Failure analysis of lattice tower like structures

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Abstract. The Experimental investigations have revealed significant mismatches between analytical estimates and experimentally measured deflections of tower structures. Conductor and groundwire tension and sag are influenced by the peak and cross-arm deflections and hence accurate estimates of the tower deflections at service loads are of key interest. This paper presents a nonlinear formulation for analysis of tower structures in an attempt to close the mismatch. The analysis includes geometric nonlinear behaviour of the tower, material nonlinearity as well as leg member buckling in compression. The nonlinear analysis reveals that the ultimate collapse loads of the towers tested to failure can be predicted. However the, deflections continue to have a mismatch despite nonlinear analysis. This demonstrates the need for inclusion of other parameters to be able to reduce the error of analytical deflection estimates.

1. Introduction

Lattice tower like structures are commonly encountered in the form of communication tower masts and transmission line towers. The property of tall heights compared with their base widths, while having limited bending action due to the lattice or truss form, make these structures unique in behaviour. The various loading conditions on towers cause specific deflection shapes and failure locations which are further dependent on properties of both members and connections. Towers are designed traditionally for reliability and security requirements by ensuring the strength limit states for individual members and connections. The designs however cannot ensure the safety against ultimate collapse of the tower as a whole. The ultimate collapse of towers are mandatorily studied through full-scale tests by clients as specified in the testing codes (IEC 60652: 2002 and IS 802:1995 – part 3). Active research on the ultimate collapse and deflection performance of transmission line towers has aimed to address several mismatches existing between test observations, analytical procedures and design provisions.

Initial research on the behaviour of transmission line towers (Al-Bermani and Kitipornchai, 1992) employed a nonlinear formulation that considered the members as thin-walled beam column elements to capture the geometric nonlinear effects. Satisfactory predictions of collapse loads were not matched by satisfactory deflection predictions, which the authors attributed due to the joints. A similar procedure was proposed to replicate and reduce the need for full-scale tests (Al-Bermani et al, 2009). The effect of hip bracing patterns on the failure of transmission towers was studied analytically with additional suggestions for appropriate bracings to adjust slenderness of leg members in different types of towers (Rao et al, 2010). A common discrepancy highlighted in the above studies is the inability to

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match deflection predictions with values measured from the full-scale tests. The primary cause is stated to be the occurrence of bolt slippage in the splices provided for joints of leg members. Two bolt slippage models were suggested – sudden and continuous slip (Kitipornchai et al, 1994). A detailed thesis on the analysis of tower structures with consideration of bolt slippage was presented by Kroeker (2000). Component level studies of bolt slippage in joints are also present in literature for further reference (Ungkurapinan et al, 2003). Modified analytically estimated deflections have also been compared with full-scale test measurements of deflections (Rao et al, 2012). Provisions for experimentally obtaining the bolt slip portion of overall deflections are present in the testing codal standards.

2. Tower modelling and design

In this paper, as truss elements were adopted for the tower models, the secondary bracings were omitted and checked for 1.5-2% of the axial force in the main leg members. This was to avoid joint instability which occurs at the joints of secondary bracing members (Fig. 1) due to no out-of plane restraint with respect to the plane of the secondary bracing. The leg members and main bracings should comply with the permissible stresses and clauses present in the design codes (IS 802:Part 1 – Section 2; ASCE Manual 52 – Guide for design of steel transmission towers; IEC 60826, 2003). Criteria for deflections of towers are not specified in either of the IEC codes or the Indian Standards. It is reported that erstwhile USSR codes contained provisions to limit tower deflections. This clause has subsequently been made obsolete given that deflections estimated by structural analysis do not match actual full-scale test deflections. Thus it would be erroneous for a designer to set deflection limits in reference to such a clause. This sharply highlights the necessity for improvements in analysis procedures of transmission line or lattice towers in general. This study aims to attempt to close this mismatch in deflections at the service load levels using nonlinear analysis of towers with inclusion of member yielding and buckling.

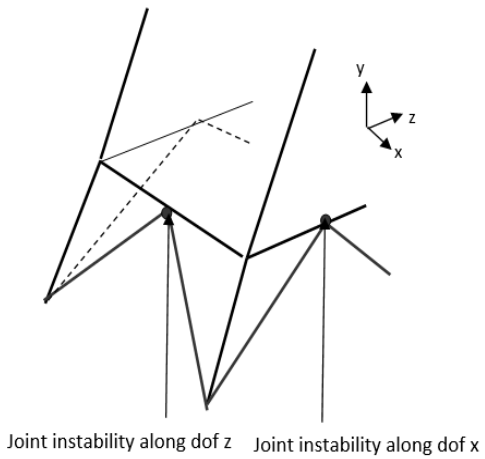


Figure 1. Joint instability at secondary bracing joints

3. Nonlinear analysis of lattice towers

In this study a formulation based on the corotated (CR) approach (Mattiasson, 1983) was employed within the updated Lagrangian form of nonlinear finite element equations. This formulation was used owing to the fact that, for truss elements in a CR approach, the local tangent stiffness matrix is 1×1

instead of 6×6. This is a consequence of the separation of strain causing deformations from rigid body motions of the total deformation in the CR approach. After applying transformations from local to global coordinate system, the global tangent stiffness matrix is given by

$$\boldsymbol{K_G = A^T\left(E^T K' E + R_{x2} B\right) A \delta p} \tag{1}$$

where $\boldsymbol{A^T}$, $\boldsymbol{B}$ and $\boldsymbol{E}$ are transformations matrices described in reference (Mattiasson, 1983), $\boldsymbol{R_{x2}}$ is the element force at the single degree of freedom after applying corotation and $\boldsymbol{\delta p}$ is the change in global displacement vector. The local tangent stiffness $\boldsymbol{K'}$ (in the corotated coordinate system) is obtained using nonlinear finite element equations in the updated Lagrangian framework. But the displacement function in this equation contains just one displacement due to the application of corotation (instead of displacements at two nodes in traditional displacement function of truss elements).

The local tangent stiffness, $\boldsymbol{K'}$ can be written as

$$\boldsymbol{K'} = \frac{A}{L}[\boldsymbol{C} + \boldsymbol{\sigma}] \tag{2}$$

where$\boldsymbol{K'}$ is the element local tangent stiffness and is a single value matrix due to the use of the corotated approach and $\boldsymbol{\sigma}$ is the element stress vector.

The above equations handle geometric nonlinear analysis. Inclusion of material nonlinearity and member buckling require modifications to the stiffness given by Eq (2). Member buckling is considered by including an initial imperfection as a small fraction of the member length (0.1%) to estimate the reduced stiffness of buckled members. This is achieved through the arc length method (Crisfield, 2000) which traces the load-deflection of members undergoing buckling. The tangent stiffness expression then includes the form lateral displacement component due to buckling (Jayachandran et al, 2004) as shown below

$$\boldsymbol{K'} = \frac{1}{\left[\left(\frac{1}{\frac{A}{L}(\boldsymbol{C}+\boldsymbol{\sigma})}\right) + \frac{F_\Delta L}{\boldsymbol{R_{x2}}}\right]} \tag{3}$$

where, with δ_c as the maximum lateral deflection

$$F_\Delta = \left[1 - \frac{1}{1 + \frac{2}{3}\left(\frac{\delta_c}{L}\right)^2}\right] \tag{4}$$

Material nonlinearity is taken as the inelastic behaviour of the members and a mixed hardening model is considered, though for static analysis a flat yielding curve would be sufficient. The state determination procedure (Bhatti, 2000) and von-Mises yield criterion for mixed hardening modify the stress component $\boldsymbol{\sigma}$ in Eq (2) to account for material nonlinearity.

$$\begin{aligned} \delta\boldsymbol{\sigma_y} &= MH\delta\mathbf{E}'^{\boldsymbol{p}} \\ \delta\boldsymbol{\alpha} &= (1-M)H\delta\mathbf{E}'^{\boldsymbol{p}} \\ f &= (\boldsymbol{\sigma} - \boldsymbol{\alpha}) - \boldsymbol{\sigma_y} \end{aligned} \tag{5}$$

In the above equations, $\delta\boldsymbol{\sigma_y}$and $\delta\boldsymbol{\alpha}$ denote the change in subsequent yield stress due to hardening and the stress shift vector respectively. $M$is the mixed hardening parameter (Axelsson and Samuelsson, 1979), H is the hardening slope and $\delta\mathbf{E}'^{\boldsymbol{p}}$ is the increment of plastic strain which is evaluated within the formulation. This combined geometric and material nonlinear analysis procedure is incremental-iterative. The results obtained using the formulation with elastic analysis and no member buckling is compared with results from ABAQUS to verify the formulation. However,

modelling member buckling in commercial FE software is tedious and this is an advantage of the formulation.

4. Results and discussion

Three examples of towers from literature are included in this study. The first example is the double circuit tower shown in Fig 2 (units in kN,mm). The tower was modelled and analysed in ABAQUS (Fig 2) considering only elasticity and this result was used to validate the formulation in the present study for nonlinear analysis of tower structures. The estimates of deflections at the groundwire (GW) and bottom panel (BP) joints on the leeward side and the comparisons of deflections with ABAQUS results are shown in Fig 3(a) and Fig 3(b). Also, the stiffness shown by the current study is 0.93 times that of commercial FE software.

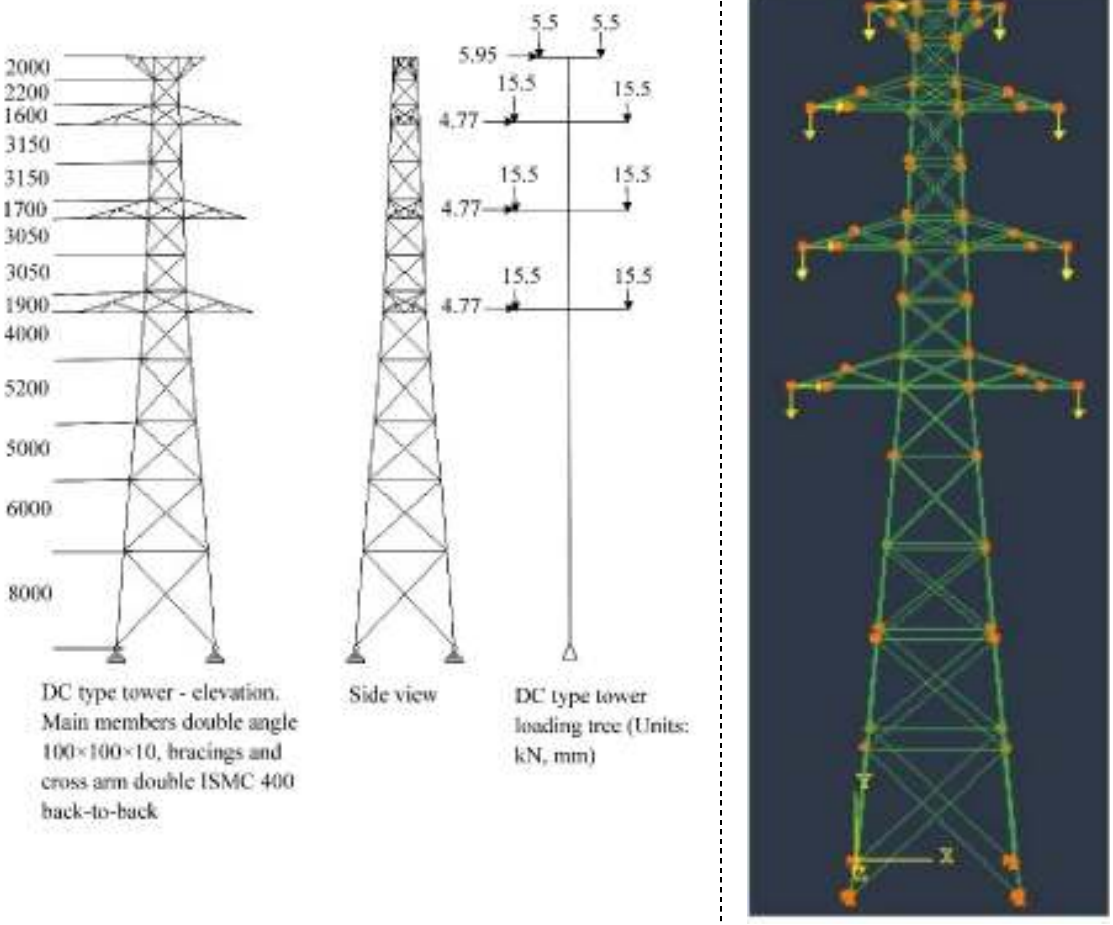


Figure 2. Double circuit tower

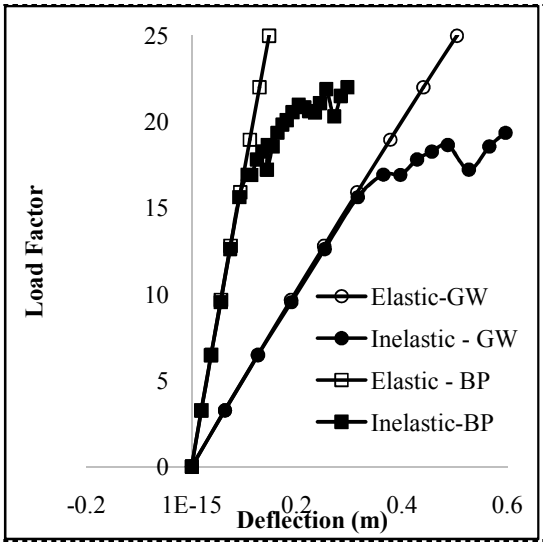


Figure 3. (a) Load-deflection comparisons of double circuit tower at groundwire and bottom panel

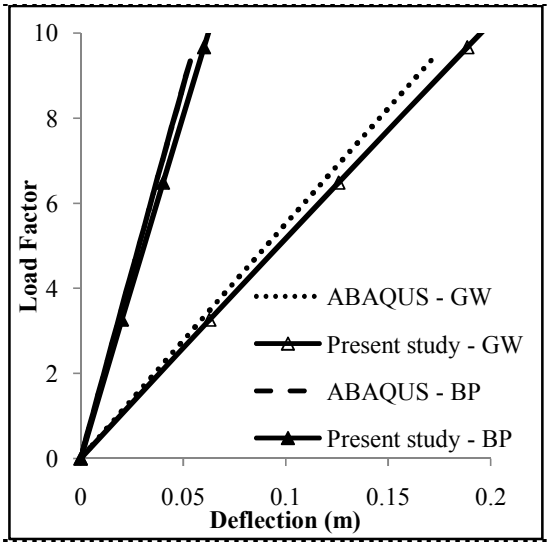


Figure 3. (b) Comparison of present study with ABAQUS results

The second example is the communication mast (Tanaka et al, 1985) in Fig 4(a) studied under vertical loads on the top nodes and an imperfection load of 0.1% of any one vertical load along the transverse direction. The axial (lb) and flexural rigidities (lb-in^2) for the members are: 7.08×10^6 and 2.16×10^6 for longerons, 2.7×10^5 and 6.43×10^4 for diagonals and battens, 1.65×10^8 and 2.2×10^8 for short longerons, 1.37×10^8 and 1.52×10^8 for plan diagonals. Fig 4(b) shows the load-deflection comparison with consideration of both member inelasticity and buckling. The softening part of the curve of elastic analysis in the study is due to the imperfection load which makes the overall tower resemble a slender column and this is not reflected in the reference curve. Member buckling is governs the ultimate collapse of the tower.

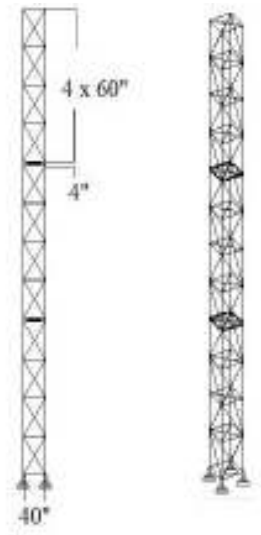


Figure 4. (a) Communication mast

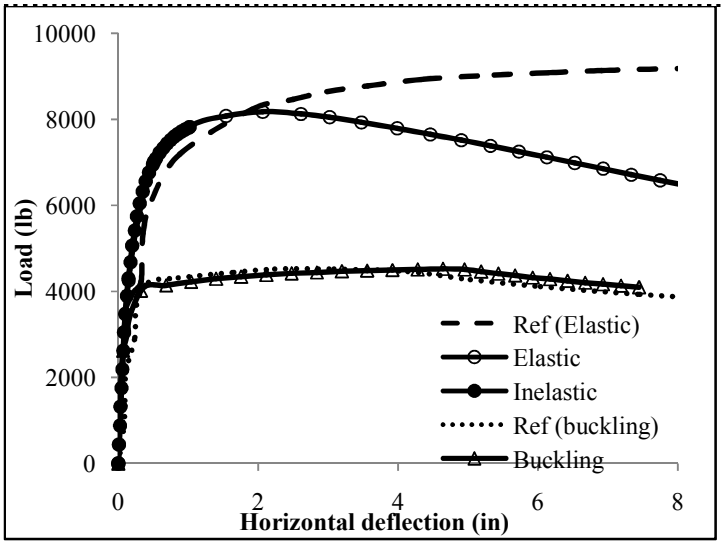
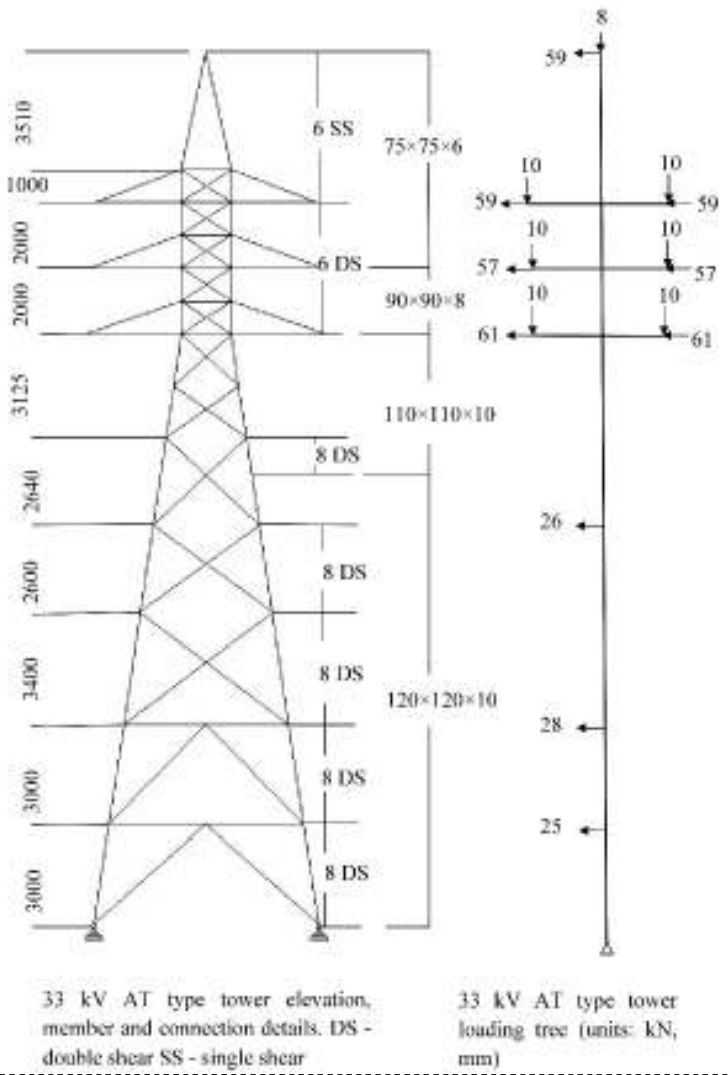


Figure 4. (b) load-deflection of communication mast

The final example is the 33 kV AT type tower (Rao et al, 2012) shown in Fig 5. The linear analysis from the reference and the ABAQUS results do not capture the ultimate collapse load level which occurs at a load factor of above 1.0 (GW – ground wire peak, C1 to C3 – top cross arm to bottom cross arm) from the current study in Fig 6. This is predicted at load factor 1.4 in analysis by buckling of bottom leg members (at 1.15 load factor in experimental test). Also, the stiffness shown by the current study is 0.87 times that of commercial FE software. The measured deflection at GW peak reported in the paper was 350mm, while that obtained from analysis is 257 mm (an error of 26%). This shows that despite nonlinear analysis, the error in deflection estimates is not bridged.



33 kV AT type tower elevation, member and connection details. DS - double shear SS - single shear

33 kV AT type tower loading tree (units: kN, mm)

Figure 5. 33 kV AT type tower.

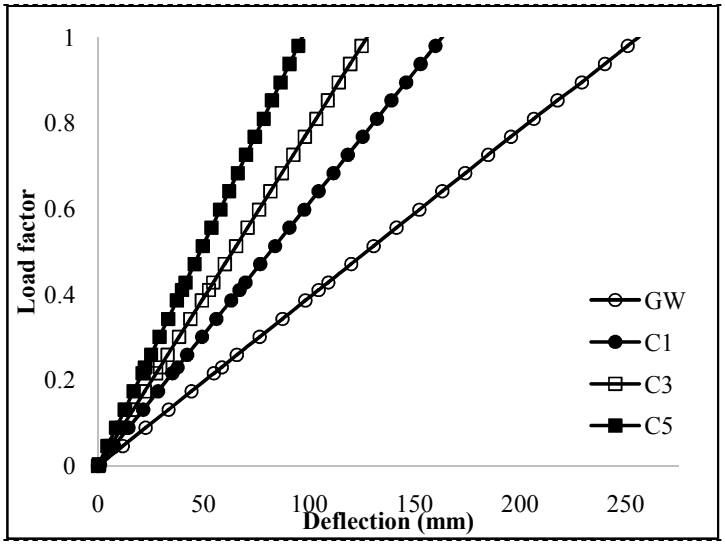


Figure 6 Load-deflection of 33 kV AT type tower

5. Conclusions

The study presents an efficient formulation for the nonlinear analysis of lattice towers based on the corotated approach and is verified through comparisons with an elastic analysis of the towers in commercial FE software ABAQUS. Determination of ultimate collapse loads of the towers is enabled by inclusion of material nonlinearity and reduced stiffness of buckled members. Experimentally measured values of ultimate load and deflections in literature showed that the ultimate loads are predicted with reasonable accuracy through nonlinear analysis. Also the cause and location of failure are predicted from the analysis. However, the analytically estimated deflections are at considerable error in relation to the test values inspite of nonlinear analysis. Thus, other effects like bolt slip and connection stiffness are necessary to enable accurate estimates of deflections through analysis. This necessitates component level studies for joint effects which can be incorporated into the overall analysis of the tower.

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