7.8 and 8.3 kN-mm respectively.

Improper Main Bracing

Main bracing elements are employed to transmit the horizontal loads as direct forces and avoid moment build-up on the main legs. Improper arrangement of these braces is liable to generate the moments in the main legs. One such example is the use of K-bracing arrangement in the tower body at the attachment of the cross-arms. The use of cross-bracing reduces the moment in the main legs about 65 to 75%. Therefore, careful attention must be paid during the selection of bracing system.

Lack of Leg Alignment

The analysis reveals that the secondary moments are sensitive to lack of leg alignment which may be caused by fabrication errors specially at the lap-joints and at the locations where intentional change of slope is introduced in the leg alignment. A 10 mm offset from the leg alignment at a lap joint in the middle part of a tower resulted in 40 kN-mm moment on the leg which is 240% of the moment at this joint in perfect alignment. This effect is produced by the normal to leg component of the axial force in the out of alignment part which is resisted by shear in the main leg. This problem can be resolved by providing an alternate path to the normal forces with the help of adequate bracing at splice joints, as a measure of precaution, and at joints where the main leg is realigned.

CONCLUSIONS

The analysis and design of transmission towers should consider both the axial force and bending moment in the main legs. These moments although dubbed as secondary and neglected in common design practice, are significantly high at locations to lead to unexpected failures. The bracing system and its arrangement play an important role in control of these moments. These moments can not be eliminated completely but can be managed to values affordable by the main leg section. It is shown that single-end bracing joint, improper bracing and lack of main leg alignment contribute to the bending moments. Provision of an alternate path, to channel the normal forces resulting from alignment offset at these locations, can lead to effective control of the secondary moments.

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