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Computational framework for mimicking prototype failure testing of transmission line towers

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ABSTRACT

Traditional analysis of transmission line tower structures gives good estimate of ultimate strength but deflection estimates are much less than actual deflections measured from tests. This is caused by additional deflections due to joint properties like bolt slip in splices and splice plate deformations. These two displacement contributing factors are difficult to include in analysis. Prototype tower testing in test beds, which provides design strength evaluation and actual deflections is expensive. This paper presents a simple analytical procedure which captures not only the ultimate load and associated deflections but also the additional deflections because of bolt slip in leg splices. The formulation incorporates a corotational updated Lagrangian procedure in finite element analysis framework. The additional deflections are integrated into a nonlinear analysis formulation used for reticulated structures which also considers member buckling and yielding. To demonstrate the applicability of the formulation, towers which were experimentally tested in test beds are studied and the results are compared with strength and deflection measurements. The formulation has been demonstrated to be a reasonable alternative to expensive proto type testing.

1. Introduction

Transmission line (TL) towers have deformation behaviour similar to tall and flexible structures. The most commonly used sections are angle sections for the leg members, cross arms and bracings and sub-bracings. Stub angles are commonly used to connected tower legs to the base. The connections between leg members are by splice plates with staggered bolting while bracings and sub-bracings are direct member connections. The loading on the tower come from wind pressure, weight of the conductors and insulator strings, and environment effects like temperature and frost upheave (in snow countries). Towers are generally modelled as pin jointed space trusses with linear elastic analysis for designing the leg members, cross arm and primary bracings omitting the sub-bracings from the model to avoid joint instability. Using beam–column elements for tower members joint instabilities due to low out of plane stiffness are avoided.

Tall towers are more flexible and deform in cantilever mode. The elastic deformation is a combination from member deflections and connection deformations like elastic bending of the splice plates and bolt slippage. When towers are very tall these large deformations cannot be predicted with accuracy by conventional analysis. Bolt slip in towers contributes to the error between actual and analytical deflections. They occur because leg members are subject to high axial forces. Bolt slip occurs within the clearance provided to bolt holes for tolerance and tend to add up when many joints are used along the height of tower leg. This makes it necessary to include bolt slip to get better predictions of tower deflections. The large deflections can alter member forces and stiffness which can be studied when tower is modelled using beam–column elements [1]. Al-Bermani and Kitipornchai [2] performed the nonlinear analysis of tower structures to predict ultimate behaviour. Albermani and Kitipornchai [3] proposed a computer simulation to reduce the need for full-scale tower testing. The nonlinear updated Lagrangian technique using thin-walled beam–column elements predicted accurately both the failure load and mode. Kitipornchai et al. [4] modelled lattice towers with the finite element method (FEM) including considerations such as connection rigidity, eccentricity and material nonlinearity. The nonlinear analysis by Albermani et al. [5] predicted the structural failures of full-scale tested transmission line towers. Rao et al. [6] studied the failures and effect of bracing patterns in full-scale towers using a finite element software. Jiang et al. [7] studied the importance of joint effects in lattice transmission towers with the help of nonlinear axial springs to consider bolt slippage. This study showed that bolt

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slip caused high second-order effects due to large deflections of loading points caused by bolt slip. Rao et al. [8] conducted failure analysis of full-scale tested towers in NE-NASTRAN software and found that ASCE, IS and BS codal provisions were unconservative compared to the test and analytical results for both leg and bracing member capacities.

Transmission towers can be included in the category of reticulated structures for the purpose of analysis and design. Study of large deformation behaviour of reticulated structures is available in literature. The relevant works include studies by Papadrakakis [9], Hill et al. [10], Thai and Kim [11], Yang et al. [12] and Ahmadizadeh and Maleek [13]. The use of thin wall beam–column elements for nonlinear analysis of transmission towers by Al-Bermani and Kitipornchai [2] provide more accurate results in the strength domain but even they cannot fully address the inaccuracy in deflection and stiffness domain. Hence this study will model transmission towers as reticulated structure.

Kitipornchai et al. [14] studied the effect of bolt slip on lattice structures assuming both sudden and gradual slip models. Ungkurapinan et al. [15] proposed empirical load–deformation relations considering bolt slip for angles connected without gussets. Reid and Hiser [16] discussed the discrete and stress based clamping models for bolted joints. Rao et al. [17] carried out experimental investigations on the effect of bolt slip on the connection behaviour. Based on their study, they stated that bolt slip in leg member connections of transmission towers causes relative rotations in the joint between the bottom and top leg members connected by the joint. This conclusion is one of the basis of the formulations presented in this paper. The authors also compared the analytical deflection estimates of full-scale tested towers to actual measured deflection after adding experimentally obtained rotations to the analytical estimates. In the present study a simplified geometry based approach to estimate the rotation in tower legs due to bolt slip is integrated into a corotational updated Lagrangian finite element framework, to improve the analytical load–deformation predictions for transmission line towers.

2. Tower analysis

Either a truss analysis using pinned jointed truss elements or a space frame analysis using beam elements or a hybrid analysis using beams for main members and truss elements for the bracing and sub-bracing elements can be used [1]. However using truss elements it is advisable to omit including sub-bracing members in the model because of planar joint instability (zero or negative stiffness in out of plane to the plane of members connected at the joint) which occur at points where sub-bracings join the main bracing members. Hence the sub-bracing members are designed outside the model for a force of 1.5–2% of the force on the main bracing members. From analysis of this model members are designed using provisions in the relevant standards (IS 802 Part 1/Sec 2: 1992 [18]). Similar provisions are mentioned in the ASCE manual 52 – Guide for design of steel transmission towers [19] and IEC 60826 – Design criteria for overhead transmission lines [20]. The loads and load combinations are obtained from IS 802 Part 1/Sec 1: 1995 [21] and IS 875 Part 3: 2000 [22]. Criteria for deflections of towers are not specified in the Indian standards. Certain provisions for tower deflections are reported available in the USSR (erstwhile) standard [23] for small-angle and straight line towers where the deflection limits for peak and cross arms are specified. One has to be careful in applying these limits since deflections from analysis are much lesser than actual deflections. When analytical deflections are compared with codal deflection limits, the designer may be at an error as the actual deflections would very much exceed these limits. This necessitates a correct estimation of actual deflections.

2.1. Nonlinear formulation

In this study a corotated-updated Lagrangian (CR-UL) formulation used for nonlinear analysis of space trusses is used. The corotated formulation offers simplicity and computational efficiency, because it separates rigid body motions of an element from its overall deformation. Particularly, for reticulated structures, as towers, which are defined by space truss elements, 6 degrees of freedom involved in element tangent stiffness matrix is reduced to one degree of freedom. The approaches are described in detail in the Ref. [24]. The final form of the global tangent stiffness $\boldsymbol{K_G}$ is given by

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

where $\boldsymbol{A}$, $\boldsymbol{B}$ and $\boldsymbol{E}$ are transformations matrices described in Ref. [24], $\boldsymbol{q_n} = \boldsymbol{R}_{x2}$ is the element force at the one degree of freedom (ending node) after applying corotation and $\delta\boldsymbol{p}$ is the change in global displacement vector. The element tangent stiffness $\boldsymbol{K}'$ for use in Eq. (1) is obtained by solving the nonlinear finite element equations based on updated Lagrangian approach with displacement shape function defined in terms of $\boldsymbol{p_n}$, the displacement along the ending node after applying corotation. The relevant equations are given below

$$\int_\Omega \left(\delta\mathbf{E}\mathbf{C}\delta\mathbf{E} + \delta\boldsymbol{u}'_{i,x}\sigma\delta\boldsymbol{u}'_{i,x}\right)d\Omega = \int_S \delta\mathbf{t_i}\delta\boldsymbol{u}'_i dS + \int_\Omega \delta\mathbf{f_i}\delta\boldsymbol{u}'_i d\Omega \tag{2}$$

$$\varphi'_2(x') = x'/L \tag{3}$$

$$\delta\boldsymbol{p_n}(x') = [\varphi'_2(x')][\delta\boldsymbol{r}_{x2}] \tag{4}$$

$$\delta\mathbf{E} = \delta\boldsymbol{p}_{n,x} + \frac{1}{2}(\delta\boldsymbol{p}_{n,x})^2 \tag{5}$$

where $\boldsymbol{\Omega}$ is the volume domain, S is the surface domain, $\boldsymbol{C}$ is the constitutive matrix, $\boldsymbol{t_i}$ is the stress vector on the boundary or traction vector, $\boldsymbol{f_i}$ is the body force vector while $\delta\boldsymbol{u}'_\mathbf{i}$ is a vector of virtual displacements in the last known configuration and $\delta\mathbf{E}$ is the change in strain. In the above equations, x in the subscript denotes differentiation with respect to x. From Eqs. (2)–(5), the element tangent stiffness is obtained as

$$\boldsymbol{K}' = \frac{A}{L}[\boldsymbol{C} + \boldsymbol{\sigma}] \tag{6}$$

where $\boldsymbol{K}'$ is the element tangent stiffness and is a 1×1 matrix due to the use of the corotated approach, A and L are the current cross-sectional area and the length of the element respectively and $\boldsymbol{\sigma}$ is the element stress vector. Member buckling is taken by assuming initial mid-span crookedness of $0.001L$. The arc length method traces the load–deflection for crooked columns to get the axial shortening and the buckling loads. This shortening is included then in expression for element tangent stiffness.

$$\boldsymbol{K}' = \frac{1}{\left[\left(\frac{1}{\frac{A}{L}(\boldsymbol{C}+\boldsymbol{\sigma})}\right) + \frac{F_\Delta L}{\boldsymbol{R}_{x2}}\right]} \tag{7}$$

where with δ_c as transverse deflection at midspan F_Δ quantifies the axial shortening.

$$F_\Delta = \left[1 - \frac{1}{1 + \frac{2}{3}\left(\frac{\delta_c}{L}\right)^2}\right] \tag{8}$$

The force carrying capacity of buckled members is reduced according to the shortening of the member with axial strain $\mathbf{E}$ and critical buckling load P_{cr} [25].

$$\boldsymbol{R}_{x2} = P_{cr}\left(1 - \frac{1}{2}\mathbf{E}\right) \tag{9}$$